Handbook of Steel Construction

Eleventh Edition

11th

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INSTITUT CANADIEN DE LA CONSTRUCTION EN ACIER

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FOREWORD

The Canadian Institute of Steel Construction is a national industry organization representing the structural steel, open-web steel joist, and steel plate fabricating industries in Canada. Formed in 1930 and granted a Federal charter in 1942, the CISC functions as a non-profit organization promoting the efficient, economic and sustainable use of fabricated steel in construction.

As a member of the Canadian Steel Construction Council, the Institute has a general interest in all uses of steel in construction. The CISC supports and actively participates in the work of the Standards Council of Canada, the Canadian Standards Association, the Canadian Commission on Building and Fire Codes and numerous other organizations, in Canada and other countries, involved in research work and the preparation of codes and standards.

Preparation of engineering plans is not a function of the CISC. The Institute provides technical information through its professional engineering staff, through the preparation and dissemination of publications, and through the medium of seminars, courses, meetings, videos, and computer programs. Architects, engineers and others interested in steel construction are encouraged to make use of CISC information services.

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PREFACE

This Handbook has been prepared and published by the Canadian Institute of Steel Construction. It is an important part of a continuing effort to provide current and practical information to assist educators, designers, fabricators, and others interested in the use of steel in construction. This Handbook is intended to be used in conjunction with the National Building Code of Canada (NBC) 2015.

The First Edition of the CISC Handbook of Steel Construction was published in 1967, with the Second through Sixth editions following each new edition of the CSA structural steel design standard, now called CSA S16-14. The Seventh Edition introduced CSA G40.21-350W as the basic steel grade for wide-flange (W) and H-pile (HP) shapes in its first printing and incorporated ASTM A992 and A572 grade 50 in its second revised printing. The Eighth Edition based on S16-01 was expanded to include Hollow Structural Sections (HSS) produced to ASTM Specification A500 grade C. The Ninth Edition incorporated S16S1-05 Supplement No. 1, while the Tenth was based on S16-09.

In this Eleventh Edition, member design tables for angles and standard channels are based on G40.21-350W grade steel, which is now commonly available. This increase brings the yield stress level for most tables of compressive and flexural resistances to 345/350 MPa. However, the yield stress of plates and angles used as connecting elements (in Part 3) remains at 300 MPa.

Part 1 is a reprint of CSA S16-14, *Design of Steel Structures*. To assist in understanding the requirements of this standard, Part 2 provides a Commentary prepared by CISC.

Part 3 contains information on bolts and welds with tables for design and evaluation of various structural framing connections. Information on imperial-series bolts has been markedly expanded in Part 3. Featured in this edition are a new design table for all-bolted single-angle connections and new design aids for shear lag in HSS tension members and for strength reduction in multi-orientation fillet welds. Bolt design data for slip-critical joints has been updated to include twist-off bolts and direct tension indicators, and data for bearing-type joints has been expanded to include twist-off bolts.

Part 4 contains information on compression members and introduces new tables of compressive resistances for wide-flange sections produced to ASTM A913 grade 65 and single-angle struts produced to grade CSA G40.21-350W steel. Part 4 also features updated design data on anchor rods, washers, and hole sizes for base plates.

In Part 5 on flexural members, the *Composite Beam Selection Tables* have been expanded to include deep W-shapes.

In Part 6, section properties and dimensions are provided for currently produced steel sections. A new table for the mechanical properties of selected ASTM steel grades has been added, as well as a table describing the common steel grades (CSA and ASTM) for building construction. Metric bolt data included in the Tenth Edition has been moved to a separate section due to lack of availability. The new Eight Edition of the CISC Code of Standard Practice leads the information found in Part 7.

The range of HSS sizes has been extended to incorporate large (Jumbo) sections. Throughout the design tables in Parts 4, 5 and 6, W-shape sections that are commonly used and readily available have been highlighted in yellow colour. It should be noted that data for welded wide-flange sections is no longer provided in this Handbook.

Permission to reprint portions of their publications, granted by the CSA Group and the American Institute of Steel Construction, is gratefully acknowledged. The contributions of Alfred F. Wong, Charles Albert, and Stephanie D'Addese, who helped in the preparation of this publication, are sincerely appreciated.

DESIGNATIONS

Standard designations should always be used to identify structural steel products on drawings and other documents. In Canada, the official designation is the metric (SI) designation, and examples of correct designations for most of the commonly used steel products are provided below. These designations should be used on all design drawings, for detailing purposes and for ordering material.

Shape		Example
W Shapes		W610x113
Miscellaneous M Shapes		M200x9.7
Standard Beams (S Shapes)		S380x64
Standard Channels (C Shapes)		C230x20
Miscellaneous Channels (MC Shapes)		MC250x12.5
Structural Tees	• Cut from W Shapes	WT155x43
	• Cut from M Shapes	MT100x4.9
Bearing Piles (HP Shapes)		HP250x62
Equal-Leg Angles		L102x102x9.5
Unequal-Leg Angles		L127x89x9.5
Plates (thickness × width)		PL8x500
Square Bars (side, mm)		Bar 25⌽
Round Bars (diameter, mm)		Bar 25Φ
Flat Bars (thickness × width)		Bar 5x60
Round Pipe (outside diameter × thickness)		DN300x9.52 †
Hollow Structural Sections	• Square	HSS152x152x9.5 CSA G40.21 Class C *
	• Rectangular	HSS152x102x9.5 CSA G40.21 Class C *
	• Round	HSS141x9.5 CSA G40.21 Class C *
Cold-Formed C-Sections		CFC305S89-326M

† *ASTM A53*

* *HSS steel grades: CSA G40.21-350W Class C or H, or ASTM A500 Grade C*

GENERAL NOMENCLATURE

Explanations of the nomenclature used in many sections of this book appear in those specific sections. In addition, the following symbols are included here for convenience. See also CSA S16-14 Clause 3.2.

A	Area
A_b	Cross-sectional area of one bolt based on nominal diameter
A_e	Effective area of section in compression to account for elastic local buckling
A_f	Flange area
A_n	Net area
A_p	Concrete pull-out area of a shear stud
A_{sc}	Cross-sectional area of a steel shear connector
A_w	Web area; shear area; effective throat area of weld
a	Centre-to-centre distance between transverse web stiffeners; depth of concrete compression zone
a/h	Aspect ratio; ratio of distance between stiffeners to web depth
B	Bearing force in a member or component under specified loads
B_f	Bearing force in a member or component under factored loads
B_r	Factored bearing resistance of a member or component
b	Width of stiffened or unstiffened compression elements; design effective width of concrete slab; overall flange width
b_l	Effective width of slab
b_{el}	width of stiffened of unstiffened compression elements
C	Ratio of connection resistance to the resistance of a single bolt or fillet weld of unit size and length (for computing the resistance of eccentrically loaded bolt or weld groups)
C_e	Euler buckling load
C_f	Compressive force in a member or component under factored loads; factored axial load
C_r	Factored compressive resistance of a member or component
C'_r	Compressive resistance of concrete acting at the centroid of the concrete area in compression
C_w	Warping torsional constant
C_y	Axial compressive load at yield stress
c	Distance from neutral axis to outer fiber of structural shape
c_s	Slip resistance factor for bolted joints (see CSA S16-14 Clause 13.12.2.2)
D	Outside diameter of circular sections; diameter of rocker or roller; stiffener factor; fillet weld size
d	Depth; overall depth of a section; diameter of bolt or stud
E	Elastic modulus of steel (200 000 MPa assumed); effective weld throat
E_c	Elastic modulus of concrete
e	End distance; lever arm between the compressive resistance, C_r, and tensile resistance, T_r
e'	Lever arm between the compressive resistance, C'_r, of concrete and tensile resistance, T_r, of steel
F_a	Acceleration-based site coefficient, as defined in the NBCC
F_{cr}	Critical plate buckling stress

F_s Ultimate shear strength

F_u Specified minimum tensile strength (MPa)

F_v Velocity-based site coefficient, as defined in the NBCC

F_y Specified minimum yield stress, yield point or yield strength (MPa)

f'_c Specified compressive strength of concrete at 28 days (MPa)

g Transverse spacing between fastener gauge lines (gauge distance)

h Clear depth of web between flanges; height of stud

I Moment of inertia

I_E Earthquake importance factor of the structure (see Clause 27 of S16-14 and the NBCC)

$I_EF_aS_a(0.2)$ Specified short-period spectral acceleration ratio (see Clause 27 of S16-14)

$I_EF_vS_a(1.0)$ Specified one-second spectral acceleration ratio (see Clause 27 of S16-14)

I_t Transformed moment of inertia of a composite beam

I_{ts} Transformed moment of inertia of a composite beam based on the modular ratio, n_s

I_x, I_y Moment of inertia about axis x-x, y-y

I_{xd} Effective deflection moment of inertia about X-X axis for cold-formed sections

J St. Venant torsional constant

j Flexural-torsional buckling parameter for cold-formed sections

K Effective length factor

K_x, K_y Effective length factor with respect to axis x-x, y-y

KL Effective length

k Distance from outer face of flange to web toe of fillet of rolled shapes

k_1 Distance from centreline of web to flange toe of fillet of rolled shapes

L Length

L_{cr} Maximum unbraced length adjacent to a plastic hinge; critical unbraced length of distortional buckling for cold-formed sections

L_u Maximum unsupported length of compression flange for which no reduction in factored moment resistance, M_r, is required (for simply-supported beams under uniform moment). See CSA S16-14 Clause 13.6(e).

L_w Length of weld segment

L_x, L_y Unsupported length with respect to axis x-x, y-y

M Mass

M_f Bending moment in a member or component under factored loads

M_{f1} Smaller factored end moment of a beam-column; factored bending moment at a point of concentrated load

M_{f2} Larger factored end moment of a beam-column

M_p Plastic moment = ZF_y

M_r Factored moment resistance of a member or component

M'_r Factored moment resistance of a member of a given unbraced length greater that L_u

M_{rc} Factored moment resistance of a composite beam

M_{rlb} Factored moment resistance based on local buckling for cold-formed sections

M_w Strength reduction factor for multi-orientation fillet welds to account for ductility incompatibility of the individual weld segments

M_y Yield moment = SF_y

m	Number of faying surfaces or shear planes in a bolted joint, equal to 1 for bolts in single shear and 2 for bolts in double shear
N	Length of bearing of an applied load
n	Number of bolts; coefficient used in calculating the factored compressive resistance of a member (see S16-14 Clause 13.3.1 and the Commentary in Part 2)
n_s	Modular ratio of modulus of elasticity of steel to age-adjusted effective modulus of elasticity of concrete, for computing shrinkage deflections of composite beams
P	Concentrated load
P_f	Factored axial load
Q_r	Sum of the factored resistances of all shear connectors between points of maximum and zero moment
q_r	Factored resistance of a shear connector
R	End reaction or concentrated transverse load applied to a flexural member
r	Radius of gyration
$\bar{r}_o$	Polar radius of gyration of a singly-symmetric section about the shear centre (see Clause 13.3.2 of S16-14)
r_u, r_v	Radius of gyration with respect to axis *u-u*, *v-v*
r_x, r_y	Radius of gyration with respect to axis *x-x*, *y-y*
r'_y	Radius of gyration of a member about its minor principal axis
r_z	Radius of gyration with respect to axis *z-z*
S	Elastic section modulus
$S_a(T)$	5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of T in seconds, as defined in the NBCC
S_x	Elastic section modulus with respect to axis *x-x*
S_y	Elastic section modulus with respect to axis *y-y*
s	Centre-to-centre spacing (pitch) between successive fastener holes in line of applied force
T	Theoretical weld throat
T_f	Tensile force in a member or component under factored loads
T_r	Factored tensile resistance of a member or component; factored tensile resistance of the steel acting at the centroid of that part of the steel area in tension
t	Thickness
U	Amplification factor for stability analysis of beam-columns
U_t	Factor to account for efficiency of the tensile area
V_f	Shear force in a member or component under factored loads
V_r	Factored shear resistance of a member or component
V_s	Slip resistance of a bolted joint
W	Total uniformly distributed load (kN); concentrated load; weld face width
w	Web thickness; load per unit of length
x_o	Horizontal coordinate of the shear centre of a section
Y_o	Vertical coordinate of the shear centre of a section
Z	Plastic section modulus of a steel section

α Angle between the geometric and principal axes of a cross-section

α_l Ratio of average stress in a rectangular compression block to the specified concrete strength

β Coefficient for weak-axis bending in beam-columns

β_x Asymmetry parameter for singly-symmetric beams (see Clause 13.6(e) of S16-14)

γ_c Density of concrete

Δ Deflection of a point of a structure

θ Angle between a weld segment and the line of applied force

κ Ratio of the smaller to the larger factored end moment, positive for double curvature and negative for single curvature

λ Non-dimensional slenderness ratio for compression members

ϕ Resistance factor

Ω Section property used in computing the flexural-torsional buckling resistance of a singly-symmetric section (see Clause 13.3.2 of S16-14)

ASTM American Society for Testing and Materials

CISC Canadian Institute of Steel Construction

CPMA Canadian Paint Manufacturers' Association (*now known as the Canadian Paint and Coatings Association*)

CSCC Canadian Steel Construction Council

CSA Canadian Standards Association

NBCC National Building Code of Canada

RCSC Research Council on Structural Connections

SSPC Steel Structures Painting Council (*now known as the Society for Protective Coatings*)

SSRC Structural Stability Research Council

PART ONE

CSA S16-14

DESIGN OF STEEL STRUCTURES

General

This Standard is reprinted with the permission of CSA Group and contains all errata and revisions approved at time of printing. The reprint includes CSA S16-14 "Design of Steel Structures" (June 2014), Errata – October 2015 and Update No. 1 – December 2016.

CSA Standards are subject to periodic review. For information on updates to S16-14, see page 1-iv.

For information on requesting interpretations, see Note (4) in the Preface to S16-14.

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Design of steel structures

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Preface

This is the eighth edition of CSA S16, *Design of steel structures*. It supersedes the previous limit states editions published in 2009, 2001, 1994, 1989, 1984, 1978, and 1974. These limit states design editions were preceded by seven working stress design editions published in 1969, 1965, 1961, 1954, 1940, 1930, and 1924. The 1969 working stress design edition was withdrawn in 1984, from which point the design of steel structures in Canada has been carried out using limit states design principles.

This Standard is appropriate for the design of a broad range of structures. It sets out minimum requirements and is expected to be used only by engineers competent in the design of steel structures. The following is a list of some of the more important changes made in this edition:

a) Clause 1.4 specifically prohibits the use of other standards for fabrication, erection and inspection.
b) The definition of "snug-tightness" has been clarified.
c) Information required on design documents has been augmented.
d) ASTM grades A500/A500M, A1085 and A913/A913M have been added as permissible steel grades for design.
e) The fire endurance design requirements have been restated to be in compliance with the *NBCC*.
f) Requirements under impulse loading have been added.
g) The initial misalignment of members at brace points has been clarified.
h) A calculation for the net area of a slotted HSS member has been given.
i) The minimum b/t for bearing stiffeners has been added.
j) The clause permitting a joist manufacturer to determine the joist resistance by testing has been removed.
k) Provisions for column stiffeners opposite a rigidly connected beam by bolting have been provided.
l) Requirements for zinc-aluminum coated assemblies have been incorporated.
m) The use of plate washers in lieu of hardened washers is permitted in oversize or slotted holes.
n) The use of non-matching electrodes is permitted with reference to W59 for locations where this is permitted.
o) Clause 24 that referred to joint surface conditions for field welding in the previous edition has been removed and is now covered in CSA W47.1.
p) The factored resistance of anchor rods in bearing has been referred to CSA A23.3 to be consistent with other Canadian design standards.
q) A clarification on fatigue calculations has been made to include bending moments due to joint eccentricities.
r) An upper limit on the design force of single-storey buildings' roof diaphragms has been provided.
s) A minimum Charpy V-notch value has been specified for weld of primary members and connections.
t) A maximum sulfur content for ASTM A913 used in seismic-resisting systems is specified.
u) Additional criteria for joint connections have been added to ductile moment-resisting frames, limited ductility moment-resisting frames, and moderately ductile concentrically braced frames.
v) The design of link beams for ductile eccentrically braced frames has been expanded.
w) Detailing information for limited ductility plate walls has been given.
x) Annex K Structural design for fire conditions has been updated.
y) The clauses related to pin-connected members have been revised to clarify the net section and resistance requirements.

A commentary on this Standard, prepared by the Canadian Institute of Steel Construction with contributions from many members of the Technical Committee, comprises Part 2 of the Institute's *Handbook of Steel Construction*.

This Standard is intended to be used with the provisions of the 2015 edition of the *National Building Code of Canada* (*NBCC*), specifically Clause 7, which references the *NBCC* for load factors, load combinations, and other loading provisions..

This Standard was prepared by the Technical Committee on Steel Structures for Buildings, under the jurisdiction of the Strategic Steering Committee for Construction and Civil Infrastructure, and has been formally approved by the Technical Committee. Annex N was prepared by the Working Group on Design and Construction of Steel Storage Racks.

This edition of the CSA S16 is dedicated to the memories of Laurie Kennedy, André Picard, and Richard Redwood, three distinguished designers, researchers, and devoted educators committed to the advancement of steel standards.

Notes:

1) *Use of the singular does not exclude the plural (and vice versa) when the sense allows.*

2) *Although the intended primary application of this Standard is stated in its Scope, it is important to note that it remains the responsibility of the users of the Standard to judge its suitability for their particular purpose.*

3) *This Standard was developed by consensus, which is defined by CSA Policy governing standardization — Code of good practice for standardization as "substantial agreement. Consensus implies much more than a simple majority, but not necessarily unanimity". It is consistent with this definition that a member may be included in the Technical Committee list and yet not be in full agreement with all clauses of this Standard.*

4) *To submit a request for interpretation of this Standard, please send the following information to* **inquiries@csagroup.org** *and include "Request for interpretation" in the subject line:*
   - a) *define the problem, making reference to the specific clause, and, where appropriate, include an illustrative sketch;*
   - b) *provide an explanation of circumstances surrounding the actual field condition; and*
   - c) *where possible, phrase the request in such a way that a specific "yes" or "no" answer will address the issue.*

   Committee interpretations are processed in accordance with the CSA Directives and guidelines governing standardization and are available on the Current Standards Activities page at standardsactivities.csa.ca.

5) *This Standard is subject to review five years from the date of publication. Suggestions for its improvement will be referred to the appropriate committee. To submit a proposal for change, please send the following information to* **inquiries@csagroup.org** *and include "Proposal for change" in the subject line:*
   - a) *Standard designation (number);*
   - b) *relevant clause, table, and/or figure number;*
   - c) *wording of the proposed change; and*
   - d) *rationale for the change.*

S16-14
Design of steel structures

1 Scope and application

1.1 General
This Standard provides rules and requirements for the design, fabrication, and erection of steel structures. The design is based on limit states. The term "steel structures" refers to structural members and frames that consist primarily of structural steel components, including the detail parts, welds, bolts, or other fasteners required in fabrication and erection. This Standard also applies to structural steel components in structures framed in other materials. The clauses related to fabrication and erection serve to show that design is inextricably a part of the design-fabrication-erection sequence and cannot be considered in isolation. For matters concerning standard practice pertinent to the fabrication and erection of structural steel not covered in this Standard, see Annex A.

1.2 Requirements
Requirements for steel structures such as bridges, antenna towers, offshore structures, and cold-formed steel structural members are given in other CSA Group Standards.

1.3 Application
This Standard applies unconditionally to steel structures, except that supplementary rules or requirements might be necessary for
a) unusual types of construction;
b) mixed systems of construction;
c) steel structures that
   i) have great height or spans;
   ii) are required to be movable or be readily dismantled;
   iii) are exposed to severe environmental conditions;
   iv) are exposed to severe loads such as those resulting from vehicle impact or explosion;
   v) are required to satisfy aesthetic, architectural, or other requirements of a non-structural nature;
   vi) employ materials or products not listed in Clause 5; or
   vii) have other special features that could affect the design, fabrication, or erection;
d) tanks, stacks, other platework structures, poles, and piling; and
e) crane-supporting structures.

1.4 Other standards
The use of other standards for the design, fabrication, erection, and/or inspection of members or parts of steel structures is neither warranted nor acceptable except where specifically directed in this Standard. The design formulas provided in this Standard may be supplemented by a rational design based on theory, analysis, and engineering practice acceptable to the regulatory authority, provided that nominal margins (or factors) of safety are at least equal to those intended in the provisions of this Standard. The substitution of other standards or criteria for fabrication, erection, and/or inspection is expressly prohibited unless specifically directed in this Standard.

1.5 Terminology

In this Standard, "shall" is used to express a requirement, i.e., a provision that the user is obliged to satisfy in order to comply with the standard; "should" is used to express a recommendation or that which is advised but not required; and "may" is used to express an option or that which is permissible within the limits of the Standard.

Notes accompanying clauses do not include requirements or alternative requirements; the purpose of a note accompanying a clause is to separate from the text explanatory or informative material.

Notes to tables and figures are considered part of the table or figure and may be written as requirements.

Annexes are designated normative (mandatory) or informative (non-mandatory) to define their application.

2 Reference publications

This Standard refers to the following publications, and where such reference is made, it shall be to the edition listed below, including all amendments published thereto.

CSA Group

A23.1/A23.2-14
Concrete materials and methods of concrete construction/Test methods and standard practices for concrete

A23.3-14
Design of concrete structures

① A344-17
User guide for steel storage racks

A660-10
Certification of manufacturers of steel building systems

B95-1962 (withdrawn)
Surface Texture (Roughness, Waviness, and Lay)

G40.20-13/G40.21-13
General requirements for rolled or welded structural quality steel/Structural quality steel

CAN/CSA-G164-M92 (withdrawn)
Hot Dip Galvanizing of Irregularly Shaped Articles

G189-1966 (withdrawn)
Sprayed Metal Coatings for Atmospheric Corrosion Protection

S136-12
North American specification for the design of cold-formed steel structural members

S136.1-12
Commentary on North American specification for the design of cold-formed steel structural members

S304-14
Design of masonry structures

S850-12
Design and assessment of buildings subject to blast loads

W47.1-09
Certification of companies for fusion welding of steel

W48-14
Filler metals and allied materials for metal arc welding

W55.3-08 (R2013)
Certification of companies for resistance welding of steel and aluminum

W59-13
Welded steel construction (metal arc welding)

W178.1-14
Certification of welding inspection organizations

W178.2-14
Certification of welding inspectors

ASTM International (American Society for Testing and Materials)

A27/A27M-10
Standard Specification for Steel Castings, Carbon, for General Application

A108-07
Standard Specification for Steel Bar, Carbon and Alloy, Cold-Finished

A148/A148M-08
Standard Specification for Steel Castings, High Strength, for Structural Purposes

A216/A216M-12
Standard Specification for Steel Castings, Carbon, Suitable for Fusion Welding, for High-Temperature Service

A307-12
Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60 000 PSI Tensile Strength

A325-10e1
Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

A325M-13
Standard Specification for Structural Bolts, Steel, Heat Treated 830 MPa Minimum Tensile Strength (Metric)

A352/A352M-06(2012)
Standard Specification for Steel Castings, Ferritic and Martensitic, for Pressure-Containing Parts, Suitable for Low-Temperature Service

① A370-15
Standard Test Methods and Definitions for Mechanical Testing of Steel Products

A490-12
Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength

A490M-12
Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints (Metric)

A500/A500M-10a
Standard Specification for Cold Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

A514/A514M-05(2009)
Standard Specification for High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding

A521/A521M-06(2011)
Standard Specification for Steel, Closed-Impression Die Forgings for General Industrial Use

A563-07a
Standard Specification for Carbon and Alloy Steel Nuts

A572/A572M-12a
Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel

A668/A668M-13
Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use

A913/A913M-11
Standard Specification for High Strength Low Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self Tempering Process

A958/A958M-10
Standard Specification for Steel Castings, Carbon and Alloy, with Tensile Requirements, Chemical Requirements Similar to Standard Wrought Grades

A992/A992M-11
Standard Specification for Structural Steel Shapes

A1011/A1011M-12b
Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength

A1085-13
Standard Specification for Cold-Formed Welded Carbon Steel Hollow Structural Sections (HSS)

F436-11
Standard Specification for Hardened Steel Washers

F959-13
Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners

F1554-07ae1
Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength

F1852-11
Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

F2280-12
Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 150 ksi Minimum Tensile Strength

CGSB (Canadian General Standards Board)
CAN/CGSB-48.9712-2014/ISO 9712:2012
Non-destructive testing — Qualification and certification of NDT personnel

CISC (Canadian Institute of Steel Construction)
Code of Standard Practice for Structural Steel (2009)

Crane-Supporting Steel Structures: Design Guide, 2nd ed. (April 2013)

Handbook of Steel Construction, 11th ed. (2015)

Hollow Structural Section: Connections and Trusses — A Design Guide, 2nd ed. (June 1997)

Moment Connections for Seismic Applications, 2nd ed. (2014)

CISC/CPMA (Canadian Institute of Steel Construction/Canadian Paint Manufacturing Association)
1-73a (1975)
A Quick-Drying One-Coat Paint for Use on Structural Steel

2-75 (1975)
A Quick-Drying Primer for Use on Structural Steel

ERF (European Racking Federation)
EN 15512:2009
Steel static storage systems. Adjustable pallet racking systems. Principles for structural design

FEMA (Federal Emergency Management Agency)
460-2005
Seismic Considerations for Steel Storage Racks Located in Areas Accessible to the Public

① **ISO/IEC (International Organization for Standardization/International Electrotechnical Commission)**
17024:2012
Conformity assessment — General requirements for bodies operating certification of persons

National Research Council Canada
National Building Code of Canada, 2015

User's Guide — NBC 2015: Structural Commentaries (Part 4)

RCSC (Research Council on Structural Connections)
Guide to Design Criteria for Bolted and Riveted Joints, 2nd ed., 2001

Specification for Structural Joints Using ASTM A325 or A490 Bolts, 2000

① **RMI (Rack Manufacturers Institute)**
RMI/ANSI MH 16.1-2012
Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks

① **SAE (Society of Automotive Engineers)**
J429-2014
Mechanical and Material Requirements for Externally Threaded Fasteners

SSPC (Society for Protective Coatings)
SP 1 (2004)
Solvent Cleaning

SP 2 (2004)
Hand Tool Cleaning

SP 3 (2004)
Power Tool Cleaning

SP 5/NACE No. 1 (2007)
White Metal Blast Cleaning

SP 6/NACE No. 3 (2007)
Commercial Blast Cleaning

SP 7/NACE No. 4 (2007)
Brush-Off Blast Cleaning

SP 10/NACE No. 2 (2007)
Near-White Blast Cleaning

SP 11 (2004)
Power Tool Cleaning to Bare Metal

SP 12/NACE No. 5
Surface Preparation and Cleaning of Metals by Waterjetting Prior to Recoating

SP 14/NACE No. 8
Industrial Blast Cleaning

Structural Stability Research Council
Guide to Stability Design Criteria for Metal Structures, 6th ed., 2010

ULC (Underwriters Laboratories of Canada)
CAN/ULC-S101-07
Standard Methods of Fire Endurance Tests of Building Construction and Materials

Other publications
Frank, K. H. and Fisher, J. W. "Fatigue Strength of Welded Cruciform Joints", *Journal of the Structural Division*, ASCE. Vol.105, ST9, pp. 1727-1740, September 1979.

Higgins, P. "Displacement based Design for Storage Racks", *ASCE/SEI Conference Proceedings*, Long Beach, CA. 2007.

3 Definitions and symbols

Δ 3.1 Definitions

The following definitions apply in this Standard:

Approved — approved by the regulatory authority.

Brace point — the point on a member or element at which it is restrained (see Clause 9).

Camber — the deviation from straightness of a member or any portion of a member with respect to its major axis.
Note: *Frequently, camber is specified and produced in a member to compensate for deflections that occur in the member when loaded (see Clause 6.3.2). Unspecified camber is sometimes referred to as bow.*

Concrete — portland cement concrete in accordance with CSA A23.1.

Deck or decking — the structural floor or roof element spanning between adjacent joists and directly supported thereby.
Note: *The terms "deck" and "decking" include cast-in-place or precast concrete slabs, profiled metal deck, wood plank or plywood, and other relatively rigid elements suitable for floor or roof construction (see Clause 16).*

Designer — the engineer responsible for the design.

Erection tolerances — tolerances related to the plumbness, alignment, and level of the piece as a whole.
Note: *The deviations are determined by considering the location of the ends of the piece (see Clause 29).*

Fabrication tolerances — tolerances allowed from the nominal dimensions and geometry, such as cutting to length, finishing of ends, cutting of bevel angles, and out-of-straightness such as sweep and camber for fabricated members (see Clause 28).

Factors —

Importance factor, *I* — a factor applied severally to loads due to snow and rain, wind, or earthquake for both the ultimate and serviceability limit states.
Note: *It is based on the importance of the structure as defined by its use and occupancy (see Clause 6.2.2).*

Load factor, α — a factor, given in Clause 7.2, applied to a specified load for the limit states under consideration that takes into account the variability of the loads and load patterns and the analysis of their effects.

Resistance factor, φ — a factor, given in Clause 13.1, applied to a specified material property or the resistance of a member, connection, or structure that, for the limit state under consideration, takes into account the variability of material properties, dimensions, quality of work, type of failure, and uncertainty in prediction of member resistance.
Note: *To maintain the simplicity of the design formulas in this Standard, the type of failure and the uncertainty in prediction of member resistance have been incorporated in the expressions of member resistance (see Annex B for a more detailed discussion).*

Fatigue limit state — the limiting case of the slow propagation of a crack within a structural element that can result either from live load effects (load-induced fatigue effect) or as the consequence of local distortion within the structure (distortion-induced fatigue effects).

Firm contact — the condition that exists on a faying surface when plies are solidly seated against each other but not necessarily in continuous contact (see Clause 23.2).

Inspector — a qualified person who acts for and on behalf of the owner or designer on all inspection and quality matters within the scope of the contract documents.

Joist shoe — the connection assembly located at the junction of the top chord and the end diagonal that allows the joist to bear on its support (see Clause 16).

Limit states — those conditions of a structure under which the structure ceases to fulfill the function for which it was designed.

Fatigue limit states — conditions that concern safety and are related to crack propagation under cyclic loading.

Serviceability limit states — conditions that restrict the intended use and occupancy of the structure and include deflection, vibration, and permanent deformation.

Ultimate limit states — conditions that concern safety and include overturning, sliding, fracturing, and exceeding load-carrying capacity.

Loads —

Companion load — a specified variable load that accompanies the principal load in a given load combination.

Factored load — the product of a specified load and its load factor.

Gravity load (newtons) — a load equal to the mass of the object (kilograms) being supported multiplied by the acceleration due to gravity, g (9.81 m/s^2).

Notional lateral load — a fictitious lateral load, as given in Clause 8.4, that allows the stability of the frame, with failure modes involving in-plane bending, to be computed based on the actual length (K = 1) for beam-columns.

Principal load — the specified variable load or rare load that dominates in a given load combination.

Specified loads ($D, E, H, L, L_{-c}, C, C_d, C_T, P, S, T$, and W) — those loads prescribed by the regulatory authority (see Clause 6.2.1).

Mill tolerances — variations allowed from the nominal dimensions and geometry with respect to cross-sectional area, non-parallelism of flanges, and out-of-straightness such as sweep or camber in the product as manufactured and given in CSA G40.20.

Modulus of elasticity of concrete — the ratio of stress to strain in the elastic range of a stress-strain curve for concrete and, with density, γ_c, between 1500 and 2500 kg/m^3, is taken as follows:

$$E_c = \left(3300\sqrt{f'_c} + 6900\right)\left(\frac{\gamma_c}{2300}\right)^{1.5}$$

For normal density concrete with compressive strength, f'_c, between 20 and 40 MPa, the modulus of elasticity may be taken as follows:

$$E_c = 4500\sqrt{f'_c}$$

Pass through force — a load or force defined by the Structural Designer that must be accommodated in the design of the structural member(s) and the connections between those designated members in addition to those loads and forces normally associated in the member and connection design of each individual interconnecting member.

Protected zone — areas of members in a seismic force resisting system that undergo large inelastic strains and in which limitations apply to fabrication and attachments. See Clause 27.1.9.

Regulatory authority — a federal, provincial/territorial, or municipal ministry, department, board, agency, or commission that is responsible for regulating by statute the use of products, materials, or services.

Resistance —

Factored resistance, ϕR — the product of the nominal resistance and the appropriate resistance factor.

Nominal resistance, R — the nominal resistance of a member, connection, or structure as calculated in accordance with this Standard and based on the specified material properties and nominal dimensions.

Segmented member — a member with a constant cross-section when axial loads are applied between in-plane lateral supports or frame connections, and a member with cross-section changes between in-plane lateral supports or frame connections.

Seismic design storey drift — the storey drift obtained from the lateral deflections obtained from a linear elastic analysis multiplied by R_dR_o/I_e (see Clause 27).

Δ **Snug tightness** — the tightness that is attained with a few impacts of an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the connected plies into firm contact.

Span of an open-web steel joist — the centre-to-centre distance of joist bearings or shoes (see Clause 16).

Sweep — the deviation from straightness of a member or any portion of a member with respect to its minor axis.

Tie joists — joists that are designed to resist gravity loads only and, in accordance with Clause 16.5.12.2, have at least one end connected to a column to facilitate erection.

Truss — a triangulated framework loaded primarily in flexure (see Clause 15).

3.2 Symbols

The following symbols are used throughout this Standard. Deviations or additional nomenclature are noted where they appear.

A = area

A_{ar} = cross-sectional area of an anchor rod based on its nominal diameter

A_b = cross-sectional area of a bolt based on its nominal diameter; cross-sectional area of a plate wall beam

A_c = transverse area of concrete between longitudinal shear planes; cross-sectional area of concrete in composite columns; cross-sectional area of a plate wall column; effective area of concrete slab

A_{cv} = critical area of two longitudinal shear planes, one on each side of the area A_c, extending from the point of zero moment to the point of maximum moment

A_e = effective area of section in compression to account for elastic local buckling (see Clause 13.3.5)

A_f = flange area

A_g = gross area

A_{gv} = gross area in shear for block failure (see Clause 13.11)

A_m = area of fusion face

A_n = net area; the tensile area of a rod

A_{ne} = effective net area reduced for shear lag

A_p = concrete pull-out area

A_r = area of reinforcing steel

A_s = area of steel section, including cover plates; area of bottom (tension) chord of a steel joist; area of a stiffener or pair of stiffeners

A_{sc} = cross-sectional area of a steel shear connector; cross-sectional of the yielding segment of the steel core of a buckling restrained brace

A_{se} = effective steel area (see Clause 18.3.2)

A_{st} = area of steel section in tension

A_w = web area; shear area; effective throat area of a weld

a = centre-to-centre distance between transverse web stiffeners; depth of the concrete compression zone

a' = length of cover plate termination

a/h = aspect ratio; ratio of distance between stiffeners to web depth

B = bearing force in a member or component under specified load

B_f = bearing force in a member or component under factored load

B_r = factored bearing resistance of a member or component

b_l = longer leg of angle in Clause 13.3.3

b_s = shorter leg of angle in Clause 13.3.3

b = overall width of flange; design effective width of concrete or cover slab

b_{el} = width of stiffened or unstiffened compression elements

b_c = width of concrete at the neutral axis specified in Clause 18.2.3; width of column flange

b_e = effective flange width in Clause 18.3.2

b_f = width of flange

C_e = Euler buckling strength

= $\pi^2 EI/L^2$

C_{ec} = Euler buckling strength of a concrete-filled hollow structural section

C_f = compressive force in a member or component under factored load; factored axial load

C_{fs} = factored sustained axial load on a composite column

C_p = nominal compressive resistance of a composite column when $\lambda = 0$ (see Clause 18.3.2)

C_r = factored compressive resistance of a member or component; factored compressive resistance of steel acting at the centroid of that part of the steel area in compression

C_{rc} = factored compressive resistance of a composite column

C_{rcm} = factored compressive resistance that can coexist with M_{rc} when all of the cross-section is in compression

C_{rco} = factored compressive resistance with $\lambda = 0$

C_r' = compressive resistance of concrete acting at the centroid of the concrete area assumed to be in uniform compression; compressive resistance of a concrete component of a composite column

C_w = warping torsional constant, mm^6

C_y = axial compressive load at yield stress

c = cohesion stress for concrete (1.0 MPa) in accordance with Clause 11.5.2 c) of CSA A23.3

c_1 = coefficient used to determine slip resistance

D = outside diameter of circular sections; diameter of rocker or roller; stiffener factor; dead load

d = depth; overall depth of a section; diameter of a bolt or stud

d_b = depth of beam

d_c = depth of column

E = elastic modulus of steel (200 000 MPa assumed); earthquake load and effects (see Clause 6.2.1)

E_c = elastic modulus of concrete

E_c' = age adjusted effective modulus of electricity of concrete

E_{ct} = effective modulus of concrete in tension

e = end distance; lever arm between the compressive resistance, C_r, and the tensile resistance, T_r; length of link in eccentrically braced frames

e' = lever arm between the compressive resistance, C_r' of concrete and tensile resistance, T_r, of steel

F = strength or stress

F_a = acceleration-based site coefficient (see Clause 27 and the *NBCC*)

F_{cr} = critical plate-buckling stress in compression, flexure, or shear

F_{cre} = elastic critical plate-buckling stress in shear

F_{cri} = inelastic critical plate-buckling stress in shear

F_e = Euler buckling stress; elastic buckling stress

F_s = ultimate shear stress

F_{sr} = allowable stress range in fatigue

F_{srt} = constant amplitude threshold stress range

F_{st} = factored axial force in the stiffener

F_u = specified minimum tensile strength

F_v = velocity-based site coefficient (see Clause 27 and the *NBCC*)

F_y = specified minimum yield stress, yield point, or yield strength

F_y' = yield level, including effect of cold-working

F_{ye} = effective yield stress of section in compression to account for elastic local buckling (see Clause 13.3.5)

F_{yr} = specified yield strength of reinforcing steel

f_c' = specified compressive strength of concrete at 28 days

f_{sr} = calculated stress range at detail due to passage of the fatigue load

G = shear modulus of steel (77 000 MPa assumed)

g = transverse spacing between fastener gauge lines (gauge distance)

H = weld leg size; permanent load due to lateral earth pressure (see Clause 6.2.1)

h = clear depth of web between flanges; height of stud; storey height
h_c = clear depth of column web
h_d = depth of steel deck
h_s = storey height
I = moment of inertia
I_b = moment of inertia of a beam
I_c = moment of inertia of a column
I_E = earthquake importance factor of the structure (see Clause 27 and the *NBCC*)
I_e = effective moment of inertia of a composite beam
I_g = moment of inertia of a cover-plated section
I_S = importance factor for snow load as defined in Table 4.1.6.2 of the *NBCC*
I_s = moment of inertia of OWSJ or truss
I_t = transformed moment of inertia of a composite beam
I_W = importance factor for wind load as defined in Table 4.1.7.1 of the *NBCC*
I_{yc} = moment of inertia of compression flange about the y-axis [see Clause 13.6 e)]
I_{yt} = moment of inertia of tension flange about the y-axis [see Clause 13.6 e)]
J = St. Venant torsional constant
K = effective length factor
K_z = effective length factor for torsional buckling
KL = effective length
k = distance from outer face of flange to web-toe of fillet of I-shaped sections; factor as specified in Clause 18.3.2
k_a = coefficient used in determining inelastic shear resistance
k_b = buckling coefficient; required stiffness of the bracing assembly
k_s = mean slip coefficient
k_v = shear buckling coefficient
L = length or span; length of longitudinal or flare bevel groove weld; live load; length of connection in direction of loading; centre-to-centre distance between columns in a plate wall; length of member between work points at truss chord centrelines in Clause 13.3.3
L_c = length of channel shear connector
L_{cr} = maximum unbraced length adjacent to a plastic hinge
L_u = longest unbraced length with which a beam will reach either $M_r = \phi M_p$ or $M_r = \phi M_y$, depending on the class of the cross-section [see Clause 13.6 e)]
L_{yr} = shortest unbraced length with which a singly symmetric beam will undergo elastic lateral-torsional buckling [see Clause 13.6 e)]
M = bending moment in a member or component under specified load
M_a = factored bending moment at one-quarter point of unbraced segment
M_b = factored bending moment at mid-point of unbraced segment
M_c = factored bending moment at three-quarter point of unbraced segment
M_f = bending moment in a member or component under factored load

M_{f1} = smaller factored end moment of a beam-column; factored bending moment at a point of concentrated load

M_{f2} = larger factored end moment of a beam-column

M_{fc} = bending moment in a girder, under factored load, at theoretical cut-off point

M_{max} = maximum factored bending moment magnitude in unbraced segment

M_p = plastic moment resistance = ZF_y

= ZF_y

M_{pb} = plastic moment of a beam

M_{pc} = plastic moment of a column

M_r = factored moment resistance of a member or component

M_{rc} = factored moment resistance of a composite beam; factored moment resistance of a column reduced for the presence of an axial load

M_u = critical elastic moment of a laterally unbraced beam

M_w = strength reduction factor for multi-orientation fillet welds to account for ductility incompatibility of the individual weld segments

M_y = yield moment resistance = SF_y

M_{yr} = yield moment resistance of a singly symmetric beam including the effects of residual stresses [see Clause 13.6 e)]

m = number of faying surfaces or shear planes in a bolted joint

= 1.0 for bolts in single shear

= 2.0 for bolts in double shear

N = length of bearing of an applied load; number of passages of moving load

N' = number of passages of moving load at which $F_{sr} = F_{srt}$

N_{fi} = number of cycles that would cause failure at stress range level i

n = number of bolts; number of shear connectors required between the point of maximum positive bending moment and the adjacent point of zero moment; parameter for compressive resistance; number of threads per inch; number of stress range cycles at a given detail for each passage of the moving load; modular ratio, E/E_c

n' = number of shear connectors required between any concentrated load and nearest point of zero moment in a region of positive bending moment

n_s = modular ratio, $E/E_c^{\prime}$

n_t = modular ratio, E/E_{ct}

P = force to be developed in a cover plate; pitch of threads; permanent effects caused by prestress (see Clause 6.2.1)

P_b = force used to design the bracing system (when two or more points are braced, the forces P_b alternate in direction)

P_f = factored axial force

p = fraction of full shear connection

Q_r = sum of the factored resistances of all shear connectors between points of maximum and zero moment

q_r = factored resistance of a shear connector

q_{rr} = factored resistance of a shear connector in a ribbed slab

q_{rs} = factored resistance of a shear connector in a solid slab

R = end reaction or concentrated transverse load applied to a flexural member; nominal resistance of a member, connection, or structure; transition radius

R_d = ductility-related force modification factor that reflects the capability of a structure to dissipate energy through inelastic behaviour (see Clause 27 and the *NBCC*)

R_o = overstrength-related force modification factor that accounts for the dependable portion of reserve strength in a structure (see Clause 27 and the *NBCC*)

R_y = factor applied to F_y to estimate the probable yield stress

r = radius of gyration

r_t = radius of gyration of a compression flange plus one-third of web area in compression due to major axis bending [see Clause 13.6 e) i)]

r_x = radius of gyration of a single-angle member about its geometric axis parallel to the connected leg in Clause 13.3.3

r_y = radius of gyration of a member about its weak axis

r_y' = radius of gyration of a member about its minor principal axis

S = elastic section modulus of a steel section; variable load due to snow (see Clause 6.2.1)

$S_a(0.2)$ = 5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of 0.2 s (see Clause 27 and the *NBCC*)

$S_a(1.0)$ = 5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of 1 s (see Clause 27 and the *NBCC*)

S_e = effective section modulus as defined in Clause 13.5 c)

s = centre-to-centre longitudinal spacing (pitch) of any two successive fastener holes; longitudinal stud spacing; vertical spacing of tie bars (see Clause 18.3.1)

T = tensile force in a member or component under specified load; load effects due to contraction, expansion, or deflection (see Clause 6.2.1); period of a structure (see Clause 27 and the *NBCC*)

T_f = tensile force in a member or component under factored load

T_r = factored tensile resistance of a member or component; in composite construction, factored tensile resistance of the steel acting at the centroid of that part of the steel area in tension

T_y = axial tensile load at yield stress

t = thickness; thickness of flange; average flange thickness of channel shear connector

t_b = thickness of beam flange

t_c = concrete or cover slab thickness; thickness of column flange

t_p = thickness of plate

U_t = factor to account for efficiency of the tensile area (see Clause 13.11)

U_1 = factor to account for moment gradient and for second-order effects of axial force acting on the deformed member

U_2 = amplification factor to account for second-order effects of gravity loads acting on the laterally displaced storey

V = shear force in a member or component under specified load

V_f = shear force in a member or component under factored load

V_h = total horizontal shear to be resisted at the junction of the steel section or joist and the slab or steel deck; shear acting at plastic hinge locations when plastic hinging occurs

V_p = plastic shear resistance = $0.55wdF_y$

V_r = factored shear resistance of a member or component

V_{re} = probable shear resistance of a steel plate wall

V_s = slip resistance of a bolted joint

V_{st} = factored shear force in column web to be resisted by stiffener

W = wind load

w = web thickness; width of plate; infill plate thickness (see Clause 20)

w' = sum of thickness of column web plus doubler plates

w_c = column web thickness

w_d = average width of flute of steel deck

w_f = width of flare bevel groove weld face

w_n = net width (i.e., gross width less design allowance for holes within the width)

X_u = ultimate strength as rated by the electrode classification number

x = subscript relating to strong axis of a member; distance from flange face to centre of plastic hinge

$\bar{x}$ = eccentricity of the weld with respect to centroid of the element

x_o, y_o = principal coordinates of the shear centre with respect to the centroid of the cross-section

y = subscript relating to weak axis of a member; distance from centroid of cover plate to neutral axis of cover-plated section; distance from centroid of the effective area of concrete slab to elastic neutral axis

Z = plastic section modulus of a steel section

z = subscript related to Z-axis of a member

α = load factor; angle of inclination from vertical (see Clause 20)

α_f = angle between shear friction reinforcement and shear plane in concrete

α_1 = ratio of average stress in rectangular compression block to the specified concrete strength

β = value used to determine bracing stiffness; angle in radians as specified in Clause 18.2.3; coefficient for bending in beam-columns, as specified in Clause 13.8.2 or Clause 18.2.4

β_x = asymmetry parameter for singly symmetric beams as specified in Clause 13.6 e)

γ = fatigue life constant

γ' = fatigue life constant at which $F_{sr} = F_{srt}$

γ_c = density of concrete

Δ_b = displacement of the bracing system at the point of support under force C_f (may be taken as Δ_o)

Δ_f = relative first-order lateral (translational) displacement of the storey due to factored loads

Δ_o = initial misalignment of the member at a brace point (see Clause 9.2)

Δ_s = deflection due to shrinkage of concrete

εf = free shrinkage strain of concrete

κ = ratio of the smaller to the larger factored end moment, positive for double curvature and negative for single curvature (see Clauses 13.6 and 13.8)

λ = non-dimensional slenderness parameter in column formula; modification factor for concrete density

λ_p = non-dimensional slenderness parameter as specified in Clause 18.3.2

μ = coefficient of friction for concrete (1.4) in accordance with Clause 11.5.2 c) of CSA A23.3

ρ = density of concrete; slenderness ratio

ρ_e = equivalent slenderness ratio of a built-up member

ρ_i = maximum slenderness ratio of the component part of a built-up member between interconnectors

ρ_o = slenderness ratio of a built-up member acting as an integral unit

ρ_v = ratio of shear friction reinforcing steel in concrete extending from the point of zero moment to the point of maximum moment

ΣC_f = sum of factored axial compressive loads of all columns in the storey

ΣV_f = sum of factored lateral loads above the storey; total first-order storey shear

σ = effective normal stress for concrete in accordance with Clause 11.5.3 of CSA A23.3

σ_{ct} = tensile stress in concrete

ϕ = resistance factor as defined in Clause 2 and specified in Clause 13.1

ω_h = non-dimensional column flexibility parameter for plate walls

ω_L = non-dimensional boundary member flexibility parameter for extreme panels of plate walls

ω_1 = coefficient to determine equivalent uniform bending effect in beam-columns (see Clause 13.8)

ω_2 = coefficient to account for increased moment resistance of a laterally unsupported doubly symmetric beam segment when subject to a moment gradient [see Clause 13.6 a)]

ω_3 = coefficient to account for modified moment resistance of a laterally unsupported singly symmetric beam segment when subject to a moment gradient [see Clause 13.6 e)]

3.3 Units

Equations and expressions appearing in this Standard are compatible with the following SI (metric) units:

a) force: N (newtons);
b) length: mm (millimetres);
c) moment: N•mm; and
d) strength or stress: MPa (megapascals).

4 Structural documents

4.1 General

The term "structural documents" may include drawings, specifications, computer output, and electronic and other data.

4.2 Structural design documents

4.2.1
The structural design documents shall show a complete design of the structure with members suitably designated and located, including such dimensions and details as necessary to permit the preparation of fabrication and erection documents. Floor levels, column centres, and offsets shall be dimensioned. Structural design drawings shall be to a scale adequate to convey the required information.

4.2.2
In addition to the information required by the applicable building code, the structural design documents shall include, but not be limited to, the following information, as applicable:
a) the design standards used;
b) the material or product standards (see Clause 5);
c) the design criteria for snow, wind, seismic, and special loads;
d) the specified live, dead loads, and superimposed dead loads;
e) the type or types of construction (see Clause 8);
f) the structural system used for seismic design and the seismic design criteria (see Clause 27);
g) the requirements for roof and floor diaphragms;
h) the design criteria for open-web steel joists (see Clause 16);
i) the design criteria for crane-supporting structures (see Annex C);
j) all load-resisting elements essential to the integrity of the completed structure and the details necessary to ensure the effectiveness of the load-resisting system in the completed structure;
k) the camber of beams, girders, and trusses;
l) the governing combinations of shears, moments, axial forces, torsions including pass through forces to be resisted by the connections;
m) the bracing required to stabilize compression elements including the size and location of stiffeners and/or reinforcement;
n) the types of bolts, the pretensioning requirements, and the designation of joints as bearing or slip-critical (see Clause 22.2);
o) the type and configuration details of structural connections that are critical for ductile seismic response; and
p) the locations and dimensions of protected zones (see Clause 27.1.9).

4.2.3
Revisions to design documents shall be clearly indicated and dated.

4.2.4
Provided that all requirements for the structural steel are shown on the structural documents, architectural, electrical, and mechanical documents may be used as supplements to the structural documents to define the detail configurations and construction information.

4.3 Fabrication and erection documents

4.3.1 Connection design details
Connection design details shall be prepared before the preparation of shop details and submitted to the structural designer for confirmation that the intent of the design is met. Connection design details shall provide details of standard and non-standard connections and other data necessary for the preparation

of shop details. Connection design details shall be referenced to the design documents, erection drawings, or both.

4.3.2 Shop details

Shop details shall

a) be prepared before fabrication and submitted to the structural designer for review;
b) provide complete information for the fabrication of various members and components of the structure, including the
   i) required material and product standards;
   ii) location, type, and size of all mechanical fasteners;
   iii) bolt installation requirements; and
   iv) welds; and
c) provide the locations and dimensions of the protected zones and a complete description of the fabrication operations that are prohibited in protected zones.

4.3.3 Erection diagrams

Erection diagrams shall be submitted to the designer for review. Erection diagrams are general arrangement drawings that should show the principal dimensions of the structure, piece marks, sizes of the members, all steel load-resisting elements essential to the integrity of the completed structure, size and type of bolts, field welds, bolt installation requirements, elevations of column bases, all necessary dimensions and details for setting anchor rods, and any other information necessary for the assembly of the structure. Erection diagrams shall provide the locations and dimensions of the protected zones and a complete description of the erection operations that are prohibited in protected zones.

4.3.4 Erection procedures

Erection procedures shall outline the construction methods, erection sequence, temporary bracing requirements, and other engineering details necessary for shipping, erecting, and maintaining the stability of the steel frame. Erection procedures shall be supplemented by drawings and sketches that identify the location of permanent and temporary load-resisting elements essential to the integrity of the partially completed structure. Erection procedures shall be submitted for review when so specified.

4.3.5 Fieldwork details

Fieldwork details shall be submitted to the designer for review. Fieldwork details shall provide complete information for modifying fabricated members in the shop or on the job site. All operations required to modify the member shall be shown on the fieldwork details. If extra materials are necessary to make modifications, shop details shall be required.

5 Material — Standards and identification

5.1 Standards

5.1.1 General

Acceptable material and product standards and specifications for use under this Standard are specified in Clauses 5.1.3 to 5.1.10. Materials and products other than those specified may be used if approved. Approval shall be based on published specifications that establish the properties, characteristics, and suitability of the material or product to the extent and in the manner of those covered in specified standards and specifications.

5.1.2 Strength levels

The yield strength, F_y, and the tensile strength, F_u, used as the basis for design shall be the specified minimum values as given in the material and product standards and specifications. The levels reported on mill test certificates shall not be used as the basis for design.

Δ 5.1.3 Structural steel

Structural steel shall meet the requirements of CSA G40.20/G40.21, ASTM A500/A500M, ASTM A1085, ASTM A572/A572M, ASTM A913/A913M, or ASTM A992/A992M. The design properties for ASTM A500/A500M products shall be determined from wall thickness equal to 90% of the nominal wall thickness.

5.1.4 Sheet steel

Sheet steel shall meet the requirements of ASTM A1011/A1011M.

Other standards for structural sheet are listed in Section A2 of CSA S136. Only structural-quality sheet standards that specify chemical composition and mechanical properties shall be acceptable for conformance with this Standard. Mill test certificates that list the chemical composition and the mechanical properties shall be available, upon request, in accordance with Clause 5.2.1 a).

5.1.5 Cast steel

Cast steel shall meet the design requirements for weldability, strength, ductility, toughness, and surface finish.

Note: *Reference standards include ASTM A27/A27M, ASTM A148/A148M, ASTM A216/A216M, ASTM A352/A352M, and ASTM A958/A959M.*

Δ 5.1.6 Forged steel

Forged steel shall meet the requirements of ASTM A521/A521M or ASTM A668/A668M.

Δ 5.1.7 Bolts and bolt assemblies

Bolts and bolt assemblies shall meet the requirements of ASTM A307, ASTM A325, ASTM A325M, ASTM A490, ASTM A490M, ASTM F1852, or ASTM F2280.

Note: *Before specifying metric bolts, the designer should check on their availability in the quantities required.*

5.1.8 Welding electrodes

Welding electrodes shall meet the requirements of CSA W48, as applicable.

5.1.9 Studs

Studs shall meet the requirements of ASTM A108.

5.1.10 Anchor rods

Anchor rods shall meet the requirements of CSA G40.20/G40.21 or ASTM F1554.

5.2 Identification

5.2.1 Methods

The specifications (including type or grade, if applicable) of the materials and products used shall be identified by the following means, except as specified in Clauses 5.2.2 and 5.2.3:

a) mill test certificates or producer's certificates satisfactorily correlated to the materials or products to which they pertain; and

b) legible markings on the material or product made by its producer in accordance with the applicable material or product standard.

5.2.2 Unidentified structural steel

Unidentified structural steel shall not be used unless approved by the building designer. If the use of unidentified steel is authorized, F_y shall be taken as 210 MPa and F_u shall be taken as 380 MPa.

5.2.3 Tests to establish identification

Unidentified structural steel may be tested to establish identification when permitted by the building designer. Testing shall be done by an approved testing agency in accordance with CSA G40.20. The test results, taking into account both mechanical properties and chemical composition, shall form the basis for classifying the steel as to specification. Once classified, the specified minimum values for steel of that specification grade shall be used as the basis for design (see Clause 5.1.2).

5.2.4 Affidavit

The fabricator, if requested, shall provide an affidavit stating that the materials and products that have been used in fabrication conform to the applicable material or product standards called for by the design drawings or specifications.

6 Design requirements

6.1 General

6.1.1 Limit states

Steel structures designed in accordance with this Standard shall be safe from collapse during construction and designed to be safe and serviceable during the useful life of the structure. Limit states define the various types of collapse and unserviceability that are to be avoided. Those concerning safety are called the ultimate limit states (strength, overturning, sliding, and fracture) or the fatigue limit state (crack propagation) and those concerning serviceability are called the serviceability limit states (deflections, vibration, and permanent deformation). The object of limit states design calculations is to keep the probability of reaching a limit state below a certain value previously established for the given type of structure. This is achieved in this Standard by the use of load factors applied to the specified loads (see Article 4.1.2.1 of the *National Building Code of Canada* [*NBCC*]) and resistance factors applied to the specified resistances (see Clause 13 and Annex B of this Standard).

The various limit states are specified in Clause 6. Some of these relate to the specified loads and others to the factored loads. Camber, provisions for expansion and contraction, and corrosion protection are further design requirements related to serviceability and durability. All limit states shall be considered in the design.

6.1.2 Structural integrity

The general arrangement of the structural system and the connection of its members shall be designed to provide resistance to disproportionate collapse as a consequence of local failure. The requirements of this Standard generally provide a satisfactory level of structural integrity for steel structures.

Note: *Further guidance can be found in the* User's Guide — NBC 2015: Structural Commentaries (Part 4).

6.2 Loads

6.2.1 Specified loads

Except as provided for in Clause 7.1, the loads and influences specified in Article 4.1.2.1 of the *NBCC* shall be considered in the design of structural steelwork, taking into consideration that the regulatory authority might specify other loads in some circumstances.

6.2.2 Importance factors based on use and occupancy

The specified snow, wind, and earthquake loads shall be multiplied by the importance factors for the different importance categories for buildings in accordance with Article 4.1.2.1 of the *NBCC*. For buildings having a Low Importance Category, the factor of 0.8 for the ultimate limit states may be applied to the live load, L.

6.3 Requirements under specified loads

6.3.1 Deflection

6.3.1.1

Steel members and frames shall be proportioned so that deflections are within acceptable limits for the nature of the materials to be supported and for the intended use and occupancy. Consideration shall be given to the differential deflections of adjacent parallel framing members in the same plane.

Note: *In the absence of a more detailed evaluation, see Annex D for recommended values for deflections.*

6.3.1.2

Roofs shall be designed to withstand any additional loads likely to occur as a result of ponding (see also Clause 6.2.1).

Note: *Further guidance can be found in the* User's Guide — NBC 2015: Structural Commentaries (Part 4).

6.3.2 Camber

6.3.2.1

Camber of beams, trusses, or girders, if necessary, shall be stipulated on the design drawings. Generally, trusses and crane girders with a span of 25 m or greater should be cambered for approximately the dead-plus-half-live-load deflection.

Note: *See Clause 16 for requirements for open-web joists, Clause 15 for requirements for trusses, and Clause 28.6 for fabrication tolerances.*

6.3.2.2

Any special camber requirements necessary to bring a loaded member into proper relation with the work of other trades shall be stipulated on the design drawings.

Note: *See also Clause 6.3.1.1. See Clause 16.12.2.5 for maximum deviation in elevation between adjacent joists.*

6.3.3 Dynamic effects

6.3.3.1

Suitable provision shall be made in the design for the effect of live loads that induce impact, vibration, or both. In severe cases, e.g., structural supports for heavy machinery that causes substantial impact or vibration when in operation, the possibility of harmonic resonance, fatigue, or unacceptable vibration shall be investigated.

6.3.3.2
Special consideration shall be given to floor systems susceptible to vibration, e.g., large open floor areas free of partitions, to ensure that such vibration is acceptable for the intended use and occupancy.
Note: *For further information, see Annex E.*

6.3.3.3
Unusually flexible structures (generally those whose ratio of height to effective resisting width exceeds 4:1) shall be investigated for lateral vibrations under dynamic wind load. Lateral accelerations of the structure shall be checked to ensure that such accelerations are acceptable for the intended use and occupancy.
Note: *Information on lateral accelerations under dynamic wind loads can be found in the* User's Guide — NBC 2015: Structural Commentaries (Part 4).

6.3.4 Resistance to fatigue
Structural steelwork shall be designed to resist the effects of fatigue under specified loads in accordance with Clause 26.

6.4 Requirements under factored loads

6.4.1 Strength
Structural steelwork shall be proportioned to resist moments and forces resulting from the application of the factored loads acting in the most critical combination, taking into account the resistance factors specified in Clause 13.1.

6.4.2 Overturning
The building or structure shall be designed to resist overturning resulting from the application of the factored loads acting in the most critical combination, taking into account the importance category of the building as specified in Clause 6.2.2 and the resistance factors specified in Clause 13.1.

6.5 Expansion and contraction
Suitable provision shall be made for expansion and contraction commensurate with the service and erection conditions of the structure.

6.6 Corrosion protection

6.6.1
Steelwork shall have sufficient corrosion protection to minimize any corrosion likely to occur in the service environment.

6.6.2
Interiors of buildings conditioned for human comfort may be generally assumed to be non-corrosive environments; however, the need for corrosion protection shall be assessed and protection shall be furnished in those buildings where it is deemed to be necessary.

6.6.3
Corrosion protection of the inside surfaces of enclosed spaces permanently sealed from any external source of oxygen shall not be necessary.

6.6.4
The minimum required thickness of steelwork situated in a non-corrosive environment and therefore not requiring corrosion protection shall be in accordance with Clause 11.

6.6.5
Corrosion protection shall be provided by means of suitable alloying elements in the steel, by protective coatings, or by other effective means, either singly or in combination.

6.6.6
Localized corrosion likely to occur from trapped water, excessive condensation, or other factors shall be minimized by suitable design and detail. Where necessary, positive means of drainage shall be provided.

6.6.7
If the corrosion protection specified for steelwork exposed to the weather, or to other environments in which progressive corrosion can occur, is likely to require maintenance or renewal during the service life of the structure, the steelwork so protected, exclusive of fill plates and shims, shall have a minimum thickness of 4.5 mm.

① 6.7 Requirements under fire conditions
The fire endurance of structural steelwork for buildings shall be determined using CAN/ULC-S101. When permitted by the regulatory authority, a performance-based fire protection analysis and design of structural steelwork shall be conducted using the methods specified in Annex K.

Note: *Annex K is an "alternative solution" that can be evaluated to determine compliance with the* NBCC *(Division A, Compliance, Objectives and Functional Statements).*

6.8 Brittle fracture
The risk of brittle fracture in steel structures subjected to tensile stresses shall be assessed.

Note: *See Annex L for guidance on material selection and details to minimize the risk of brittle fracture.*

6.9 Requirements under impulse loading
Structural steelwork that has been determined by the authority having jurisdiction to be potentially subjected to impulse loads shall follow design concepts and details that will mitigate collapse.

Notes:
1) *Annex L provides recommendations to prevent brittle fracture.*
2) *CSA S850 provides guidelines to account for blast loads.*

7 Factored loads and safety criterion

7.1 Safety during erection and construction
Suitable provision shall be made for loads imposed on the steel structure during its erection. During subsequent construction, suitable provision shall be made to support the construction loads on the steel structure with an adequate margin of safety.

7.2 Safety criterion and effect of factored loads for the ultimate limit states

7.2.1
The structural steelwork shall be designed to have sufficient strength or stability, or both, such that factored resistance is greater than or equal to the effect of factored loads, as follows:

$\phi R \geq \Sigma \alpha_i S_i$

where the factored resistance is determined in accordance with the applicable clauses of this Standard and the effect of factored loads for the ultimate limit states is determined in accordance with Division B, Article 4.1.3.2 of the *NBCC*.

7.2.2
The effect of factored loads in force units shall be determined from the structural effect due to the specified loads, including importance factors due to use and occupancy (see Clause 6.2), multiplied by the load factors, α, for load combination cases in accordance with Division B, Article 4.1.3.2 of the *NBCC*.

8 Analysis of structure

8.1 General
In proportioning the structure to meet the design requirements of Clause 6, the methods of analysis specified in Clause 8 shall be used. The distribution of internal forces and bending moments shall be determined both under the specified loads to satisfy the requirements of serviceability and fatigue specified in Clause 6 and under the factored loads to satisfy strength and overturning requirements specified in Clause 7.

8.2 Types of construction

8.2.1 General
Three basic types of construction and associated design assumptions, i.e., "rigidly connected", "simple", and "semi-rigid" (see Clauses 8.2.2 to 8.2.4) may be used for all or part of a structure under this Standard. The distribution of internal forces and bending moments throughout the structure shall depend on the type or types of construction chosen and the forces to be resisted.

8.2.2 Rigidly connected and continuous construction
In this construction, the beams, girders, and trusses are rigidly connected to other frame members or are continuous over supports. Connections shall be generally designed to resist the bending moments and internal forces calculated by assuming that the angles between intersecting members remain unchanged as the structure is loaded.

8.2.3 Simple construction
Simple construction assumes that the ends of beams, girders, and trusses are free to rotate under load in the plane of loading. Resistance to lateral loads, including stability effects, shall be ensured by a suitable system of bracing or plate walls or by the design of part of the structure as rigidly connected or semi-rigid construction.

8.2.4 Semi-rigid (partially restrained) construction

8.2.4.1
In this construction, the angles between connected members change under applied bending moments and redistribute the moments between members while maintaining sufficient capacity to resist lateral loads and to provide adequate stability of the framework in accordance with Clause 8.4.

8.2.4.2
The design and construction of semi-rigid frameworks shall meet the following requirements:

a) The positive and negative moment/rotation response of the connections up to their maximum capacity shall have been established by test and either published in the technical literature or be available from a reputable testing facility.

b) The design of the structure shall be based on either linear analysis employing the secant stiffness of connections at ultimate load or incremental analyses following the non-linear test response of the connections.

c) Consideration shall be given to the effects of repeated vertical and horizontal loading and load reversals, with particular regard to incremental strain in connections and low-cycle fatigue.

8.3 Analysis methods

8.3.1 Elastic analysis
Under a particular loading combination, the forces and moments throughout all or part of the structure may be determined by an analysis that assumes that individual members behave elastically.

① 8.3.2 Plastic analysis
Under a particular loading combination, the forces and moments throughout all or part of the structure may be determined by a plastic analysis, provided that

a) the steel used has $F_y \le 0.85F_u$ and exhibits the stress-strain characteristics necessary to achieve moment redistribution;

b) the width-to-thickness ratios of member cross-section elements meet the requirements of Class 1 sections as specified in Clause 11.2;

c) the members are braced laterally in accordance with the requirements of Clause 13.7;

d) web stiffeners are supplied on a member at a point of load application where a plastic hinge would form;

e) splices in beams or columns are designed to transmit 1.1 times the maximum calculated moment under factored loads at the splice location or $0.25M_p$, whichever is greater;

f) members are not subject to repeated heavy impact or fatigue; and

g) the influence of inelastic deformation on the strength of the structure is taken into account (see Clause 8.4).

8.4 Stability effects

8.4.1
The translational load effects produced by notional lateral loads, applied at each storey, equal to 0.005 times the factored gravity loads contributed by that storey, shall be added to the lateral loads for each load combination. The notional lateral loads shall be applied in both orthogonal directions independently when the three-dimensional effects of loading are included in the analysis of the structure.

8.4.2
The analyses referred to in Clause 8.3 shall include the sway effects in each storey produced by the vertical loads acting on the structure in its displaced configuration. The second-order effects that are due to the relative translational displacement (sway) of the ends of a member shall be determined from a second-order analysis. Elastic second-order effects may be accounted for by amplifying translational load effects obtained from a first-order elastic analysis by the factor

$$U_2 = \frac{1}{1 - \left[\frac{\Sigma C_f \Delta_f}{\Sigma V_f h}\right]}$$

Note: *For combinations including seismic loads, see Clause 27.1.8.2 for the expression of U_2.*

9 Stability of structures and members

9.1 Stability of structures
The structural system shall be adequate to
a) resist the forces caused by factored loads;
b) transfer the factored loads to the foundations;
c) transfer forces from walls, floors, or roofs acting as shear-resisting elements or diaphragms to adjacent lateral-load-resisting elements; and
d) resist torsional effects.

See also Clause 8.4.

9.2 Stability of members

9.2.1 Initial misalignment at brace point
The initial misalignment of the member at a brace point, Δ_O, shall be taken such that the offset of that brace point relative to the adjacent brace points from the alignment shown on the drawings corresponds to the out-of-alignment tolerance specified in Clause 29.3.

9.2.2 Displacement of bracing systems
The displacement of the bracing system at the brace point, Δ_b, is the sum of the brace deformation, the brace connection deformation, and the brace support displacement. This displacement is due to the brace force and any other forces acting on the brace and shall be calculated in the direction perpendicular to the braced member at the brace point.

9.2.3 Function of bracing
Bracing systems provide lateral support to columns, the compression flange of beams and girders, or the compression chords of joists or trusses.

Bracing systems, including bracing members and their connections and supports, shall be proportioned to resist the forces that develop at the brace points and limit the lateral displacement of the brace points.

Bracing for beams shall provide lateral restraint to the compression flange, except that at cantilevered ends of beams and beams subject to double curvature, the restraint shall be provided at both top and bottom flanges unless otherwise accounted for in the design.

9.2.4 Twisting and lateral displacements

Twisting and lateral displacements shall be prevented at the supports of a member or element unless accounted for in the design.

9.2.5 Simplified analysis

Bracing systems shall be proportioned to have a strength perpendicular to the longitudinal axis of the braced member in the plane of buckling equal to at least 0.02 times the factored compressive force at each brace point in the member or element being braced, unless a detailed analysis is carried out in accordance with Clause 9.2.6 to determine the appropriate strength and stiffness of the bracing system. Any other forces acting on the bracing member shall also be taken into account. The displacement Δ_b shall not exceed Δ_o.

9.2.6 Detailed analysis

9.2.6.1 Second-order method

Forces acting in the member bracing system and its deformations shall be determined by means of a second-order elastic analysis of the member and its bracing system. This analysis shall include the most critical initial deformed configuration of the member and shall consider forces due to external loads. In the analysis, hinges may be assumed at brace points in the member or element being braced.

The displacement Δ_b shall not exceed Δ_o unless a greater value can be justified by analysis.

9.2.6.2 Direct method

Unless a second-order analysis is carried out in accordance with Clause 9.2.6.1 or a simplified analysis is carried out in accordance with Clause 9.2.5, bracing systems shall be proportioned at each brace point to have a factored resistance in the direction perpendicular to the longitudinal axis of the braced member in the plane of buckling equal to at least

$$P_b = \frac{\beta\left(\Delta_o + \Delta_b\right)C_f}{L_b}$$

where

P_b = force used to design the bracing system (when two or more points are braced, the forces P_b alternate in direction)

β = 2, 3, 3.41, 3.63, or 4 for 1, 2, 3, 4, or more equally spaced braces, respectively, unless a lesser value can be justified by the analysis

Δ_o = initial misalignment

Δ_b = displacement of the bracing system, assumed to be equal to Δ_o for the initial calculation of P_b

C_f = maximum factored compression in the segments bound by the brace points on either side of the brace point under consideration

L_b = length between braces

For flexural members, P_b shall be increased, as appropriate, when loads are applied above the shear centre or for beams in double curvature.

After P_b and any other forces acting on the bracing member are applied, the calculated displacement of the bracing system, Δ_b, shall not exceed Δ_o unless justified by analysis.

9.2.7 Slabs or decks

When bracing of the compression flange is effected by a slab or deck, the slab or deck and the means by which the calculated bracing forces are transmitted between the flange or chord and the slab or deck shall be adequate to resist a force in the plane of the slab or deck. This force, which shall be taken as at least 0.05 times the maximum force in the flange or chord unless a lesser amount can be justified by analysis, shall be considered to be uniformly distributed along the length of the compression flange or chord.

9.2.8 Accumulation of forces

Consideration shall be given to the probable accumulation of forces, C_f, when the bracing system restrains more than one member. When members are erected with random out-of-straightness, the initial misalignment may be taken as

$$(0.2 + 0.8 / \sqrt{n})\Delta_o$$

where

n = number of members or elements being braced

This reduction shall not be applied when member initial misalignments are dependent on each other and are likely to be in the same direction and of the same magnitude.

9.2.9 Torsion

Bracing systems for beams, girders, and columns designed to resist loads causing torsion shall be proportioned in accordance with Clause 14.10. Special consideration shall be given to the connection of asymmetric sections such as channels, angles, and Z-sections.

10 Design lengths and slenderness ratios

10.1 Simple span flexural members

Beams, girders, and trusses may be designed on the basis of simple spans, whose length may be taken as the distance between the centres of gravity of supporting members. Alternatively, the span length of beams and girders may be taken as the actual length of such members measured between centres of end connections. The length of trusses designed as simple spans may be taken as the distance between the extreme working points of the system of triangulation employed. The design of columns or other supporting members shall provide for the effect of any significant moment or eccentricity arising from the manner in which a beam, girder, or truss is connected or supported.

10.2 Continuous span flexural members

Beams, girders, or trusses having full or partial end restraint due to continuity or cantilever action shall be proportioned to carry all moments, shears, and other forces at any section, assuming the span, in general, to be the distance between the centres of gravity of the supporting members. Supporting members shall be proportioned to carry all moments, shears, and other forces induced by the continuity of the supported beam, girder, or truss.

10.3 Members in compression

10.3.1 General

A member in compression shall be designed on the basis of its effective length, KL (the product of the effective length factor, K, and the unbraced length, L).

Unless otherwise specified in this Standard, the unbraced length, L, shall be taken as the length of the compression member between the centres of restraining members. The unbraced length may differ for different cross-sectional axes of a compression member. At the bottom storey of a multi-storey structure or for a single-storey structure, L shall be taken as the length from the top of the base plate to the centre of restraining members at the next higher level.

The effective length factor, K, depends on the potential failure modes, whether by bending in-plane or buckling, as specified in Clauses 10.3.2 and 10.3.3.

Note: *See also Clause 9 on the effectiveness of the brace or support point.*

10.3.2 Failure mode involving bending in-plane

The effective length shall be taken as the actual length ($K = 1.0$) for beam-columns that would fail by in-plane bending, provided that, when applicable, the sway effects, including notional load effects, are included in the analysis of the structure to determine the end moments and forces acting on the beam-columns.

10.3.3 Failure mode involving buckling

The effective length for axially loaded columns that would fail by buckling and for beam-columns that would fail by out-of-plane (lateral-torsional) buckling shall be based on the rotational and translational restraint afforded at the ends of the unbraced length (see Annexes F and G).

10.4 Slenderness ratios

10.4.1 General

The slenderness ratio of a member in compression shall be taken as the ratio of the effective length, KL, to the corresponding radius of gyration, r. The slenderness ratio of a member in tension shall be taken as the ratio of the unbraced length, L, to the corresponding radius of gyration.

10.4.2 Maximum slenderness ratio

10.4.2.1

The slenderness ratio of a member in compression shall not exceed 200.

10.4.2.2

Except as specified in Clauses 15.2.7 and 16.5.6.1, the slenderness ratio of a member in tension shall not exceed 300. This limit may be waived if other means are provided to control flexibility, sag, vibration, and slack in a manner commensurate with the service conditions of the structure or if it can be shown that such factors are not detrimental to the performance of the structure or of the assembly of which the member is a part.

11 Width (or diameter)-to-thickness — Elements in compression

11.1 Classification of sections

11.1.1

For the purposes of this Standard, structural sections shall be designated as Class 1, 2, 3, or 4, depending on the maximum width (or diameter)-to-thickness ratios of the elements subject to compression, and as otherwise specified in Clauses 11.1.2 and 11.1.3, as follows:

a) Class 1 sections permit attainment of the plastic moment and subsequent redistribution of the bending moment;
b) Class 2 sections permit attainment of the plastic moment but need not allow for subsequent moment redistribution;
c) Class 3 sections permit attainment of the yield moment; and
d) Class 4 sections generally have elastic local buckling of elements in compression as the limit state of structural resistance.

11.1.2

Class 1 sections, when subject to flexure, shall have an axis of symmetry in the plane of loading and, when subject to axial compression, shall be doubly symmetric.

11.1.3

Class 2 sections, when subject to flexure, shall have an axis of symmetry in the plane of loading unless the effects of asymmetry of the section are included in the analysis.

11.2 Maximum width (or diameter)-to-thickness ratios of elements subject to compression

The maximum width (or diameter)-to-thickness ratios of elements subject to axial compression shall be as specified in Table 1 and those of elements subject to flexural compression shall be as specified in Table 2, for the specified section classification.

Sections that exceed the limits presented in Table 1 or Table 2 shall be classified as Class 4 sections. The factored axial compressive resistance of Class 4 sections shall be calculated in accordance with Clause 13.3.5. The factored bending resistance of Class 4 sections shall be calculated in accordance with Clause 13.5.

11.3 Width and thickness

11.3.1

For elements supported along only one edge parallel to the direction of compressive force, the width, b_{el}, shall be taken as follows:

a) plates: shall be the distance from the free edge to the first row of fasteners or line of welds;
b) legs of angles, flanges of channels and Z's, and stems of T's: the full nominal dimension; and
c) flanges of beams and T's: one-half of the full nominal dimension.

11.3.2

For elements supported along two edges parallel to the direction of compressive force, the width shall be taken as follows:

a) flange or diaphragm plates in built-up sections: the width, b_{el}, shall be the distance between adjacent lines of fasteners or lines of welds;
b) flanges, b_{el}, and webs, h, of rectangular hollow sections (HSS) shall be the nominal outside dimension less four times the wall thickness;
c) webs of built-up sections: the width, h, shall be the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; and
d) webs of hot-rolled sections: the width, h, shall be the clear distance between flanges.

11.3.3

The thickness of elements, t or w, shall be taken as the nominal thickness. For tapered flanges of rolled sections, the thickness shall be taken as the nominal thickness halfway between a free edge and the corresponding face of the web.

12 Gross and net areas

12.1 Application

Members in tension shall be proportioned on the basis of the areas associated with the potential failure modes. Members in compression shall be proportioned on the basis of the gross area associated with the potential failure mode.

Note: *For beams and girders, see Clause 14.*

12.2 Gross area

Gross area shall be calculated by summing the products of the thickness and the gross width of each element (flange, web, leg, plate), as measured normal to the axis of the member.

12.3 Net area

12.3.1 General

The net area, A_n, shall be determined by summing the critical net areas, A_n, of each segment along a potential path of minimum resistance calculated as follows:

a) for a segment normal to the force (i.e., in direct tension):
$A_n = w_n t$
b) for a segment inclined to the force between openings (e.g., bolt holes) but not parallel to the force:

$$A_n = w_n t + \frac{s^2 t}{4g}$$

12.3.2 Allowance for bolt holes

In calculating w_n, the width of bolt holes shall be taken as 2 mm larger than the specified hole dimension. If drilled holes are used, this allowance may be waived.

12.3.3 Effective net area — Shear lag

12.3.3.1

When fasteners transmit load to each of the cross-sectional elements of a member in tension in proportion to their respective areas, the effective net area shall be taken as the net area, i.e., $A_{ne} = A_n$.

12.3.3.2

When bolts transmit load to some but not all of the cross-sectional elements and when the critical net area includes the net area of unconnected elements, the effective net area shall be taken as follows:

a) for WWF, W, M, or S shapes with flange widths not less than two-thirds the depth, and for structural tees cut from these shapes, when only the flanges are connected with three or more transverse lines of fasteners:
   $A_{ne} = 0.90A_n$

b) for angles connected by only one leg with
   i) four or more transverse lines of fasteners:
      $A_{ne} = 0.80A_n$
   ii) fewer than four transverse lines of fasteners:
      $A_{ne} = 0.60A_n$

c) for all other structural shapes connected with
   i) three or more transverse lines of fasteners:
      $A_{ne} = 0.85A_n$
   ii) two transverse lines of fasteners:
      $A_{ne} = 0.75A_n$

12.3.3.3

When a tension load is transmitted by welds, the effective net area, A_{ne}, shall be computed as the sum of the effective net areas of the elements, A_{n1}, A_{n2}, and A_{n3}, as applicable, but shall not exceed A_g. The net areas of the connected plate elements shall be defined as follows:

a) for elements connected by transverse welds, A_{n1}:
   $A_{n1} = wt$

b) for elements connected by longitudinal welds along two parallel edges, A_{n2}:
   i) when $L \geq 2w$:
      $A_{n2} = 1.00wt$
   ii) when $2w > L \geq w$:
      $A_{n2} = 0.50wt + 0.25Lt$
   iii) when $w > L$:
      $A_{n2} = 0.75Lt$

   where
   L = average length of welds on the two edges
   w = plate width (distance between welds)

c) for elements connected by a single longitudinal weld, A_{n3}:
   i) when $L \geq w$:

$$A_{n3} = \left(1 - \frac{\bar{x}}{L}\right)wt$$

   ii) when $w > L$:
      $A_{n3} = 0.50Lt$

   where
   $\bar{x}$ = eccentricity of the weld with respect to centroid of the connected element

L = length of weld in the direction of the loading

The outstanding leg of an angle shall be considered connected by the (single) line of weld along the heel.

12.3.3.4

When round or rectangular HSS members are slotted and welded to a plate, the effective net area, A_{ne}, of the HSS member under concentric tension shall be taken as follows:

$$A_{ne} = A_n\left(1.1 - \frac{\bar{x}'}{L_w}\right) \geq 0.8A_n, \text{ when } \frac{\bar{x}'}{L_w} > 0.1$$

$$A_{ne} = A_n, \text{ when } \frac{\bar{x}'}{L_w} \leq 0.1$$

where

$\bar{x}'$ = the distance between the centre of gravity of half of the HSS cross section taken from the edge of the connection plate

L_w = the length of a single weld segment on the HSS (the usual case has the total weld length being $4L_w$)

12.3.3.5

Larger values of the effective net area may be used if justified by test or rational analysis, but shall not exceed A_g.

12.3.4 Angles

For angles, the gross width shall be the sum of the widths of the legs minus the thickness. The gauge for holes in opposite legs shall be the sum of the gauges from the heel of the angle minus the thickness.

12.3.5 Plug or slot welds

In calculating the net area of a member across plug or slot welds, the weld metal shall not be taken as adding to the net area.

12.4 Pin-connected members in tension

12.4.1 Effective net areas

Two effective net areas shall be computed as follows

a) The effective net area for tension, A_{net} shall be taken as $2tb_e$

b) The effective net area for shear rupture, A_{nes} shall be taken as $2t(a + d/2)$

where

a = shortest distance parallel to the tensile force from the edge of the pin hole to the end of the tension member pin plate

b_e = $2t + 16$ mm but not to exceed the actual distance from the edge of the hole to the edge of the part normal to the tensile force

d = diameter of pin

12.4.2 Detail requirements

The hole of the pin shall be located on the longitudinal member axis as defined by the centroid of the member cross section. The diameter of a pin hole shall be not more than 1 mm larger than the diameter of the pin when relative movement between connected parts under full service loads is required. At the centre of the pin hole the width of the plate, measured normal to the direction of the force shall be not less than $2b_e + d$. The distance from the edge of the hole to the edge of the pin plate on either side of the axis of the member axis, measured at an angle of 45° or less to the axis of the member, shall be not less than a.

13 Member and connection resistance

13.1 Resistance factors

Unless otherwise specified, resistance factors, ϕ, applied to resistances specified in this Standard shall be taken as follows:

a) structural steel: $\phi = 0.90$ and $\phi_u = 0.75$;
b) reinforcing steel bars: $\phi_r = 0.85$;
c) bolts: $\phi_b = 0.80$;
d) shear connectors: $\phi_{sc} = 0.80$;
e) beam web bearing, interior: $\phi_{bi} = 0.80$ (see Clause 14.3.2);
f) beam web bearing, end: $\phi_{be} = 0.75$ (see Clause 14.3.2);
g) bearing of bolts on steel: $\phi_{br} = 0.80$;
h) weld metal: $\phi_w = 0.67$;
i) anchor rods: $\phi_{ar} = 0.67$; and
j) concrete: $\phi_c = 0.65$.

The factored resistances so determined, in order to meet the strength requirements of this Standard, shall be greater than or equal to the effect of factored loads determined in accordance with Clause 7.2.

13.2 Axial tension

The factored tensile resistance, T_r, developed by a member subjected to an axial tensile force shall be taken as follows:

a) the least of
   i) $T_r = \phi A_g F_y$;
   ii) T_r = resistance determined using Clause 13.11; and
   iii) $T_r = \phi_u A_{ne} F_u$; and
b) for pin connections (excluding eyebars), the least of
   i) $T_r = \phi A_g F_y$;
   ii) $T_r = \phi_u A_{net} F_u$; and
   iii) $T_r = 0.6\phi_u A_{nes} F_u$,
   where A_{net} and A_{nes} are defined in Clause 12.4.1.

13.3 Axial compression

13.3.1 Flexural buckling of doubly symmetric shapes

The factored axial compressive resistance, C_r, of doubly symmetric shapes meeting the requirements of Table 1 shall be taken as

$$C_r = \frac{\phi A F_y}{(1 + \lambda^{2n})^{\frac{1}{n}}}$$

where

n = 1.34 for hot-rolled, fabricated structural sections and hollow structural sections manufactured in accordance with CSA G40.20, Class C (cold-formed non-stress-relieved), ASTM A500, or ASTM A1085

= 2.24 for doubly symmetric welded three-plate members with flange edges oxy-flame-cut and hollow structural sections manufactured in accordance with CSA G40.20, Class H (hot-formed or cold-formed stress-relieved) and ASTM A1085 with Supplement S1

$$\lambda = \sqrt{\frac{F_y}{F_e}}$$

where

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

Doubly symmetric shapes that can be governed by torsional buckling shall also meet the requirements of Clause 13.3.2.

13.3.2 Flexural, torsional, or flexural-torsional buckling

The factored compressive resistance, C_r, of asymmetric, singly symmetric, and cruciform or other doubly symmetric sections not covered under Clause 13.3.1 shall be computed using the expressions given in Clause 13.3.1 with a value of $n = 1.34$ and the value of F_e taken as follows:

a) for doubly symmetric sections (e.g., cruciform) and point symmetric sections (e.g., Z-sections), the least of F_{ex}, F_{ey}, and F_{ez};

b) for singly symmetric sections (e.g., double angles, channels, and T-sections), with the y-axis taken as the axis of symmetry, the lesser of F_{ex} and F_{eyz}
where

$$F_{eyz} = \frac{F_{ey} + F_{ez}}{2\Omega}\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}\Omega}{\left(F_{ey} + F_{ez}\right)^2}}\right]$$

c) for asymmetric sections (e.g., bulb angles), the smallest root of

$$\left(F_e - F_{ex}\right)\left(F_e - F_{ey}\right)\left(F_e - F_{ez}\right) - F_e^2\left(F_e - F_{ey}\right)\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2\left(F_e - F_{ex}\right)\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0$$

where

F_{ex}, F_{ey}, and F_{ez} are calculated with respect to the principal axes:

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{K_x L_x}{r_x}\right)^2}$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L_y}{r_y}\right)^2}$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z L_z)^2} + GJ\right)\frac{1}{A\bar{r}_o^2}$$

where

K_z = effective length factor for torsional buckling, conservatively taken as 1.0

$\bar{r}_o^2 = x_o^2 + y_o^2 + r_x^2 + r_y^2$

$\Omega = 1 - \left[\frac{x_o^2 + y_o^2}{\bar{r}_o^2}\right]$

where

x_o, y_o = principal coordinates of the shear centre with respect to the centroid of the cross-section

Note: *For equal-leg double angles connected back-to-back to a common gusset plate, flexural-torsional buckling is not a controlling limit state.*

13.3.3 Single-angle members in compression

13.3.3.1 General

The factored compressive resistance, C_r, of single-angle members may be calculated neglecting the effects of eccentricity if the appropriate slenderness as specified in Clause 13.3.3.2 or 13.3.3.3 is used, provided that

a) members are loaded at the ends in compression through the same one leg;
b) members are attached by welding or by minimum two-bolt connections; and
c) there are no intermediate transverse loads.

The factored compressive resistance, C_r, of single-angle members meeting the requirements of Table 1 shall be taken as

$$C_r = \frac{\phi A F_y}{(1 + \lambda^{2n})^{\frac{1}{n}}}$$

where

$n = 1.34$

$\lambda = \sqrt{\frac{F_y}{F_e}}$

$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$

13.3.3.2 Individual members and planar trusses

For equal-leg angles or unequal-leg angles with leg length ratios (b_l/b_s) less than 1.7 and connected through the longer leg that are individual members or are members of planar trusses with adjacent web members attached to the same side of the gusset plate or chord:

a) $0 \le \frac{L}{r_x} \le 80$: $\frac{KL}{r} = 72 + 0.75\frac{L}{r_x}$
b) $\frac{L}{r_x} > 80$: $\frac{KL}{r} = 32 + 1.25\frac{L}{r_x} \le 200$

For unequal-leg angles with leg length ratios (b_l/b_s) less than 1.7 and connected through the shorter leg, KL/r shall be increased by adding $4[(b_l/bs)^2 - 1]$. KL/r shall be not less than $0.95L/r_y'$

where

L = length of member between work points at truss chord centrelines

b_l = longer leg of angle

b_s = shorter leg of angle

r_x = radius of gyration of single-angle member about geometric axis parallel to connected leg

r_y' = radius of gyration of single-angle member about minor principal axis

13.3.3.3 Box and space trusses

For equal-leg angles or unequal-leg angles with leg length ratios (b_l/b_s) less than 1.7 and connected through the longer leg that are members of box or space trusses with adjacent web members attached to the same side of the gusset plate or chord:

a)

$$0 \le \frac{L}{r_x} \le 75 : \frac{KL}{r} = 60 + 0.8\frac{L}{r_x}$$

b)

$$\frac{L}{r_x} > 75 : \frac{KL}{r} = 45 + \frac{L}{r_x} \le 200$$

For unequal-leg angles with leg length ratios (b_l/b_s) less than 1.7 and connected through the shorter leg, KL/r shall be increased by adding $6\,[(b_l/bs)^2 - 1]$. KL/r shall be not less than $0.82L/r_y'$.

13.3.3.4 Other members

Single-angle members with different end conditions from those described in Clause 13.3.3.1, leg length ratios (b_l/b_s) greater than 1.7, adjacent web members attached to opposite sides of the gusset plate or chord, or transverse loading shall be designed for compressive resistance, C_r, with Clause 13.3.2, accounting for the effects of eccentricity.

13.3.4 Segmented members in compression

The factored compressive resistance of segmented columns shall be determined using a rational method. Notional loads need not be applied between in-plane lateral supports.

13.3.5 Members in compression subjected to elastic local buckling

The factored compressive resistance, C_r, for sections that exceed the width (or diameter)-to-thickness ratios specified in Table 1 shall be determined as either

a)

$$C_r = \frac{\phi A_e F_y}{(1 + \lambda^{2n})^{\frac{1}{n}}}$$

where

$$\lambda = \sqrt{\frac{F_y}{F_e}}$$

with an effective area, A_e, calculated using reduced element widths meeting the maximum width-to-thickness ratio specified in Table 1; or

b)

$$C_r = \frac{\phi A F_{ye}}{\left(1 + \lambda_{ye}^{2n}\right)^{\frac{1}{n}}}$$

where

$$\lambda_{ye} = \sqrt{\frac{F_{ye}}{F_e}}$$

with an effective yield stress, F_{ye}, determined from the maximum width (or diameter)-to-thickness ratio meeting the limit specified in Table 1.
The elastic buckling stress, F_e, shall be calculated using Clause 13.3.1, 13.3.2, or 13.3.3, as applicable, and using gross section properties.

13.4 Shear

13.4.1 Webs of flexural members with two flanges

13.4.1.1 Elastic analysis

The factored shear resistance, V_r, developed by the web of a flexural member shall be taken as

$$V_r = \phi A_w F_s$$

where

A_w = shear area (dw for rolled shapes and hw for girders, $2ht$ for rectangular HSS)

F_s = as follows:

a) for unstiffened webs:

i) when $\frac{h}{w} \le \frac{1014}{\sqrt{F_y}}$:

$F_s = 0.66F_y$

ii) when $\frac{1014}{\sqrt{F_y}} < \frac{h}{w} \le \frac{1435}{\sqrt{F_y}}$:

$$F_s = \frac{670\sqrt{F_y}}{(h/w)}$$

iii) when $\frac{h}{w} > \frac{1435}{\sqrt{F_y}}$:

$$F_s = \frac{961\ 200}{(h/w)^2}$$

b) for stiffened webs:

i) when $\frac{h}{w} \le 439\sqrt{\frac{k_v}{F_y}}$:

$F_s = 0.66F_y$

ii) when $439\sqrt{\frac{k_v}{F_y}} < \frac{h}{w} \le 502\sqrt{\frac{k_v}{F_y}}$:

$F_s = F_{cri}$

iii) when $502\sqrt{\frac{k_v}{F_y}} < \frac{h}{w} \le 621\sqrt{\frac{k_v}{F_y}}$:

$F_s = F_{cri} + k_a(0.50F_y - 0.866F_{cri})$

iv) when $621\sqrt{\frac{k_v}{F_y}} < \frac{h}{w}$:

$F_s = F_{cre} + k_a(0.50F_y - 0.866\ F_{cre})$

where

k_v = shear buckling coefficient, as follows:

1) when $a / h < 1$

$$k_v = 4 + \frac{5.34}{(a/h)^2}$$

2) when $a / h \ge 1$

$k_v = 5.34 + \frac{4}{(a/h)^2}$

where

a / h = aspect ratio = ratio of the distance between stiffeners to web depth

where

$F_{cri} = 290\frac{\sqrt{F_y k_v}}{(h/w)}$

K_a = aspect coefficient

$= \frac{1}{\sqrt{1+(a/h)^2}}$

$F_{cre} = \frac{180\,000\ k_v}{(h/w)^2}$

13.4.1.2 Combined shear and moment in stiffened web beams

Transversely stiffened web members depending on tension field action to carry shear shall be proportioned to satisfy the requirements of Clause 14.6 for combined shear and moment.

13.4.1.3 Tubular members and concrete-filled tubular members

The shear resistance, V_r, of Class 1 and 2 tubular members and concrete-filled tubular members where local wall buckling is prevented shall be taken as

$V_r = 0.66\phi\ (A/2)F_y$

where

A = cross-sectional area of the tubular member portion of the concrete-filled member

13.4.2 Plastic analysis

In structures designed on the basis of a plastic analysis as defined in Clause 8.3.2, the factored shear resistance, V_r, developed by the web of a flexural member subjected to shear shall be taken as

$V_r = 0.8\phi A_w F_s$

where F_s is determined in accordance with Clause 13.4.1.1.

13.4.3 Webs of flexural members not having two flanges

The factored shear resistance for cross-sections not having two flanges (e.g., solid rectangles, rounds, and T_s) shall be determined by rational analysis. The factored shear stress at any location in the cross-section shall be taken as not greater than $0.66\phi\ F_y$ and shall be reduced where shear buckling is a consideration.

13.4.4 Pins

The total factored shear, V_r, resistance of the nominal area of pins shall be taken as

$V_r = 0.66\phi A F_y$

13.4.5 Gusset plates and coped beams

The shear resistance of gusset plates and the shear resistance at the ends of coped beams shall be computed in accordance with Clause 13.11.

13.5 Bending — Laterally supported members

The factored moment resistance, M_r, developed by a member subjected to uniaxial bending moments about a principal axis where effectively continuous lateral support is provided to the compression flange, or where the member has no tendency to buckle laterally, shall be taken as follows:

a) for Class 1 and Class 2 sections (except that singly symmetric I-sections and T-sections shall not yield under service loads):

$$M_r = \phi Z F_y$$
$$= \phi M_p$$

b) for Class 3 sections:

$$M_r = \phi S F_y$$
$$= \phi M_y$$

c) for Class 4 sections:

i) when both the web and compression flange slenderness exceed the limits for Class 3 sections, the value of M_r shall be determined in accordance with CSA S136. The calculated value, F_y', applicable to cold-formed members, shall be determined using only the values for F_y and F_u that are specified in the relevant structural steel material standard;

ii) when the flanges meet the requirements of Class 3 but the web slenderness exceeds the limit for Class 3, the requirements of Clause 14 shall apply; and

iii) when the web meets the requirements of Class 3 but the flange slenderness exceeds the limit for Class 3, M_r shall be calculated as follows:

$M_r = \phi\ S_e\ F_y$

where

S_e = effective section modulus determined using an effective flange width of $670t/\sqrt{F_y}$ for flanges supported along two edges parallel to the direction of stress and an effective width of $200t/\sqrt{F_y}$ for flanges supported along one edge parallel to the direction of stress. For flanges supported along one edge, b_{el}/t shall not exceed 60.

Alternatively, the moment resistance may be calculated using an effective yield stress determined from the flange width-to-thickness ratio meeting the Class 3 limit.

① 13.6 Bending — Laterally unsupported members

Where continuous lateral support is not provided to the compression flange of a member subjected to uniaxial strong axis bending, the factored moment resistance, M_r, of a segment between effective brace points shall be determined as follows:

a) For doubly symmetric Class 1 and 2 sections, except closed square and circular sections:

i) when $M_u > 0.67M_p$:

$$M_r = 1.15\phi M_p\left[1 - \frac{0.28M_p}{M_u}\right] \le \phi M_p$$

ii) when $M_u \le 0.67M_p$:

$M_r = \phi M_u$

where the critical elastic moment of the unbraced segment, M_u, is given by

$$M_u = \frac{\omega_2 \pi}{L}\sqrt{EI_yGJ + \left(\frac{\pi E}{L}\right)^2 I_yC_w}$$

where

$$\omega_2 = \frac{4M_{max}}{\sqrt{M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2}} \le 2.5$$

where

C_w = warping torsional constant, taken as 0 for rectangular hollow structural sections

J = St. Venant torsional constant

L = length of unbraced segment of beam

M_{max} = maximum factored bending moment magnitude in unbraced segment

M_a = factored bending moment at one-quarter point of unbraced segment

M_b = factored bending moment at midpoint of unbraced segment

M_c = factored bending moment at three-quarter point of unbraced segment

ω_2 = coefficient to account for increased moment resistance of a laterally unsupported doubly symmetric beam segment when subject to a moment gradient

Where the bending moment distribution within the unbraced segment is effectively linear, the equivalent moment factor, ω_2, may be taken as

$1.75 + 1.05\kappa + 0.3\kappa^2 \le 2.5$

where

κ = ratio of the smaller factored moment to the larger factored moment at opposite ends of the unbraced length (positive for double curvature and negative for single curvature)

For unbraced beam segments loaded above the shear centre between brace points, where the method of load delivery to the member provides neither lateral nor rotational restraint to the member, the associated destabilizing effect shall be taken into account using a rational method. For loads applied at the level of the top flange, in lieu of a more accurate analysis, M_u may be determined using $\omega_2 = 1.0$ and using an effective length, for pinned-ended beams, equal to 1.2 L and, for all other cases, 1.4 L.

b) For doubly symmetric Class 3 and 4 sections, except closed square and circular sections, and for channels:
   i) when $M_u > 0.67M_y$:

   $$M_r = 1.15\phi M_y\left[1 - \frac{0.28M_y}{M_u}\right]$$

   but not greater than ϕM_y for Class 3 sections and the value specified in Clause 13.5 c) iii) for Class 4 sections; and
   ii) when $M_u \le 0.67M_y$:
   $M_r = \phi M_u$
   where M_u and ω_2 are as specified in Item a) ii).

c) For closed square and circular sections, M_r shall be determined in accordance with Clause 13.5.

d) For cantilever beams, a rational method of analysis taking into account the lateral and torsional restraint conditions at the supports and tip of the cantilever, as well as the loading conditions and the flexibility of the backspan, shall be used.

e) For singly symmetric (monosymmetric) Class 1, 2, or 3 I-sections and T-sections, lateral-torsional buckling strength shall be checked separately for each flange that experiences compression under factored loads at any point along its unbraced length, as follows (except that these sections shall not yield under service loads):
   i) when $M_u > M_{yr}$:

   $$M_r = \phi\left[M_p - \left(M_p - M_{yr}\right)\left(\frac{L - L_u}{L_{yr} - L_u}\right)\right] \le \phi M_p$$

   except for Class 3 sections, as well as Class 1 and 2 T-sections where at any point within the unbraced segment the stem tip is in compression, where M_p is replaced with M_y

   where

   $M_{yr} = 0.7S_xF_y$ with S_x taken as the smaller of the two potential values

   L_{yr} = length L obtained by setting $M_u = M_{yr}$

   $L_u = 1.1r_t\sqrt{E/F_y} = \frac{490r_t}{\sqrt{F_y}}$

   where

   $r_t = \dfrac{b_c}{\sqrt{12\left(1 + \frac{h_c w}{3b_c t_c}\right)}}$

   where

   h_c = depth of the web in compression
   b_c = width of compression flange
   t_c = thickness of compression flange

ii) when $M_u \le M_{yr}$:
$M_r = \phi M_u$
where the critical elastic moment of the unbraced segment, M_u, is given by

$$M_u = \frac{\omega_3 \pi^2 E I_y}{2L^2}\left[\beta_x + \sqrt{\beta_x^2 + 4\left(\frac{GJL^2}{\pi^2 E I_y} + \frac{C_w}{I_y}\right)}\right]$$

and where in lieu of more accurate values the section properties βx and C_w may be evaluated as

$$\beta_x = 0.9(d-t)\left(\frac{2I_{yc}}{I_y} - 1\right)\left(1 - \left(\frac{I_y}{I_x}\right)^2\right)$$

$$C_w = \frac{I_{yc} I_{yt} (d-t)^2}{I_y}$$

where
β_x = asymmetry parameter for singly symmetric beams
I_{yc} = moment of inertia of the compression flange about the y-axis
I_{yt} = moment of inertia of the tension flange about the y-axis

and when singly symmetric beams are in single curvature
ω_3 = ω_2 for beams with two flanges
= 1.0 for T-sections

in all other cases
ω_3 = $\omega_2 (0.5 + 2 (I_{yc}/I_y)^2)$ but ≤ 1.0 for T-sections

For unbraced beam segments loaded above the section mid-height and between brace points, where the method of load delivery to the member provides neither lateral nor rotational restraint to the member, the associated destabilizing effect shall be taken into account using a rational method.
For other singly symmetric shapes, a rational method of analysis shall be used.

f) For biaxial bending, the member shall meet the following requirement:

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \le 1.0$$

13.7 Lateral bracing for members in structures analyzed plastically

Members in structures or portions of structures in which the distributions of moments and forces have been determined by a plastic analysis shall be braced to resist lateral and torsional displacement at all hinge locations. However, bracing shall not be required at the location of the last hinge to form in the failure mechanism assumed as the basis for proportioning the structure. The laterally unsupported distance, L_{cr}, from braced hinge locations to the nearest adjacent point on the frame similarly braced shall not exceed the following:

a) for static plastic analysis and for seismic design in accordance with Clauses 27.3 and 27.7.9.3:

$$\frac{L_{cr}}{r_y} = \frac{25\,000 + 15\,000\,\kappa}{F_y}$$

b) for seismic design in accordance with Clauses 27.2 and 27.9:

$$\frac{L_{cr}}{r_y} = \frac{17\,250 + 15\,500\,\kappa}{F_y}$$

where κ is as specified in Clause 13.6 a).

Except as specified in Items a) and b), the maximum unsupported length of members in structures analyzed plastically need not be less than that permitted for the same members in structures analyzed elastically.

13.8 Axial compression and bending

13.8.1 General

In Clause 13.8, a distinction is made between braced and unbraced frames. A frame without bracing is classified as unbraced. A frame with bracing is classified as braced if its sway stiffness is at least five times that of the frame with only the existing moment connections and without the bracing; otherwise, it is classified as unbraced. For members not contributing through bending to the lateral strength and stability of the structure, the conditions applicable to braced frames may be used.

Note: *For segmented members, the in-plane compressive resistance may be determined assuming pinned end connections. See Clause 13.3.4.*

13.8.2 Member strength and stability — Class 1 and Class 2 sections of I-shaped members

Members required to resist both bending moments and an axial compressive force shall be proportioned so that

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \le 1.0$$

where

C_f and M_f = the maximum load effects, including stability effects as specified in Clause 8.4

β = $0.6 + 0.4\lambda_y \le 0.85$

The capacity of the member shall be examined for

a) cross-sectional strength (members in braced frames only) with $\beta = 0.6$, in which case
   i) C_r shall be as specified in Clause 13.3, with the value $\lambda = 0$;
   ii) M_r shall be as specified in Clause 13.5 (for the appropriate class of section); and
   iii) U_{1x} and U_{1y} shall be as specified in Clause 13.8.4, but not less than 1.0;
b) overall member strength, in which case
   i) C_r shall be as specified in Clause 13.3, with the value $K = 1$, except that for uniaxial bending, C_r shall be based on the axis of bending (see also Clause 10.3.2);
   ii) M_r shall be as specified in Clause 13.5 (for the appropriate class of section);
   iii) U_{1x} and U_{1y} shall be taken as 1.0 for members in unbraced frames; and
   iv) U_{1x} and U_{1y} shall be as specified in Clause 13.8.4 for members in braced frames; and
c) lateral torsional buckling strength, when applicable, in which case
   i) C_r shall be as specified in Clause 13.3 and based on weak-axis or torsional-flexural buckling (see also Clause 10.3.3);
   ii) M_{rx} shall be as specified in Clause 13.6 (for the appropriate class of section);
   iii) M_{ry} shall be as specified in Clause 13.5 (for the appropriate class of section);
   iv) U_{1x} and U_{1y} shall be taken as 1.0 for members in unbraced frames;
   v) U_{1x} shall be as specified in Clause 13.8.4, but not less than 1.0, for members in braced frames; and

vi) U_{1y} shall be as specified in Clause 13.8.4 for members in braced frames.
In addition, the member shall meet the following requirement:

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0$$

where M_{rx} and M_{ry} are as specified in Clause 13.5 or 13.6, as appropriate.

13.8.3 Member strength and stability — All classes of sections except Class 1 and Class 2 sections of I-shaped members

Members required to resist both bending moments and an axial compressive force shall be proportioned so that

$$\frac{C_f}{C_r} + \frac{U_{1x}M_{fx}}{M_{rx}} + \frac{U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

where all terms are as specified in Clause 13.8.2.

The capacity of the member shall be examined for the following cases in the manner specified in Clause 13.8.2:

a) cross-sectional strength (members in braced frames and tapered members only);
b) overall member strength; and
c) lateral-torsional buckling strength.

In addition, for braced frames, the member shall meet the following requirement:

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0$$

where M_{rx} and M_{ry} are as specified in Clause 13.5 or 13.6, as appropriate.

13.8.4 Value of U_1

In lieu of a more detailed analysis, the value of U_1 for the axis under consideration, accounting for the second-order effects due to the deformation of a member between its ends, shall be taken as

$$U_1 = \left[\frac{\omega_1}{1 - \frac{C_f}{C_e}}\right]$$

where ω_1 is as specified in Clause 13.8.5 and

$$C_e = \frac{\pi^2 EI}{L^2}$$

13.8.5 Values of ω_1

Unless otherwise determined by analysis, the following values shall be used for ω_1:

a) for members not subjected to transverse loads between supports:
$\omega_1 = 0.6 - 0.4\kappa \geq 0.4$
where
κ = ratio of the smaller factored moment to the larger factored moment at opposite ends of the member length (positive for double curvature and negative for single curvature)

b) for members subjected to distributed loads or a series of point loads between supports:
$\omega_1 = 1.0$

c) for members subjected to a concentrated load or moment between supports:
$\omega_1 = 0.85$
For the purpose of design, members subjected to a concentrated load or moment between supports (e.g., segmented columns) may be considered to be divided into segments at the points of load (or moment) application. Each segment shall then be treated as a member that depends on its own flexural stiffness to prevent sidesway in the plane of bending considered and ω_1 shall be taken as 0.85. In calculating the slenderness ratio for use in Clause 13.8, the total length of the compression member shall be used.

Note: *For references to more exact methods often justified for crane-supporting columns and similar applications, see Annex C.*

13.9 Axial tension and bending

13.9.1

Members required to resist both bending moments and an axial tensile force shall be proportioned so that

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \le 1.0$$

where M_r is as specified in Clause 13.5.

13.9.2

Additionally, the following shall apply to laterally unsupported members:

a)

$$\frac{M_f}{M_r} - \frac{T_f Z}{M_r A} \le 1.0 \text{ for Class 1 and Class 2 sections}$$

b)

$$\frac{M_f}{M_r} - \frac{T_f S}{M_r A} \le 1.0 \text{ for Class 3 and Class 4 sections}$$

where M_r is as specified in Clause 13.6.

13.10 Load bearing

The factored bearing resistance in newtons, B_r, developed by a member or portion of a member subjected to bearing shall be taken as follows:

a) on the contact area of accurately cut or fitted parts:
$B_r = 1.50\phi F_y A$

b) on expansion rollers or rockers:

$$B_r = 0.00026\phi\left(\frac{R_1}{1-\frac{R_1}{R_2}}\right)LF_y^2$$

where

F_y = specified minimum yield point of the weaker part in contact

R_1 and L = radius and length, respectively, of the roller or rocker

R_2 = radius of the groove of the supporting plate

13.11 Block shear — Tension member, beam, and plate connections

The factored resistance for a potential failure involving the simultaneous development of tensile and shear component areas shall be taken as follows:

$$T_r = \phi_u \left[U_t A_n F_u + 0.6 A_{gv} \frac{(F_y + F_u)}{2} \right]$$

where

a) U_t is an efficiency factor and $U_t = 1.0$ is used for symmetrical blocks or failure patterns and concentric loading or is taken from the following for specific applications:

Connection type	U_t
Flange-connected T_s	1.0
Angles connected by one leg and stem-connected T_s	0.6
Coped beams	
One bolt line	0.9
Two bolt lines	0.3

b) A_n is the net area in tension, as specified in Clause 12; and

c) A_{gv} is the gross area in shear.

For steel grades with $F_y > 460$ MPa, $(F_y + F_u)/2$ shall be replaced with F_y in the determination of T_r.

The second term of the expression in this Clause may be used to calculate the potential plate tear-out resistance of one or more bolts along parallel planes tangent to the bolt hole(s) and directed towards the edge of the plate.

13.12 Bolts and local connection resistance

13.12.1 Bolts in bearing-type connections

13.12.1.1 General

For bolts subject to shear or tension, ϕ_b, shall be taken as 0.80.

Δ 13.12.1.2 Bolts in bearing and shear

The factored resistance developed at the bolts in a bolted joint subjected to bearing and shear shall be taken as the lesser of

a) the factored bearing resistance at bolt holes B_r (except for long slotted holes loaded perpendicular to the slot), B_r, as follows:

$B_r = 3\phi_{br} n t d F_u$

b) the factored bearing resistance perpendicular to long slotted holes, B_r, as follows:

$B_r = 2.4\phi_{br} n t d F_u$

where

ϕ_{br} = 0.8

F_u = tensile strength of the connected material

The reduced bearing resistance of holes close to the edge in the direction of the loading shall be accounted for by appropriate consideration of the resistance requirements of Clause 13.11; or

Note: *See also Clauses 13.2 and 13.11 for resistances of bolted parts and Clause 22.3 for limiting end and edge distances.*

c) the factored shear resistance of the bolts, V_r, as follows:

$V_r = 0.60\phi_b nmA_bF_u$

For lap splices with $L \geq 760$ mm, where L is the joint length between centres of end fasteners:

$V_r = 0.50\phi_b nmA_bF_u$

When the bolt threads are intercepted by a shear plane, the factored shear resistance shall be taken as $0.70V_r$.

Note: *The specified minimum tensile strength, F_u, for bolts is given in the relevant ASTM Standard, e.g., for*

a) *ASTM A325M, F_u is* 830 MPa;
b) *ASTM A490M, F_u is* 1040 MPa;
c) *ASTM A325 or ASTM F1852 bolts 1 inch or less in diameter, F_u is* 825 MPa;
d) *ASTM A325 or ASTM F 1852 bolts greater than 1 inch in diameter, F_u is* 725 MPa;
e) *ASTM A490 or ASTM F2280 bolts, F_u is* 1035 MPa; *and*
f) *ASTM A307 Grade A bolts with heavy hex nuts as appropriate, per ASTM A563, F_u =* 410 MPa.

13.12.1.3 Bolts in tension

The factored tensile resistance, T_r, that can be developed by a bolt in a joint subjected to factored tensile force, T_f, shall be taken as

$T_r = 0.75\phi_b A_b F_u$

The calculated factored tensile force, T_f, is independent of the pretension and shall be taken as the sum of the external load plus any tension caused by prying action.

Note: *See also Clause 26.5 for bolts in tension subjected to load combinations involving fatigue.*

13.12.1.4 Bolts in combined shear and tension

A bolt in a joint that is required to develop resistance to both tension and shear shall be proportioned so that

$$\left(\frac{V_f}{V_r}\right)^2 + \left(\frac{T_f}{T_r}\right)^2 \leq 1$$

where V_r is as specified in Clause 13.12.1.2 and T_r is as specified in Clause 13.12.1.3.

13.12.2 Bolts in slip-critical connections

13.12.2.1 General

For a slip-critical connection under the forces and moments produced by specified loads, slip of the assembly shall not occur. In addition, the effects of factored loads shall not exceed the resistances of the connection as specified in Clause 13.12.1.

13.12.2.2 Shear connections

The slip resistance, V_s, of a bolted joint, subjected to shear, V, shall be taken as

$V_s = 0.53c_s k_s mnA_b F_u$

where

c_s = the resistance factor for slip resistance of bolted joints

k_s = the mean slip coefficient as determined by tests carried out in accordance with "Testing method to Determine the Slip Coefficient for Coatings Used in Bolted Joints", Annex A, of RCSC *Specification for Structural Joints Using High-Strength Bolts*

See Table 3 for values of k_s and c_s.

When long slotted holes are used in slip-critical connections, slip resistance shall be taken as 0.75 V_s.

① 13.12.2.3 Connections in combined shear and tension

The bolts in a bolted joint, required to develop resistance to both tension, T, and shear, V, shall be proportioned so that the following relationship is satisfied for the specified loads:

$$\frac{V}{V_s} + 1.9\frac{T}{nA_bF_u} \le 1.0$$

where V_s is the slip resistance specified in Clause 13.12.2.2.

13.13 Welds

13.13.1 General

The resistance factor, ϕ_w, for welded connections shall be taken as 0.67.

Note: *See Table 4 for matching electrode classifications for CSA G40.21 steels.*

13.13.2 Shear

13.13.2.1 Complete and partial joint penetration groove welds, and plug and slot welds

The factored shear resistance shall be taken as the lesser of

a) for the base metal:
$V_r = 0.67\phi_w A_m F_u$

b) for the weld metal:
$V_r = 0.67\phi_w A_w X_u$
where
A_m = shear area of effective fusion face
A_w = area of effective weld throat, plug, or slot

① 13.13.2.2 Fillet welds

The factored resistance for direct shear and tension- or compression-induced shear shall be taken as

$$V_r = 0.67\phi_w A_w X_u(1.00 + 0.50\sin^{1.5}\theta)M_w$$

where

θ = angle, in degrees, of axis of weld segment with respect to the line of action of applied force (e. g., 0° for a longitudinal weld and 90° for a transverse weld)

M_w = strength reduction factor for multi-orientation fillet welds. For joints with a single weld orientation, M_w = 1.0; for joints with multiple weld orientations, for each segment
$M_w = \frac{0.85 + \theta_1/600}{0.85 + \theta_2/600}$

where

θ_1 = orientation of the weld segment under consideration

θ_2 = orientation of the weld segment in the joint that is nearest to 90°

Weld returns that are not accounted for in the joint capacity need not be considered a weld segment in the context of this Clause.

When an overmatched electrode is used, the value of X_u in this Clause shall not exceed the value of X_u of the matching electrode.

13.13.2.3 Flare bevel groove welds for open-web steel joists

The factored resistance for direct shear and tension- or compression-induced shear shall be taken as

$V_r = 0.67\phi_w A_w F_u$

where

A_w = $0.50 w_f L$ (or as established by procedure qualification tests)

where

w_f = width of flare bevel groove weld face

F_u = least ultimate tensile strength of the components in the joint

13.13.3 Tension normal to axis of weld

13.13.3.1 Complete joint penetration groove weld made with matching electrodes

The factored tensile resistance shall be taken as that of the base metal.

13.13.3.2 Partial joint penetration groove weld made with matching electrodes

The factored tensile resistance shall be taken as

$T_r = \phi_w A_n F_u \le \phi A_g F_y$

where

A_n = nominal area of fusion face normal to the tensile force

When overall ductile behaviour is desired (member yielding before weld fracture), the following shall apply:

$A_nF_u > A_gF_y$

13.13.3.3 Partial joint penetration groove weld combined with a fillet weld, made with matching electrodes

The factored tensile resistance shall be taken as

$$T_r = \phi_w\sqrt{(A_nF_u)^2 + (A_wX_u)^2} \le \phi A_gF_y$$

where

A_g = gross area of the components of the tension member connected by the welds

13.13.4 Compression normal to axis of weld

13.13.4.1 Complete and partial joint penetration groove welds made with matching electrodes

The compressive resistance shall be taken as that of the effective area of base metal in the joint. For partial joint penetration groove welds, the effective area in compression shall be taken as the nominal area of the fusion face normal to the compression plus the area of the base metal fitted in contact bearing (see Clause 28.5).

13.13.4.2 Cross-sectional properties of continuous longitudinal welds

Continuous longitudinal welds made with matching electrodes may be considered as contributing to the cross-sectional properties A, S, Z, and I of the cross-section.

13.13.4.3 Welds for hollow structural sections

The provisions of Annex L of CSA W59 may be used for hollow structural sections.

13.14 Welds and high-strength bolts in combination

The factored shear resistance of a joint that combines welds and bolts in the same plane, $V_{r,joint}$, shall be taken as the largest of

a) $V_{friction} + V_{r,trans} + 0.85V_{r,long}$;

b) $V_{friction} + V_{r,long} + 0.5V_{r,bolt}$; and

c) $V_{r,bolt}$.

where

$V_{friction}$ = plate friction resistance component

= $0.25V_s$ when the bolts are pretensioned in accordance with Clause 23.7

= 0 when the bolts are not pretensioned

$V_{r,trans}$ = transverse weld resistance component

= V_r determined from Clause 13.13.2.2 for $\theta = 90°$

$V_{r,long}$ = longitudinal weld resistance component

= V_r determined from Clause 13.13.2.2 for $\theta = 0°$

$V_{r,bolt}$ = bolt shear resistance component

= V_r determined from Clause 13.12.1.2

14 Beams and girders

14.1 Proportioning

Beams and girders consisting of rolled shapes (with or without cover plates), hollow structural sections, or fabricated sections shall be proportioned on the basis of the properties of the gross section or the modified gross section. No deduction need be made for fastener holes in webs or flanges unless the reduction of flange area by such holes exceeds 15% of the gross flange area, in which case the excess shall be deducted. The effect of openings other than holes for fasteners shall be considered in accordance with Clause 14.3.3.

14.2 Flanges

14.2.1

Flanges of welded girders should consist of a single plate or a series of plates joined end-to-end by complete penetration groove welds.

14.2.2

Flanges of bolted girders shall be proportioned so that the total cross-sectional area of cover plates does not exceed 70% of the total flange area.

14.2.3

Fasteners or welds connecting flanges to webs shall be proportioned to resist horizontal shear forces due to bending combined with any loads that are transmitted from the flange to the web other than by direct bearing. Spacing of fasteners or intermittent welds in general shall be in proportion to the intensity of the shear force and shall not exceed the maximum for compression or tension members, as applicable, in accordance with Clause 19.

14.2.4

Partial-length flange cover plates shall be extended beyond the theoretical cut-off point and the extended portion shall be connected with sufficient fasteners or welds to develop a force in the cover plate at the theoretical cut-off point not less than

$$P = \frac{AM_{fc}y}{I_g}$$

where

P = required force to be developed in cover plate

A = area of cover plate

M_{fc} = moment due to factored loads at theoretical cut-off point

y = distance from centroid of cover plate to neutral axis of cover-plated section

I_g = moment of inertia of cover-plated section

Additionally, for welded cover plates, the longitudinal welds connecting the cover-plate termination to the beam or girder shall be designed to develop the force, P, within a length, a', measured from the actual end of the cover plate, determined as follows:

a) when there is a continuous weld equal to or larger than three-fourths of the cover-plate thickness across the end of the plate and along both edges of the cover plate, a' shall be taken as the width of the cover plate;

b) when there is a continuous weld smaller than three-fourths of the cover-plate thickness across the end of the plate and along both edges, a' shall be taken as 1.5 times the width of the cover plate; and

c) when there is no weld across the end of the plate but there are continuous welds along both edges, a' shall be taken as 2 times the width of the cover plate.

14.3 Webs

14.3.1 Maximum slenderness

The slenderness ratio, h/w, of a web shall not exceed $83\,000/F_y$

where

F_y = specified minimum yield point of the compression flange steel

This limit may be waived if analysis indicates that buckling of the compression flange into the web will not occur at factored load levels.

14.3.2 Web crippling and yielding

The factored bearing resistance of the web shall be taken as follows:

a) for interior loads (concentrated load applied at a distance from the member end greater than the member depth), the smaller of
   i) $B_r = \phi_{bi} w (N + 10t) F_y$
   ii) $B_r = 1.45 \phi_{bi} w^2 \sqrt{F_y E}$

b) for end reactions, the smaller of
   i) $B_r = \phi_{be} w (N + 4t) F_y$
   ii) $B_r = 0.60 \phi_{be} w^2 \sqrt{F_y E}$

where

ϕ_{bi} = 0.80

ϕ_{be} = 0.75

N = length of bearing

Where the bearing resistance of the web is exceeded, bearing stiffeners shall be used (see Clause 14.4).

14.3.3 Openings

14.3.3.1

Except as specified in Clause 14.1, the effect of all openings in beams and girders shall be considered in the design. At all points where the factored shear or moments at the net section would exceed the capacity of the member, adequate reinforcement shall be added to the member at these points to provide the required strength and stability.

14.3.3.2

Unreinforced circular openings may be located in the web of unstiffened prismatic Class 1 and Class 2 beams or girders without considering net section properties, provided that

a) the load is uniformly distributed;
b) the section has an axis of symmetry in the plane of bending;
c) the openings are located within the middle third of the depth and the middle half of the span of the member;
d) the spacing between the centres of any two adjacent openings, measured parallel to the longitudinal axis of the member, is a minimum of 2.5 times the diameter of the larger opening; and
e) the factored maximum shear at the support does not exceed 50% of the factored shear resistance of the section.

14.3.3.3

If the forces at openings are determined by an elastic analysis, the procedure shall be in accordance with published, recognized principles.

14.3.3.4

The strength and stability of the member in the vicinity of openings may be determined on the basis of assumed locations of plastic hinges, such that the resulting force distributions satisfy the requirements of equilibrium, provided that the analysis is carried out in accordance with Items a), b), and f) of Clause 8.3.2. However, for I-type members, the width-to-thickness ratio of the flanges may meet only the requirements of Class 1 or 2 sections, provided that the webs meet the width-to-thickness limit of Class 1 sections.

14.3.4 Effect of thin webs on moment resistance

When the web slenderness ratio, h/w, exceeds $1900/\sqrt{M_f/\phi S}$, the flange shall meet the width-to-thickness ratios of Class 3 sections in accordance with Clause 11, and the factored moment resistance of the beam or girder, M_r' shall be determined as follows:

$$M_r' = M_r\left[1 - 0.0005\frac{A_w}{A_f}\left(\frac{h}{w} - \frac{1900}{\sqrt{M_f/\phi S}}\right)\right]$$

where

M_r = factored moment resistance determined in accordance with Clause 13.5 or 13.6, but not to exceed ϕM_y

When an axial compressive force acts on the girder in addition to the moment, the constant 1900 in the expression for M_r' shall be reduced by the factor $(1 - 0.65C_f/\phi C_y)$ (see also Clause 11.2).

14.4 Bearing stiffeners

14.4.1

Pairs of bearing stiffeners on the webs of single-web beams and girders shall be required at points of concentrated loads and reactions wherever the bearing resistance of the web is exceeded (see Clause 14.3.2). Bearing stiffeners shall also be required at unframed ends of single-web girders having web depth-to-thickness ratios greater than $1100\sqrt{F_y}$. Box girders may employ diaphragms designed to act as bearing stiffeners.

14.4.2

Bearing stiffeners shall bear against the flange or flanges through which they receive their loads and shall extend approximately to the edge of the flange plates or flange angles. They shall be designed as columns in accordance with Clause 13.3, assuming that the column section consists of the pair of stiffeners and a centrally located strip of the web equal to not more than 25 times its thickness at the interior stiffeners or a strip equal to not more than 12 times its thickness when the stiffeners are located at the end of the web. The effective column length, KL, shall be taken as not less than three-fourths of the length of the stiffeners in calculating the ratio KL/r. Only that portion of the stiffeners outside of the angle fillet or the flange-to-web welds shall be considered effective in bearing. Angle bearing stiffeners shall not be crimped. Bearing stiffeners shall be connected to the web to develop the full force required to be carried by the stiffener into the web or vice versa. The stiffeners shall conform to Clause 11.2 (see Table 1) and have a width to thickness ratio that satisfies $\frac{b}{t} \leq \frac{200}{\sqrt{F_y}}$.

14.5 Intermediate transverse stiffeners

14.5.1

Intermediate transverse stiffeners, when used, shall be spaced to suit the shear resistance determined in accordance with Clause 13.4, except that at girder end panels or at panels adjacent to large openings, the tension-field component shall be taken as zero unless means are provided to anchor the tension field.

14.5.2

Except as specified in Clause 14.5.1, the maximum distance between stiffeners, when required, shall not exceed the values shown in Table 5.

Δ 14.5.3

Intermediate transverse stiffeners may be furnished singly or in pairs. Width-to-thickness ratios shall meet the requirements of Clause 11. The moment of inertia of the stiffener, or pair of stiffeners if so furnished, shall be not less than $(h/50)^4$ taken about an axis in the plane of the web. The gross area, A_s, of intermediate stiffeners, or pairs of stiffeners if so furnished, shall be as follows:

$$A_s = \frac{aw}{2}\left[1 - \frac{a/h}{\sqrt{1+(a/h)^2}}\right]CYD$$

where

a = centre-to-centre distance of adjacent stiffeners (i.e., panel length)

w = web thickness

h = web depth

C = $\left[1 - \frac{310\,000 k_v}{F_y(h/w)^2}\right]$ but not less than 0.10

where

k_v = shear buckling coefficient (see Clause 13.4.1.1)

F_y = specified minimum yield point of web steel

Y = ratio of specified minimum yield point of web steel to specified minimum yield point of stiffener steel

D = stiffener factor

= 1.0 for stiffeners furnished in pairs

= 1.8 for single-angle stiffeners

= 2.4 for single-plate stiffeners

When the greatest shear, V_f, in an adjacent panel is less than that permitted by Clause 13.4.1.1, this gross area requirement may be reduced by multiplying by the ratio V_f/V_r.

14.5.4

Intermediate transverse stiffeners shall be connected to the web for a shear transfer per pair of stiffeners (or per single stiffener when so furnished), in newtons per millimetre of web depth, h, not less than $1 \times 10^{-4} h F_y^{1.5}$, except that when the largest calculated shear, V_f, in the adjacent panels is less than V_r, this shear transfer may be reduced in the same proportion. However, the total shear transfer shall not be less than the value of any concentrated load or reaction required to be transmitted to the web through the stiffener. Fasteners connecting intermediate transverse stiffeners to the web shall be spaced not more than 300 mm from centre-to-centre. If intermittent fillet welds are used, the clear distance between welds shall not exceed 16 times the web thickness or four times the weld length.

14.5.5

When intermediate stiffeners are used on only one side of the web, the stiffeners shall be attached to the compression flange. Intermediate stiffeners used in pairs shall have at least a snug fit against the compression flange. When stiffeners are cut short of the tension flange, the distance cut short shall be equal to or greater than four times but not greater than six times the girder web thickness. Stiffeners should be clipped to clear girder flange-to-web welds.

14.6 Combined shear and moment

Transversely stiffened girders depending on tension-field action to carry shear shall be proportioned such that

a) $0.727\frac{M_f}{M_r} + 0.455\frac{V_f}{V_r} \le 1.0$

b) $\frac{M_f}{M_r} \le 1.0$

c) $\frac{V_f}{V_r} \le 1.0$

where

M_r = value determined in accordance with Clause 13.5 or 13.6, as applicable

V_r = value determined in accordance with Clause 13.4

14.7 Rotational restraint at points of support

Beams and girders shall be restrained against rotation about their longitudinal axes at points of support.

14.8 Copes

14.8.1
The effect of copes on flexural yielding, local web buckling, and lateral torsional buckling resistance of a beam or girder shall be taken into account.

14.8.2
The effect of copes in reducing the net area of the web available to resist transverse shear and the effective net area of potential paths of minimum resistance shall be taken into account (see Clause 13.11).

14.9 Lateral forces
The flanges of beams and girders supporting cranes or other moving loads shall be proportioned to resist any lateral forces produced by such loads.

14.10 Torsion

14.10.1
Beams and girders subjected to torsion shall have sufficient strength and rigidity to resist the torsional moment and forces in addition to other moments or forces. The connections and bracing of such members shall be adequate to transfer the reactions to the supports.

14.10.2
The factored resistance of I-shaped members subject to combined flexure and torsion may be determined from moment-torque interaction diagrams that take into account the normal stress distribution due to flexure and warping torsion and the St. Venant torsion. Assumed normal stress distributions shall be consistent with the class of section.

14.10.3
Members subject to torsional deformations required to maintain compatibility of the structure need not be designed to resist the associated torsional moments, provided that the structure satisfies the requirements of equilibrium.

14.10.4
For all members subject to loads causing torsion, the torsional deformations under specified loads shall be limited in accordance with Clause 6.3.1.1. For members subject to torsion or to combined flexure and torsion, the maximum combined normal stress, as determined by an elastic analysis, arising from warping torsion and bending due to the specified loads shall not exceed F_y.

15 Trusses

15.1 Analysis

15.1.1 Simplified method
The simplified method assumes that all members are pin-connected and loads are only applied at the panel points, except that bending effects due to transverse loads applied between panel points are

assessed by taking into account any continuity of the members. This method may be used when compression members are at least Class 3.

15.1.2 Detailed method

The detailed method accounts for the actual loading and joint fixity. The detailed method shall be used for trusses

a) with panels adjacent to abrupt changes in the slope of a chord;
b) with Vierendeel panels;
c) with panels at abrupt changes in transverse shear; or
d) designed for fatigue.

15.2 General requirements

15.2.1 Effective lengths of compression members

The effective length for buckling in the plane of the truss shall be taken as the distance between the lines of intersection of the working points of the web members and the chord. The effective length for buckling perpendicular to the plane of the truss shall be equal to the distance between the points of lateral support. For built-up members, see Clause 19. For single-angle members, see also Clause 13.3.3.

Note: *For the effective lengths of compression members in trusses comprising hollow structural sections, see CISC's* Hollow Structural Section: Connections and Trusses — A Design Guide.

15.2.2 Joint eccentricities

Bending moments due to joint eccentricities shall be taken into account. The eccentricity of work points at a joint or at a support shall be taken into account.

15.2.3 Stability

Trusses shall be braced to ensure their lateral stability. Brace members that support compression chords at discrete points shall meet the requirements of Clause 9.2. Ends of compression chords that are not attached to a supporting member shall be braced laterally, unless it can be demonstrated that the support is not necessary.

15.2.4 Chord members

Splices may occur at any point in chord members.

15.2.5 Web members

The factored resistances of the first compression web member subject to transverse shear, and its connections, shall be determined with their respective resistance factors, ϕ, multiplied by 0.85.

The bending moments due to truss geometric distortions of end compression web members of bottom bearing trusses shall be included in the design. The simplified method may be used.

Splices may occur at any point in web members.

15.2.6 Compression chord supports

Truss web members that provide support to a compression chord in the plane of the truss shall be designed for an additional force equal to 0.02 of the chord force, unless the brace force has been determined by rigorous analysis.

15.2.7 Maximum slenderness ratio of tension chords

The maximum slenderness ratio shall be limited to 240, except when other means are provided to control flexibility, sag, vibration, and slack in a manner commensurate with the service conditions of the structure.

15.2.8 Deflection and camber

Except for the deflection due to flexural deformation of Vierendeel panels, deflections may be determined from the axial deformations of the truss members. For camber, see Clause 6.3.2.

15.3 Composite trusses

Trusses designed to act compositely with the slab or cover slab shall also meet the requirements of Clause 17.

16 Open-web steel joists

16.1 Scope

Clause 16 specifies requirements for the design, manufacture, transportation, and erection of open-web steel joists used in the construction of buildings. Joists intended to act compositely with the deck slab shall also meet the requirements of Clause 17. Clause 16 shall be used only for the design of joists having an axis of symmetry in the plane of the joist.

16.2 General

Open-web steel joists are steel trusses of relatively low mass with parallel or slightly pitched chords and triangulated web systems proportioned to span between walls or structural supporting members, or both, and to provide direct support for floor or roof deck. In general, joists are manufactured on a production line that employs jigs, with certain details of the members being standardized by the individual manufacturer. When specified, joists can be designed to provide lateral support to compression elements of beams or columns, to participate in lateral-load-resisting systems, or as continuous joists, cantilevered joists, or joists having special support conditions.

16.3 Materials

Steel used for joists shall be a weldable structural grade meeting the requirements of Clause 5.1. Structural members cold-formed to shape may use the effect of cold-forming in accordance with Clause A7 of CSA S136. The calculated value of F_y' shall be determined using only the values for F_y and F_u that are specified in the relevant structural steel material standard. Yield levels reported on mill test certificates or determined in accordance with Clause F3 of CSA S136 shall not be used as the basis for design.

16.4 Design documents

16.4.1 Building structural design documents

The building structural design documents shall include, as a minimum, the following:

a) all the loads carried by the joists, such as the uniformly distributed specified live and total dead loads, unbalanced loading conditions, any concentrated loads, and any special loading conditions, e.g., non-uniform snow loads, ponding loads, horizontal loads, end moments, net uplift, downward wind load, bracing forces to provide lateral support to compression elements of beams or columns, and allowances for mechanical equipment;

b) joist spacing, deflection limits and camber (see Clause 6.3.2), joist depth, and shoe depth;
c) where joists are not supported on steel members, maximum bearing pressures or sizes of bearing plates;
d) anchorage requirements in excess of the requirements of Clause 16.5.12;
e) bracing required by Clause 16.5.6.2 (if any);
f) method for and spacing of attachments of steel deck to the top chord (the documents shall indicate the special cases where the deck is incapable of supplying lateral support to the top chord [see Clause 16.8.1]);
g) minimum moment of inertia to provide satisfactory design criteria for floor vibrations, if applicable (see Clause 6.3.3.2);
h) any other information necessary for designing and supplying the joists; and
i) a note that no drilling, cutting, or welding is to be done unless approved by the building designer.

Note: *The building drawings should include a note warning that attachments for mechanical, electrical, and other services should be made using approved clamping devices or U-bolt-type connectors.*

16.4.2 Joist design documents

Joist design documents prepared by the joist manufacturer shall show, as a minimum, the
a) specified loading;
b) factored member loads;
c) material specification;
d) member sizes;
e) dimensions;
f) spacers;
g) welds;
h) shoes;
i) anchorages;
j) bracing;
k) bearings;
l) field splices;
m) bridging locations;
n) camber; and
o) coating type.

16.5 Design

16.5.1 Loading for open-web steel joists

The factored moment and shear resistances of open-web steel joists at every section shall be not less than the moment and shear due to the loading conditions specified by the building designer in the documents described in Clause 16.4.1 a) or to the factored dead load plus the following factored live load conditions, considered separately:
a) for floor joists, an unbalanced live load applied on any continuous portion of the joist to produce the most critical effect on any component;
b) for roof joists, an unbalanced loading condition with 100% of the snow load plus other live loads applied on any continuous portion of the joist and 50% of the snow load on the remainder of the joist to produce the most critical effect on any component;
c) for roof joists, wind uplift;
d) for roof joists, 100% of the snow load plus 40% of the downward wind load (companion load) (1.5S + 0.4W); and

e) the appropriate factored concentrated load (from the *NBCC*) applied at any one panel point to produce the most critical effect on any component.

16.5.2 Design assumptions

Open-web steel joists shall be designed for loads acting in the plane of the joist applied to the top chord assumed to be prevented from lateral buckling by the deck. For the purpose of determining axial forces in all members, members may be assumed to be pin-connected and the loads may be replaced by statically equivalent loads applied at the panel points.

The resistance of the deck connections as well as the resistance of the deck shall be verified by the joist designer to ensure that adequate lateral support is provided to the top chord of a joist as determined in accordance with Clause 9.2.7. When additional stability elements are necessary, they shall be designed in accordance with Clause 9.2.6.2.

16.5.3 Member and connection resistance

Member and connection resistance shall be calculated in accordance with Clause 13, except as otherwise specified in Clause 16.

16.5.4 Width-to-thickness ratios

Note: *Clause 16.5.4 is applicable for members made of more than one shape.*

16.5.4.1

The width-to-thickness ratios of compressive elements of hot-formed sections and cold-formed HSS shall be governed by Clause 11. The width-to-thickness ratios of compressive elements of cold-formed sections shall be governed by CSA S136.

16.5.4.2

For the purpose of determining the appropriate width-to-thickness ratio of compressive elements supported along one edge, any stiffening effect of the deck or the joist web shall be neglected.

16.5.5 Bottom chord

Δ 16.5.5.1

The bottom chord shall be continuous and, when in tension, may be designed as an axially loaded tension member unless subject to eccentricities in excess of those permitted under Clause 16.5.10.4 or subject to applied load between panel points. The governing radius of gyration of the tension chord or any component thereof shall be not less than 1/240 of the corresponding unsupported length. For joists with the web in the y-plane, the unsupported length of chord for computing L_x/r_x shall be taken as the panel length centre-to-centre of panel points and the unsupported length of chord for calculating L_y/r_y shall be taken as the distance between bridging lines connected to the tension chord. Joist shoes, when anchored, may be assumed to be equivalent to bridging lines. A bottom chord subjected to concentrated loads between panel points shall be designed, when the chord is in tension, in accordance with Clause 13.9 and, when the chord is in compression, in accordance with Clause 16.5.6.3, respectively.

Δ 16.5.5.2

The bottom chord shall be designed in accordance with Clause 16.5.6.3 for the resulting compressive forces when

a) net uplift is specified;

b) joists are made continuous or cantilevered;
c) end moments are specified; or
d) it provides lateral support to compression elements of beams or columns.

Bracing, when required, shall be provided in accordance with Clause 9.2. For joists with net uplift, a single line of bottom-chord bridging shall be provided at each end of the joists near the first bottom chord panel points unless the ends of the bottom chord are otherwise restrained. [See also Clause 16.7.9 a)].

16.5.6 Top chord

16.5.6.1

The top chord shall be continuous and may be designed for axial compressive force alone when
a) the panel length does not exceed 610 mm;
b) concentrated loads are not applied between the panel points; and
c) not subject to eccentricities in excess of those permitted under Clause 16.5.10.4.

When the panel length exceeds 610 mm, the top chord shall be designed as a continuous member subject to combined axial and bending forces.

16.5.6.2

The slenderness ratio, KL/r, of the top chord or of its components shall not exceed 90 for interior panels or 120 for end panels. The governing KL/r shall be the maximum value determined by the following:
a) for the x-x (horizontal) axis, L_x shall be the centre-to-centre distance between panel points and K shall be taken as 0.9;
b) for the y-y (vertical) axis, L_y shall be the centre-to-centre distance between the attachments of the deck. The spacing of attachments shall be not more than the design slenderness ratio of the top chord times the radius of gyration of the top chord about its vertical axis and not more than 1000 mm.K shall be taken as 1.0; and
c) for the z-z (skew) axis of individual components, L_z shall be the centre-to-centre distance between panel points or spacers, or both, and K shall be taken as 0.9. Decking shall not be considered to fulfill the function of batten plates or spacers for top chords consisting of two separated components, where r = the appropriate radius of gyration.

16.5.6.3

Compression chords shall be proportioned such that

$$\frac{C_f}{C_r}+\frac{M_f}{M_r}\leq 1.0$$

where
M_r = value specified in Clause 13.5
C_r = value specified in Clause 13.3

At the panel point, C_r may be taken as ϕAF_y and Clause 13.5 a) may be used to determine M_r, provided that the chord meets the requirements of a Class 2 section and $M_f/M_p < 0.25$.

For top chords with panel lengths not exceeding 610 mm, M_f resulting from any uniformly distributed loading may be neglected.

The chord shall be assumed to be pinned at the joist supports.

16.5.6.4
Top chords in tension whose panel lengths exceed 610 mm shall be designed in accordance with Clause 13.9.

16.5.6.5
When welding is used to attach steel deck to the chord of a joist, the flat width of any chord component in contact with the deck shall be at least 5 mm larger than the nominal design dimensions of the deck welds, measured transverse to the longitudinal axis of the chord.

16.5.6.6
When mechanical fasteners are used to attach steel deck to the chord of a joist, the minimum chord thickness shall be specified by the designer.

16.5.7 Webs

16.5.7.1
Webs shall be designed in accordance with Clause 13 to resist the shear at any point due to the factored loads specified in Clause 16.5.1. Particular attention shall be paid to possible reversals of force in each web member.

16.5.7.2
The length of a web member shall be taken as the distance between the intersections of the neutral axes of the web member and the chords. For buckling in the plane of the web, the effective length factor shall be taken as 0.9 if the web consists of individual members. For all other cases, the effective length factor shall be taken as 1.0.

16.5.7.3
The factored resistances of the first compression web member subject to transverse shear, and its connections, shall be determined with their respective resistance factors, ϕ, multiplied by 0.85.

16.5.7.4
The vertical web members of a joist with a modified Warren geometry shall be designed to resist an axial force equal to the calculated sum of the compressive force in the web member plus 0.02 times the force in the compression chord at that location.

16.5.7.5
The slenderness ratio of a web member in tension need not be limited.

16.5.7.6
The slenderness ratio of a web member in compression shall not exceed 200.

16.5.8 Spacers and battens
Compression members consisting of two or more sections shall be interconnected so that the slenderness ratio of each section calculated using its least radius of gyration is less than or equal to the design slenderness ratio of the built-up member. Spacers or battens shall be an integral part of the joist.

16.5.9 Connections and splices

16.5.9.1
Component members of joists shall be connected by welding, bolting, or other approved means.

16.5.9.2
Connections and splices shall develop the factored loads without exceeding the factored member resistances specified in Clause 16. Butt-joint splices shall develop the factored tensile resistance, T_r, of the member.

16.5.9.3
Splices may occur at any point in chord or web members.

16.5.9.4
Members connected at a joint should have their centroidal axes meet at a point. Where this is impractical and eccentricities are introduced, such eccentricities may be neglected if they do not exceed the following:
a) for continuous web members, the greater of the two distances measured from the neutral axis of the chord member to the extreme fibres of the chord member; and
b) for non-continuous web members, the distance measured from the neutral axis to the back (outside face) of the chord member.

When the eccentricity exceeds these limits, provision shall be made for the effects of the total eccentricity. Eccentricities assumed in design shall be taken as the maximum fabrication tolerances and shall be included with the shop details.

16.5.10 Bearings

16.5.10.1
Bearings of joists shall be proportioned so that the factored bearing resistance of the supporting material is not exceeded.

16.5.10.2
Where a joist bears, with or without a bearing plate, on solid masonry or concrete support, the bearing shall meet the requirements of CSA S304.1 for masonry and CSA A23.3 for concrete.

16.5.10.3
Where a joist bears on a structural steel member, the end of the shoe shall extend at least 65 mm beyond the edge of the support, except that when the available bearing area is restricted, this distance may be reduced, provided that the shoe is adequately proportioned and anchored to the support.

16.5.10.4
The joist shoe and the end panel of the joist shall be proportioned to include the effect of the eccentricity between the centre of the bearing and the intersection of the centroidal axes of the chord and the end diagonal.

16.5.10.5
Bottom bearing joists shall have their top and bottom chords held adequately in position at the supports.

16.5.11 Anchorage

16.5.11.1
Joists shall be properly anchored to withstand the effects of the combined factored loads, including net uplift. As a minimum, the following shall be provided:

a) when anchored to masonry or concrete:
   i) for floor joists, a 10 mm diameter rod at least 300 mm long embedded horizontally; and
   ii) for roof joists, a 20 mm diameter anchor rod 300 mm long embedded vertically with a 50 mm, 90° hook or a 20 mm diameter headed anchor rod; and

b) when supported on steel, one 20 mm diameter bolt, or a pair of fillet welds satisfying the minimum size and length requirements of CSA W59; the connection shall be capable of withstanding a horizontal load equal to 10% of the reaction of the joist.

16.5.11.2
Tie joists may have their top and bottom chords connected to a column. Unless otherwise specified by the building designer, tie joists shall have top and bottom chord connections that are each at least equivalent to those required by Clause 16.5.12.1. Either the top or bottom connection shall utilize a bolted connection.

16.5.11.3
Where joists are used as a part of a frame, the joist-to-column connections shall be designed to carry the moments and forces due to the factored loads.

16.5.12 Deflection

16.5.12.1
Steel joists shall be proportioned so that deflection due to specified loads is within acceptable limits for the nature of the materials to be supported and the intended use and occupancy. Such deflection limits shall be as specified in Clause 6.3.1 unless otherwise specified by the building designer.

16.5.12.2
The deflection shall be calculated based on truss action, taking into account the axial deformation of all of the components of the joists.

16.5.13 Camber
Unless otherwise specified by the building designer, the nominal camber shall be 0.002 of the span. Negative cambers to satisfy roof drainage requirements shall be designed for appropriate rainwater ponding loads.

Note: *For manufacturing tolerances, see Clause 16.10.9. For maximum deviation between adjacent joists, or joists and adjacent beams or walls, see Clause 16.12.2.5. For special camber requirements, see Clause 6.3.2.2.*

16.5.14 Vibration
The building designer shall give special consideration to floor systems where unacceptable vibration can occur. When requested, the joist manufacturer shall supply joist properties and details to the building designer (see Annex E).

16.5.15 Welding
Welding shall meet the requirements of Clause 24. Specific welding procedures for joist fabrication shall be developed and meet the requirements of CSA W47.1.

16.6 Stability during construction
Means shall be provided to support joist chords against lateral movement and to hold the joist in the vertical or specified plane during construction.

16.7 Bridging

16.7.1 General
Bridging transverse to the span of joists may be used to meet the requirements of Clause 16.6 and also to meet the slenderness ratio requirements for chords. Bridging shall not be considered "bracing" as described in Clause 9.2.

16.7.2 Installation
All bridging and bridging anchors shall be completely installed before any construction loads, except for the weight of the workers necessary to install the bridging, are placed on the joists.

16.7.3 Types
Unless otherwise specified or approved by the building designer, the joist manufacturer shall supply bridging that may be of the diagonal or horizontal type.

16.7.4 Diagonal bridging
Diagonal bridging consisting of crossed members running from the top chord to the bottom chord of adjacent joists shall have a slenderness ratio, L/r, of not more than 200, where L is the length of the diagonal bridging member or one-half of this length when crossed members are connected at their point of intersection and r is the least radius of gyration. All diagonal bridging shall be connected adequately to the joists by bolts or welds.

16.7.5 Horizontal bridging
A line of horizontal bridging shall consist of a continuous member perpendicular to the joist span attached to either the top chord or the bottom chord of each joist. Horizontal bridging members shall have a slenderness ratio of not more than 300.

16.7.6 Attachment of bridging
Attachment of diagonal and horizontal bridging to joist chords shall be by welding or mechanical means capable of resisting an axial load of at least 3 kN in the attached bridging member. Welds shall meet the minimum length requirements specified in CSA W59.

16.7.7 Anchorage of bridging

Each line of bridging shall be adequately anchored at each end to sturdy walls or to main components of the structural frame, if practicable. Otherwise, diagonal and horizontal bridging shall be provided in combination between adjacent joists near the ends of bridging lines.

16.7.8 Bridging systems

Bridging systems, including sizes of bridging members and all necessary details, shall be shown on the erection diagrams. If a specific bridging system is required by the design, the design drawings shall show all information necessary for the preparation of shop details and erection diagrams.

16.7.9 Spacing of bridging

Diagonal and horizontal bridging shall be spaced so that the unsupported length of the chord between bridging lines or between laterally-supported ends of the joist and adjacent bridging lines does not exceed

a) $170r$ for chords in compression; and
b) $240r$ for chords always in tension

where

r = applicable chord radius of gyration about its axis in the plane of the web.

Ends of joists anchored to supports may be assumed to be equivalent to bridging lines. If ends of joists are not so anchored before the deck is installed, the distance from the face of the support to the nearest bridging member in the plane of the bottom chord shall not exceed $120r$. There shall not be less than one line of horizontal or diagonal bridging attached to each joist spanning 4 m or more. If only a single line of bridging is required, it shall be placed at the centre of the joist span. If bridging is not used on joists less than 4 m in span, the ends of such joists shall be anchored to the supports to prevent overturning of the joist during placement of the deck.

16.8 Decking

16.8.1 Decking to provide lateral support

Decking shall bear directly on the top chord of the joist. If not sufficiently rigid to provide lateral support to the compression chord of the joist, the compression chord of the joist shall be braced laterally in accordance with Clause 9.2.

16.8.2 Deck attachments

Attachments considered to provide lateral support to top chords shall meet the requirements of Clause 9.2.3. The spacing of attachments shall not exceed

a) the design slenderness ratio of the top chord times the radius of gyration of the top chord about its vertical axis; and
b) 1 m.

16.8.3 Diaphragm action

Where decking is used in combination with joists to form a diaphragm for the purpose of transferring lateral applied loads to vertical bracing systems, special attachment requirements shall be fully specified on the building design drawings.

16.8.4 Cast-in-place slabs

Cast-in-place slabs used as decking shall have a minimum thickness of 65 mm. Forms for cast-in-place slabs shall not cause lateral displacement of the top chords of joists during installation of the forms or the placing of the concrete. Non-removable forms shall be positively attached to top chords by means of welding, clips, ties, wedges, fasteners, or other suitable means at intervals not exceeding 1 m; however, there shall be at least two attachments in the width of each form at each joist. Forms and their method of attachment shall be such that the cast-in-place slab, after hardening, is capable of furnishing lateral support to the joist chords.

16.8.5 Installation of steel deck

16.8.5.1

To facilitate attachment of the steel deck, the location of the top chord of the joist shall be confirmed by marking the deck at suitable intervals or by other means.

16.8.5.2

The installer of the steel deck to be fastened to joists by arc spot welding shall be a company that is certified by the Canadian Welding Bureau to the requirements of CSA W47.1.

The welding procedures shall meet the requirements of CSA W47.1.

The welders shall meet the requirements of CSA W47.1 for arc spot welding.

16.9 Shop coating

Joists shall have a shop coating meeting the requirements of Clause 28.7.3.3, unless otherwise specified by the building designer.

16.10 Manufacturing tolerances

16.10.1

The tolerance on the specified depth of the manufactured joist shall be ± 7 mm.

16.10.2

The deviation of a panel point from the design location, measured along the length of a chord, shall not exceed 13 mm. The centroidal axes of the bottom chord and the end diagonals carrying transverse shear should meet at the first bottom panel point even when the end diagonal is an upturned bottom chord (see Clause 16.5.10.4).

16.10.3

The deviation of a panel point from the design location, measured perpendicular to the longitudinal axis of the chord and in the plane of the joist, shall not exceed 7 mm.

16.10.4

The connections of web members to chords shall not deviate laterally more than 3 mm from that assumed in the design.

16.10.5
The sweep of a joist or any portion of the length of the joist, upon completion of manufacture, shall not exceed 1/500 of the length on which the sweep is measured.

16.10.6
The tilt of bearing shoes shall not exceed 1 in 50 measured from a plane perpendicular to the plane of the web and parallel to the longitudinal axis of the joist.

16.10.7
The tolerance on the specified shoe depth shall be ± 3 mm.

16.10.8
The tolerance on the specified length of the joist shall be ± 7 mm. The connection holes in a joist shall not vary from the detailed location by more than 2 mm for joists 10 m or less in length or by more than 3 mm for joists more than 10 m in length.

16.10.9
The tolerance in millimetres on the nominal or specified camber shall be $\pm\left(6 + \frac{L}{4000}\right)$.

The minimum camber in a joist shall be 4 mm. The range in camber for joists of the same span shall be 20 mm.

16.11 Inspection and quality control

16.11.1 Inspection
Material and quality of work shall be accessible for inspection at all times by qualified inspectors representing the building designer. Random in-process inspection shall be carried out by the manufacturer and all joists shall be thoroughly inspected by the manufacturer before shipping. Third-party welding inspection shall be in accordance with Clause 30.5.

16.11.2 Identification and control of steel
Steel used in the manufacture of joists shall be identified in the manufacturer's plant as to its specification (and grade, where applicable) by suitable markings, recognized colour-coding, or a system devised by the manufacturer that will ensure to the satisfaction of the building designer that the correct material is being used.

16.11.3 Quality control
Upon request by the building designer, the manufacturer shall provide evidence of having suitable quality control measures to ensure that the joists meet all specified requirements. When testing is part of the manufacturer's normal quality control program, the loading criteria shall be 1.0/0.9 times the factored loads for the specific joist design.

16.12 Handling and erection

16.12.1 General
Care shall be exercised to avoid damage during strapping, transport, unloading, site storage, stacking, and erection. Dropping of joists shall be avoided. Special precautions shall be taken when erecting long, slender joists, and hoisting cables should not be released until the member is stayed laterally by at least

one line of bridging. Joists shall have all bridging attached and permanently fastened in place before the application of any loads. Construction loads shall be adequately distributed so as not to exceed the capacity of any joist. Field welding shall not cause damage to joists, bridging, deck, and supporting steel members.

16.12.2 Erection tolerances

16.12.2.1
The maximum sweep of a joist or a portion of the length of a joist upon completion of erection shall not exceed the limit specified in Clause 16.10.5 and shall be in accordance with the requirements of Clause 29.

16.12.2.2
All members shall be free from twists, sharp kinks, and bends.

16.12.2.3
The deviation of joists as erected from the location in the plan shown on the erection diagrams shall not exceed 15 mm.

16.12.2.4
The deviation of the bottom chord with respect to the top chord, normal to the specified plane of the web of a joist, shall not exceed 1/50 of the depth of the joist.

16.12.2.5
The maximum deviation in elevation between the tops of any three adjacent joists shall not be greater than 0.01 times the joist spacing and not greater than 25 mm. The deviation is the vertical offset from the top of the centre joist to the line joining the tops of the centres of the adjacent joists. The maximum shall also apply to joists adjacent to beams or walls.

17 Composite beams, trusses, and joists

17.1 Application
Clause 17 shall apply to composite beams consisting of steel sections, trusses, or joists interconnected with either a reinforced concrete slab or a steel deck with a concrete cover slab. Trusses and joists designed to act compositely with the slab or cover slab shall also meet the requirements of Clauses 15 and 16, respectively. The minimum slab or cover slab thickness shall be 65 mm unless the adequacy of a lesser thickness has been established by appropriate tests.

17.2 Definitions
The following definitions apply in Clause 17:

Cover slab — the concrete above the flutes of the steel deck. All flutes are filled with concrete so as to form a ribbed slab.

Effective cover slab thickness, *t* — the minimum thickness of concrete measured from the top of the slab to the top of the steel deck.

Δ **Effective slab thickness, *t*** — the overall slab thickness, provided that the slab is cast
a) with a flat underside;
b) on corrugated steel forms having a height of corrugation not greater than 0.25 times the overall slab thickness; or
c) on fluted steel forms whose profile has the following characteristics:
   i) the minimum concrete rib width is 125 mm;
   ii) the maximum rib height is 40 mm but not more than 0.4 times the overall slab thickness; and
   iii) the average width between ribs does not exceed 0.25 times the overall slab thickness nor 0.2 times the minimum width of concrete ribs.

In all other cases, "effective slab thickness" means the overall slab thickness minus the height of the corrugation or the flute .

Flute — the portion of the steel deck that forms a valley.

Rib — the portion of the concrete slab that is formed by the flute.

Slab — a reinforced cast-in-place concrete slab at least 65 mm in effective thickness. The area equal to the effective width times the effective slab thickness should be free of voids or hollows except for those specifically permitted in the definition of effective slab thickness.

Steel deck — a load-carrying steel deck consisting of a
a) single fluted element (non-cellular deck); or
b) two-element section consisting of a fluted element in conjunction with a flat sheet (cellular deck).

Steel joist — an open-web steel joist suitable for composite design (see Clause 16).

Steel section — a steel structural section with a solid web or webs suitable for composite design. Web openings may be used only if their effects are fully investigated and accounted for in the design.

Steel truss — a steel truss suitable for composite design (see Clause 15).

17.3 General

17.3.1 Deflections

Calculation of deflections shall take into account the effects of creep of concrete, shrinkage of concrete, and increased flexibility resulting from partial shear connection and from interfacial slip. These effects shall be established by test or analysis, where practicable. Consideration shall also be given to the effects of full or partial continuity in the steel beams and concrete slabs in reducing calculated deflections.

In lieu of tests or analysis, the effects of partial shear connection and interfacial slip, creep, and shrinkage may be assessed as follows:
a) for increased flexibility resulting from partial shear connection and interfacial slip, the deflections shall be calculated using an effective moment of inertia given by

   $I_e = I_s + 0.85p^{0.25}(I_t - I_s)$

   where

   I_s = moment of inertia of a steel beam, or of a steel joist or truss adjusted to include the effect of shear deformations, which may be taken into account by decreasing the moment of inertia based on the cross-sectional areas of the top and bottom chords by 15% or by a more detailed analysis

   p = fraction of full shear connection

= 1.00 for full shear connection

I_t = transformed moment of inertia of composite beam based on the modular ratio $n = E/E_c$

b) for creep, elastic deflections caused by dead loads and long-term live loads, as calculated in Item a), need to be increased by 15%; and

c) for shrinkage of concrete, using a selected free shrinkage strain, strain compatibility between the steel and concrete, and an age-adjusted effective modulus of elasticity of concrete as it shrinks and creeps, the deflection of a simply supported composite beam, joist, or truss shall be calculated as follows:

$$\Delta_s = \frac{L^2}{8}\psi = \frac{L^2}{8}c\frac{\varepsilon_f A_c y}{n_s I_{es}}$$

where

L = span of the beam, joist, or truss

ψ = curvature along length of the beam, joist, or truss due to shrinkage of concrete

c = empirical coefficient used to match theory with test results (accounting for cracking of concrete in tension, the non-linear stress-strain relationship of concrete, and other factors)

ε_f = free shrinkage strain of concrete

A_c = effective area of concrete slab

y = distance from centroid of effective area of concrete slab to centroidal axis of the composite beam, joist, or truss

n_s = modular ratio, E/E'_c

where

E'_c = $E_c/(1+\chi\phi)$

= age-adjusted effective modulus of elasticity of concrete

where

χ = aging coefficient of concrete

ϕ = creep coefficient of concrete

I_{es} = $I_s + 0.85p^{0.25}(I_{ts} - I_s)$

= effective moment of inertia of composite beam, truss, or joist based on the modular ratio n_s

where

I_{ts} = transformed moment of inertia based on the modular ratio n_s

Note: *For typical values of c, ε_f, χ, and ϕ, see Annex H.*

17.3.2 Vertical shear

The web area of steel sections or the web system of steel trusses and joists shall be proportioned to carry the total vertical shear, V_f.

17.3.3 End connections

End connections of steel sections, trusses, and joists shall be proportioned to transmit the total end reaction of the composite beam.

17.3.4 Steel deck

The maximum depth of the deck shall be 80 mm and the average width of the minimum flute shall be 50 mm. A steel deck may be of a type intended to act compositely with the cover slab in supporting applied load.

17.4 Design effective width of concrete

17.4.1

Slabs or cover slabs extending on both sides of the steel section or joist shall be deemed to have a design effective width, *b*, equal to the lesser of

a) 0.25 times the composite beam span; or
b) the average distance from the centre of the steel section, truss, or joist to the centres of adjacent parallel supports.

17.4.2

Slabs or cover slabs extending on one side only of the supporting section or joist shall be deemed to have a design effective width, *b*, not greater than the width of the top flange of the steel section or top chord of the steel joist or truss plus the lesser of

a) 0.1 times the composite beam span; or
b) 0.5 times the clear distance between the steel section, truss, or joist and the adjacent parallel support.

17.5 Slab reinforcement

17.5.1 General

Slabs shall be adequately reinforced to support all loads and to control both cracking transverse to the composite beam span and longitudinal cracking over the steel section or joist. Reinforcement shall not be less than that required by the specified fire-resistance design of the assembly.

17.5.2 Parallel reinforcement

Reinforcement parallel to the span of the beam in regions of negative bending moment of the composite beam shall be anchored by embedment in concrete that is in compression. The reinforcement of slabs that are to be continuous over the end support of steel sections or joists fitted with flexible end connections shall be given special attention. Reinforcement at the ends of beams supporting ribbed slabs perpendicular to the beam shall be not less than two 15M bars or equivalent.

17.5.3 Transverse reinforcement — Concrete slab on metal deck

Unless it is known from experience that longitudinal cracking caused by composite action directly over the steel section or joist is unlikely, additional transverse reinforcement or other effective means shall be provided. Such additional reinforcement shall be placed in the lower part of the slab and anchored so as to develop the yield strength of the reinforcement. The area of such reinforcement shall be not less than 0.002 times the concrete area being reinforced and shall be uniformly distributed.

17.5.4 Transverse reinforcement — Ribbed slabs

17.5.4.1

Where the ribs are parallel to the beam span, the area of transverse reinforcement shall be not less than 0.002 times the concrete cover slab area being reinforced and shall be uniformly distributed.

17.5.4.2
Where the ribs are perpendicular to the beam span, the area of transverse reinforcement shall be not less than 0.001 times the concrete cover slab area being reinforced and shall be uniformly distributed.

17.6 Interconnection

17.6.1
Except as permitted by Clauses 17.6.2 and 17.6.4, interconnection between steel sections, trusses, or joists and slabs or steel decks with cover slabs shall be attained by the use of shear connectors as specified in Clause 17.7.

17.6.2
Uncoated steel sections, trusses, or joists that support slabs and are totally encased in concrete shall not require interconnection by means of shear connectors, provided that
a) a minimum of 50 mm of concrete covers all portions of the steel section, truss, or joist except as specified in Item c);
b) the cover in Item a) is reinforced to prevent spalling; and
c) the top of the steel section, truss, or joist is at least 40 mm below the top and 50 mm above the bottom of the slab.

17.6.3
Studs may be welded through a maximum of two steel sheets in contact, each not more than 1.71 mm in overall thickness, including coatings (1.52 mm in nominal base steel thickness plus zinc coating not greater than nominal 275 g/m^2). Otherwise, holes for placing studs shall be made through the sheets as necessary. Welded studs shall meet the requirements of CSA W59.

17.6.4
Methods of interconnection other than those specified in Clause 17.7 that have been adequately demonstrated by test and verified by analysis may be used to effect the transfer of forces between the steel section, truss, or joist and the slab or steel deck with cover slab. In such cases, the design of the composite member shall conform, to the extent practicable, to the design of a similar member employing shear connectors.

17.6.5
The diameter of a welded stud shall not exceed 2.5 times the thickness of the part to which it is welded unless test data satisfactory to the designer are provided to establish the capacity of the stud as a shear connector.

17.7 Shear connectors

17.7.1 General
The resistance factor, ϕ_{sc}, to be used with the shear resistances specified in Clause 17.7 shall be taken as 0.80. The factored shear resistance, q_r, of other shear connectors shall be established by tests acceptable to the designer.

17.7.2 End-welded studs

17.7.2.1

End-welded studs shall be headed or hooked with $h/d \geq 4$. The projection of a stud in a ribbed slab, based on its length prior to welding, shall be at least two stud diameters above the top surface of the steel deck. The factored resistance of end-welded studs shall be as specified in Clauses 17.7.2.2 and 17.7.2.4.

17.7.2.2

In solid slabs,

$$q_{rs} = 0.50\phi_{sc} A_{sc} \sqrt{f'_c E_c} \leq \phi_{sc} A_{sc} F_u$$

where

q_{rs} = factored shear resistance

F_u = 450 MPa for commonly available studs (CSA W59 Type B studs)

Δ 17.7.2.3

In ribbed slabs with ribs parallel to the beam,

a) when $3.0 > w_d/h_d \geq 1.50$:

$$q_{rr} = q_{rs}\left[0.75 + 0.167\left(\frac{w_d}{h_d} - 1.5\right)\right] \leq q_{rs}$$

b) when $w_d/h_d < 1.50$:

$$q_{rr} = \phi_{sc}\left[0.92\frac{w_d}{h_d}dh(f'_c)^{0.8} + 11sd(f'_c)^{0.2}\right] \leq 0.75q_{rs}$$

where

s = longitudinal stud spacing

17.7.2.4

In ribbed slabs with ribs perpendicular to the beam

a) when h_d = 75 mm:

$$q_{rr} = 0.35\phi_{sc}\rho A_p\sqrt{f'_c} \leq q_{rs}$$

b) when h_d = 38 mm:

$$q_{rr} = 0.61\phi_{sc}\rho A_p\sqrt{f'_c} \leq q_{rs}$$

where

A_p = concrete pullout area, taking the deck profile and stud burnoff into account. For a single stud, the apex of the pyramidal pullout area, with four sides sloping at 45°, shall be taken as the centre of the top surface of the head of the stud. For a pair of studs, the pullout area has a ridge extending from stud to stud

ρ = 1.0 for normal-density concrete (2150 to 2500 kg/m^3)

= 0.85 for semi-low-density concrete (1850 to 2150 kg/m^3)

17.7.2.5

The longitudinal spacing of stud connectors in solid slabs and in ribbed slabs when ribs of formed steel deck are parallel to the beam shall be not less than six stud diameters. The spacing of studs shall not exceed 1000 mm (see also Clause 17.8).

The transverse spacing of stud connectors shall be not less than four stud diameters.

17.7.3 Channel connectors

In solid slabs of normal-density concrete with $f'_c \geq 20$ MPa and a density of at least 2300 kg/m³, the following shall apply:

$$q_{rs} = 45\phi_{sc}\ (t + 0.5w)L_c\sqrt{f'_c}$$

The spacing of the shear connectors shall be in accordance with Clause 17.9.8.

17.8 Ties

Mechanical ties shall be provided between the steel section, truss, or joist and the slab or steel deck to prevent separation. Shear connectors may serve as mechanical ties if suitably proportioned. The maximum spacing of ties shall not exceed 1000 mm. The average spacing in a span shall not exceed 600 mm or be greater than that required to achieve any specified fire-resistance rating of the composite assembly.

17.9 Design of composite beams with shear connectors

17.9.1

The composite beam shall consist of steel section, truss or joist, shear connectors, ties, and slab or steel deck with cover slab.

The flat width of the top chord or that of a component member of the top chord shall be not less than $1.4d + 20$ mm

where
d = diameter of the stud connector

17.9.2

The properties of the composite section shall be based on the maximum effective area (equal to effective width times effective thickness), neglecting any concrete area in tension. If a steel truss or joist is used, the area of its top chord shall be neglected in determining the properties of the composite section and only Clause 17.9.3 a) shall apply.

Δ 17.9.3

The factored moment resistance, M_{rc}, of the composite section with the slab or cover slab in compression shall be calculated as follows, where $\phi = 0.90$, the resistance factor for concrete, $\phi_c = 0.65$, and $\alpha_1 = 0.85 - 0.0015 f'_c$ (but not less than 0.67):

a) Case 1 — full shear connection and plastic neutral axis in the slab, i.e., $Q_r \geq \phi A_s F_y$ and $\phi A_s F_y \leq \alpha_1 \phi_c b t f'_c$
where
Q_r = sum of the factored resistances of all shear connectors between points of maximum and zero moment
$M_{rc} = T_r e' = \phi A_s F_y e'$

where

e' = the lever arm and is calculated from the equation

$a = \frac{\phi A_s F_y}{\alpha_1 \phi_c b f'_c}$

b) Case 2 — full shear connection and plastic neutral axis in the steel section, i.e., $Q_r \geq \alpha_1 \phi_c b t f'_c$ and $\alpha_1 \phi_c b t f'_c < \phi A_s F_y$

$M_{rc} = C_r e + C'_r e'$

where

$C_r = \frac{\phi A_s F_y - C'_r}{2}$

$C'_r = \alpha_1 \phi_c b t f'_c$

c) Case 3 — partial shear connection, i.e., $Q_r < \alpha_1 \phi_c b t f'_c$ and $\phi A_s F_y$

$M_{rc} = C_r e + C'_r e'$

where

$$C_r = \frac{\phi A_s F_y - C'_r}{2}$$

$C'_r = Q_r$

where

e' = the lever arm and is calculated from the equation

$$a = \frac{C'_r}{\alpha_1 \phi_c b f'_c}$$

17.9.4

No composite action shall be assumed in calculating

a) flexural strength when Q_r is less than 0.4 times the lesser of $\alpha_1 \phi_c b t f'_c$ and $\phi A_s F_y$; and

b) deflections when Q_r is less than 0.25 times the lesser of $\alpha_1 \phi_c b t f'_c$ and $\phi A_s F_y$.

17.9.5

For full shear connection, the sum of the factored resistances of all shear connectors distributed between the point of maximum bending moment and each adjacent point of zero moment, Q_r, shall equal or exceed the total horizontal shear, V_h, at the junction of the steel section, truss, or joist and the concrete slab or steel deck, calculated as $V_h = \phi A_s F_y$ or $V_h = \alpha_1 \phi_c b t f'_c$ for Cases 1 and 2, as specified in Items a) and b), respectively, of Clause 17.9.3.

17.9.6

For partial shear connection, the total horizontal shear, V_h, as specified in Clause 17.9.3 c), shall be calculated as $V_h = Q_r$.

17.9.7

Composite beams employing steel sections and concrete slabs may be designed as continuous members. The factored moment resistance of the composite section, with the concrete slab in the tension area of the composite section, shall be the factored moment resistance of the steel section alone, except that when sufficient shear connectors are placed in the negative moment region, suitably anchored concrete slab reinforcement parallel to the steel sections and within the design effective width of the concrete slab may be included in calculating the properties of the composite section. The total horizontal shear, V_h, to be resisted by shear connectors between the point of maximum negative bending moment and each adjacent point of zero moment shall be taken as $\phi_r A_r F_{yr}$.

17.9.8

The number of shear connectors to be located on each side of the point of maximum bending moment (positive or negative, as applicable), distributed between that point and the adjacent point of zero moment, shall be not less than

$$n = \frac{V_h}{q_r}$$

Shear connectors may be spaced uniformly, except that in a region of positive bending the number of shear connectors, n', required between any concentrated load applied in that region and the nearest point of zero moment shall be not less than

$$n' = n\left(\frac{M_{f1} - M_r}{M_f - M_r}\right)$$

where

M_{f1} = positive bending moment under factored load at concentrated load point

M_r = factored moment resistance of the steel section alone

M_f = maximum positive bending moment under factored load

17.9.9

In the end panels of composite joists and trusses, the top chord shall be designed to resist all factored forces, ignoring any composite action unless adequate shear connectors are placed over the seat or along a top chord extension to carry horizontal shear. Studs shall not be placed closer than their height to the end of the concrete slab.

17.9.10

The shear that is to be developed on the longitudinal shear surfaces, A_{cv}, of composite beams with solid slabs or with cover slabs and steel deck parallel to the beam shall be taken as

$$V_u = \Sigma q_r - \alpha_1 \phi_c f'_c A_c - \phi_r A_r F_{yr}$$

where

A_r = area of longitudinal reinforcement within the concrete area, A_c

For normal-weight concrete, the factored shear resistance along any potential longitudinal shear surfaces in the concrete slab shall be taken as

$$V_r = \left(0.80\phi_r A_r F_{yr} + 2.76\phi_c A_{cv}\right) \leq 0.50\phi_c f'_c A_{cv}$$

where

A_r = area of transverse reinforcement crossing shear planes, A_{cv}

17.10 Design of composite beams without shear connectors

17.10.1

Uncoated steel sections or joists supporting concrete slabs and encased in concrete in accordance with Clause 17.6.2 may be proportioned based on the assumption that the composite section supports the total load.

17.10.2
The properties of the composite section for determination of load-carrying capacity shall be calculated using ultimate strength methods, neglecting any area of concrete in tension.

17.10.3
As an alternative method of design, encased simple-span steel sections or joists may be proportioned based on the assumption that the steel section, truss, or joist alone supports 0.90 times the total load.

17.11 Unshored beams
For composite beams that are unshored during construction, the stresses in the tension flange of the steel section, truss, or joist due to the loads applied before the concrete strength reaches $0.75f'_c$ plus the stresses at the same location due to the remaining specified loads considered to act on the composite section shall not exceed F_y.

17.12 Beams during construction
The steel section, truss, or joist alone shall be proportioned to support all factored loads applied prior to hardening of the concrete without exceeding its calculated capacity under the conditions of lateral support or shoring, or both, to be furnished during construction.

18 Composite columns

18.1 Resistance prior to composite action
The factored resistance of the steel member prior to the attainment of composite action shall be determined in accordance with Clause 13.

18.2 Concrete-filled hollow structural sections

18.2.1 General

18.2.1.1 Scope
Clause 18.2 applies to composite members consisting of steel hollow structural sections completely filled with concrete, provided that

a) the width-to-thickness ratio of the walls of rectangular hollow structural sections does not exceed $\frac{1350}{\sqrt{F_y}}$;

b) the outside diameter-to-thickness ratio of circular hollow structural sections does not exceed $28\,000/F_y$; and

c) the concrete strength is between 20 and 80 MPa for axially loaded columns and between 20 and 40 MPa for columns subjected to axial compression and bending.

18.2.1.2 Axial load on concrete
The axial load assumed to be carried by the concrete at the top level of a column shall be only that portion applied by direct bearing on concrete. At the bottom of a column, a base plate or other means shall be provided for load transfer. At intermediate floor levels, direct bearing on the concrete shall not be considered necessary.

18.2.1.3 Composite action in bending

Full composite resistance as specified in Clause 18.2.3 may be developed at the ends of concrete-filled hollow structural members in bending or combined axial-bending, e.g., at column bases, only if the connection is able to transfer the forces from both the steel and concrete elements to the adjacent structural elements.

Δ 18.2.2 Compressive resistance

The factored compressive resistance of a composite concrete-filled hollow structural section shall be taken as

$$C_{rc} = (\tau\phi A_s F_y + \tau'\alpha_1\phi_c A_c f'_c)(1 + \lambda^{2n})^{-1/n}$$

where

τ = τ'

= 1.0, except for circular hollow structural sections with a height-to-diameter ratio (L/D) of less than 25 for which

$$\tau = \frac{1}{\sqrt{1 + \rho + \rho^2}}$$

and

$$\tau' = 1 + \left(\frac{25\rho^2\tau}{D/t}\right)\left(\frac{F_y}{\alpha_1 f'_c}\right)$$

where

ρ = 0.02 (25 – L/D)

α_1 = 0.85 – 0.0015f'_c (but not less than 0.73)

$$\lambda = \sqrt{\frac{C_p}{C_{ec}}}$$

where

$C_p = C_{rc}$, computed with $\phi = \phi_c = 1.0$ and $\lambda = 0$

$$C_{ec} = \frac{\pi^2 EI_e}{(KL)^2}$$

where

$$EI_e = EI_s + \frac{0.6E_c I_c}{1 + C_{fs}/C_f}$$

where

I_s and I_c = moment of inertia of the steel and concrete areas, respectively, as computed with respect to the centre of gravity of the cross-section

E_c = modulus of elasticity of concrete as defined in Clause 3

C_{fs} = sustained axial load on the column

C_f = total axial load on the column

n = 1.80

18.2.3 Bending resistance

The factored bending resistance of a composite concrete-filled hollow structural section shall be taken as

$$M_{rc} = C_r e + C_r' e'$$

where

a) for a rectangular hollow structure section:

$$C_r = \frac{\phi A_s F_y - C'_r}{2}$$

$$C_r' = 1.18\ \alpha_1 \phi_c a\ (b - 2t)\ f_c'$$

$$C_r + C_r' = T_r$$

$$= \phi A_{st} F_y$$

Note: *The concrete in compression is taken to have a rectangular stress block of intensity f'_c over a depth of a.*

b) for a circular hollow structural section:

$$C_r = \phi F_y \beta \frac{Dt}{2}$$

$$C_r' = 1.18\alpha_1 \phi_c f_c' \left[\frac{\beta D^2}{8} - \frac{b_c}{2}\left(\frac{D}{2} - a\right)\right]$$

$$e = b_c\left[\frac{1}{(2\pi - \beta)} + \frac{1}{\beta}\right]$$

$$e' = b_c\left[\frac{1}{(2\pi - \beta)} + \frac{b_c^2}{1.5\beta D^2 - 6b_c(0.5D - a)}\right]$$

where

β = value in radians found from the recursive equation

$$\beta = \frac{\phi A_s F_y + 0.295\alpha_1 \phi_c D^2 f_c' \left[\sin(\beta/2) - \sin^2(\beta/2)\tan(\beta/4)\right]}{\left(0.148\alpha_1 \phi_c D^2 f_c' + \phi D t F_y\right)}$$

$$b_c = D\sin\left(\frac{\beta}{2}\right)$$

$$a = \frac{b_c}{2}\tan\left(\frac{\beta}{4}\right)$$

Conservatively, M_{rc} may be taken as

$$M_{rc} = (Z - 2th_n^2)\phi F_y + \left[\frac{2}{3}(0.5D - t)^3 - (0.5D - t)h_n^2\right]1.18\alpha_1 \phi_c f_c'$$

where

Z = the plastic modulus of the steel section alone

$$h_n = \frac{1.18\alpha_1 \phi_c A_c f_c'}{2.36D\alpha_1 \phi_c f_c' + 4t(2\phi F_y - 1.18\alpha_1 \phi_c f_c')}$$

α_1 = value as defined in Clause 18.2.2

18.2.4 Axial compression and bending

Composite concrete-filled hollow structural sections required to resist both bending moments and axial compression shall be proportioned analogously to members conforming to Clause 13.8.2 so that

$$\frac{C_f}{C_{rc}} + \frac{\beta\omega_1 M_f}{M_{rc}\left(1 - \frac{C_f}{C_{ec}}\right)} \le 1.0 \text{ and}$$

$$\frac{M_f}{M_{rc}} \le 1.0$$

where

$$\beta = \frac{C_{rco} - C_{rcm}}{C_{rco}}$$

where

C_{rco} = factored compressive resistance with $\lambda = 0$

C_{rcm} = $1.18\,\alpha_1\phi_c A_c f'_c$

where

α_1 = value as defined in Clause 18.2.2

M_{rc} = value as defined in Clause 18.2.3

18.3 Partially encased composite columns

Note: *The Canam Group Inc. holds patents on the partially encased composite columns described in this Clause. Canam Group Inc. will make available any patent rights to interested applicants, wherever located, either as a free licence or on reasonable terms and conditions.*

18.3.1 General

Clause 18.3 applies to doubly symmetrical composite members consisting of three-plate built-up steel H-sections, with plain tie bars welded between the flange tips at regular intervals, in which the cells between the column flanges and the web are completely filled with concrete in the field during construction, provided that

a) concrete is of normal density and has a compressive strength, f'_c, between 20 and 70 MPa;
b) $A_s + A_r \le 0.20$ of the gross cross-sectional area;
c) the full width of flange, b_f, is between 0.9 and 1.1 times the section depth, d;
d) the flanges and the web are of equal thickness, t;
e) the flange width-to-thickness ratio is not greater than 32;
f) a pair of continuous fillet welds, sufficient to develop the shear yield capacity of the web, connects the web to each flange;
g) the vertical spacing of tie bars, s, does not exceed the lesser of 500 mm or two-thirds of the least dimension of the cross-section. The area of a tie bar shall be taken as the greatest of
   i) 63 mm^2;
   ii) $0.01 b_f t$; and
   iii) 0.5 mm^2 per mm of tie bar spacing;
h) the tie bars are welded to the flanges to develop the yield strength of the tie bars and the cover of the tie bars is at least 30 mm;
i) out-of-straightness of the flanges, as measured between any two adjacent ties along the column edges, does not exceed 0.005 times the tie spacing;
j) the specified yield strength of structural steel, F_y, does not exceed 350 MPa;

k) the specified yield strength of reinforcement, F_{yr}, does not exceed 400 MPa; and
l) the clear height-to-width ratio of the column does not exceed 14.

① 18.3.2 Compressive resistance

The factored compressive resistance of a partially encased three-plate built-up composite column shall be taken as

$$C_{rc} = (\phi A_{se} F_y + 0.95\alpha_1 \phi_c A_c f'_c + \phi_r A_r F_{yr})(1 + \lambda^{2n})^{-1/n}$$

where
A_{se} = effective area of the steel section
= $(d - 2t + 2b_e)t$

where

$$b_e = \frac{b_f}{\left(1 + \lambda_p^3\right)^{1/1.5}} \le b_f$$

where

$$\lambda_p = \frac{b_f}{t}\sqrt{\frac{F_y}{720\,000k}}$$

where

$$k = \frac{0.9}{(s / b_f)^2} + 0.2(s / b_f)^2 + 0.75$$

α_1 = value specified in Clause 18.2.2

A_r = area of longitudinal reinforcement

$$\lambda = \sqrt{\frac{C_p}{C_{ec}}}$$

where
C_p = C_{rc} computed with ϕ, ϕ_c, and $\phi_r = 1.0$ and $\lambda = 0$

C_{ec} = value specified in Clause 18.2.2

n = 1.34

18.3.3 Bending resistance

The factored bending resistance of a partially encased three-plate built-up composite column shall be taken as

$$M_{rc} = C_r e + C'_r e'$$

where

$$C_r = \frac{\phi A_s F_y - C'_r}{2}$$

$C_r + C'_r$ = T_r
= $\phi A_{st} F_y$

C'_r = $1.18\alpha_1 \phi_c a\,(b - t)\, f'_c$ for strong axis bending

$C_r' = 1.18\alpha_1\phi_c a\,(b - 2t)\,f_c'$ for weak axis bending

Note: *The concrete in compression is taken to have a rectangular stress block of intensity f_c' over a depth of a.*

18.3.4 Axial compression and bending

Partially encased three-plate built-up composite columns required to resist both bending moments and axial compression shall be proportioned so that

$$\frac{C_f}{C_{rc}} + \frac{M_{fx}}{M_{rcx}} + \frac{M_{fy}}{M_{rcy}} \le 1$$

18.3.5 Special reinforcement for seismic zones

① 18.3.5.1

Columns larger than 500 mm in depth in buildings where the specified one-second spectral acceleration ratio ($I_E F_v S_a(1.0)$) is greater than 0.30 shall be reinforced with longitudinal and transverse bars.

18.3.5.2

The longitudinal bars specified in Clause 18.3.5.1 shall

a) have an area not less than 0.005 times the total gross cross-sectional area;

b) be at least two in number in each cell; and

c) be positioned against the tie bars and at a spacing not greater than the tie spacing, s.

18.3.5.3

The transverse bars specified in Clause 18.3.5.1 shall

a) be U-shaped 15M bars arranged to provide corner support to at least every alternate longitudinal bar in such a way that no unsupported longitudinal bar is farther than 150 mm clear from a laterally supported bar;

b) have ends welded to the web of the steel shape, in line with the ends of the transverse bars located in the opposite cell, or ends anchored within the concrete core located on the opposite side of the web; and

c) have a vertical spacing not greater than the tie spacing, s, or 16 times the diameter of the smallest longitudinal bar.

18.4 Encased composite columns

18.4.1 General

Clause 18.4 applies to doubly symmetrical steel columns encased in concrete, provided that

a) the steel shape is a Class 1, 2, or 3 section;

b) $A_s \ge 0.04$ of the gross cross-sectional area;

c) $A_s + A_r \le 0.20$ of the gross cross-sectional area;

d) the concrete is of normal density and has a compressive strength, f_c', between 20 and 55 MPa;

e) the specified yield strength of structural steel, F_y, does not exceed 350 MPa; and

f) the specified yield strength of reinforcement, F_{yr}, does not exceed 400 MPa.

① 18.4.2 Compressive resistance

The factored compressive resistance of a steel concrete-encased composite column shall be taken as

$$C_{rc} = (\phi\, A_s F_y + \alpha_1 \phi_c A_c f_c' + \phi_r A_r F_{yr})(1 + \lambda^{2n})^{-1/n}$$

where

α_1 = value specified in Clause 18.2.2

A_r = value specified in Clause 18.3.2

λ = value specified in Clause 18.3.2

n = value specified in Clause 18.3.2

18.4.3 Reinforcement

18.4.3.1

The concrete encasement shall be reinforced with longitudinal bars and lateral ties extending completely around the structural steel core. The clear cover shall be not less than 40 mm.

The longitudinal bars shall

a) be continuous at framed levels when considered to carry load;
b) have an area not less than 0.01 times the total gross cross-sectional area;
c) be located at each corner; and
d) spaced on all sides not further apart than the lesser of $525t / \sqrt{F_y}$ and one-half the least dimension of the composite section.

18.4.3.2

The lateral ties shall

a) be 15M bars, except that 10M bars may be used when no side dimension of the composite section exceeds 500 mm; and
b) have a vertical spacing not exceeding the least of the following:
   i) two-thirds of the least side dimension of the cross-section;
   ii) 16 longitudinal bar diameters; or
   iii) 500 mm.

18.4.4 Columns with multiple steel shapes

Where the composite cross-section includes two or more steel shapes, the steel shapes shall be considered built-up members subject to the requirements of Clause 19 until the concrete strength reaches $0.75f'_c$.

18.4.5 Load transfer

The portion of the total axial load resisted by the concrete shall be developed by direct bearing at connections. The bearing strength of concrete may be taken as $1.95\phi_c\alpha_1 f'_c A_L$, where A_L is the loaded area, provided that the concrete is restrained against lateral expansion.

18.4.6 Bending resistance

The bending resistance of encased composite columns may be determined according to the Structural Stability Research Council's *Guide to Stability Design Criteria for Metal Structures*.

19 Built-up members

19.1 General

Components of built-up members shall be joined for the applied forces and other minimum connection requirements specified in this Clause.

Note: *The use of fillet welds or partial penetration welds, instead of complete joint penetration welds, is encouraged. If undermatching is permitted per CSA W59, this also needs to be considered. This will provide better ductility, improve fracture resistance, minimize lamellar tearing, and minimize distortion of the overall built-up section.*

19.2 Members in compression

19.2.1

All components of built-up compression members and the transverse spacing of their lines of connecting bolts or welds shall meet the requirements of Clauses 10 and 11.

19.2.2

Component parts that are in contact with one another at the ends of built-up compression members shall be connected by

a) bolts spaced longitudinally not more than four diameters apart for a distance equal to 1.5 times the width of the member; or

b) continuous welds having a length of not less than the width of the member.

19.2.3

Unless closer spacing is required for transfer of load or sealing inaccessible surfaces, the longitudinal spacing in-line between intermediate bolts or the clear longitudinal spacing between intermittent welds for the outside plate component of built-up compression members shall not exceed the following, where t is the thickness of the outside plate:

a) when the bolts or intermittent welds are staggered on adjacent lines: $525t / \sqrt{F_y}$, but not more than 450 mm; and

b) when the bolts on all gauge lines or intermittent welds along the component edges are not staggered: $330t / \sqrt{F_y}$, but not more than 300 mm.

19.2.4

Compression members composed of two or more shapes in contact or separated from one another shall be interconnected in such a way that the slenderness ratio of any component, based on its least radius of gyration and the distance between interconnections, shall not exceed that of the built-up member. The compressive resistance of the built-up member shall be based on

a) the slenderness ratio of the built-up member with respect to the appropriate axis, when the buckling mode does not involve relative deformation that produces shear forces in the interconnectors; or

b) an equivalent slenderness ratio, with respect to the axis orthogonal to that in Item a), when the buckling mode involves relative deformation that produces shear forces in the interconnectors, taken as follows:

$$\rho_e = \sqrt{\rho_o^2 + \rho_i^2}$$

where

ρ_e = equivalent slenderness ratio of the built-up member

ρ_o = slenderness ratio of the built-up member acting as an integral unit

ρ_i = maximum slenderness ratio of component part of the built-up member between interconnectors

For built-up members composed of two interconnected shapes, e.g., back-to-back angles or channels, in contact or separated only by filler plates, the maximum slenderness ratio of component parts between fasteners or welds shall be based on an effective length factor of 1.0 when the fasteners are snug-tight bolts and 0.65 when welds or pretensioned bolts are used.

For built-up members composed of two interconnected shapes separated by lacing or batten plates, the maximum slenderness ratio of component parts between fasteners or welds shall be based on an effective length factor of 1.0 for both snug-tight and pretensioned bolts and for welds.

For compound compression members, connections at the ends and interconnectors should be capable of transferring the shears and moments through a rigid connection up to the factored load levels.

19.2.5
For starred angle compression members interconnected at least at the one-third points, Clause 19.2.4 need not apply.

19.2.6
The fasteners and interconnecting parts, if any, of members identified in Clause 19.2.4 shall be proportioned to resist a force equal to 0.01 times the total force in the built-up member.

19.2.7
Spacing requirements of Clauses 19.2.3, 19.3.3, and 19.3.4 might not always provide a continuous tight fit between components in contact. When the environment is such that corrosion could be a serious problem, it is possible that the spacing of bolts or welds will need to be less than the specified maximum.

19.2.8
Open sides of compression members built up from plates or shapes shall be connected to each other by lacing, batten plates, or perforated cover plates.

19.2.9
Lacing shall provide a complete triangulated shear system and may consist of bars, rods, or shapes. Lacing shall be proportioned to resist a shear normal to the longitudinal axis of the member of not less than 0.025 times the total axial load on the member plus the shear from transverse loads, if any.

19.2.10
The slenderness ratio of lacing members shall not exceed 140. The effective length for single lacing shall be the distance between connections to the main components; for double lacing connected at the intersections, the effective length shall be half of that distance.

19.2.11
Lacing members shall be inclined preferably to the longitudinal axis of the built-up member at an angle of not less than 45°.

19.2.12

Lacing systems shall have diaphragms in the plane of the lacing and as near to the ends as practicable, as well as at intermediate points where lacing is interrupted. Such diaphragms may be plates (tie plates) or shapes.

19.2.13

End tie plates used as diaphragms shall have a length not less than the distance between the lines of bolts or welds connecting them to the main components of the member. Intermediate tie plates shall be at least one-half the specified length of end tie plates. The thickness of tie plates shall be at least 1/60 of the width between lines of bolts or welds connecting them to the main components, and the longitudinal spacing of the bolts or clear longitudinal spacing between welds shall not exceed 150 mm. At least three bolts shall connect the tie plate to each main component or a total length of weld not less than one-third the length of tie plate shall be used.

19.2.14

Shapes used as diaphragms shall be proportioned and connected to transmit a longitudinal shear equal to 0.05 times the axial compression in the member from one main component to the other.

19.2.15

Perforated cover plates may be used in lieu of lacing and tie plates on open sides of built-up compressive members. The net width of such plates at access holes may be assumed to resist axial load, provided that

a) the width-to-thickness ratio is as specified in Clause 11;
b) the length of the access hole does not exceed twice its width;
c) the clear distance between access holes in the direction of load is not less than the transverse distance between lines of bolts or welds connecting the perforated plate to the main components of the built-up member; and
d) the periphery of the access hole has a minimum radius of 40 mm at all points.

19.2.16

Battens consisting of plates or shapes may be used on open sides of built-up compression members that do not carry primary bending in addition to axial load. Battens shall be provided at the ends of the member, at locations where the member is laterally supported along its length, and elsewhere as required by Clause 19.2.4.

19.2.17

Battens shall have a length of not less than the distance between lines of bolts or welds connecting them to the main components of the member and shall have a thickness of not less than 1/60 of this distance if the batten consists of a flat plate. Battens and their connections shall be proportioned to resist the following simultaneously:

a) a longitudinal shear force $V_f = \dfrac{0.025\,C_f d}{na}$; and
b) a moment $M_f = 0.025\,C_f d/2n$

where

d = longitudinal centre-to-centre distance between battens

n = number of parallel planes of battens

a = distance between lines of bolts or welds connecting the batten to each main component

19.3 Members in tension

19.3.1
Members in tension composed of two or more shapes, plates, or bars separated from one another by intermittent fillers shall have the components interconnected at fillers spaced so that the slenderness ratio of any component between points of interconnection shall not exceed 300.

19.3.2
Members in tension composed of two plate components in contact or a shape and a plate component in contact shall have the components interconnected so that the spacing between connecting bolts or clear spacing between welds does not exceed the lesser of 36 times the thickness of the thinner plate or 450 mm (see Clause 19.2.3).

19.3.3
Members in tension composed of two or more shapes in contact shall have the components interconnected so that the spacing between connecting bolts or the clear spacing between welds does not exceed 600 mm, except where it can be determined that a greater spacing would not affect the satisfactory performance of the member (see Clause 19.2.3).

19.3.4
Members in tension composed of two separated main components may have perforated cover plates or tie plates on the open sides of the built-up member. Tie plates, including end tie plates, shall have a length of not less than two-thirds of the transverse distance between bolts or welds connecting them to the main components of the member and shall be spaced so that the slenderness ratio of any component between the tie plates does not exceed 300. The thickness of tie plates shall be at least 1/60 of the transverse distance between the bolts or welds connecting them to the main components, and the longitudinal spacing of the bolts or welds shall not exceed 150 mm. Perforated cover plates shall meet the requirements of Items b), c), and d) of Clause 19.2.15.

19.4 Open box-type beams and grillages
Two or more rolled beams or channels used side by side to form a flexural member shall be connected at intervals of not more than 1500 mm. Through-bolts and separators may be used, provided that, in beams having a depth of 300 mm or more, not fewer than two bolts are used at each separator location. When concentrated loads are carried from one beam to the other or distributed between the beams, diaphragms having sufficient stiffness to distribute the load shall be bolted or welded between the beams. The design of members shall provide for torsion resulting from any unequal distribution of loads. Where beams are exposed, they shall be sealed against corrosion of interior surfaces or spaced sufficiently far apart to permit cleaning and coating.

20 Plate walls

20.1 General

20.1.1 Definition
A plate wall is a lateral-force-resisting structural system consisting of a framework of columns and beams, with relatively thin infill plates in the plane of the frame connected all around to the

surrounding members. Frame connections between the beams and columns may be moment-resisting or simple shear connections.

20.1.2 Lateral resistance

Lateral storey shears are considered to be carried by a combination of frame action, if applicable, and post-buckling tension fields that develop in the infill plates parallel to the direction of the principal tensile stresses. Axial forces and moments develop in the beams and columns of plate walls as a result of the

a) response of the wall to the overall bending and shear; and
b) tension field action in the adjacent infill plates.

① 20.2 Seismic applications

Under seismic loading, plate walls shall also meet the requirements of Clause 27.9 or 27.10, as appropriate.

① 20.3 Analysis and design

Forces and moments in the members and connections, including those resulting from tension field action, may be determined from a plane frame analysis, with the infill plates represented by a series of inclined pin-ended strips.

20.4 Angle of inclination

20.4.1

When the aspect ratio of the panel lies within the limits $0.6 \le L/h \le 2.5$, the angle of inclination from the vertical, α, of the inclined pin-ended strips may be taken as 40°. Otherwise, it shall be determined as follows and shall be between 38° and 45°:

$$\tan^4\alpha = \frac{1 + \frac{wL}{2A_c}}{1 + wh\left(\frac{1}{A_b} + \frac{h^3}{360 I_c L}\right)}$$

where

w = infill plate thickness

L = centre-to-centre distance between columns

A_c = cross-sectional area of column

h = storey height

A_b = cross-sectional area of beam

I_c = moment of inertia of column

20.4.2

A single angle of inclination taken as the average for all the panels may be used to analyze the entire plate wall.

20.5 Limits on column and beam flexibilities

20.5.1

The column flexibility parameter at each panel, ω_h, shall be determined as follows and shall not exceed 2.5:

$$\omega_h = 0.7h\left(\frac{w}{2LI_c}\right)^{0.25}$$

This requirement is met by providing columns with moments of inertia, I_c, greater than or equal to $0.0031wh^4/L$.

20.5.2

The boundary member flexibility parameter for the extreme panels, ω_L, shall be determined as follows:

a) not exceed 2.5 at the top panel of the plate wall;
b) not exceed 2.0 at the bottom panel of the plate wall; and
c) be greater than $0.84\omega_h$:

$$\omega_L = 0.7\left(\left(\frac{h^4}{I_c} + \frac{L^4}{I_b}\right)\frac{w}{4L}\right)^{0.25}$$

These requirements are met by providing a beam with a moment of inertia, I_b, greater than or equal to $\dfrac{wL^4}{650L - (wh^4/I_c)}$ for the top beam and $\dfrac{wL^4}{267L - (wh^4/I_c)}$ for the bottom beam, if present. See also Clause 20.9.2.

20.6 Infill plates

The factored tensile resistance of the inclined infill plate strips shall be calculated in accordance with Clause 13.2.

①

20.7 Beams

Beams shall be Class 1 or 2, except as required by Clause 27.9.3.1. Beams shall be proportioned to resist bending moments and axial compressive forces in accordance with Clause 13.8. Infill plates shall not be deemed to provide lateral support to adjacent beams.

20.8 Columns

Columns shall be Class 1 sections and proportioned to resist bending moments and axial forces in accordance with Clause 13.8 or 13.9, as appropriate. Infill plates shall not be deemed to provide lateral support to adjacent columns.

20.9 Anchorage of infill plates

20.9.1

At the top panel, the vertical component of the infill plate tension field shall be anchored to a beam meeting the requirements of Clause 20.5.2.

20.9.2

At the bottom panel, the vertical component of the infill plate tension field shall be anchored by connecting the infill plate directly to the substructure or to a beam that meets the requirements of Clause 20.5.2.

20.9.3

At the bottom panel, the horizontal component of the infill plate tension field shall be transferred to the substructure.

20.10 Infill plate connections

Infill plates shall be connected to the surrounding beams and columns. These connections and, if required, any infill plate splices shall be in accordance with Clause 13.12 or 13.13. The factored ultimate tensile strength of the infill plate strips shall be developed by the connections.

21 Connections

21.1 Alignment of members

Axially-loaded members that meet at a joint shall have their centroidal axes intersect at a common point if practicable. Bending resulting from joint eccentricity shall be taken into account.

21.2 Unrestrained members

Except as otherwise indicated in the structural design documents, all connections of beams, girders, and trusses shall be designed and detailed as flexible and ordinarily may be proportioned for the reaction shears only. Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, inelastic action at the specified load levels in the connection is permitted.

21.3 Restrained members

When beams, girders, or trusses are subject to both reaction shear and end moment due to full or partial end restraint or to continuous or cantilever construction, their connections shall be designed for the combined effect of shear, bending, and axial load.

When beams are rigidly framed to the flange of an I-shaped column and the distance from the end of the column to the top flange of the beam is greater than the depth of the column, stiffeners shall be provided on the column web if the following bearing and tensile resistances of the column are exceeded:

a) opposite the compression flange of the beam:

$$B_r = \phi_{bi} w_c (t_b + 10t_c) F_{yc} < \frac{M_f}{d_b}$$

except when the column has a Class 3 or 4 web, in which case the following shall apply:

$$B_r = \frac{640\,000 \phi_{bi} w_c (t_b + 10t_c)}{(h_c / w_c)^2}$$

b) opposite the tension flange of the beam when the connected element is

i) welded to the column:

$$T_r = 7\phi t_c^2 F_{yc} < \frac{M_f}{d_b}$$

ii) bolted to the column with two rows of bolts centered about the web of column and the tension flange of the beam:

$$T_r = \phi 2 t_c^2 F_{yc}\left[\sqrt{\frac{b_c}{g}} + \frac{e + c_b}{g}\right] < \frac{M_f}{d_b}$$

where

w_c = thickness of column web

t_b = thickness of beam flange

t_c = thickness of column flange

b_c = width of column flange, but not to be taken as greater than $0.25(9g - 5w_c)$

g = bolt gauge, spacing of the tension bolts transverse to long axis of the column

c_b = bolt spacing between two bolt rows in tension, taken parallel to the long axis of the column, but not to be taken as greater than $2\sqrt{b_c \times g}$

e = distance from the free end of the unstiffened column to the nearest bolt row in tension, but e is not to be taken as greater than $\sqrt{b_c \times g}$

F_{yc} = specified yield point of column

d_b = depth of beam

h_c = clear depth of column web

The stiffener or pair of stiffeners opposite either beam flange shall develop a force, F_{st}, equal to

$(M_f/d_b) - B_r$.

Stiffeners shall also be provided on the web of columns, beams, or girders if V_r calculated from Clause 13.4.2 is exceeded, in which case the stiffener or stiffeners shall transfer a shear force, V_{st}, equal to

$V_f - 0.8\phi A_w F_s$

The stiffeners shall be connected so that the force in the stiffener is transferred through the stiffener connection. When beams frame to one side of the column only, the stiffeners need not be longer than one-half of the depth of the column. When an axial tension or compression force is acting on the beam, its effects (additive only) shall be considered in the design of the stiffeners.

When beams are rigidly framed to the flange of an I-shaped column and the distance from the end of the column to the top flange of the beam is less than or equal to the depth of the column, the requirement of stiffeners shall be evaluated by rational analysis. In lieu of rational analysis, stiffeners shall be provided.

21.4 Connections of tension or compression members

The connections at ends of compression members not finished to bear or of tension members shall be designed for the full factored load effect.

21.5 Bearing joints in compression members

Where columns or other compression members bear on bearing plates or are finished to bear at splices, there shall be sufficient fasteners or welds to hold all parts securely in place to provide a satisfactory level of structural integrity (see Clauses 6.1.2, 28.5, and 29.3.9). The flanges of single web members shall be connected.

21.6 Lamellar tearing

Corner or T-joint details of rolled structural members or plates involving transfer of tensile forces in the through-thickness direction resulting from shrinkage due to welding executed under conditions of restraint shall be avoided where possible. If this type of connection cannot be avoided, measures shall be taken to address the possibility of lamellar tearing.

21.7 Placement of fasteners and welds

Except in members subject to fatigue (see Clause 26) and in braces subject to seismic loads (see Clause 27.5.4.1), disposition of fillet welds to balance the forces about the neutral axis or axes for end connections of single-angle, double-angle, or similar types of axially loaded members shall not be required. Eccentricity between the centroidal axes of such members and the gauge lines of bolted end connections may also be neglected. In axially loaded members subject to fatigue, the fasteners or welds in end connections shall have their centroid on the centroidal axis of the member unless provision is made for the effect of the resulting eccentricity.

21.8 Fillers

21.8.1 Fillers in bolted connections

21.8.1.1

When load-carrying fasteners pass through fillers with a total thickness greater than 19 mm, the fillers shall be extended beyond the splice material and the filler extension shall be secured by sufficient fasteners to distribute the total force in the connected element uniformly over the combined cross-section of the connected element and the filler. , If the filler extension is not provided and/or the filler is not secured by sufficient fasteners, an equivalent number of fasteners shall be included in the connection.

21.8.1.2

When load-carrying fasteners pass through fillers with a total thickness between 6.4 and 19 mm, the shear capacity of the fasteners shall be reduced to account for bending in the fasteners by R_v, as follows:

$R_v = 1.1 - 0.0158t$

where
t = thickness of the fillers

Alternatively, the fillers shall be extended beyond the splice material and the filler extension shall be secured by sufficient fasteners to distribute the total force in the connected element uniformly over the combined cross-section of the connected element and the filler or an equivalent number of fasteners shall be included in the connection.

21.8.1.3

When load-carrying fasteners pass through fillers with a total thickness less than or equal to 6.4 mm, the shear capacity of the fasteners need not be reduced.

21.8.2 Fillers in welded connections

In welded construction, any filler with a total thickness greater than 6 mm shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate load, applied at the surface of the filler, as an eccentric load. Welds that connect the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler that is 6 mm or less in thickness shall have its edges made flush with the edges of the splice plate and the required weld size shall be equal to the thickness of the filler plate plus the size necessary to transmit the splice plate load.

21.9 Welds in combination

If two or more of the general types of weld (groove, fillet, plug, or slot) are combined in a single connection, the effective capacity of each shall be calculated separately with reference to the axis of the group to determine the factored resistance of the combination.

21.10 Fasteners and welds in combination

21.10.1 New connections

The strength of a joint that combines welds and bolts in the same plane shall be proportioned in accordance with Clause 13.14.

21.10.2 Existing connections

The strength of a joint that combines welds and bolts in the same plane shall be proportioned in accordance with Clause 13.14.

The loads that are being carried by the existing welds and/or bolts at the time that the new fasteners are installed shall be considered when determining the strength of the joint.

21.11 High-strength bolts (in slip-critical joints) and rivets in combination

In making alterations, rivets and high-strength bolts in slip-critical joints may be considered as sharing forces caused by specified dead and live loads.

21.12 Connected elements under combined tension and shear stresses

Except as noted elsewhere in this Standard, welded connection plates under combined normal stress, σ_n, and shear stress, τ, shall be proportioned such that $\tau \leq 0.66\phi F_y$ and $\sigma_n \leq \sigma_{nR}$, where

when $\tau \leq 0.5\phi F_y$, $\sigma_{nR} = \phi F_y$

when $\tau > 0.5\phi F_y$, $\sigma_{nR} = 25/4\ (0.66\phi F_y - \tau)$

22 Design and detailing of bolted connections

22.1 General

Clause 22 deals primarily with ASTM A325, ASTM A325M, ASTM A490, ASTM A490M, ASTM F1852, and ASTM F2280 bolt assemblies and equivalent fasteners. The bolts may be required to be installed to a specified minimum tension, depending on the type of connection.

22.2 Design of bolted connections

22.2.1 Use of snug-tightened high-strength bolts

Snug-tightened high-strength bolts may be used in connections other than those specified in Clause 22.2.2 (see Clause 23.6).

22.2.2 Use of pretensioned high-strength bolts

Pretensioned high-strength bolts (ASTM A325, ASTM A325M, ASTM A490, ASTM A490M, ASTM F1852, and ASTM F2280) shall be used in

a) slip-critical connections where slippage cannot be tolerated (e.g., connections subject to fatigue or frequent load reversal, or connections in structures that have rigorous deflection or stiffness limit states);
b) shear connections, when required by Clause 27.1;
c) all elements resisting crane loads;
d) connections subject to impact or cyclic loading;
e) connections where the bolts are subject to tensile loading (see Clause 13.12.1.3); and
f) connections using oversize or long slotted holes (unless specifically designed to accommodate movement).

22.2.3 Joints subject to fatigue loading

Joints subject to fatigue loading shall be proportioned in accordance with Clause 26.

22.2.4 Effective bearing area

The effective bearing area of bolts shall be the nominal diameter multiplied by the length in bearing. For countersunk bolts, half of the depth of the countersink shall be deducted from the bearing length.

22.2.5 Fastener components

Δ 22.2.5.1 Structural bolt assemblies

Except as specified in Clause 22.2.5.4, bolts, nuts, and washers for structural bolt assemblies shall meet the requirements of ASTM A325, ASTM A325M, ASTM A490, ASTM A490M, ASTM F1852, or ASTM F2280.

22.2.5.2 Galvanized bolt assemblies

Galvanized ASTM A325 and ASTM A325M bolt assemblies shall meet the galvanizing requirements of ASTM A325 and ASTM A325M.

22.2.5.3 Zinc/aluminum coated bolt assemblies

Zinc/aluminum coated ASTM A325, ASTM A325M, ASTM A490, and ASTM A490M bolt assemblies shall meet the coating requirements of ASTM F1136.

22.2.5.4 Alternatives to ASTM A325, ASTM A325M, ASTM A490, and ASTM A490M bolt assemblies

Other fasteners may be used if they meet the chemical and mechanical requirements of ASTM A325, ASTM A325M, ASTM A490, or ASTM A490M and have body diameters and bearing areas under the head and nut specified in those Standards. Such fasteners may differ in other dimensions and their use shall be subject to the approval of the designer.

22.3 Detailing of bolted connections

22.3.1 Minimum pitch

The minimum distance between centres of bolt holes shall be 2.7 times the bolt diameter.

22.3.2 Minimum edge distance
The minimum distance from the centre of a bolt hole to an edge shall be as specified in Table 6.

22.3.3 Maximum edge distance
The maximum distance from the centre of any bolt to the nearest edge of parts in contact shall be 12 times the thickness of the outside connected part, but not greater than 150 mm.

22.3.4 Minimum end distance
In the connection of tension members having more than two bolts in a line parallel to the direction of load, the minimum end distance (from the centre of the end fastener to the nearest end of the connected part) shall be governed by the edge distance values specified in Table 6. In members having one or two bolts in the line of load, the end distance shall be not less than 1.5 bolt diameters.

22.3.5 Bolt holes

22.3.5.1
Holes may be punched, sub-punched, sub-drilled and reamed, or drilled, as permitted by Clause 28.4. The nominal diameter of a hole shall be not more than 2 mm greater than the nominal bolt size. This requirement may be waived to permit the use of the following bolt diameters and hole combinations in bearing-type or slip-critical connections:
a) 3/4 in diameter bolt or an M20 bolt in a 22 mm hole;
b) a 7/8 in diameter bolt or an M22 bolt in a 24 mm hole; and
c) a 1 in diameter bolt or an M24 bolt in a 27 mm hole.

Oversized or slotted holes may be used with high-strength bolts 16 mm in diameter and larger when approved by the designer.

① 22.3.5.2
Joints that use enlarged or slotted holes shall be proportioned in accordance with Clauses 13.11, 13.12, and 23 and meet the following requirements:
a) Oversize holes shall be 4 mm larger than bolts 22 mm and less in diameter, 6 mm larger than bolts 24 mm in diameter, and 8 mm larger than bolts 27 mm and greater in diameter. Oversize holes shall not be used in bearing-type connections but may be used in any or all plies of slip-critical connections. Hardened washers shall be used under heads or nuts adjacent to the plies containing oversize holes.
b) Short slotted holes shall be 2 mm wider than the bolt diameter and have a length that does not exceed the oversize diameter requirements of Item a) by more than 2 mm. Short slotted holes may be used in any or all plies of slip-critical or bearing-type connections and without regard to direction of loading in slip-critical connections, but shall be normal to the direction of the load in bearing-type connections. For pretensioned bolts, hardened washers shall be used under heads or nuts adjacent to the plies containing the slotted holes.
c) Long slotted holes shall be 2 mm wider than the bolt diameter, shall have a length greater than that allowed in Item b) (but not more than 2.5 times the bolt diameter in only one of the connected parts at an individual faying surface of either a slip-critical or bearing-type connection), and may be used in
   i) slip-critical connections without regard to the direction of loading (slip resistance shall be decreased in accordance with Clause 13.12.2.2); and
   ii) bearing-type connections with the long dimension of the slot normal to the direction of loading, provided that structural plate washers or a continuous bar not less than 8 mm in

thickness covers long slots that are in the outer plies of joints. The plate washers or bar shall have a size sufficient to completely cover the slot after installation. Plate washers or bars shall not be required for bearing-type connections in double shear having long slotted holes in the inner ply only.

22.3.5.3

The maximum and minimum edge distance for bolts in slotted or oversize holes (as permitted in Clause 22.3.5.1) shall meet the requirements of Clauses 22.3.2 to 22.3.4, assuming that the fastener can be placed at any extremity of the slot or hole.

23 Installation and inspection of bolted joints

23.1 Connection fit-up

When assembled, all joint surfaces, including those adjacent to bolt heads, nuts, and washers, shall be free of scale (tight mill scale excepted), burrs in excess of 2 mm in height, dirt, and foreign material that could prevent firm contact of the parts. Connections using high-strength bolts shall be in firm contact when assembled and shall not be separated by gaskets or compressible materials.

Δ 23.2 Surface conditions for slip-critical connections

The condition of the contact surfaces for slip-critical connections, as specified in Table 3, shall be as follows:

a) For clean mill scale, the surfaces shall be free of oil, paint, lacquer, or any other coating for all areas within the bolt pattern and for a distance beyond the edge of the bolt hole that is the greater of 25 mm or the bolt diameter.
b) For Classes A and B, the blast-cleaning and the coating application shall be the same as those used in the tests to determine the mean slip coefficient.
c) For hot-dip galvanized surfaces, galvanizing shall be done in accordance with CAN/CSA-G164 and the surface subsequently roughened by hand wire-brushing. Power wire-brushing shall not be used.
d) For all other coatings, the surface preparation and coating application for the joint shall be the same as those used in the tests to determine the mean slip coefficient.

Coated joints shall not be assembled before the coatings have cured for the minimum time used in the tests to determine the mean slip coefficient.

23.3 Minimum bolt length

The length of bolts shall be such that the point of the bolt will be flush with or outside the face of the nut when completely installed.

23.4 Use of washers

23.4.1

ASTM F436 hardened washers shall be used under the turned element

a) as required by Clause 23.4.2;
b) for pretensioned ASTM F1852 and ASTM F2280 bolts; and
c) for bolt arbitration inspection procedures.

① **23.4.2**

When high strength bolts are pretensioned, ASTM F436 hardened washers shall
a) be used to cover oversize or slotted holes (see Clause 22.3.5);
b) be used with ASTM F959 washers, as applicable;
c) be placed under the head and nut when used with steel having a specified minimum yield point of less than 280 MPa and the bolts are either ASTM A490, ASTM A490M, or ASTM F2280; and
d) be not less than 8 mm in thickness when either ASTM A490, ASTM A490M, or ASTM F2280 bolts greater than 26 mm in diameter are used in oversize and slotted holes, except that ASTM F436 washers in combination with a 10 mm plate washer covering the holes may be used.

23.4.3

If necessary, washers may be clipped on one side to a point not closer than 7/8 of the bolt diameter from the centre of the washer hole.

23.4.4

ASTM F436 bevelled washers shall be used to compensate for lack of parallelism where, in the case of ASTM A325, ASTM A325M, and ASTM F1852 bolts, an outer face of bolted parts has more than a 5% slope with respect to a plane normal to the bolt axis. In the case of ASTM A490, ASTM A490M, and ASTM F2280 bolts, bevelled washers shall be used to compensate for any lack of parallelism due to the slope of the outer faces.

23.5 Storage of fastener components for pretensioned bolt assemblies

Fastener components shall
a) be stored in closed containers;
b) be returned to protected storage at the end of the work shift when not incorporated into the work;
c) not have the as-delivered condition altered in any fashion, including cleaning; and
d) not be incorporated into the work if rust or dirt resulting from plant or job site conditions accumulates unless they are cleaned, relubricated, and requalified with a bolt tension calibrator.

ASTM F1852 and ASTM F2280 bolt assemblies shall not be relubricated, except by the manufacturer.

23.6 Snug-tightened bolt assemblies

Snug-tightened bolted assemblies shall have the following two conditions:
a) High-strength fastener assemblies that are not required to be pretensioned shall be installed in properly aligned holes to a snug-tight condition as a minimum (for slotted holes, see Clause 22.3.5.2).
b) Fastener assemblies incorporating ASTM A307 bolts shall only be snug-tightened. Where so specified by the designer, additional security from working loose of ASTM A307 assemblies shall be provided by the use of lock washers, locknuts, jam nuts, thread burring, welding or other methods so approved.

23.7 Pretensioned high-strength bolt assemblies

23.7.1 Installation procedure

Pretensioned bolts shall be installed to at least the minimum bolt tensions specified in Table 7, in accordance with the following procedure:
a) After the holes in a joint are aligned, sufficient bolts shall be placed to secure the member.

b) Bolts shall be placed in the remaining open holes and snug-tightened, with joint assembly progressing systematically from the most rigid part of the joint to its free edges (re-snugging may be necessary in large joints).
c) When all bolts are snug-tight, each bolt in the joint shall be pretensioned, with pretensioning progressing systematically from the most rigid part of the joint to its free edges in a manner that will minimize relaxation of previously pretensioned bolts.

23.7.2 Turn-of-nut method

After the snug-tightening procedure is completed, each bolt in the connection shall be pretensioned additionally by the applicable amount of relative rotation specified in Table 8. During this operation there shall be no rotation of the part not turned by the wrench unless the bolt and nut are match-marked to enable the amount of relative rotation to be determined.

23.7.3 Use of ASTM F959 washers

When ASTM F959 washers are used (also known as direct tension indicator washers), the pretension of the bolt in accordance with Table 7 shall be verified using a tension calibrator. Prior to installation of ASTM F959 bolt assemblies, a sample of not fewer than three complete bolt assemblies of each combination of diameter, length, grade, and lot to be used in the work shall be placed individually in a bolt-tension calibrator at the site of installation to verify that the pretensioning method develops a tension that is equal to or greater than 1.05 times the minimum tensions specified in Table 7. The pre-installation verification procedure shall be performed at the start of the work and whenever the lot of fastener assembly is changed.

23.7.4 Use of ASTM F1852 and ASTM F2280 bolts

Prior to installation of ASTM F1852 and ASTM F2280 bolt assemblies in joints requiring pretension, a sample of not fewer than three complete bolt assemblies of each combination of diameter, length, grade, and lot to be used in the work shall be placed individually in a bolt-tension calibrator at the site of installation to verify that the pretensioning method develops a tension that is equal to or greater than 1.05 times the minimum tensions specified in Table 7. The pre-installation verification procedure shall be performed at the start of the work and whenever the lot of fastener assembly is changed.

During the snug-tightening procedure, care shall be taken to avoid severing the splined ends. Bolts with severed ends shall be replaced. After the snug-tightening procedure is completed, each bolt in the joint shall be pretensioned.

23.8 Inspection procedures

23.8.1

The inspector shall determine that the requirements of Clauses 23.1 to 23.6 are met. Tensioning of bolts shall be observed during their installation to ascertain that the proper procedures are employed. In addition, the following shall apply:
a) for snug-tight connections, the inspection need ensure only that the bolts have been tightened sufficiently to bring the connected elements into firm contact;
b) for bolts pretensioned by the turn-of-nut method, the turned element of all bolts shall be visually examined for evidence that they have been pretensioned;
c) for ASTM F959 washers, the washers shall be inspected to ensure that adequate deformations have been achieved in accordance with the manufacturer's installation procedures; and

d) for ASTM F1852 and ASTM F2280 bolt assemblies, the splined ends shall be inspected for twist-off.

Note: *For pretensioned connections, see Annex I if there is disagreement concerning the results of inspection of bolt-tensioning procedures.*

23.8.2

Bolt tensions exceeding those specified in Table 7 shall not be cause for rejection.

24 Welding

24.1 Arc welding

Arc welding shall be designed in accordance with

a) Clause 13.13 for factored resistance of welds under static loading with matching electrode (see CSA W59 for locations and conditions where non-matching is permissible); and

b) Clause 26 for resistance to fatigue loading, with matching electrode (see CSA W59 for locations and conditions where non-matching is permissible).

For all other aspects of welding, the requirements of CSA W59 shall be followed.

24.2 Resistance welding

The resistance of resistance-welded joints shall be in accordance with CSA W55.3. Quality assurance and weld process control procedures shall be as specified in CSA W55.3.

24.3 Fabricator and erector qualification

Fabricators and erectors responsible for welding structures fabricated or erected under this Standard shall be certified by the Canadian Welding Bureau to the requirements of CSA W47.1 (Division 1 or Division 2), CSA W55.3, or both, as applicable. Part of the work may be sublet to a Division 3 fabricator or erector; however, the Division 1 or Division 2 fabricator or erector shall retain responsibility for the sublet work.

25 Column bases and anchor rods

25.1 Loads

Suitable provision shall be made to transfer factored axial loads, including uplift, shears, and moments, to footings and foundations. Forces present during construction and in the finished structure shall be resisted.

25.2 Minimum number of anchor rods

Columns shall be fitted with at least four anchor rods. When four non-colinear anchor rods for erection safety are not feasible, special precautions shall be taken.

25.3 Resistance

25.3.1 Concrete in compression

The compressive resistance of concrete shall be determined in accordance with Clause 10.8 of CSA A23.3. When compression exists over the entire base plate area, the bearing pressure on the concrete may be assumed to be uniform over an area equal to the width of the base plate multiplied by the length minus $2e$, where e is the eccentricity of the column load. Where eccentricity exists about both column axes, the width of the base plate shall also be reduced by twice the eccentricity in that direction.

25.3.2 Tension

25.3.2.1 Anchor rods

The factored tensile resistance of an anchor rod shall be taken as

$$T_r = \phi_{ar} A_n F_u$$

where

ϕ_{ar} = 0.67

A_n = the tensile area of the rods

= $0.85A_g$

25.3.2.2 Pull-out

The pull-out resistance shall be determined in accordance with CSA A23.3, Annex D. Full anchorage shall be obtained when the factored pull-out resistance of the concrete is equal to or greater than the factored tensile resistance of the rods.

The determination of the pull-out value shall account for single and group anchor behaviour.

25.3.3 Shear

① 25.3.3.1 Shear transfer mechanisms

Shear resistance may be developed by friction between the base plate and the foundation unit or by bearing of the anchor rods or shear lugs against the concrete. The appropriate requirements of CSA A23.3, Clause 11 and Annex D, shall be met for

a) anchor rods bearing against the concrete;
b) loads are transferred by friction;
c) shear lugs bearing against the concrete; and
d) shear acting toward a free edge of concrete.

25.3.3.2 Anchor rods in bearing

The factored bearing resistance of an anchor rod shall be determined by CSA A23.3, Annex D. The thickness of the grout layer under the base plate shall be taken into account, in accordance with CSA A23.3.

25.3.3.3 Anchor rods in shear

The factored shear resistance of an anchor rod shall be taken as

$$V_r = 0.60\phi_{ar} A_{ar} F_u$$

where

A_{ar} = cross-sectional area of the anchor rod based on its nominal diameter

When the rod threads are intercepted by the shear plane, the factored shear resistance shall be taken as $0.70V_r$.

Δ 25.3.4 Anchor rods in shear and tension

An anchor rod required to develop resistance to both tension and shear shall be proportioned so that

$(V_f/V_r)^2 + (T_f/T_r)^2 \leq 1$

where

V_r = the lesser of the factored shear resistance of the anchor rod or the portion of the total shear per rod resisted by bearing of the anchor rods on the concrete

T_r = the lesser of the factored tension resistance of the anchor rod or the factored pull-out resistance of the concrete

25.3.5 Anchor rods in tension and bending

An anchor rod required to develop resistance to both tension and bending shall be proportioned to meet the requirements of Clause 13.9.1. The tensile and moment resistances, T_r and M_r, shall be based on the properties of the cross-section at the critical section. M_r shall be taken as $\phi_{ar}SF_y$.

25.3.6 Moment on column base

The moment resistance of a column base shall be taken as the couple formed by the tensile resistance determined in accordance with Clause 25.3.2 and by the concrete compressive resistance determined in accordance with Clause 25.3.1.

25.4 Fabrication and erection

25.4.1 Fabrication

25.4.1.1 Base plate holes

Base plate holes may be drilled, machined, or thermally cut. The surfaces of thermally cut holes shall meet the requirements of Clause 28.2.

Holes in base plates for anchor rods shall be of sufficient size to meet or exceed the placement tolerances for anchor rods. The Designer shall provide details of corrective work if base plate holes are to be adjusted to suit as-cast locations of anchor rods.

25.4.1.2 Bases resting on masonry or concrete

The bottom surfaces of bearing plates and column bases that rest on masonry or concrete foundations and are grouted to ensure full bearing need not be planed.

25.4.1.3 Rolled steel bearing plates

Finishing of steel-to-steel contact bearing surfaces shall meet the requirements of Clauses 28.5 and 29.3.9. Plates 55 mm or less thick may be used without machining. Plates more than 55 mm thick may be straightened by pressing or machined at bearing locations.

25.4.2 Erection

25.4.2.1 Setting column bases

Column bases shall be set on level finished floors, pre-grouted levelling plates, levelling nuts, or shim packs that are adequate to transfer the construction loads. Steel shim packs may remain in place unless otherwise specified by the Designer.

25.4.2.2 Tensioning of anchor rods

Nuts on anchor rods need be installed only to a snug-tight condition unless otherwise specified by the designer. If pre-tension is required, the method of tensioning and the pre-tension value shall be defined by the designer.

26 Fatigue

① 26.1 General

In addition to meeting the fatigue requirements of Clause 26, all members and connections shall meet the requirements for the static load conditions using the factored loads. Specified loads shall be used for all fatigue calculations. This is calculated using ordinary elastic analysis and the principles of mechanics of materials and includes stresses that may result from bending moments due to joint eccentricities. A specified load less than the maximum specified load but acting with a greater number of cycles can govern and therefore shall be considered. Members and connections subjected to fatigue loading shall be designed, detailed, and fabricated so as to avoid abrupt changes in cross-sections and other sources of stress concentration. The life of the structure shall be taken as 50 years, unless otherwise specified by the owner.

26.2 Proportioning

In the absence of more specific requirements by the owner or designer, the requirements of Clause 26 shall be used to proportion members and parts. Fatigue resistance shall be provided only for repetitive loads.

26.3 Live-load-induced fatigue

26.3.1 Calculation of stress range

The controlling stress feature in load-induced fatigue is the range of stress to which the element is subjected. This is calculated using ordinary elastic analysis and the principles of mechanics of materials and includes stresses that may result from the bending moments due to joint eccentricities. More sophisticated analysis shall be required only in cases not covered by Table 9, e.g., major access holes and cut-outs. Stress range is the algebraic difference between the maximum stress and minimum stress at a given location; thus, only live load induces a stress range.

The load-induced fatigue requirements of Clause 26 need be applied only at locations that undergo a net applied tensile stress. Stress ranges that are completely in compression need not be investigated for fatigue.

26.3.2 Design criteria

For load-induced fatigue and constant amplitude fatigue loading, the following design requirement shall apply:

$$F_{sr} \geq f_{sr}$$

where

F_{sr} = fatigue resistance

$$= \left(\frac{\gamma}{nN}\right)^{1/3} \geq F_{srt}$$

$$= \left(\frac{\gamma'}{nN}\right)^{1/5} \le F_{srt}$$

where

γ and γ' = fatigue life constants (see Clause 26.3.4)

n = number of stress range cycles at given detail for each application of load

N = number of applications of load

F_{srt} = constant amplitude threshold stress range (Clauses 26.3.3 and 26.3.4)

f_{sr} = calculated stress range at the detail due to passage of the fatigue load including stresses due to eccentricities

① 26.3.3 Cumulative fatigue damage

The total damage that results from variable amplitude fatigue loading shall satisfy

$$\sum\left[\frac{(nN)_i}{N_{fi}}\right] \le 1.0$$

where

$(nN)_i$ = number of expected stress range cycles at stress range level i, f_{sri}

N_{fi} = number of cycles that would cause failure at the stress range f_{sri}, obtained from Figure 1 for the appropriate fatigue category. Alternatively, it may be calculated as follows:

$N_{fi} = \gamma f_{sri}^{-3}$ for $f_{sri} \ge F_{srt}$

and

$N_{fi} = \gamma' f_{sri}^{-5}$ for $f_{sri} \le F_{srt}$

The summation shall include both stress cycles above and below F_{srt}.

The fatigue constant γ' shall be as specified in Table 10.

26.3.4 Fatigue constants and detail categories

The fatigue constants γ, γ', nN', and F_{srt} shall be as specified in Table 10 and shown in Figure 1. The detail categories shall be obtained from Table 9 and are illustrated in Figure 2.

For high-strength bolts, see also Clause 13.12.1.3.

26.3.5 Limited number of cycles

Except for fatigue-sensitive details with high stress ranges (probably with stress reversal), special considerations beyond those specified in Clause 26.1 need not apply in the event that the number of stress range cycles, nN, over the life of the structure, expected to be applied at a given detail, is less than the greater of γ / f_{sr}^3 or 20 000.

26.4 Distortion-induced fatigue

26.4.1

Members and connections shall be detailed to minimize distortion-induced fatigue that can occur in regions of high strain at the interconnection of members undergoing differential displacements.

Whenever practicable, all components that make up the cross-section of the primary member shall be fastened to the interconnection member.

26.4.2

Plate girders with $h / w > 3150 / \sqrt{F_y}$ shall not be used under fatigue conditions.

26.5 High-strength bolts

A high-strength bolt subjected to tensile cyclic loading shall be pretensioned to the minimum preload specified in Clause 23.7. Connected parts shall be arranged so that prying forces are minimized. The prying force per bolt shall not exceed 30% of the externally applied load.

The permissible maximum applied nominal axial stress, including amplification by prying under specified loads, based on the nominal area of the bolt, shall not exceed 214 MPa for ASTM A325, ASTM A325M, and ASTM F1852 bolts and 262 MPa for ASTM A490, ASTM A490M, and ASTM F2280 bolts.

The total maximum cyclic service load that may be applied to a bolt is calculated as the product of the permissible maximum nominal stress above and the nominal area of a bolt. Thus calculated, the service load per bolt, including the amplification by prying, shall not exceed this maximum applied service load on a pretensioned bolt.

27 Seismic design

27.1 General

27.1.1 Scope

Clause 27 specifies requirements for the design of members and connections in the seismic-force-resisting system of steel-framed building structures. With the exception of Clause 27.11, Clause 27 applies to buildings for which seismic design loads are based on a ductility-related force modification factor, R_d, greater than 1.5. Clause 27 shall be applied with the requirements of the *NBCC*. Alternatively, the maximum anticipated seismic loads may be determined from non-linear time-history analyses using appropriate structural models and ground motions. Height restrictions shall not apply when the seismic forces are determined from non-linear time-history analyses or to buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) less than 0.35, unless otherwise specified in Clause 27 or the *NBCC*.

Clause 27 may be applied to structures other than building structures provided that the structure includes a clearly defined seismic-force-resisting system and that a level of safety and seismic performance comparable to that required by Clause 27 for building structures is provided.

27.1.2 Capacity design

Unless otherwise specified in Clause 27, seismic-force-resisting systems shall be designed according to capacity design principles to resist the maximum anticipated seismic loads, but such loads need not exceed the values corresponding to $R_d R_o = 1.3$.

In capacity design,

a) specific elements or mechanisms are designed and detailed to dissipate energy;
b) all other elements are sufficiently strong for this energy dissipation to be achieved;
c) structural integrity is maintained;

d) elements and connections in the horizontal and vertical load paths are designed to resist the seismic loads;
e) diaphragms and collector elements are capable of transmitting the loads developed at each level to the vertical seismic-force-resisting system; and
f) these loads are transmitted to the foundation.

Connections along the horizontal load path that are designed for forces corresponding to $R_dR_o = 1.3$ shall have a ductile governing ultimate limit state.

27.1.3 Seismic load path

Any element that significantly affects the load path or the seismic response shall be considered in the analysis and shown on the structural drawings.

27.1.4 Members and connections supporting gravity loads

Structural members and their connections that are not considered to form part of the seismic-force-resisting system shall be capable of supporting gravity loads when subjected to seismically induced deformations.

Splices in gravity columns not part of the seismic-force-resisting system shall have a factored shear resistance in both orthogonal axes equal to the sum of $0.2ZF_y/h_s$ of the columns above and below the splices.

Note: *The gravity loads to be supported are those considered in combination with the earthquake loading.*

27.1.5 Material requirements

27.1.5.1

Steel used in the energy-dissipating elements described in Clauses 27.2 to 27.10 shall comply with Clauses 5.1.3 and 8.3.2 a). F_y shall not exceed 350 MPa unless the suitability of the steel is determined by testing or other rational means. F_y shall not exceed 450 MPa in columns in which the only expected inelastic behaviour is at the column base. Other material may be used if approved by the regulatory authority.

Note: *F_y is the specified minimum yield stress. See Clause 5.1.2.*

27.1.5.2

When the specified short-period spectral acceleration ratio ($I_EF_aS_a(0.2)$) is greater than 0.55, rolled shapes with flanges 40 mm or thicker, or plates and built-up shapes over 51 mm in thickness, used in energy-dissipating elements or welded parts, shall have a minimum average Charpy V-notch impact test value of 27 J at 20 °C, unless it can be demonstrated that tensile stresses, including local effects, are not critical. The impact tests shall be conducted in accordance with CSA G40.21, with the following exceptions:

a) the central longitudinal axis of the test specimens in rolled shapes shall be located as near as practicable to midway between the inner flange surface and the centre of the flange thickness at the intersection with the web mid-thickness; and
b) one impact test sample shall be taken from each 15 tonnes or less of shapes produced from each heat, or from each ingot for shapes rolled from ingots.

27.1.5.3

This Clause applies to welds in primary members and connections where the specified short-period spectral acceleration ratio ($I_EF_aS_a(0.2)$) is greater than 0.35.

All welds shall be made with filler metals that have a minimum average Charpy V-notch impact test value of 27 J at a test temperature equal to or lower than −18 °C as certified in accordance with CSA W48 or a manufacturer's certificate of conformance.

In addition, demand critical welds as designated below shall be made with filler metals that have a minimum average Charpy V-notch impact test value of 54 J at +20 °C, except that where the structure in service is exposed to temperatures lower than +10 °C, the maximum testing temperature shall be 20 °C above the 2.5% January design temperature as defined in Appendix C, Division B of the *NBCC*. Demand critical welds shall include

a) groove welds in column splices;
b) welds at column-to-base plate connections when plastic hinging or net section fracture in tension is expected at the column bases;
c) except when Item e) applies, complete joint penetration groove welds joining beam flanges and beam webs to columns in moment connections for Type D and MD moment-resisting frames;
d) except when Item e) applies, complete joint penetration groove welds joining beam flanges to columns in moment connections for Type LD moment-resisting frames and Type D plate walls;
e) when moment connections are designed in accordance with the CISC *Moment Connections for Seismic Applications*, all demand critical welds designated therein;
f) welds joining link beam flanges and webs to columns in Type D eccentrically braced frames;
g) welds joining webs and flanges in built-up tubular link beams in Type D eccentrically braced frames; and
h) welds joining infill plates to perimeter frame members in Type D plate walls.

The requirements of this Clause may be waived when the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is less than or equal to 0.55 and the welds are loaded primarily in shear.

Note: *The maximum testing temperature for demand critical welds in structures exposed to low temperatures is based on a service temperature taken as* 10 °C *above the 2.5% January design temperature as defined in Appendix C, Division B of the* NBCC.

27.1.5.4

When T-joint or corner-joint details susceptible to through-thickness tensile stresses resulting from welding executed under conditions of restraint cannot be avoided, measures shall be taken to minimize the possibility of lamellar tearing in accordance with CSA W59.

① 27.1.6 Bolted connections

Bolted connections shall

a) have pretensioned high-strength bolts;
b) when designed as bearing-type connections, have surfaces of Class A or better, or provide the equivalent slip resistance by increasing the number of bolts, bolt size, bolt strength, or any combination thereof;
c) not be considered to share load with welds;
d) not have long slotted holes;
e) not have short slotted holes unless the load is normal to the slot; and
f) have end distances in the line of seismic force not less than two bolt diameters when the bearing force due to seismic load exceeds 75% of the bearing resistance (see Clause 13.12.1.2).

The requirements of this Clause may be waived when fastener and connection details conform to those of a tested assembly.

27.1.7 Probable yield stress

The probable yield stress shall be taken as R_yF_y. The value of R_y shall be taken as 1.1 and the product R_yF_y as not less than 460 MPa for HSS sections or 385 MPa for other sections, unless the probable yield stress, taken as an average yield stress, is obtained in accordance with CSA G40.20.

Width-to-thickness limits of energy-dissipating elements shall be based on F_y, with F_y taken as not less than 300 MPa for angles and 350 MPa for other sections.

27.1.8 Stability effects

27.1.8.1

The effects of notional loads and P-delta effects shall be taken into account when sizing the energy-dissipating elements or mechanisms of the seismic-force-resisting system. Notional loads and P-delta effects shall also be considered when determining the limiting forces corresponding to $R_dR_o = 1.3$. Notional loads and P-delta effects need not be considered when determining member forces induced by yielding of the energy-dissipating elements or mechanisms of the seismic-force-resisting system.

The notional loads shall be calculated in accordance with Clause 8.4.1.

27.1.8.2

When the provisions of the *User's Guide — NBC 2015: Structural Commentaries (Part 4)* are applied in calculating P-delta effects, the value of U_2 in Clause 8.4.2 may be taken as

$$U_2 = 1 + \left(\frac{\sum C_f R_d \Delta_f}{\sum V_f h} \right)$$

Structural stiffness shall be provided such that U_2 does not exceed 1.4.

27.1.9 Protected zones

Structural and other attachments that could introduce metallurgical notches or stress concentrations shall not be used in areas designated as protected zones unless engineered and forming part of the design system or forming part of a test assembly that satisfies the physical test requirements of Clauses 27.2.5.1,27.7.8.1, and 27.8.6. Discontinuities created by fabrication or erection operations shall be repaired.

Welded shear studs and decking attachments that penetrate the beam flange shall not be placed on the beam flanges within protected zones unless approved by the designer. Arc-spot welds necessary to secure decking to beam flanges may be used.

Protected zones shall be indicated on the structural design documents and shop details (see Clauses 4.2.2,4.3.2, and 4.3.3).

27.2 Type D (ductile) moment-resisting frames, R_d = 5.0, R_o = 1.5

27.2.1 General

① 27.2.1.1

Ductile moment-resisting frames can develop significant inelastic deformation through plastic hinging in beams a short distance from the face of columns. Plastic hinges in columns are permitted to develop only at the base and at the top of a continuous column stack.

Note: *Plastic deformation in joints is limited by Clause 27.2.4. See Clause 27.11.2 for cantilever column structures.*

27.2.1.2

Energy-dissipating elements shall be proportioned and braced to enable them to undergo large plastic deformations.

27.2.1.3

In Clauses 27.2.2 to 27.2.4, the effects of bearing of slabs on column flanges shall be considered in determining the flexural resistance of, and the loading produced by, composite beams.

① 27.2.2 Beams

Beams are expected to develop plastic hinges typically at a short distance from the face of columns (see Clause 27.2.5) and shall

a) be Class 1 sections; and
b) be laterally braced in accordance with Clause 13.7 b) unless alternative bracing is demonstrated as satisfactory in accordance with Clause 27.2.5.1. The value of κ shall be based on the bending moment distribution for combined gravity and seismic loads. The bending moments due to seismic load may be taken as varying linearly from a maximum at one end of the beam to zero at the other, unless another value can be justified.

The forces acting on other members and connections due to beam plastic hinging shall be calculated using $1.1R_y$ times the nominal flexural resistance, ZF_y, except when connections and associated design procedures referenced in Annex J are selected.

Beams need not meet the requirements of this Clause when plastic hinges are expected to develop near the top of columns instead of in the beams, as permitted in Clause 27.2.1. However, these beams shall meet the requirements of Clause 27.2.3.1 for non-dissipating elements adjacent to plastic hinges in columns.

27.2.3 Columns

27.2.3.1

Columns shall be Class 1 or 2. When a column is expected to develop plastic hinging, it shall be Class 1 and meet the following requirements:

a) the column shall be laterally braced in accordance with Clause 13.7 b), using κ = 0.0, unless other values of κ can be justified by analysis;
b) when the specified one-second spectral acceleration ratio ($I_EF_vS_a(1.0)$) is greater than 0.30, the factored axial load shall not exceed $0.30AF_y$ for all seismic load combinations; and
c) the column shall meet the requirements of Clause 27.2.8.

Non-dissipating structural elements adjacent to plastic hinges in columns shall be able to resist forces corresponding to $1.1R_y$ times the nominal flexural resistance of the columns. This nominal flexural resistance shall be taken as $1.18M_{pc}(1 - C_f/R_yC_y)$, but shall not be greater than the nominal plastic moment resistance of the column, M_{pc}, where C_f is as specified in Clause 27.2.3.2.

① **27.2.3.2**

Columns shall resist the gravity loads together with the forces induced by plastic hinging of the beams as projected at the column centrelines. The following shall apply at each beam-to-column intersection:

$$\sum M'_{rc} \geq \sum\left(1.1R_yM_{pb} + V_h\left(x + \frac{d_c}{2}\right)\right)$$

where

$\Sigma M'_{rc}$ = sum of the column factored flexural resistances projected at the intersection of the beam and column centrelines

and

$$M'_{rc} = 1.18\phi M_{pc}\left(1 - \frac{C_f}{\phi C_y}\right) \leq \phi M_{pc}$$

where

M_{pb} = nominal plastic moment resistance of the beam

V_h = shear acting at that beam plastic hinge location due to gravity loads on the beam plus moments equal to $1.1R_yM_{pb}$ at beam hinge locations

x = distance from the centre of a beam plastic hinge to the column face, which shall correspond to that of the assembly used to demonstrate performance in accordance with Clause 27.2.5.1

M_{pc} = nominal plastic moment resistance of the column

C_f = axial force from gravity loads plus the summation of V_h acting at and above the level under consideration

Columns need not meet the requirements of this Clause when plastic hinges are expected to develop near the top of columns instead of in the beams, as permitted in Clause 27.2.1.

27.2.3.3

When the axial force calculated in accordance with Clause 27.2.3.2 is tensile, column splices having partial-joint-penetration groove welds shall

a) be capable of resisting twice the calculated tensile force;

b) have flange connections that are each capable of resisting at least $0.5A_fR_yF_y$, where A_f is the flange area of the smaller column at the splice; and

c) be located at least one-fourth of the clear distance between beams but not less than 1 m from the beam-to-column joint.

① 27.2.4 Joint panel zone

① **27.2.4.1**

When plastic hinges form in adjacent beams, the panel zone shall resist forces arising from beam moments at the column faces of

$$\Sigma\left(1.1R_yM_{pb} + V_hx\right)$$

where the summation is for both beams at a joint, and M_{pb}, V_h, and x are as specified in Clause 27.2.3.2.

When plastic hinges are expected to develop near the top of columns instead of in the beams, as permitted in Clause 27.2.1, panel zones shall resist forces arising from moments corresponding to plastic hinge moments of $1.1R_y$ times the nominal flexural resistance of the column.

27.2.4.2

The horizontal shear resistance of the column joint panel zone shall be taken as either

a) $V_r = 0.55\phi d_c w' F_{yc}\left[1 + \frac{3\, b_c t_c^2}{d_c d_b w'}\right] \leq 0.66\phi d_c w' F_{yc}$; or

b) $V_r = 0.55\phi d_c w' F_{yc}$

where the subscripts b and c denote the beam and the column, respectively, and w' is the thickness of the column web plus the thickness of the doubler plates, when used.

27.2.4.3

The following requirements shall also apply:

a) Where the specified short-period spectral acceleration ratio ($I_E F_a S_a$ (0.2)) is equal to or greater than 0.55, and the joint panel zones are designed in accordance with Clause 27.2.4.2 a), the sum of panel zone depth and width divided by the panel zone thickness shall not exceed 90 and the effects of panel-zone deformations on frame stability shall be accounted for.

b) Joint panel zones designed in accordance with Clause 27.2.4.2 b) shall satisfy the width-to-thickness limit of Clause 13.4.1.1 a) i).

c) Doubler plates shall be groove- or fillet-welded to the column flanges to develop their full shear resistance.

d) When doubler plates are placed against the column web and continuity plates are used, the doubler plates shall be fillet welded to the continuity plates to develop the proportion of the total force transmitted to the doubler plate. When continuity plates are not used, the doubler plates shall extend above and below the level of the beam flanges and be fillet welded across the top and bottom edges to develop the proportion of the total force transmitted to the doubler plate.

e) When doubler plates are placed away from the column web, they shall be placed symmetrically in pairs and welded to continuity plates to develop the proportions of the total force transmitted to the doubler plate.

f) In calculating width-to-thickness ratios, doubler plate thickness may be included with web thickness only when the doubler plate is connected to the column web near the centre of the panel.

27.2.4.4

Other requirements may apply for the beam-to-column connections selected in accordance with Clause 27.2.5.1.

27.2.5 Beam-to-column joints and connections

27.2.5.1

The beam-to-column joint shall maintain a strength at the column face of at least the nominal plastic moment resistance of the beam, M_{pb}, through a minimum interstorey drift angle of 0.04 radians under cyclic loading. When reduced beam sections are used, or when local buckling limits the flexural strength of the beam, the beam need only achieve $0.8M_{pb}$ at the column face when an interstorey drift angle of 0.04 radians is developed under cyclic loading.

Beam-to-column connections shall satisfy the requirements in this Clause by one of the following:
a) use of connections designed and detailed in accordance with the CISC *Moment Connections for Seismic Applications*; or
b) demonstration of the connection performance through at least two physical qualifying cyclic connection tests as described and referenced in Annex J.

27.2.5.2
The factored resistance of the beam web-to-column connection shall equal or exceed the effects of gravity loads combined with shears induced by moments of $1.1R_yZF_y$ acting at plastic hinge locations. Other requirements may apply for the beam-to-column connections selected in accordance with Clause 27.2.5.1.

27.2.5.3
In single-storey buildings, when the column frames into the underside of the beam and plastic hinging is expected near the top of a column, the connection shall meet the requirements of Clause 27.2.5.1.

27.2.6 Bracing
The following bracing requirements shall apply:
a) Beams, columns, and beam-to-column joints shall be braced by members proportioned in accordance with Clause 9.2 where $C_f = 1.1R_yF_y$ times the cross-sectional area in compression. The possibility of complete load reversals shall be considered.
b) When plastic hinges occur in the beam, lateral bracing at the joints shall be provided at least at the level of one beam flange. If bracing is not provided at the level of both beam flanges, the transverse moments produced by the forces that would otherwise be resisted by the lateral bracing shall be included in the seismic load combinations. Attachments in the hinging area shall meet the requirements of Clause 27.2.8.
c) When plastic hinges occur near the top of the column, lateral bracing at the joints shall be provided at the level of both beam flanges.
d) When no lateral support can be provided to the joint at the level considered, the following shall apply:
   i) the column maximum slenderness ratio shall not exceed 60; and
   ii) transverse moments produced by the forces otherwise resisted by the lateral bracing shall be included in the seismic load combinations.

27.2.7 Fasteners
Fasteners connecting the separate elements of built-up flexural members shall have resistance adequate to support forces corresponding to moments of $1.1R_yZF_y$ at the plastic hinge locations.

27.2.8 Protected zones
The regions at each end of the beams subject to inelastic deformations and in columns where inelastic deformations are anticipated shall be designated as protected zones and meet the requirements of Clause 27.1.9.

The protected zone of the beams shall be defined as the area from the face of the column flange to one-half of the beam depth beyond the theoretical hinge point. Abrupt changes in beam flange cross-sections shall be avoided in protected zones, unless specially detailed reduced beam sections are provided that satisfy Clause 27.2.5. Bolt holes in beam webs, when detailed in accordance with the individual connection requirements of this Standard, may be used.

Where the theoretical hinge point falls at the base of the column, the protected zone of the columns shall be defined as the area from the face of the base plate to one-half of the column depth beyond the theoretical hinge point or the column depth, whichever is greater. Where the theoretical hinge point falls within the column below the beam, the protected zone of the columns shall be defined as the area from the underside of the beam to one-half of the column depth beyond the theoretical hinge point or the column depth, whichever is greater.

27.3 Type MD (moderately ductile) moment-resisting frames, $R_d = 3.5$, $R_o = 1.5$

Moderately ductile moment-resisting frames can develop a moderate amount of inelastic deformation through plastic hinging in the beams at a short distance from the face of columns. The requirements of Clause 27.2 shall apply to such frames, except that

a) with respect to Clause 27.2.2,
   i) the beams shall be Class 1 or 2 sections; and
   ii) the bracing shall meet the requirements of Clause 13.7 a);
b) with respect to Clause 27.2.3.1 b), the factored axial load shall not exceed $0.50AF_y$; and
c) with respect to Clause 27.2.5.1, the minimum interstorey drift angle shall be 0.03 radians.

27.4 Type LD (limited-ductility) moment-resisting frames, $R_d = 2.0$, $R_o = 1.3$

27.4.1 General

Limited-ductility moment-resisting frames can develop a limited amount of inelastic deformation through plastic hinging in the beams, columns, or joints. This system may be used in buildings

a) not exceeding 60 m in height where the specified short-period spectral acceleration ratio ($I_EF_aS_a$ (0.2)) is greater than or equal to 0.35 but less than or equal to 0.75; and
b) not exceeding 30 m in height where the specified short-period spectral acceleration ratio ($I_EF_aS_a$ (0.2)) is greater than 0.75 or where the specified one-second spectral acceleration ratio ($I_EF_vS_a$ (1.0)) is greater than 0.30.

27.4.2 Beams and columns

27.4.2.1

Beams shall be Class 1 or 2. Columns shall be Class 1. Except at roof level, beams shall frame into the columns.

27.4.2.2

When the specified short-period spectral acceleration ratio ($I_EF_aS_a$ (0.2)) is greater than 0.55 or the building is greater than 60 m in height, columns shall satisfy the requirements of Clause 27.2.3.2. However, when Clause 27.2.3.2 is applied, the term $1.1R_yM_{pb}$ may be replaced by R_yM_{pb} and columns may be Class 2. In addition, the beams shall be designed so that for each storey, the storey shear resistance is not less than that of the storey above.

27.4.3 Column joint panel zone

The horizontal shear resistance of the column joint panel zone shall be that specified in Clause 27.2.4.2.

27.4.4 Beam-to-column connections

27.4.4.1

The beam-to-column joints shall meet the requirements of Clause 27.2.5.1, except that the minimum interstorey drift angle shall be 0.02 radians.

Beam-to-column connections shall satisfy the requirements in this Clause by one of the following:

a) use of connections designed and detailed in accordance with Clause 27.4.4.2;
b) use of connections designed and detailed in accordance with the CISC *Moment Connections for Seismic Applications*; or
c) demonstration of the connection performance through at least two physical qualifying cyclic connection tests as described and referenced in Annex J.

① 27.4.4.2

With respect to Clause 27.4.4.1 a):

a) Columns shall be I-shaped sections.
b) The beam flanges shall be directly welded to the column flanges.
c) Beam-to-column connections shall have a moment resistance equal to R_yM_{pb}, except that, when the controlling limit state is ductile, the moment resistance need not exceed the effect of the gravity loads combined with the seismic load multiplied by 2.0.
d) Beam-to-column connections designed for a moment resistance of R_yM_{pb} shall have a welded web connection.
e) Weld backing bars and run-off tabs shall be removed and repaired with reinforcing fillet welds. Top-flange backing bars may remain in place if continuously fillet welded to the column flange on the edge below the complete joint penetration groove weld. Neither partial-joint-penetration groove welds nor fillet welds shall be used to resist tensile forces in the connections.
f) The tensile resistance of the column flange shall be taken as $0.6T_n$ as specified in Clause 21.3.
g) When columns frame under the beams, the roles of beam and column shall be reversed.

Note: *Beam-to-column connections with a welded web connection and complete-penetration groove welds made with matching electrodes in accordance with Clause 13.13.3.1 between the beam flanges and the column flanges are considered to have a moment-resistance equal to R_yM_{pb}.*

27.4.4.3

Beam-to-column connections shall resist shear forces resulting from the gravity load together with shears corresponding to the moments at the beam ends equal to those specified in Clause 27.4.4.2 c).

27.5 Type MD (moderately ductile) concentrically braced frames, R_d = 3.0, R_o = 1.3

27.5.1 General

Moderately ductile concentrically braced frames can dissipate moderate amounts of energy through yielding of bracing members.

27.5.2 Bracing systems

27.5.2.1 General

Moderately ductile concentrically braced frames include

a) tension-compression bracing systems (see Clause 27.5.2.3);
b) chevron braced systems (see Clause 27.5.2.4);

c) tension-only bracing systems (see Clause 27.5.2.5); and
d) other systems, provided that stable inelastic response can be demonstrated.

Knee bracing and K-bracing, including those systems in which pairs of braces meet a column on one side between floors, are not considered to be moderately ductile concentrically braced frames.

27.5.2.2 Proportioning

At all levels of any planar frame, the diagonal bracing members along any braced column line shall be proportioned in such a way that the ratio of the sum of the horizontal components of the factored tensile brace resistances in opposite directions is between 0.75 and 1.33.

27.5.2.3 Tension-compression bracing

Except where the specified short-period spectral acceleration ratio ($I_E F_a S_a$ (0.2)) is less than 0.35, tension-compression concentric bracing systems shall not exceed 40 m in height. In addition, when the height exceeds 32 m, the factored seismic forces for the ultimate limit states shall be increased by 3% per metre of height above 32 m.

Tension-compression bracing, in which pairs of braces meet a column at one or two points on one side between horizontal diaphragms, may be used provided that the columns meet the requirements of Clause 27.5.6.

27.5.2.4 Chevron bracing

Chevron bracing systems comprise pairs of braces, located either above or below a beam, that meet the beam at a single point within the middle half of the span. Chevron bracing systems shall meet the requirements of Clause 27.5.2.3.

The beams to which the chevron bracing is attached shall
a) be continuous between columns;
b) have both top and bottom flanges laterally braced at the brace connection; and
c) resist bending moments due to gravity loads (assuming no vertical support is provided by the bracing members) in conjunction with bending moments and axial forces induced by forces of T_u and C_u' in the tension and compression bracing members, respectively. In the case of buildings not exceeding four storeys, the tension brace force may be taken as $0.6T_u$, provided that the beam is a Class 1 section. When braces are connected to the beam from above, the case where the brace compression force is equal to C_u shall also be considered.

The beam-to-column connections shall resist the forces corresponding to the loading described in Item c) for beams. However, when the tension brace force is less than T_u, the connections shall resist the gravity loads combined with forces associated with the attainment of R_y times the nominal flexural resistance of the beam at the brace connection.

The lateral braces at the brace connection shall resist a transverse load of 0.02 times the beam flange yield force.

Note: *See Clause 27.5.3.4 for the probable tensile, compressive, and post-buckling compressive resistances of bracing members, T_u, C_u, and C'_u, respectively.*

① 27.5.2.5 Tension-only bracing

The braces in tension-only bracing systems are designed to resist, in tension, 100% of the seismic loads and are connected at beam-to-column intersections. In addition, except where the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is less than 0.35,

a) the structure shall not exceed 20 m in height and, when the height exceeds 16 m, the factored seismic forces for ultimate limit states shall be increased by 3% per metre of height above 16 m;
b) all columns are continuous and of constant cross-section over the building height; and
c) the column splices are proportioned for the full moment resistance of the cross-section and for a shear force of $2.0ZF_y/h_s$, where Z is the plastic modulus of the column and h_s is the storey height.

Although the braces are proportioned on the basis of tension loading only, this system shall meet the other requirements of Clause 27, including Clauses 27.5.3 to 27.5.7.

27.5.3 Diagonal bracing members

Note: *Where possible, at every storey, the two discontinuous bracing members in every X-bracing bay should be fabricated and installed from the same heat.*

27.5.3.1 Brace slenderness

The slenderness ratio, KL/r, of bracing members shall not exceed 200.

When the specified short-period spectral acceleration ratio ($I_E F_a S_a$ (0.2)) is equal to or greater than 0.75 or the specified 1 s spectral acceleration ratio ($I_E F_v S_a(1.0)$) is equal to or greater than 0.30, the slenderness ratio of HSS bracing members shall not be less than 70.

Note: *The effects of translational and rotational restraints at the brace ends or along the brace length should be accounted for in the calculation of KL.*

① 27.5.3.2 Width (diameter)-to-thickness ratios

When the specified short-period spectral acceleration ratios ($I_E F_a S_a$ (0.2)) are equal to or greater than 0.35, width-to-thickness ratios shall not exceed the following limits:

a) when $KL/r \leq 100$:
   i) for rectangular and square HSS: $330/\sqrt{F_y}$;
   ii) for circular HSS: $10\,000/F_y$;
   iii) for legs of angles and flanges of channels: $145/\sqrt{F_y}$; and
   iv) for other elements: Class 1;
b) when $KL/r = 200$
   i) for HSS members: Class 1;
   ii) for legs of angles: $170/\sqrt{F_y}$; and
   iii) for other elements: Class 2; and
c) when $100 < KL/r < 200$, linear interpolation may be used.

When the specified short-period acceleration ratio ($I_E F_a S_a$ (0.2)) is less than 0.35, HSS shall be Class 1 and all other sections shall be Class 1 or 2. The width-to-thickness ratio for legs of angles shall not exceed $170/\sqrt{F_y}$.

Back-to-back legs of double-angle bracing members for which buckling out of the plane of symmetry governs, the width-to-thickness ratio shall not exceed $200/\sqrt{F_y}$ irrespective of the specified short-period acceleration ratio ($I_E F_a S_a$ (0.2)).

27.5.3.3 Built-up bracing members

For buildings with specified short-period spectral acceleration ratios [$I_E F_a S_a$ (0.2)] equal to or greater than 0.35, the slenderness ratio of the individual parts of built-up bracing members, as defined in Clause 19.2.4, shall not be greater than 0.5 times the governing effective slenderness ratio of the member as a whole. If overall buckling of the brace does not induce shear in the stitch fasteners that connect the separate elements of built-up bracing members, the slenderness ratio of the individual parts shall not exceed 0.75 times the governing effective slenderness ratio of the member as a whole.

If overall buckling of the brace induces shear in the stitch fasteners, the stitch fasteners shall have a resistance adequate to support one-half of the yield load of the larger component being joined, with this force assumed to act at the centroid of the smaller member. Bolted stitch connections shall not be located in the anticipated plastic hinge regions of bracing members.

27.5.3.4 Probable brace resistances

For the purpose of evaluating forces on connections and other members upon yielding and buckling of the bracing members in capacity design, the probable tensile resistance of bracing members, T_u, shall be taken as equal to $A_gR_yF_y$; the probable compressive resistance of bracing members, C_u, shall be taken as equal to the lesser of $A_gR_yF_y$ and $1.2C_r/\phi$, where C_r is computed using R_yF_y ; and the probable post-buckling compressive resistance of bracing members, C_u', shall be taken as equal to the lesser of $0.2A_gR_yF_y$ and C_r/ϕ, where C_r is computed using R_yF_y.

Each of the two loading conditions,

a) the compression acting braces attaining their probable compressive resistance, C_u; and
b) the compression acting braces attaining their probable buckled resistance, C_u', shall be considered as occurring in conjunction with the tension acting braces developing their probable tensile resistance, T_u.

For chevron bracing, when plastic hinging in the beam is permitted by Clause 27.5.2.4 c) or 27.6.2.2, the brace tensile force need not exceed the greater of that corresponding to plastic hinging in the beam and that corresponding to C_u of the compression brace.

When the forces corresponding to $R_dR_o = 1.3$ are computed, the redistribution of forces due to brace buckling shall be considered.

27.5.4 Brace connections

27.5.4.1 Eccentricities

Eccentricities in connections of braces to gusset plates or other supporting elements shall be minimized.

27.5.4.2 Resistance

The factored resistance of brace connections shall equal or exceed both the probable tensile resistance of the bracing members in tension, T_u, and the probable compressive resistance of the bracing members in compression, C_u, specified in Clause 27.5.3.4. For chevron bracing, the brace tension force may be reduced as specified in Clause 27.5.2.4.

The net section fracture resistance of the brace shall be adequate to resist the tension resistance, T_u. The net section factored resistance of the brace may be multiplied by R_y/ϕ_u, where R_y shall not exceed 1.2 for HSS and 1.1 for other shapes. This multiplier shall not be applied to the factored resistance of any cross-section reinforcement.

27.5.4.3 Ductile hinge rotation

Brace members or connections, including gusset plates, shall be detailed to provide ductile rotational behaviour, either in or out of the plane of the frame, depending on the governing effective brace slenderness ratio. When rotation is anticipated in the bracing member, the factored flexural resistance of the connections shall equal or exceed $1.1ZR_yF_y$ of the bracing member and the net section factored bending resistance of an unreinforced brace may be multiplied by R_y/ϕ. This requirement may be satisfied in the absence of axial load.

27.5.5 Columns, beams, and connections other than brace connections

27.5.5.1

The factored resistance of columns, beams, and connections other than brace connections shall equal or exceed the effects of gravity loads and the brace forces corresponding to the brace probable resistances specified in Clause 27.5.3.4. For chevron bracing, the beams shall be designed in accordance with Clause 27.5.2.4 and the brace tension force may be reduced as specified in Clause 27.5.3.4.

27.5.5.2

Columns in multi-storey buildings using the systems specified in Items a) to c) of Clause 27.5.2.1 shall be continuous and of constant cross-section over a minimum of two storeys, except as required by Clause 27.5.2.5.

Columns outside of the braced bays shall meet the requirements of Class 1, 2, or 3 flexural members.

Columns in braced bays shall meet the requirements of Class 1 or 2 beam-columns. Column resistances in the braced bays shall satisfy the requirements of Clause 13.8 including an additional bending moment in the direction of the braced bay of $0.2ZF_y$ in combination with the computed bending moments and axial loads. Splices in columns in braced bays shall be designed to provide the required axial, shear, and flexural resistances including the effects of the additional bending moments in the direction of the braced bays of $0.2ZF_y$ acting either in the same or the opposite directions at the column ends.

27.5.5.3

Partial-joint-penetration groove weld splices in columns subject to tension shall meet the requirements of Items a) and b) of Clause 27.2.3.3.

27.5.6 Columns with braces intersecting between horizontal diaphragms

27.5.6.1

Columns with braces intersecting at one or two points between horizontal diaphragms may be used provided that they also satisfy the requirements of this Clause.

27.5.6.2

Columns shall resist the simultaneous effects of

a) the gravity loads;
b) the axial loads, shear forces, and bending moments induced by yielding and buckling of the bracing members at the design storey drift as obtained from non-linear incremental analysis, assuming that yielding develops in the tension-acting bracing members located at any one level along the height of the storey; and
c) an out-of-plane transverse load at each brace-to-column connection equal to 2% of the factored axial compression load in the columns below the connection.

27.5.6.3

Horizontal struts shall be provided between columns at the brace-to-column connection levels in the plane of the bracing bents for transferring loads between tension-acting braces along the height of the storey assuming that the compression-acting braces attain their probable post-buckling resistance.

27.5.7 Protected zones

The protected zone of bracing members shall

a) be designated to include the full brace length;
b) be designated to include elements that connect braces to beams and columns; and
c) meet the requirements of Clause 27.1.9.

Splices shall not be used in bracing members.

27.6 Type LD (limited-ductility) concentrically braced frames, R_d = 2.0, R_o = 1.3

27.6.1 General

Concentrically braced frames of limited ductility can dissipate limited amounts of energy through yielding of bracing members. The requirements of Clause 27.5 shall be met, except as modified by Clauses 27.6.2 to 27.6.6.

27.6.2 Bracing systems

① 27.6.2.1 Tension-compression bracing

Except where the specified short-period spectral acceleration ratio ($I_EF_aS_a(0.2)$) is less than 0.35, tension-compression concentric bracing systems shall not exceed 60 m in height. In addition, when the height exceeds 48 m, the factored seismic forces for ultimate limit states shall be increased by 2% per metre of height above 48 m.

Tension-compression bracing, in which pairs of braces meet a column on one side between floors, may be used in limited-ductility concentrically braced frames provided that the columns meet the requirements of Clause 27.6.6.

① 27.6.2.2 Chevron bracing

Except where the specified short-period spectral acceleration ratio ($I_EF_aS_a$ (0.2)) is less than 0.35, tension-compression concentric bracing systems shall not exceed 60 m in height. In addition, when the height exceeds 48 m, the factored seismic forces for ultimate limit states shall be increased by 2% per metre of height above 48 m.

Beams in chevron bracing of 20 m or less in height need not meet the requirements of Clause 27.5.2.4 c) provided that the beams and the beam-to-column connections are proportioned to resist the forces that develop when buckling of the compression brace occurs and provided that when the braces are connected to the beam from below, the beam is a Class 1 section and has adequate nominal resistance to support the tributary gravity loads assuming no vertical support is provided by the bracing members.

27.6.2.3 Tension-only bracing

Except where the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is less than 0.35, tension-only systems shall

a) not exceed 40 m in height and, when the height exceeds 32 m, the factored seismic forces for ultimate limit states shall be increased by 3% per metre of height above 32 m; and
b) in multi-storey structures, have all columns fully continuous and of constant cross-section over a minimum of two storeys.

27.6.3 Diagonal bracing members

27.6.3.1

In single- and two-storey structures, the slenderness ratio of bracing members connected and designed in accordance with Clause 27.5.2.5 shall not exceed 300.

27.6.3.2

The requirements of Clause 27.5.3.2 may be modified as follows:

a) when the brace slenderness ratio exceeds 200 (as permitted by Clause 27.6.3.1), the width-to-thickness limits of Clause 27.5.3.2 need not apply; and
b) for buildings less than 40 m in height and with specified short-period spectral acceleration ratios ($I_E F_a S_a$ (0.2)) less than 0.45, braces need not be more compact than Class 2. The width-to-thickness ratio of the legs of angles shall not exceed $170 / \sqrt{F_y}$.

27.6.4 Bracing connections

The requirements of Clause 27.5.4.3 shall not apply to buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) less than 0.55 if the brace slenderness ratio is greater than 100.

① 27.6.5 Columns, beams, and other connections

For buildings with specified one-second spectral acceleration ratios ($I_E F_v S_a(1.0)$) not greater than 0.30, the design forces for column splices in Clause 27.1.4 need not be taken into account.

27.6.6 Columns with braces intersecting between horizontal diaphragms

Columns with braces intersecting at 4 points or less between horizontal diaphragms may be used provided that they meet the requirements of Clause 27.5.6.

27.7 Type D (ductile) eccentrically braced frames, $R_d = 4.0$, $R_o = 1.5$

27.7.1 General

Ductile eccentrically braced frames can dissipate energy by yielding of links.

27.7.2 Link beam

27.7.2.1

The link beam shall contain a segment (the link) designed to yield, either in flexure or in shear, prior to yielding of other parts of the eccentrically braced frame.

27.7.2.2
The link beam shall be either
a) a segment of the beam, for beams with an I-section or a built-up tubular rectangular cross-section; or
b) a modular link distinct from the rest of the beam. A modular link shall be either
   i) an end-plate connected link fabricated from a I-shaped section connected to the beam with unstiffened end-plate moment connections; or
   ii) a web connected link consisting of a built-up cross-section made of two C-sections connected back-to-back to the beam web, where the C-sections are channels or wide-flange cross-sections with the flanges cut flush with the web on one side.

27.7.2.3
A link shall be provided at least at one end of each brace. A link shall not be required in roof beams of frames over five storeys in height.

① 27.7.2.4
Link beams shall be Class 1 and designed for the coexisting shears, bending moments, and axial forces. Link beams may have Class 2 flanges and Class 1 webs when $e \le 1.6M_p/V_p$, where e is the length of the link and $V_p = 0.55wdF_y$, for links with wide-flange cross-sections, or $0.55(2w)dF_y$, for links with built-up tubular cross-sections and links with back-to-back C-sections.

27.7.2.5
The web or webs of the link shall be of uniform depth and have no penetrations, splices, attachments, reinforcement, or doubler plates, other than the stiffeners required by Clause 27.7.6.

For links with built-up tubular rectangular cross-sections, complete-joint-penetration groove welds shall be used to connect the webs to the flanges. Inaccessible backing bars need not be removed in these joints.

27.7.2.6
Flanges of built-up tubular links shall satisfy $b / t \le 285 / \sqrt{F_y}$, where b is the clear flange width. Webs shall satisfy $h / w \le 750 / \sqrt{F_y}$. The moment of inertia of built-up tubular links associated to horizontal, out-of-plane bending shall not be less than 0.67 times the link moment of inertia associated to bending in the vertical plane.

27.7.2.7
For web connected modular links, the flanges of the two C-sections shall be interconnected at both flange levels such that the clear longitudinal spacing between interconnections does not exceed 2.0 times the width of the flange of the individual C-sections.

When plates are used to reinforce the flanges of the C-sections in web connected modular links,
a) the flange reinforcement plates shall be continuously welded along their two longitudinal edges over the full length of the C-sections; and
b) the reinforced flanges shall satisfy Class 1 limit for flanges of I-sections in Table 2, where b_{el} is taken as the average of the C-section flange width and the flange reinforcement plate width and t is taken as the thickness of an equivalent flange having a moment of inertia for bending in the plane of the frame equal to that of the reinforced flange.

27.7.3 Link resistance

27.7.3.1 Factored link resistance

The factored shear resistance of the link shall be taken as the lesser of

$\phi V_p'$ and $2\phi M_p'/e$

where

$$V_p' = V_p\sqrt{1 - \left(\frac{P_f}{AF_y}\right)^2}$$

where

V_p = $0.55wdF_y$ for links with wide-flange cross-sections

= $0.55(2w)dF_y$ for links with built-up tubular cross-sections and modular links with back-to-back C-sections

P_f = axial force in the link

= C_f or T_f

A = gross area of the link beam

$$M_p' = 1.18M_p\left(1 - \frac{P_f}{AF_y}\right) \le M_p$$

e = length of the link (see Clause 27.7.4)

27.7.3.2 Probable link resistance

The nominal shear resistance of the link shall be taken equal to the lesser of V_p' and $2M_p'/e$, as defined in Clause 27.7.3.1, except that when P_f is equal to T_f, V_p' is given by

$$V_p' = V_p\sqrt{1 + \left(\frac{P_f}{AF_y}\right)^2}$$

The probable shear resistance of the link shall be taken equal to 1.3 R_y times the nominal link resistance except for links with built-up tubular cross-sections for which the probable shear resistance of the link shall be taken equal to 1.45 R_y times the nominal link resistance.

27.7.4 Link length

① 27.7.4.1

For end-plate connected modular links, the length of the link e shall be taken as the distance between the end plates. For web connected modular links, the length of the link e shall be taken as the distance between the innermost rows of bolts or vertical welds of the web connections.

For links that consist of a segment of the beams, the length of the link e shall be taken as the clear distance between the ends of two braces. When a link is directly connected to a column, the link length is measured from the column face or from the link-to-column connection reinforcement.

27.7.4.2
The link length shall be not less than the depth of the link beam. When $P_f/(AF_y) > 0.15$, the link length shall be as follows:

a) when $\frac{A_w}{A} \geq 0.3\frac{V_f}{P_f}$:

$$e \leq \left[1.15 - 0.5\frac{P_f}{V_f}\frac{A_w}{A}\right]\left(\frac{1.6M_p}{V_p}\right)$$

b) when $\frac{A_w}{A} < 0.3\frac{V_f}{P_f}$:

$$e \leq \frac{1.6M_p}{V_p}$$

where

A_w = area of web

= $(d - 2t)w$ for links with wide-flange cross-sections

= $(d-2t)(2w)$ for links with tubular cross-sections and modular links with back-to-back C-sections

27.7.4.3
The length of modular links shall also be as follows:

$$e \leq \frac{1.6M_p}{V_p}$$

27.7.5 Inelastic link rotation
The inelastic component of the rotation of the link segment relative to the rest of the beam, the inelastic link rotation, taken as the rotation associated to an inelastic drift equal to three times the elastic drift determined under factored seismic loading, Δf, shall not exceed the following limits:

a) when $e \leq 1.6M_p/V_p$: 0.08 radians;

b) when $e \geq 2.6M_p/V_p$: 0.02 radians; and

c) when $1.6M_p/V_p < e < 2.6M_p/V_p$,linear interpolation may be used.

27.7.6 Link stiffeners

27.7.6.1 Links with wide-flange cross-sections

27.7.6.1.1
Full-depth web stiffeners shall be provided on both sides of the beam web at the ends of the link. The stiffeners shall have a combined width of not less than $b - 2w$ and a thickness of not less than $0.75w$ or 10 mm, whichever is larger.

27.7.6.1.2
Intermediate link web stiffeners shall be full depth and when

a) $e \leq 1.6M_p/V_p$, spaced at intervals not exceeding $(30w - 0.2d)$ when the inelastic link rotation is 0.08 radians or $(52w - 0.2d)$ when the inelastic link rotation is 0.02 radians or less (for intermediate inelastic link rotations, spacing shall be determined by linear interpolation);

b) $2.6M_p/V_p < e < 5M_p/V_p$, placed at a distance of $1.5b$ from each end of the link;

c) $1.6M_p/V_p < e < 2.6M_p/V_p$, provided as in Items a) and b); and
d) $e \geq 5M_p/V_p$, are not required.

27.7.6.1.3

Intermediate web stiffeners shall be required on only one side of the web for link beams less than 650 mm in depth and on both sides of the web for beams 650 mm or greater in depth. One-sided stiffeners shall have a thickness of not less than w or 10 mm, whichever is larger, and a width of not less than $0.5(b - 2w)$.

27.7.6.1.4

Fillet welds connecting stiffeners to the beam web shall be continuous and develop a stiffener force of A_sF_y. The welds shall be terminated a distance of five times the link web thickness from the transition radius between the web and the flanges of the link.

Fillet welds connecting intermediate stiffeners to the flanges shall develop a force of $0.50A_sF_y$. Welds connecting the stiffeners at the link ends to the flanges shall develop a force of A_sF_y.

27.7.6.2 Links with built-up tubular cross-sections

27.7.6.2.1

Full-depth web stiffeners shall be provided on one side of each link web at the diagonal brace connection. These stiffeners shall have a combined width not less than $(b - 2w)$ and a thickness not less than $0.75w$ or 13 mm, whichever is larger.

① 27.7.6.2.2

Intermediate link web stiffeners shall be full depth and when
a) $e \leq 1.6M_p/V_p$ and $0.64\ (E/F_y)^{0.5} < h/w \leq 1.67\ (E/F_y)^{0.5}$ spaced at intervals not exceeding $20w-(d-2t)/8$, on one side of each web; and
b) $h/w \leq 0.64\ (E/F_y)^{0.5}$, are not required.

27.7.6.2.3

Fillet welds connecting stiffeners to the beam web shall be continuous and develop a stiffener force of A_sF_y.

27.7.6.3 Modular links

27.7.6.3.1

Link stiffeners of modular links shall be designed and detailed in accordance with Clause 27.7.6.1, except that
a) For end-plate modular links, the end-plates shall be considered as end stiffeners.
b) For web connected modular links, end stiffeners shall be provided on the exterior side of each C-section and have a width of not less than $0.5(b - 2w)$ and a thickness of not less than $0.75w$ or 10 mm, whichever is larger. End stiffeners shall be located at the location of the inner vertical weld for welded web connections and at a distance of 1.5 d inside of the innermost row of bolts for bolted web connections.
c) For web connected modular links, intermediate web stiffeners shall be provided on the exterior side of each C-section and shall have a width of not less than $0.5(b - 2w)$ and a thickness of not less than $0.75w$ or 10 mm, whichever is larger.

27.7.7 Lateral support for link

Except for links with built-up tubular rectangular cross-sections for which lateral bracing is not required, lateral support shall be provided to both top and bottom flanges at the ends of a link. These lateral supports shall have factored resistance equal to at least $0.06btR_yF_y$.

27.7.8 Link beam-to-column connection

27.7.8.1

When a link is directly connected to a column, the link-to-column connection shall be demonstrated by physical tests as being capable of undergoing cyclic inelastic rotation equal to at least 1.2 times the inelastic component of the rotation as specified in Clause 27.7.5.

Note: *Physical testing procedures to be used to demonstrate the required behaviour are referenced in Annex J.*

27.7.8.2

The demonstration of performance required by Clause 27.7.8.1 may be waived when

a) a link is separated from a column by a short distance in which the beam is reinforced to ensure elastic behaviour of the connection and the beam within this length remains elastic under the forces corresponding to the probable resistance of the link (see Clause 27.7.3.2);

b) the link length does not exceed $1.6M_p/V_p$; and

c) full-depth web stiffeners are provided at the end of the reinforced section.

27.7.8.3

Except for connections designed in accordance with Clauses 27.7.8.1 and 27.7.8.2, link beam-to-column connections may be designed for shear and torsion only. The factored torsional resistance shall equal or exceed $0.02btdF_y$.

27.7.9 Beam outside the link

27.7.9.1

The beam outside the link shall be Class 1 or 2.

27.7.9.2

The beam outside the link shall resist forces corresponding to the probable resistance of the link (see Clause 27.7.3.2). When subject to these forces, the beam resistance may be taken as the factored resistance multiplied by R_y/ϕ when the link and the beam outside the link are part of the same beam piece.

27.7.9.3

The beam outside of the link shall be provided with sufficient lateral support to maintain stability of the beam under the forces defined in Clause 27.7.9.2. If yielding is anticipated at the link end of this outer beam segment, bracing shall be provided in accordance with Clause 13.7 a). Lateral bracing shall be provided to both top and bottom flanges and have factored resistances at least equal to $0.02btR_yF_y$.

27.7.9.4

When welded shear studs are used to transfer horizontal seismic loads from a concrete slab to the beam, shear studs shall not be placed within a distance from the link end equal to

a) four times the overall slab deck thickness for solid slabs or for ribbed slabs with ribs parallel to the beam; or

b) two times the spacing of the ribs for ribbed slabs with ribs perpendicular to the beam.

27.7.10 Modular link-to-beam connections

27.7.10.1
Modular link-to-beam connection shall be demonstrated by physical tests as being capable of undergoing cyclic inelastic rotation equal to at least 1.2 times the inelastic rotation specified in Clause 27.7.5.

Note: *Physical testing procedures to be used to demonstrate the required behaviour are referenced in Annex J.*

27.7.10.2
Modular link-to-beam connections designed to resist forces corresponding to the probable resistance of the link (see Clause 27.7.3.2) may be considered as achieving the requirements of Clause 27.7.10.1 provided that they satisfy Clause 27.7.10.3 or 27.7.10.4, as applicable.

27.7.10.3 End-plate connected modular links
End connections shall be bolted unstiffened end-plate moment connections designed and detailed in accordance with the CISC *Moment Connections for Seismic Applications*. The depth limits for the beam in this publication do not apply. Effects of axial forces shall be included in the design procedure.

27.7.10.4 Web connected modular links
Webs of the back-to-back C-sections shall be connected to the web of the beam by means of either welded or bolted connections. The connections shall be designed in accordance with established design procedures for eccentrically loaded connections that account for the load-deformation response of the welds or bolts, as applicable, and the eccentricity of the load with respect to the instantaneous centre of rotation. When doubler plates are used for the webs of the C-sections, the doubler plates shall not extend into the link length.

27.7.11 Diagonal braces

27.7.11.1
Diagonal brace sections shall be Class 1 or 2.

27.7.11.2
Each diagonal brace and its end connections shall have a factored resistance to support axial force and moment produced by the link developing its probable resistance (see Clause 27.7.3.2).

27.7.12 Brace-to-beam connection
No part of the brace-to-beam connection shall extend into the link. The intersection of the brace and beam centrelines shall be at the end of or within the link. If the brace is designed to resist a portion of the link end moment, full end restraint shall be provided. The beam shall not be spliced within or adjacent to the connection between beam and brace.

27.7.13 Columns

27.7.13.1
Column sections shall be Class 1 or 2.

27.7.13.2

Columns shall be designed to resist the cumulative effect of yielding links together with the gravity loads. The link forces shall be taken as the probable resistances of the links except in storeys below the top two storeys, where the link forces may be taken as 0.90 times the probable resistance of the links.

Column sections in braced bays shall be Class 1 or 2. Column resistances in the braced bays shall satisfy the requirements of Clause 13.8 assuming an additional bending moment in the direction of the braced bay of $0.2ZF_y$ in combination with the computed bending moments and axial loads. In the top two storeys, the additional bending moment shall be taken as equal to $0.4ZF_y$.

27.7.13.3

Splices in columns in braced bays shall be designed to provide the required axial, shear, and flexural resistances including the effects of the additional bending moments in the direction of the braced bays of $0.2ZF_y$ acting either in the same or the opposite directions at the column ends.

Splices that incorporate partial-joint-penetration groove welds shall be located at least one-fourth of the clear distance between beams but not less than 1 m from the beam-to-column joints. When tension occurs in columns due to the link-induced forces, column splices having partial-joint-penetration groove welds shall be designed in accordance with Items a) and b) of Clause 27.2.3.3.

27.7.14 Protected zone

Link beams shall be designated as a protected zone. The protected zone shall extend to one-half of the depth of the beam beyond the ends of the link beams. Welding on link beams may be used for attachment of link stiffeners. The protected zone shall meet the requirements of Clause 27.1.9.

27.8 Type D (ductile) buckling restrained braced frames, $R_d = 4.0$, $R_o = 1.2$

27.8.1 General

Ductile buckling restrained braced frames can develop significant inelastic deformation through axial yielding in tension and compression of the core of the buckling restrained bracing members.

27.8.2 Bracing systems

Knee bracing and K-bracing, including systems in which pairs of braces meet a column on one side between floors, shall not be considered to be buckling-restrained braced frames.

Except where the specified short-period spectral acceleration ratio ($I_EF_aS_a(0.2)$) is less than 0.35, buckling restrained braced frames shall not exceed 40 m in height unless stable inelastic response can be demonstrated.

27.8.3 Bracing members

27.8.3.1

The braces shall consist of a structural steel core and a system that restrains the steel core from buckling. The steel core shall be designed to resist the entire axial force in the brace. The factored axial tensile and compression resistances (T_r and C_r, respectively) of the steel core to be used for design of the core shall be taken as follows:

$T_r = C_r = \phi A_{sc}F_{ysc}$

where

A_{sc} = cross-sectional area of the yielding segment of the steel core

F_{ysc} = specified minimum yield strength of the steel core, or actual yield strength of the steel core determined from the average of 2 coupon tests. The coupons shall be taken from the actual plate that the steel core is fabricated from. The long axis of the coupons shall be parallel to the long axis of the core. The coupons shall be tested in accordance with CSA G40.20

27.8.3.2

Splices shall not be used in the steel core. Plates used in the steel core that are 50 mm thick or greater shall satisfy the minimum notch toughness requirements of Clause 27.1.5.

27.8.3.3

The buckling restraining system shall be able to resist, without buckling, the forces and deformations that will develop in the brace at deformations corresponding to 2.0 times the seismic design storey drift.

27.8.3.4

The probable tensile, T_{ysc}, and compressive, C_{ysc}, resistances of the bracing members, including strain hardening, friction, and other effects, shall be taken as follows:

$$T_{ysc} = \omega A_{sc} R_y F_{ysc}$$

$$C_{ysc} = \beta\omega A_{sc} R_y F_{ysc}$$

where

ω = a strain hardening adjustment factor obtained by dividing the maximum tension force developed in the buckling restrained brace in the qualification testing specified in Clause 27.8.6, up to a deformation corresponding to 2.0 times the seismic design storey drift, by $A_{sc}R_yF_{ysc}$

β = a friction adjustment factor obtained by dividing the maximum compression force developed in the buckling restrained brace in the qualification testing specified in Clause 27.8.6, up to a deformation corresponding to 2.0 times the seismic design storey drift, by $\omega A_{sc}R_yF_{ysc}$

R_y may be taken as equal to 1.0 if F_{ysc} is determined from a coupon test as part of the qualification testing specified in Clause 27.8.6.

27.8.4 Brace connections

The factored resistance of brace connections shall equal or exceed the probable tensile and compressive resistances of the bracing members.

The design of connections shall include consideration of local and overall buckling and shall be consistent with the bracing forces and details considered in the qualification testing required by Clause 27.8.6.

27.8.5 Beams, columns, and connections other than brace connections

27.8.5.1

The factored resistance of beams, columns, and connections other than brace connections shall equal or exceed the effect of gravity forces and the brace connection forces specified in Clause 27.8.4, assuming the redistribution of loads when the bracing members develop their probable tensile and compressive resistances.

27.8.5.2
Columns in multi-storey buildings shall be continuous and of constant cross-section over a minimum of two storeys.

Column sections outside of the braced bays shall be Class 1, 2, or 3.

Column sections in braced bays shall be Class 1 or 2. Column resistances in the braced bays shall satisfy the requirements of Clause 13.8 assuming an additional bending moment in the direction of the braced bay of $0.2ZF_y$ in combination with the computed bending moments and axial loads. Splices in columns in braced bays shall be designed to provide the required axial, shear, and flexural resistances including the effects of the additional bending moments in the direction of the braced bays of $0.2ZF_y$ acting either in the same or the opposite directions at the column ends.

27.8.5.3
Partial-joint-penetration groove weld splices in columns subject to tension shall meet the requirements of Items a) and b) of Clause 27.2.3.3.

27.8.6 Testing
Individual buckling restrained brace members and buckling restrained braced frames shall be able to develop their resistance without buckling and with positive strain hardening up to deformations corresponding to 2.0 times the seismic design storey drift and shall exhibit values of ω and β greater than 1.0. Satisfaction of these requirements shall be demonstrated by physical testing as described in Annex J. Qualifying test results shall consist of at least two successful cyclic tests, one a test of a brace subassemblage (including brace connection rotational demands at the specified performance) and the other a uniaxial or subassembly test. Both requirements may be based on

a) tests reported in research or documented tests performed for other projects; or
b) tests conducted specifically for the project.

27.8.7 Protected zone
The steel core of bracing members and the elements that connect the steel core to beams and columns shall be designated as protected zones and shall meet the requirements of Clause 27.1.9.

27.9 Type D (ductile) plate walls, R_d = 5.0, R_o = 1.6

27.9.1 General
Ductile plate walls are composed of infill plates framed by rigidly connected columns and beams. They can develop significant inelastic deformation by the yielding of the infill plates and plastic hinging in beams a short distance from the face of columns. Plastic hinges in columns shall be allowed only at the base and shear yielding in columns shall be prevented.

The requirements of Clause 20 shall apply unless otherwise specified by Clause 27.9.

27.9.2 Infill plates

27.9.2.1 Shear resistance
The infill plate shall be designed to resist 100% of the applied factored storey shear force. The factored shear resistance of infill plates shall be taken as

$V_r = 0.4\phi F_y wL \sin 2\alpha$

27.9.2.2 Probable yield force

The forces acting on other members and connections due to yielding of the infill plates shall be calculated as R_y times the tension yield resistance of the infill plates, but these forces need not exceed the value corresponding to $R_dR_o = 1.3$.

27.9.2.3 Perforated infill plates

Unreinforced circular perforations may be located in infill plates provided that

a) the perforations are of equal diameter, D, and are regularly spaced vertically and horizontally over the entire area of the infill plates to form a regular grid of staggered holes to allow development of continuous diagonal tension fields at 45°;

b) the shortest centre-to-centre distance between the perforations, S_{diag}, is such that $D/S_{diag} \leq 0.6$;

c) the distance between the first holes and infill plate connections to the surrounding beams and columns is at least D, but does not exceed $(D + 0.7S_{diag})$; and

d) a minimum of four horizontal and four vertical lines of holes is used.

The factored shear resistance of infill plates with circular perforations shall be taken as

$$V_r = 0.4(1 - 0.7D/S_{diag})\phi F_y w L_i$$

27.9.2.4 Infill plates with corner cut-outs

Quarter-circular cut-outs may be located at the upper corners of the infill plates if

a) the infill plates are connected to a reinforcement arching plate that follows the edge of the cut-outs and are designed to allow development of the full strength of the solid infill plate;

b) the radius of the corner cut-outs is less than one-third of the infill plate clear height; and

c) beams and columns are designed to resist the compression or tension axial forces acting at the end of the arching reinforcement.

27.9.3 Beams

27.9.3.1

Beams shall be Class 1 sections braced in accordance with Clause 13.7 b).

27.9.3.2

Beams at every storey shall have sufficient flexural resistance such that at least 25% of the applied factored storey shear force is resisted by beams and columns forming a moment-resisting frame. Axial loads in beams and gravity load effects on beams need not be considered in calculating this resistance.

27.9.3.3

Beam resistances shall meet the requirements of Clause 13.8, considering the axial loads and bending moments induced by the gravity and lateral loads and the tension force in the infill plate determined in accordance with Clause 27.9.2.2. The effects of the tension force in the infill plate acting on the beams and the columns shall be considered in the calculation of the beam axial loads.

27.9.4 Columns

27.9.4.1

Columns shall be Class 1 sections braced in accordance with Clause 13.7 b).

27.9.4.2
Columns shall resist the effects of gravity loads together with the axial loads, shear forces, and bending moments due to the tension forces in the infill plates as determined in accordance with Clause 27.9.2.2, as well as the forces induced by the beams as determined in accordance with Clause 27.9.7.2.

27.9.4.3
Column splices shall develop the full flexural resistance of the smaller section at the splice, together with the shear force consistent with plastic hinging at column ends, assuming double curvature. Splices shall be located as close as practicable to one-fourth of the storey height above the floor.

27.9.4.4
The columns shall be stiffened so that plastic hinging forms in the columns above the base plate or foundation beam.

27.9.5 Minimum stiffness for beams and columns
Beams and columns shall have sufficient flexural stiffness so that the entire infill plate is yielded at the design storey drift.

The requirement of this Clause may be satisfied by applying Clauses 20.5.1 and 20.5.2.

27.9.6 Column joint panel zones
The horizontal shear resistance of the column joint panel zone shall meet the requirements of Clauses 27.2.4.2 and 27.2.4.3.

27.9.7 Beam-to-column joints and connections

27.9.7.1
Beam-to-column joints and connections shall meet the requirements of Clause 27.4.4, except that the moment resistance in Clause 27.4.4.2 c) shall be taken equal to $1.1R_yM_{pb}$.

27.9.7.2
The factored resistance of the beam web-to-column connection shall equal or exceed the effects of gravity loads and tension forces in the infill plates, as determined in accordance with Clause 27.9.2.2, acting above and below the beams, combined with shears induced by moments of $1.1R_yM_{pb}$ acting at plastic hinge locations. The moments acting in the beam plastic hinges may be taken as $1.18(1.1R_yM_{pb})$ $(1-C_f/\phi C_y)$, where C_f is the beam axial load due to the tension forces in the infill plates and C_y is the axial yield resistance of the beam.

27.9.8 Protected zones
Infill plates, the region at each end of the beams subject to inelastic straining, and column bases where inelastic deformations are anticipated shall be designated as protected zones and shall meet the requirements of Clause 27.1.9. The protected zone of beams shall be defined as the area from the face of the column flange to one-half of the beam depth beyond the theoretical hinge point. Bolt holes in beam webs, when detailed in accordance with the individual connection requirements of this Standard, may be used.

27.10 Type LD (limited-ductility) plate walls, R_d = 2.0, R_o = 1.5

27.10.1 General

Limited-ductility plate walls are composed of infill plates framed by columns and beams that may be connected rigidly or by simple connections. They can develop limited inelastic deformation by the yielding of the infill plates and plastic hinging in the beams, columns, or joints. Except where the specified short-period spectral acceleration ratio ($I_EF_aS_a$ (0.2)) is less than 0.35, the height of the structure shall be limited to 60 m

The requirements of Clause 20 and Clause 27.9.8 apply unless otherwise specified by Clause 27.10.

27.10.2 Infill plates

27.10.2.1

The factored shear resistance of infill plates shall be determined in accordance with Clause 27.9.2.1 and the forces acting on other members and connections due to yielding of the infill plates shall be determined in accordance with Clause 27.9.2.2.

27.10.2.2

Infill plate splices shall be designed to resist forces determined in accordance with Clause 27.9.2.2 and proportioned so as not to inhibit the formation of a uniform tension field in the panel.

27.10.3 Beams

27.10.3.1

Beams shall be Class 1 or Class 2 sections braced in accordance with Clause 13.7 a).

27.10.3.2

Beams shall meet the requirements of Clause 27.9.3.3.

① 27.10.4 Columns

Clause 27.9.4 shall apply, except that in applying Clause 27.9.4.2 the forces induced by the beams shall be determined using Clause 27.10.6.2.

Where the specified one-second spectral acceleration ratio $I_EF_vS_a$ (1.0) is less than 0.30, the design forces for the column splices in Clause 27.1.4 need not be taken into account.

27.10.5 Column joint panel zones

If rigid beam-to-column connections are used, the horizontal shear resistance of the column joint panel zone shall meet the requirements of Clause 27.2.4.2.

27.10.6 Beam-to-column joints and connections

27.10.6.1

If rigid beam-to-column connections are used, they shall have a moment resistance equal to R_yM_{pb}.

27.10.6.2
The factored resistance of the beam web-to-column connection shall equal or exceed the effects of gravity loads and tension forces in the infill plates, as determined in accordance with Clause 27.9.2.2, acting above and below the beams. If rigid beam-to-column connections are used, the design forces shall include shears induced by moments of R_yM_{pb} acting at plastic hinge locations and these moments may be taken as $1.18(R_yM_{pb})(1-C_f/\phi C_y)$, where C_f and C_y are as defined in Clause 27.9.7.2.

27.11 Conventional construction, R_d = 1.5, R_o = 1.3

27.11.1
Structural systems in this category have some capacity to dissipate energy through localized yielding and friction that inherently exists in traditional design and construction practices. Except as otherwise specified in Clause 27.11, the requirements of Clauses 27.1 to 27.10 and 27.12 shall not apply to these systems.

Diaphragms and connections of primary framing members and diaphragms of the seismic-load-resisting system of steel-framed buildings with specified short-period spectral acceleration ratios ($I_E F_aS_a(0.2)$) greater than 0.45 designed to resist seismic loads based on a force reduction factor, R_d, of 1.5 shall be
a) proportioned so that the expected connection failure mode is ductile; or
b) designed to resist gravity loads combined with the seismic load multiplied by R_d.

The connection design load need not exceed the gross section strength of the members being joined, as determined using the probable yield stress R_yF_y.

27.11.2
Cantilever column structures composed of single or multiple beam-columns fixed at the base and pin-connected or free at the upper ends shall
a) have Class 1 section columns;
b) have U_2 not greater than 1.25; and
c) have base connections designed to resist a moment of $1.1R_y$ times the nominal flexural resistance of the column, but need not exceed the value corresponding to R_dR_o = 1.0.

27.11.3
When the specified short-period spectral acceleration ratio ($I_EF_aS_a(0.2)$) is greater than or equal to 0.35, seismic force resisting systems other than cantilever column structures as specified in Clause 27.11.2 and not part of an assembly occupancy building as specified in the *NBCC*, may exceed 15 m in height if
a) all factored seismic forces for ultimate limit states are increased linearly by 2% per metre of height above 15 m, without exceeding forces corresponding to R_dR_o = 1.3;
b) the height does not exceed 40 m when the specified short-period acceleration ratio ($I_EF_aS_a(0.2)$) is greater than 0.75 or the specified one-second spectral acceleration ratio ($I_EF_vS_a(1.0)$) is greater than 0.30;
c) the height does not exceed 60 m when the specified short-period spectral acceleration ratio ($I_EF_aS_a$ (0.2)) is greater than or equal to 0.35 but less than or equal to 0.75;
d) the seismic forces and deformations are determined using the Dynamic Analysis Procedure described in the *NBCC*;
e) the requirements of Clauses 27.1.3 to 27.1.8 are satisfied;
f) beams, columns, and I-shaped or HSS bracing members are Class 1 or Class 2 sections;

g) for bracing members with slenderness equal to or less than 200, the width-to-thickness ratios is less than $170 / \sqrt{F_y}$ for the legs of angles and flanges of channels and $670 / \sqrt{F_y}$ for the webs of channels;
h) the columns are designed to resist in compression the effects of gravity loads combined with 1.30 times the member factored seismic forces, where the seismic induced axial loads for columns that are part of two or more intersecting seismic-force-resisting systems are obtained from analysis of the structure independently in any two orthogonal directions for 100% of the earthquake loads applied in one direction plus 30% of the earthquake loads in the perpendicular direction;
i) connections are designed to resist the effects of gravity loads combined with 1.30 times the member factored seismic forces, without exceeding the gross section strength of the members being joined, as determined using the probable yield stress R_yF_y;
j) connections are designed and detailed such that the governing failure mode is ductile when the member gross section strength does not control the connection design loads;
k) the factored seismic forces for diaphragms are determined for forces corresponding to $R_oR_d = 1.3$; and
l) compression members of the seismic-force-resisting system that are intersected by bracing members at an unbraced location are designed for an additional out-of-plane transverse force equal to 10% of the axial load carried by the compression members at that intersection point.

27.12 Special seismic construction

Other framing systems and frames that incorporate special bracing, ductile truss segments, seismic isolation, or other energy-dissipating devices shall be designed on the basis of published research results or design guides, observed performance in past earthquakes, or special investigation. A level of safety and seismic performance comparable to that required by Clause 27 shall be provided.

28 Shop and field fabrication and coating

28.1 Cambering, curving, and straightening

Cambering, curving, and straightening may be done by mechanical means, local application of heat, or both. The temperature of heated areas as measured by approved methods shall not exceed the limits specified in CSA W59.

28.2 Thermal cutting

Thermal cutting shall be performed by guided machine where practicable. Thermally-cut edges shall meet the requirements of CSA W59. Re-entrant corners shall be free from notches and have the largest practicable radii, with a minimum radius of 14 mm.

28.3 Sheared or thermally cut edge finish

28.3.1

Planing or finishing of sheared or thermally cut edges of plates or shapes shall not be required unless noted on the drawings or included in a stipulated edge preparation for welding.

28.3.2

The use of sheared edges in the tension area shall be avoided in locations subject to plastic hinge rotation at factored loading. Sheared edges, if used, shall be finished smooth by grinding, chipping, or planing. The requirements of this Clause shall be noted on design drawings and on shop details where applicable.

28.3.3

All burrs over 2 mm in height shall be removed. Projections and burrs under 2 mm in height shall be removed

a) when needed for proper fit-up for welding; and
b) when they create a hazard during or after construction.

28.4 Fastener holes

28.4.1 Drilled and punched holes

Unless otherwise shown on design documents or as specified in Clause 22.3.5, holes

a) shall be made 2 mm larger than the nominal diameter of the fastener;
b) may be punched when the thickness of the material is not greater than the nominal fastener diameter plus 4 mm;
c) shall be either drilled from the solid or sub-punched or sub-drilled and reamed when the material is greater than the nominal fastener diameter plus 4 mm; and
d) shall be drilled in CSA G40.21-700Q or ASTM A514 steels more than 13 mm thick.

28.4.2 Holes at plastic hinges

In locations subject to plastic hinge rotation at factored loading, fastener holes in the tension area shall be either sub-punched and reamed or drilled full size. This requirement shall be noted on design drawings and shop details.

28.4.3 Thermally cut holes

Thermally cut holes produced by guided machine may be used in statically loaded structures if the actual hole size does not exceed the nominal hole size by more than 1 mm. Gouges not exceeding 1.5 mm deep may be permitted along edges of thermally cut slots. Manually cut fastener holes may be permitted only with the approval of the designer.

28.4.4 Alignment

Drifting done during assembly to align holes shall not distort the metal or enlarge holes. Holes in adjacent parts shall match well enough to permit easy entry of bolts. Holes, except oversize or slotted holes, may be enlarged to admit bolts by a moderate amount of reaming. However, gross mismatch of holes shall be cause for rejection.

28.5 Joints in contact bearing

Joints in compression that depend on contact bearing shall have the bearing surfaces prepared to a common plane by milling, sawing, or other suitable means. Surface roughness shall have a roughness height rating not exceeding 500 (12.5 µm), as specified in CSA B95, unless otherwise specified by the designer.

When shop assembled, such joints shall have at least 75% of the entire contact area in bearing. A separation not exceeding 0.5 mm shall be considered acceptable as bearing. The separation of any remaining portion shall not exceed 1 mm. A gap of up to 3 mm may be packed with non-tapered steel shims to meet the requirements of this Clause. Shims need not be other than mild steel, regardless of the grade of the main material.

28.6 Member tolerances

28.6.1
Structural members consisting primarily of a single rolled shape shall be straight within the tolerances allowed in CSA G40.20, except as specified in Clause 28.6.4.

28.6.2
Built-up bolted structural members shall be straight within the tolerances allowed for rolled wide-flange shapes in CSA G40.20, except as specified in Clause 28.6.4.

28.6.3
Dimensional tolerances of welded structural members shall be those specified in CSA W59, unless otherwise specified by the designer.

28.6.4
The out-of-straightness of fabricated compression members shall not exceed 0.001 of the axial length between points that are to be laterally supported.

28.6.5
Beams with bow within the straightness tolerance shall be fabricated so that, after erection, the bow due to rolling or fabrication shall be upward.

28.6.6
Completed members shall be free from twists, bends, and open joints. Sharp kinks or bends shall be cause for rejection.

28.6.7
A variation of 1 mm is permissible in the overall length of members with both ends finished for contact bearing.

28.6.8
Members without ends finished for contact bearing that are to be framed to other steel parts of the structure may have a variation from the detailed length not greater than 2 mm for members 10 m or less in length and not greater than 4 mm for members more than 10 m in length.

28.7 Cleaning, surface preparation, and shop coating

28.7.1 General
Steelwork need not be coated unless required by Clause 6.6 or otherwise specified by the designer.

28.7.2 Uncoated steel

28.7.2.1
Steelwork need not be cleaned of oil, grease, dirt, and other foreign matter unless encased in concrete or otherwise specified by the designer.

28.7.2.2
Steelwork to be encased in concrete need not be coated. Steelwork that is designed to act compositely with reinforced concrete and depends on natural bond for interconnection shall not be coated.

28.7.3 Coated steel

28.7.3.1 General
The requirements of the coating system, including surface preparation, minimum finished coating thickness, and coating or performance specifications, shall be specified to meet service conditions. The primer and subsequent coats shall be compatible. Coatings shall be applied thoroughly and evenly to dry, clean surfaces.

28.7.3.2 Surface preparation
Steelwork shall be cleaned of all loose mill scale, loose rust, weld slag and flux deposit, oil, grease, dirt, other foreign matter, and excessive weld spatter prior to application of the coating. When specified, special surface preparation prior to coating shall meet the requirements of SSPC SP 1; SSPC SP 2; SSPC SP 3; SSPC SP 5/NACE No. 1; SSPC SP 6/NACE No. 3; SSPC SP 7/NACE No. 4; SSPC SP 10/NACE No. 2; SSPC SP 11; SSPC SP 12; or SSPC SP 14, as applicable.

28.7.3.3 One-coat systems
Steelwork to be coated shall, at a minimum, be given a one-coat paint intended to withstand exposure to an essentially non-corrosive atmosphere for a period not exceeding six months in accordance with CISC/CPMA 1-73a, unless otherwise specified.

A one-coat shop primer intended to withstand exposure to an essentially non-corrosive atmosphere for a period not exceeding 12 months shall comply with CISC/CPMA 2-75, unless otherwise specified by the designer.

28.7.3.4 Inaccessible surfaces
Surfaces that will be inaccessible after assembly shall be cleaned or cleaned and coated, as necessary, prior to assembly. Inside surfaces of enclosed spaces that will be entirely sealed off from any external source of oxygen need not be coated.

28.7.3.5 Field coating
Unless otherwise specified by the designer, the cleaning of steelwork in preparation for field coating, touch-up of shop coat, spot-coating of field fasteners, and general field coating shall not be considered part of the erection work.

28.7.4 Special surfaces

28.7.4.1
Coated-faying surfaces in high-strength bolted slip-critical joints shall meet the requirements of Clause 23.3.

28.7.4.2
For members in compression, surfaces that are finished to bear shall be cleaned before assembly but shall not be coated unless otherwise specified by the designer.

28.7.4.3
Joints that are to be welded shall be kept free of all foreign matter, including paint, primer, or other coatings that could be detrimental to achieving a sound weldment.

28.7.5 Metallic zinc coatings

28.7.5.1
Material to be hot-dip galvanized shall comply with CAN/CSA-G164.

28.7.5.2
Material to be zinc metallized shall comply with CSA G189.

29 Erection

29.1 Temporary conditions

29.1.1 General
Suitable provisions shall be made in accordance with this Standard to ensure that an adequate margin of safety exists in the uncompleted structure and members during erection. (See also Clause 4.3.4.)

29.1.2 Temporary loads
Suitable provisions shall be made to ensure that the loads incurred during steel erection can be safely sustained for their duration and without permanent deformation or other damage to any member of the steel frame and other building components supported thereby.

Temporary loads can include but are not limited to loads due to wind, equipment, equipment operation, and storage of construction materials.

29.1.3 Temporary bracing
Temporary bracing shall be employed whenever necessary to withstand all loads to which the structure may be subject during steel erection. Temporary bracing shall be left in place undisturbed as long as necessary for the safety and integrity of the structure.

29.1.4 Adequacy of temporary connections
As erection progresses, the work shall be securely bolted or welded to resist safely all dead, wind, and erection loads and to provide necessary structural integrity.

29.2 Alignment
Permanent welding or bolting shall not be performed until as much of the structure as will be stiffened thereby has been suitably aligned.

29.3 Erection tolerances

29.3.1 General

The steel framework shall be erected true and plumb within the specified tolerances. The tolerances specified in Clauses 29.3.2 to 29.3.11 are the maximum allowable tolerances for a given member.

Note: *A member tolerance can be limited to less than the allowed tolerance due to a stricter tolerance controlling the member to which it is framed into or to a member that it supports.*

29.3.2 Elevation of base plates

Column base plates shall be considered to be at their proper elevation if the following tolerances are not exceeded:

a) for single- and multi-storey buildings designed as simple construction as specified in Clause 8.3: ±5 mm from the specified elevation; and
b) for single- and multi-storey buildings designed as continuous construction as specified in Clause 8.2 or as partially restrained construction as specified in Clause 8.4: ±3 mm from the specified elevation.

29.3.3 Plumbness of columns

Unless otherwise specified by the designer, columns shall be considered plumb if their verticality does not exceed the following tolerances:

a) for exterior columns of multi-storey buildings: 1/1000, but not more than 25 mm toward or 50 mm away from the building line in the first 20 storeys, plus 2 mm for each additional storey, up to a maximum of 50 mm toward or 75 mm away from the building line over the full height of the building;
b) for columns adjacent to elevator shafts: 1/1000, but not more than 25 mm in the first 20 storeys, plus 1 mm for each additional storey, up to a maximum of 50 mm over the full height of the elevator shaft; and
c) for all other columns: 1/500.

Column plumbness shall be measured from the actual column centreline at the base of the column to its centreline at the next adjacent storey. Deviation from straightness of the erected column shall meet the requirements of Clause 28.6.

29.3.4 Horizontal alignment of members

Unless otherwise specified by the designer, spandrel beams shall be considered aligned when the offset of one end relative to the other from the alignment shown on the drawings does not exceed $L/1000$. However, the offset need not be less than 3 mm and shall not exceed 6 mm.

Other members shall be considered aligned when the offset of one end relative to the other from the alignment shown on the drawings does not exceed $L/500$. However, the offset need not be less than 3 mm and shall not exceed 12 mm.

29.3.5 Elevations of members

The elevations of the ends of members shall be within 10 mm of the specified member elevation. Allowances shall be made for initial base elevation, column shortening, differential deflections, temperature effects, and other special conditions, but the maximum deviation from the specified slope shall not exceed $L/500$. The difference from the specified elevation between member ends that meet at a joint shall not exceed 6 mm.

29.3.6 Crane runway beams

Unless otherwise required by the operational characteristics of the crane, crane runway beams and monorail beams shall be erected within the following tolerances:

a) The slope of a member shall not exceed $L/1000$. However, the difference in elevation of the ends need not be less than 3 mm and shall not exceed 6 mm. The difference in elevation of opposite points on two parallel runway beams shall not exceed 1/1000 of the distance between the runway beams and shall not exceed 6 mm.
b) The offset of one end of the member relative to the other from the horizontal alignment shown on the drawings shall not exceed $L/500$. However, the offset need not be less than 3 mm and shall not exceed 8 mm.
c) The distance between the ends of two parallel runway beams shall not deviate by more than 1/500 of the span of the runway beam. However, the difference in the distances between the runway beam ends need not be less than 3 mm and shall not exceed 10 mm.

29.3.7 Alignment of braced members

Members such as columns, beams, trusses, and open web steel joists that are braced between their supports shall be erected in such a way that the fabrication tolerances specified in this Standard are maintained.

29.3.8 Members with adjustable connections

Members with adjustable connections (e.g., shelf angles, sash angles, and lintels) shall be considered to be within tolerances when the following requirements are met:

a) Each piece shall be level within $L/1000$; however, the difference in elevation of the ends need not be less than 3 mm and shall not exceed 6 mm.
b) Adjoining ends of members shall be aligned vertically and horizontally within 2 mm.
c) The location of the members both vertically and horizontally shall be within 10 mm of the location established by the dimensions on the drawings.

29.3.9 Column splices

Column splices and other compression joints that depend on contact bearing as part of the splice resistance shall, after alignment, have a maximum allowable separation of 6 mm. Any gap exceeding 1.5 mm shall be packed with non-tapered steel shims. Shims need not be other than mild steel, regardless of the grade of the main material.

29.3.10 Welded joint fit-up

The fit-up of joints that are to be field-welded shall be within the tolerances shown on the erection diagrams and shall not exceed the tolerances specified in CSA W59 when welding is completed.

29.3.11 Bolted joint fit-up

Bolted joint fit-up shall meet the requirements of Clause 28.4.4.

30 Inspection

30.1 General

Material and quality of work shall at all times be subject to inspection by qualified inspectors who represent and are responsible to the designer. The inspection shall cover shop work and field erection work to ensure compliance with this Standard.

30.2 Co-operation

Insofar as possible, all inspections shall be made in the fabricator's shop. The fabricator shall co-operate with the inspector and permit access for inspection to all places where work is being done. The inspector shall co-operate in avoiding undue delay in the fabrication or erection of the steelwork.

30.3 Rejection

Material or quality of work not meeting the requirements of this Standard may be rejected at any time during the progress of work once non-compliance is established.

30.4 Inspection of high-strength bolted joints

The inspection of high-strength bolted joints shall be performed in accordance with Clause 23.8.

① 30.5 Welding inspection

30.5.1 Extent of examination

30.5.1.1 General

The fabricator or erector shall visually inspect all welds. Non-destructive examination of welds (other than visual inspection) shall be completed by the fabricator or erector when specified by the owner. Third-party welding inspection (visual and/or non-destructive) shall be performed when required by the owner.

30.5.1.2 Competency of fabricator or erector inspection personnel

Personnel performing weld quality control for the fabricator or erector shall be competent to perform the assigned weld quality control tasks. The required competency of personnel performing visual weld inspection tasks shall be defined and documented by the fabricator or erector based on their processes. Records of personnel competency shall be maintained by the fabricator or erector.

30.5.1.3 Competency of personnel performing non-destructive testing when performed by the Fabricator or Erector (not including visual inspection)

Competency of personnel performing non-destructive testing, other than visual, shall be in accordance with CAN/CGSB-48.9712/ISO 9712. The record of compliance with these requirements shall be documented.

30.5.1.4 Competency for third-party personnel performing visual and/or non-destructive testing on behalf of the fabricator or erector

If the fabricator or erector elects to subcontract any visual or non-destructive inspection to a third party to complete on their behalf, then any-third party personnel performing such inspection shall meet the requirements specified in Clauses 30.5.2.2 to 30.5.2.5.

30.5.2 Competency of inspection personnel

30.5.2.1 General

The required competency of personnel performing visual weld inspection tasks shall be defined and documented by the fabricator or erector based on their processes. Records of personnel competency shall be maintained by the fabricator or erector.

30.5.2.2 Competency of all personnel performing non-destructive testing (not including visual inspection)

Competency of all personnel performing non-destructive testing, other than visual, shall be in accordance with CAN/CGSB-48.9712/ISO 9712.

30.5.2.3 Competency for third-party personnel performing non-destructive testing (visual inspection only)

The competency of third-party visual inspection personnel shall meet the requirements of CSA W178.2 or AWS QC1. AWS inspectors shall have evidence of an eye exam showing 20/20 vision corrected or uncorrected within the last 2 years.

30.5.2.4 Compliance records

Compliance with the requirements of CAN/CGSB-48.9712/ISO 9712 shall be documented.

30.5.2.5 Non-destructive testing personnel

Non-destructive testing personnel referenced in Causes 30.5.2.2 and 30.5.2.3 shall meet the requirements of Level 2 or 3 of CSA W178.2, AWS QC1, or CAN/CGSB-48.9712/ISO 9712 as applicable. Level 1 personnel (or a CAWI under AWS QC1) may only perform the applicable tasks under the direct supervision of Level 2 or 3 personnel.

Note: *For personnel certified under AWS QC1, a CWI or SCWI is equivalent to an inspector certified to Level II of CSA W178.2.*

30.5.3 Acceptance criteria

The fabricator or erector shall ensure that all welds under their responsibility comply with the CSA W59. When third-party welding inspection is required by the owner, such verification shall be completed by the fabricator or erector prior to third-party inspection.

Unless otherwise specified, the acceptance criteria for all welds shall be in accordance with CSA W59.

30.6 Identification of steel by marking

In the fabricator's plant, steel used for main components shall at all times be marked to identify its specification (and grade, if applicable). This shall be done by suitable markings or by recognized colour coding, except that cut pieces identified by piece mark and contract number need not continue to carry specification identification markings when it has been satisfactorily established that such cut pieces meet the required material specifications.

Table 1
Maximum width (or diameter)-to-thickness ratios: Elements in axial compression
(See Clauses 11.2, 13.3.1, 13.3.3.1, 13.3.5, and 14.4.2.)

Description of elements	Limits
Elements supported along one edge such as	
Flanges of I-sections, T-sections, and channels	$\frac{b_{el}}{t} \le \frac{200}{\sqrt{F_y}}$

(Continued)

Table 1 (Concluded)

Description of elements	Limits
Stiffeners of Plate-girders	
Legs of angles	$\frac{b_{el}}{t} \leq \frac{250}{\sqrt{F_y}}$
Element supported along one edge and restrained by a plate that is substantially stiffer than the element itself, such as: Stems of T-sections	$\frac{b_{el}}{t} \leq \frac{340}{\sqrt{F_y}}$
Element supported along two edges, such as: Flanges of rectangular hollow sections Flange cover plates and diaphragm plates between lines of fasteners or welds, web of I-shape sections. Web supported on both edges	$\frac{b_{el}}{t} \leq \frac{670}{\sqrt{F_y}}$
Perforated cover plates	$\frac{b_{el}}{t} \leq \frac{840}{\sqrt{F_y}}$
Circular hollow sections	$\frac{D}{t} \leq \frac{23\ 000}{F_y}$

①

Table 2
Maximum width (or diameter)-to-thickness ratios: Elements in flexural compression
(See Clauses 11.2 and 27.7.2.7.)

Description of elements	Section classification limits		
	Class 1	Class 2	Class 3
Element supported along one edge and under flexural compression, such as Flanges of I-sections or T-sections under bending about the major axis Plates projecting from element in compression elements Outstanding legs of pairs of angles in continuous contact with an axis of symmetry in the plane of loading	$\frac{b_{el}}{t} \le \frac{145}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{170}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{200}{\sqrt{F_y}}$
Element supported along one edge under compressive stress due to flexural bending but with a part in tension, such as Stems of T-sections Flange of I-section under flexure around the minor axis	$\frac{b_{el}}{t} \le \frac{145}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{170}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{340}{\sqrt{F_y}}$
Element supported along two edges mainly under compressive stress due to flexural bending, such as Flanges of rectangular hollow sections	$\frac{b_{el}}{t} \le \frac{420}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{670}{\sqrt{F_y}}$
Element supported along two edges mainly under compressive stress due to flexural bending, such as Flanges of box sections Flange cover plates and diaphragm plates between lines of fasteners or welds	$\frac{b_{el}}{t} \le \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{670}{\sqrt{F_y}}$
Element supported along two edges and subjected to combined axial compression and bending about the major axis, such as Webs of I-sections	$\frac{h}{w} \le \frac{1100}{\sqrt{F_y}}\left(1 - 0.39\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1700}{\sqrt{F_y}}\left(1 - 0.61\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left(1 - 0.65\frac{C_f}{\phi C_y}\right)$
Web of I-section subjected to compression due to combined member axial compression and bending about the minor axis:			

(Continued)

Table 2 (Concluded)

Description of elements	Section classification limits: Class 1	Class 2	Class 3
a) For $C_f > 0.4\phi C_y$	$\frac{h}{w} \le \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \le \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left(1 - 0.65\frac{C_f}{\phi C_y}\right)$
b) For $C_f \le 0.4\phi C_y$	$\frac{h}{w} \le \frac{1100}{\sqrt{F_y}}\left(1 - 1.31\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1700}{\sqrt{F_y}}\left(1 - 1.73\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left(1 - 0.65\frac{C_f}{\phi C_y}\right)$
Web of I-section subjected to compression due to combined member axial compression and bending about both principal axes, with $\frac{M_{fy}}{S_y} > \frac{0.9M_{fx}}{S_x}$ See Note 2.	$\frac{h}{w} \le \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \le \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left(1 - 0.65\frac{C_f}{\phi C_y}\right)$
Circular hollow sections	$\frac{D}{t} \le \frac{13\,000}{F_y}$	$\frac{D}{t} \le \frac{18\,000}{F_y}$	$\frac{D}{t} \le \frac{66\,000}{F_y}$

Notes:

1) *Elements with ratios exceeding Class 3 limits are Class 4 sections.*

2) *If $\frac{M_{fy}}{S_y} \le \frac{0.9M_{fx}}{S_x}$, the limits for elements supported along two edges and subjected to combined axial compression and bending about the major axis shall apply.*

Δ

Table 3
Values of k_s and c_s
(See Clauses 13.12.2.2 and 23.2.)

Contact surface of bolted parts: Class	Description	k_s	c_s: Turn-of-nut: A325 and A325M* bolts	A490 and A490M* bolts	Other: F959, F1852, and F2280
A	Unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel or hot-dipped galvanized and roughened surfaces	0.30	1.00	0.92	0.78
B	Unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel	0.52	1.04	0.96	0.81

* *Bolts are installed by the turn-of-nut method.*

Note: *Class A and Class B coatings are those coatings that provide a mean slip coefficient, k_s, of not less than 0.30 and 0.52, respectively.*

Table 4
Matching electrode ultimate tensile strengths for CSA G40.21 steels
(See Clause 13.13.1.)

Matching electrode ultimate tensile strength* MPa	G40.21 Grades, MPa						
	260	300	350	380	400	480	700
430	X	X†					
490	X	X	X‡	X			
550					X‡		
620						X	
820							X

* *The electrode ultimate tensile strength is ten times the first two digits of the electrode classification in CSA W48.*
† *For HSS only.*
‡ *For unpainted applications using "A" or "AT" steels where the deposited weld metal is to have atmospheric corrosion resistance or colour characteristics, or both, similar to the base metal, the requirements of Clauses 5.2.1.4 and 5.2.1.5 of CSA W59 shall apply.*

Note: *For matching conditions of ASTM steels, see Table 11-1 or 12-1 of CSA W59.*

Table 5
Maximum intermediate transverse stiffener spacing
(See Clause 14.5.2.)

Web depth-to-thickness ratio, h/w	Maximum distance between stiffeners, a, in terms of clear web depth, h
≤ 150	$3h$
> 150	$\frac{67\,500\,h}{(h/w)^2}$

Table 6
Minimum edge distance for bolt holes, mm
(See Clauses 22.3.2 and 22.3.4.)

Bolt diameter		Minimum edge distance	
mm	in	At sheared edge	At rolled or sawn edges, or edges cut by gas*, plasma, laser, or water jet
—	5/8	28	22
16	—	28	22
—	3/4	32	25
20	—	34	26
—	7/8	38†	28
22	—	38	28
24	—	42	30
—	1	44†	32
27	—	48	34
—	1-1/8	51	38
30	—	52	38
—	1-1/4	57	41
36	—	64	46
Over 36	Over 1-1/4	1.75 × diameter	1.25 × diameter

* *Gas-cut edges shall be smooth and free from notches. The edge distance in this column may be decreased by 3 mm when the hole is at a point where calculated stress under factored loads is not more than 0.3 of the yield stress.*
† *At the ends of beam-framing angles, this distance may be 32 mm.*

Table 7
Minimum bolt tension, kN
(See Clauses 23.7.1, 23.7.3, 23.7.4, 23.8.2, and I.1.)

Bolt diameter		Minimum bolt tension*	
mm	in	A325, A325M, and F1852 bolts	A490, A490M, and F2280 bolts
—	1/2	53	67
—	5/8	85	107
16	—	91	114
—	3/4	125	157
20	—	142	178
—	7/8	174	218
22	—	176	220
24	—	205	257
—	1	227	285
27	—	267	334
—	1-1/8	249	356
30	—	326	408
—	1-1/4	316	454
—	1-3/8	378	538
36	—	475	595
—	1-1/2	458	658

** Equal to 70% of the specified minimum tensile strength.*

Table 8
Nut rotation from snug-tight condition*
(See Clauses 23.7.2and I.1.)

Disposition of outer faces of bolted parts	Bolt length†	Turn
Both faces normal to bolt axis or one face normal to axis and other face sloped 1:20 max. (bevelled washers not used)‡	Up to and including 4 diameters	1/3
	Over 4 diameters and not exceeding 8 diameters or 200 mm	1/2
	Exceeding 8 diameters or 200 mm	2/3
Both faces sloped 1:20 max. from normal to bolt axis (bevelled washers not used)‡	All lengths of bolts	3/4

** Nut rotation is rotation relative to a bolt regardless of whether the nut or bolt is turned. The tolerance on rotation is 30° over or under. This Table applies to coarse-thread heavy-hex structural bolts of all sizes and lengths used with heavy-hex semi-finished nuts.*
† Bolt length is measured from the underside of the head to the extreme end of point.
‡ Bevelled washers are necessary when A490, A490M, or F2280 bolts are used.

Table 9
Detail categories for load-induced fatigue
(See Clauses 26.3.1 and 26.3.4.)

General condition	Situation	Detail category	Illustrative example (see Figure 2)
Plain members	Base metal		
	• with rolled or cleaned surfaces. Flame-cut edges with a surface roughness not exceeding 1000 (25 µm) as specified by CSA B95	A	1, 2
	• of unpainted weathering steel	B	
	• at re-entrant corners of copes with a radius ≥ 35 mm and ground smooth	E1	2a
	• at net section of eyebar heads and pin plates	E	
Built-up members	Base metal and weld metal in components, without attachments, connected by		3, 4, 5, 7
	• continuous full-penetration groove welds with backing bars removed; or	B	
	• continuous fillet welds parallel to the direction of applied stress;	B	
	• continuous full-penetration groove welds with backing bars in place;or	B1	
	• continuous partial-penetration groove welds parallel to the direction of applied stress.	B1	
	Base metal at ends of partial-length cover plates		
	• bolts in slip-critical connections;	B	22
	• narrower than the flange, with or without end welds, or wider than the flange with end welds		
	– flange thickness ≤ 20 mm	E	7
	– flange thickness > 20 mm	E1	7
	• wider than the flange without end welds	E1	7
Groove-welded splice connections with weld soundness established by NDT and all required grinding in the direction of the applied stresses	Base metal and weld metal at full-penetration groove-welded splices		
	• of plates of similar cross-sections with welds ground flush	B	8, 9
	• with 600 mm radius transitions in width with welds ground flush	B	11
	• with transitions in width or thickness with welds ground to provide slopes not steeper than 1.0 to 2.5		10, 10a
	• G40.21-700Q and 700QT base metal	B1	
	• other base metal grades	B	
	• with or without transitions having slopes not greater than 1.0 to 2.5, when weld reinforcement is not removed	C	8, 9, 10, 10a

(Continued)

Table 9 (Continued)

General condition	Situation	Detail category	Illustrative example (see Figure 2)
	• at weld access holes		
	– of rolled members	C	
	– of built-up members	D	
Longitudinally loaded groove-welded attachments	Base metal at details attached by full- or partial-penetration groove welds		
	When the detail length in the direction of applied stress is		
	• less than 50 mm	C	6, 18
	• between 50 mm and 12 times the detail thickness, but less than 100 mm	D	18
	• greater than either 12 times the detail thickness or 100 mm		
	– detail thickness < 25 mm	E	18
	– detail thickness ≥ 25 mm	E1	18
	• with a transition radius, R, with the end welds ground smooth, regardless of detail length		12
	– $R \geq 600$ mm	B	
	– 600 mm > $R \geq 150$ mm	C	
	– 150 mm > $R \geq 50$ mm	D	
	– $R < 50$ mm	E	
	• with a transition radius, R, with the end welds not ground smooth	E	12
Transversely loaded groove-welded attachments with weld soundness established by NDT and all required grinding transverse to the direction of stress	Base metal at detail attached by full-penetration groove welds with a transition radius, R		12
	• to flange, with equal plate thickness and weld reinforcement removed		
	– $R \geq 600$ mm	B	
	– 600 mm > $R \geq 150$ mm	C	
	– 150 mm > $R \geq 50$ mm	D	
	– $R < 50$ mm	E	
	• to flange, with equal plate thickness and weld reinforcement not removed, or to web		
	– $R \geq 150$ mm	C	
	— 150 mm > $R \geq 50$ mm	D	
	– $R < 50$ mm	E	
	• to flange, with unequal plate thickness and weld reinforcement removed		
	– $R \geq 50$ mm	D	
	– $R < 50$ mm	E	

(Continued)

Table 9 (Continued)

General condition	Situation	Detail category	Illustrative example (see Figure 2)
	– to flange, for any transition radius with unequal plate thickness and weld reinforcement not removed	E	
Fillet-welded connections with welds normal to the direction of stress	Base metal		
	• at details other than transverse stiffener-to-flange or transverse stiffener-to-web connections	C*	19
	• at the toe of transverse stiffener-to-flange and transverse stiffener-to-web welds	C1	6
Fillet-welded connections with welds normal and/or parallel to the direction of stress	Shear stress on weld throat	E	16
Longitudinally loaded fillet-welded attachments	Base metal at details attached by fillet welds		
	• when the detail length in the direction of applied stress is		
	– less than 50 mm, and stud-type shear connectors	C	13, 14, 15, 18, 20
	– between 50 mm and 12 times the detail thickness, but less than 100 mm	D	14, 18, 20
	– greater than either 12 times the detail thickness or 100 mm		7, 14, 16, 18, 20
	• detail thickness < 25 mm	E	
	• detail thickness ≥ 25 mm	E1	
	• with a transition radius, R, with the end of welds ground smooth, regardless of detail length		12
	– $R \geq 50$ mm	D	
	– $R < 50$ mm	E	
	• with a transition radius with the end of welds not ground smooth	E	12
Transversely loaded fillet-welded attachments with welds parallel to the direction of primary stress	Base metal at details attached by fillet welds		12
	• with a transition radius, R, with the end of welds ground smooth		
	– $R \geq 50$ mm	D	
	– $R < 50$ mm	E	
	• with any transition radius with end of welds not ground smooth	E	
Mechanically fastened connections	Base metal		17
	• at gross section of high-strength bolted slip-critical connections, except axially loaded joints in which out-of-plane bending is induced in connected materials	B	
	• at net section of high-strength bolted non-slip-critical connections	B	

(Continued)

Table 9 (Concluded)

General condition	Situation	Detail category	Illustrative example (see Figure 2)
	• at net section of non-pretensioned bolted connections	D	
	• at net section of riveted connections	D	
Anchor rods and threaded parts	Tensile stress range on the tensile stress area of the threaded part, including effects of bending	E	
Fillet-welded HSS to base plate	Shear stress on fillet weld	E1	21
A325, A325M, and F1852 bolts in axial tension	Tensile stress on area A_b	See Clause 13.12.1.3	
A490, A490M, and F2280 bolts in axial tension	Tensile stress on area A_b		

Note: *The fatigue resistance of fillet welds transversely loaded is a function of the effective throat and plate thickness. See Frank and Fisher (1979).*

$$F_{sr} = F_{sr}^{c}\left[\left(0.06 + 0.79H / t_p\right) / \left(0.64 t_p^{\ 1/6}\right)\right]$$

where

H = *weld leg size*

C_{sr} = *fatigue resistance for Category C as determined in accordance with Clause 26.3.3. This assumes no penetration at the weld root*

t_p = *plate thickness*

Table 10
Fatigue constants for detail categories
(See Clauses 26.3.3 and 26.3.4.)

Detail category	Fatigue life constant, γ	Constant amplitude threshold stress range, F_{srt}, MPa	nN'	Fatigue life constant, γ'
A	8190×10^9	165	1.82×10^6	223×10^{15}
B	3930×10^9	110	2.95×10^6	47.6×10^{15}
B1	2000×10^9	83	3.50×10^6	13.8×10^{15}
C	1440×10^9	69	4.38×10^6	6.86×10^{15}
C1	1440×10^9	83	2.52×10^6	9.92×10^{15}
D	721×10^9	48	6.52×10^6	1.66×10^{15}
E	361×10^9	31	12.1×10^6	0.347×10^{15}
E1	128×10^9	18	21.9×10^6	0.0415×10^{15}

Figure 1
Fatigue constants for detail categories
(See Clauses 26.3.3 and 26.3.4.)

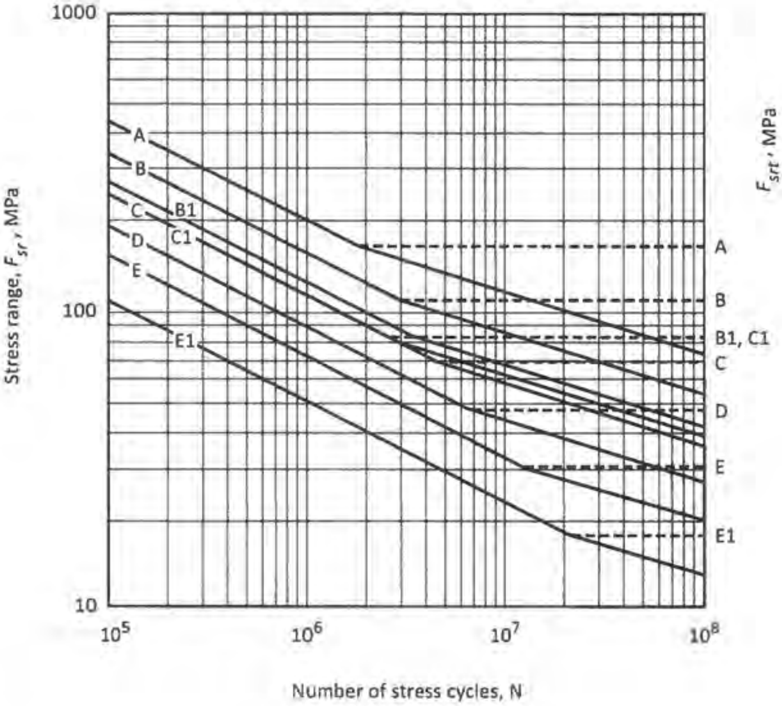


Figure 2
Illustrative examples of detail categories
(See Clause 26.3.4 and Table 9.)

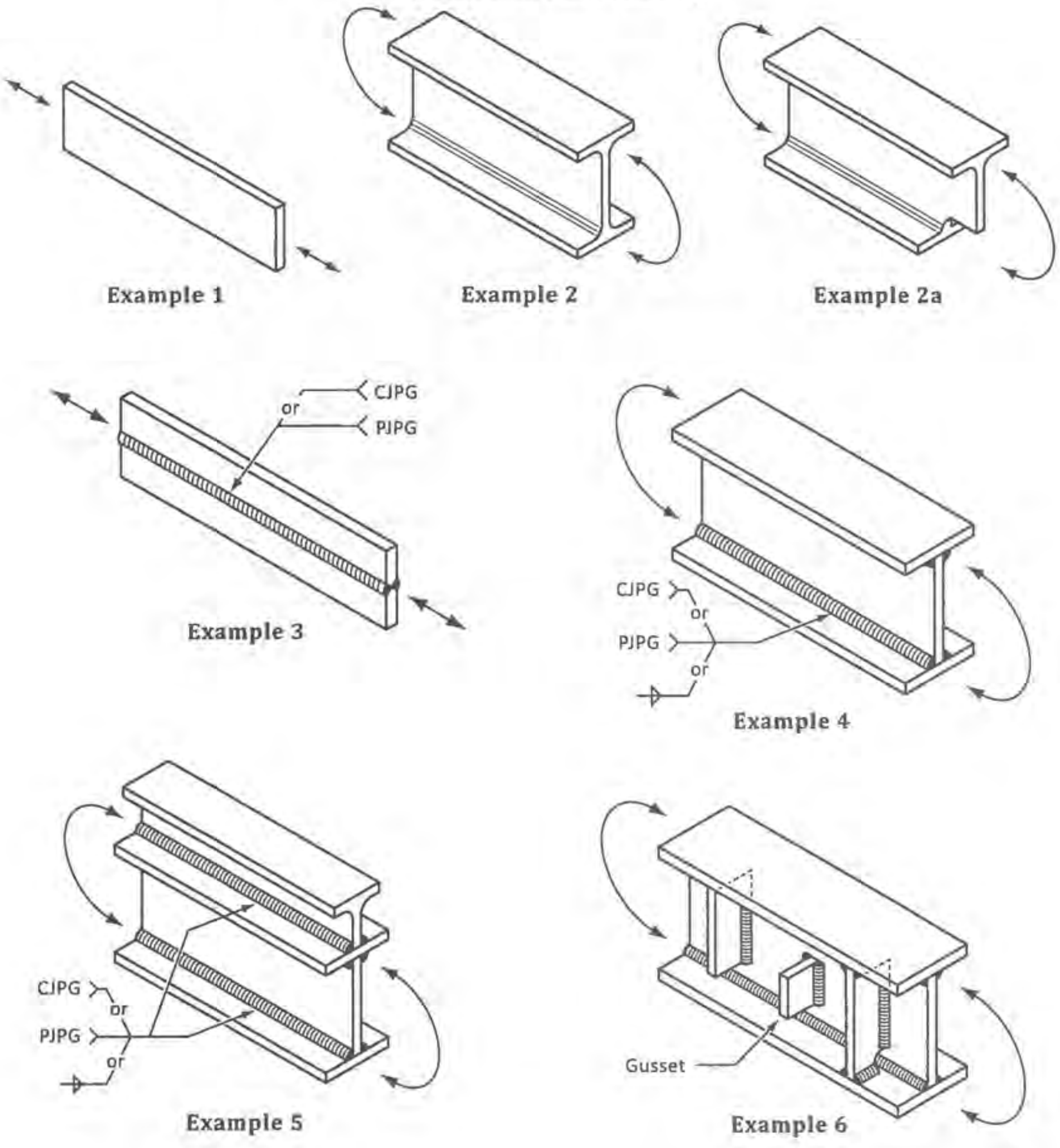


Figure 2 (Continued)

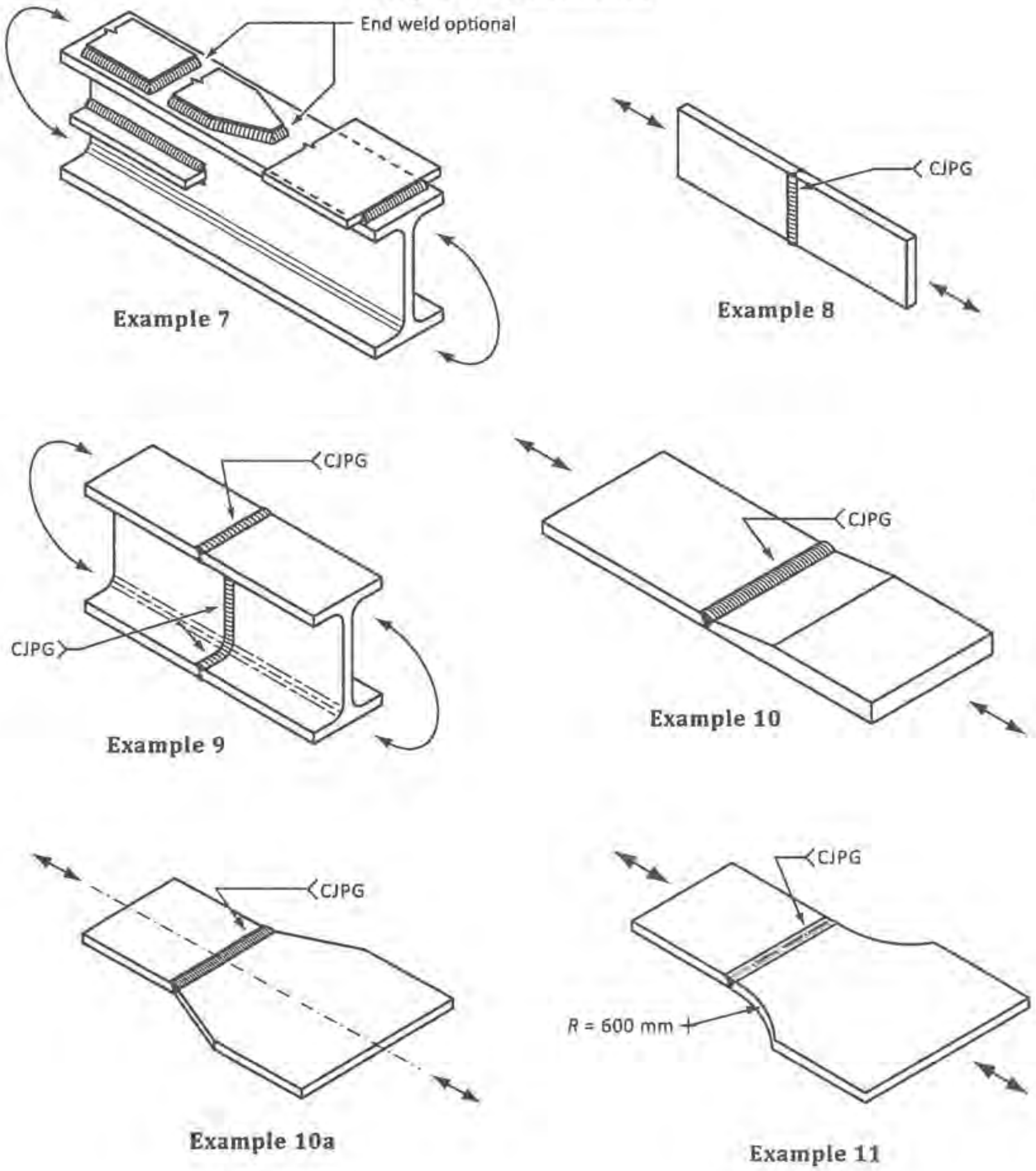
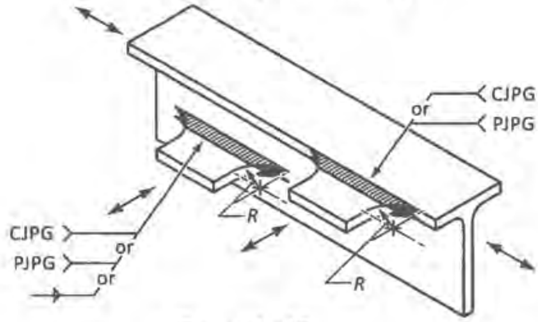
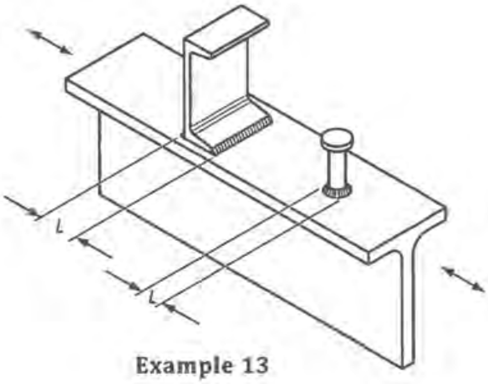


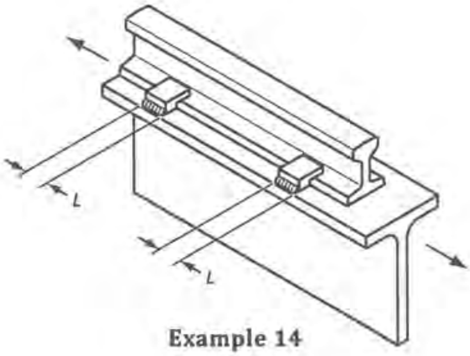
Figure 2 (Continued)



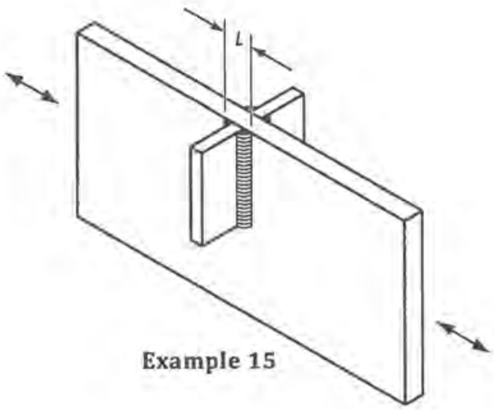
Example 12



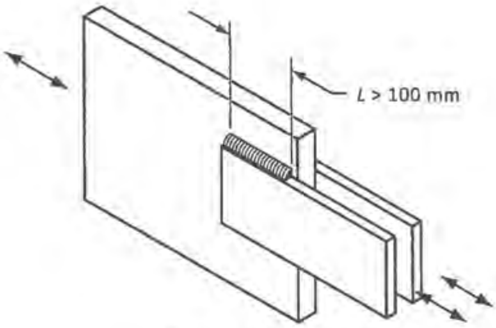
Example 13



Example 14

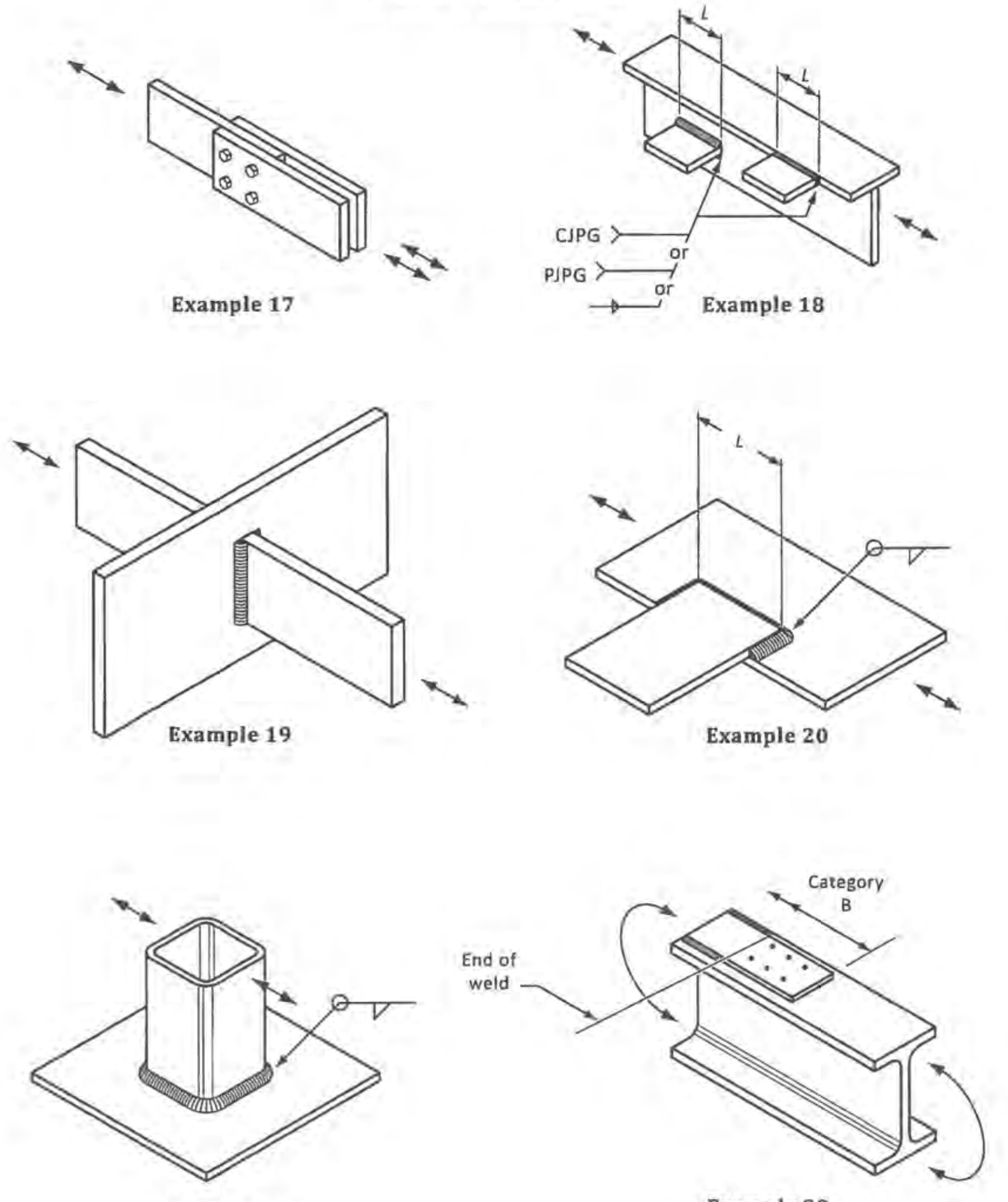


Example 15



Example 16

Figure 2 (Concluded)



Annex A (informative)
Standard practice for structural steel

Note: *This informative Annex has been written in mandatory language to facilitate adoption by anyone wishing to do so.*

A.1 General

Matters concerning standard practice not covered by this Standard but pertinent to the fabrication and erection of structural steel (e.g., classification of material and contract documents) shall be in accordance with the CISC's *Code of Standard Practice for Structural Steel* unless otherwise clearly specified in the plans and specifications issued to the bidders.

Annex B (informative)
Margins of safety

Note: *This Annex is an informative (non-mandatory) part of this Standard.*

B.1

Code writers now use limit states design to provide a practical level of reliability over the lifetime of a structure. One of the advantages of limit states design is that by using load and resistance factors based on the statistical variation of the loads and resistances, a relatively uniform degree of reliability is obtained in the design of structures across a variety of configurations and load conditions. At the same time, economies accrue in limit states design since structures or portions of them are not designed for excessive safety, either due to the unrealistic load combinations or inaccurate modelling based on the assumed elastic behaviour of structural steel components used in the past. Moreover, by changing the reliability index in limit states design, greater or lesser safety can be assigned on a quantitative basis to entire structures or to components.

B.2

The load and resistance factors in limit states design, derived to give the desired reliability index, are related to the calculated probability of failure and are based on the statistical variations of the loads and resistances.

B.3

Limit states design was first introduced in the *NBCC*, 1975, where the reliability index for steel buildings as a whole was taken as 3.0. A greater reliability index was used for connectors so that the probability of the connector failing before the member as a whole was reduced and the more ductile mode of failure of the member was favoured. This was done to make the connections stronger than the members they joined. In the current *NBCC* and this edition of this Standard, the reliability index for steel buildings as a whole remains 3.0, and indices greater than this value are used for connections.

B.4

The development of member resistance factors used in the first limited states design standard, CSA S16.1-1974, is discussed in Kennedy and Gad Aly (1980) and others in Kennedy and Baker (1984). Since then, other resistance factors have been introduced based on statistical analyses of the resistances. That of 0.67 for welds was confirmed in the 1994 edition of CAN/CSA-S16.1, when the strength of transverse fillet welds was recognized to be 1.50 times that of longitudinal fillet welds (Lesik and Kennedy, 1990). Other resistance factors have been introduced for shear connectors, anchor rods, bearing of bolts on steel, and reinforcing bars, as well as $\phi b = 0.80$ for high-strength bolts in shear and tension (Kennedy, 1999a) and $\phi bi = 0.80$ and $\phi be = 0.75$ for bearing on webs of interior loads and end reactions, respectively (Kennedy, et al., 1998; Kennedy, 1999b). Enhanced target reliability indices for calculating resistance factors of 4.5 were used for welds and bolts and 3.5 for bearing on webs. In this edition of the Standard, a new resistance factor for the block shear, net section rupture, and bolt tear-out limit states, $\phi u = 0.75$, has been introduced based on recent research by Driver et al. (2006) and Cai and Driver (2010). A review of resistance factors used in this Standard is presented by Schmidt and Bartlett (2002).

B.5

Cai, Q. and Driver, R.G. (2010). "Prediction of bolted connection capacity for block shear failures along atypical paths". *AISC Engineering Journal* Fourth Quarter, 213–221 .

Driver, R.G., Grondin, G.Y., and Kulak, G.L. (2006). *"Unified block shear equation for achieving consistent reliability". Journal of Constructional Steel Research*, 62 (3), March, 210–222.

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Kennedy, S.J., Kennedy, D.J.L., and Medhekar, M.S. (1998). "The bearing resistance of webs: Further studies of the post-buckling strength". *Proceedings of the Annual Conference*, Structural Stability Research Council, Atlanta, GA, September 21–23, 25–41.

Lesik, D.F. and Kennedy, D.J.L., (1990). "Ultimate strength of fillet welded connections loaded in plane". *Canadian Journal of Civil Engineering*, 17 (1), February, 55–67.

Schmidt, B.J. and Bartlett, F.M. (2002). "Review of resistance factor for steel: Resistance distributions and resistance factor calibration". *Canadian journal of Civil Engineering*, 29 (1), February, 109–118.

Annex C (normative)
Crane-supporting structures

Note: *This Annex is a normative (mandatory) part of this Standard.*

C.1 General

Steel structures that support overhead cranes and hoists require special consideration in order to provide safe and serviceable structures. Electrically-operated top-running overhead travelling cranes, underslung cranes, and monorails impose repetitive loads that can lead to the development and propagation of fatigue cracks in the crane-supporting structure. These loads shall be accounted for in the design and construction of the crane-supporting structure. Conditions that apply to these steel structures, where any component is subjected to fatigue loads as specified in Clause 26, are given in this Annex.

The requirements of this Standard for design for fatigue shall apply. The structural design shall take into account, among other factors, appropriate methods of analysis, rotational restraints at crane runway beam supports, crane load eccentricities, distortion leading to fatigue cracking, welded details, built-up column section details, bracing systems, deflections, and details related to crane rails. The construction specifications shall include (but not necessarily be limited to) requirements for materials, detailing, fabrication, erection, bearing and contact surfaces, dimensional tolerances, crane rail installation, and shop and field inspection.

The designer shall determine the loading parameters and the appropriate number of loading cycles at each level of load by analyzing the duty cycles for the design life of the structure, in addition to other crane details that are necessary to design the structure. This information shall be included in the structural design documents.

Note: *For design information and information to be shown on the structural design documents, see the CISC's* Crane-Supporting Steel Structures: Design Guide.

Annex D (informative)
Recommended maximum values for deflections for specified design live, snow, and wind loads

Note: *This Annex is an informative (non-mandatory) part of this Standard.*

D.1 General

Table D.1 provides deflection criteria for floor or roof members as a fraction of the span and for lateral drift as a fraction of the storey height. These criteria are related to the serviceability limit states. Although the criteria refer to specified live, snow, and wind loads, the designer should consider the inclusion of specified dead loads in some instances. For example, non-permanent partitions, which are classified by the *NBCC* as dead load, should be part of the loading considered under this Annex if they are likely to be applied to the structure after the completion of finishes susceptible to cracking.

D.2 Wind

Some building materials augment the rigidity provided by the steelwork; therefore, the deflections calculated for bare steel structures under wind loads can be somewhat reduced. The more common structural and non-structural elements that contribute to the stiffness of a building are masonry walls, certain types of curtain walls, masonry partitions, and concrete around steel members. Provided that the materials augmenting rigidity are accounted for in the analysis for wind loads, the deflections for comparison to the limits in Table D.1 can be reduced by a maximum of 15%. The deflections used for strength and stability calculations should not be reduced. In tall and slender structures (height greater than four times the width), the wind effects should be determined by means of dynamic analysis or wind tunnel tests.

Table D.1
Deflection criteria
(See Clauses D.1 and D.2.)

Building type	Deflection	Specified loading	Application	Maximum
Industrial	Vertical	Live, snow	Members supporting inelastic roof coverings	$L/240$
		Live, snow	Members supporting elastic roof coverings	$L/180$
		Live, snow	Members supporting floors	$L/300$
		Maximum wheel loads (no impact)	Crane runway girders for crane capacity of 225 kN and over	$L/800$
		Maximum wheel loads (no impact)	Crane runway girders for crane capacity under 225 kN	$L/600$
	Lateral	Crane lateral	Crane runway girders	$L/600$
		Crane lateral or wind	Storey drift*	$h/400$ to $h/200$
All others	Vertical	Live, snow	Members of floors and roofs supporting construction and finishes susceptible to cracking	$L/360$
		Live, snow	Members of floors and roofs supporting construction and finishes not susceptible to cracking	$L/300$
	Lateral	Wind	Building drift due to all effects	$h/400$
		Wind	Storey drift (relative horizontal movement of any two consecutive floors) in buildings in cladding and partitions without special provision to accommodate building frame deformation	$h/500$
		Wind	Storey drift, with special provision to accommodate building frame deformation	$h/400$

Legend:

h = storey height

L = length or span

* *The permissible drift of industrial buildings depends on such factors as wall construction, building height, and the effect of deflection on the operation of the crane. Where the operation of the crane is sensitive to lateral deflections, a lateral deflection of less than h/400 may be necessary.*

Annex E (informative)
Floor vibrations

Note: *This informative Annex has been written in mandatory language to facilitate adoption by anyone wishing to do so.*

E.1 General

The development of floors of lighter construction, longer spans, and less inherent damping can sometimes result in disturbing floor vibrations during normal human activity. The specific vibration characteristics of the floor should be evaluated by the building designer.

Such an evaluation shall, at a minimum, consider the following:

a) the characteristics and nature of the forcing excitations, e.g., walking and rhythmic activities (see also the *NBCC*);

b) acceptance criteria for human comfort (depending on the use and occupancy of the floor area);

c) a determination of the natural frequency of the floor framing systems, including the effect of continuity;

d) the modal damping ratio; and

e) the effective floor weights.

For guidance, see Murray, et al. (1997) and Commentary I, *User's Guide — NBC 2010: Structural Commentaries (Part 4)*.

E.2 Light-framed construction

For guidance on vibrations due to walking on light-framed construction made of light steel members and wood deck, see Applied Technology Council (1999).

E.3 Bibliography

Applied Technology Council (1999). *Minimizing floor vibration.* ATC Design Guide 1, Applied Technology Council, Redwood City, California.

Murray, T.M., Allen, D.E. and Ungar, E.E. (1997). *Floor vibrations due to human activity.* Steel Design Guide Series 11. American Institute of Steel Construction, Chicago; Canadian Institute of Steel Construction, Toronto.

Annex F (informative)
Effective lengths of columns

Note: *This Annex is an informative (non-mandatory) part of this Standard.*

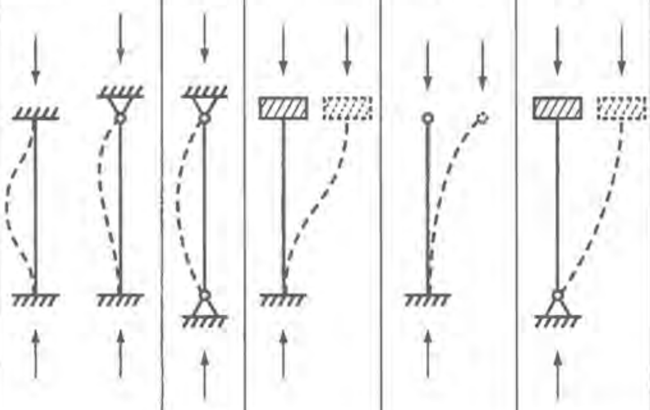
F.1

The slenderness ratio of a member whose failure mode involves buckling is defined as the ratio of the effective length to the applicable radius of gyration. The effective length, KL, may be thought of as the actual unbraced length, L, multiplied by a factor, K, so that the product, KL, is equal to the length of a pin-ended column of equal capacity to the actual member. The effective length factor, K, of a column of finite unbraced length therefore depends on the conditions of restraint afforded to the column at its braced locations.

F.2

A variation in K between 0.65 and 2.0 will apply to the majority of cases likely to be encountered in actual structures. Figure F.1 illustrates six idealized cases in which joint rotation and translation are either fully realized or non-existent.

Figure F.1
Effective lengths of columns
(See Clause F.2.)

	a)	b)	c)	d)	e)	f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.0	2.0

End condition code		Rotation fixed, translation fixed
		Rotation free, translation fixed
		Rotation fixed, translation free
		Rotation free, translation free

Annex G (informative) ***Criteria for estimating effective column lengths in continuous frames***

Note: *This Annex is an informative (non-mandatory) part of this Standard.*

G.1

Because this Standard requires the in-plane behaviour of beam columns to be based on their actual lengths (provided that, when applicable, the sway effects are included in the analysis of the structure [see Clause 8.4]), this Annex applies only to cases related to buckling, i.e., to axially loaded columns and beam columns failing by out-of-plane buckling.

G.2

Figure G.1 is a nomograph applicable to cases in which the equivalent I/L of adjacent girders that are rigidly attached to the columns is known; it is based on the assumption that all columns, in the portion of the framework considered, reach their individual critical load simultaneously. This is a conservative assumption made in the interest of simplification.

G.3

The equation on which the nomograph is based is as follows:

$$\frac{G_U G_L}{4}(\pi / K)^2 + \frac{G_U + G_L}{2}\left(1 - \frac{\pi / K}{\tan\pi / K}\right) + 2\left[\frac{\tan\pi / 2K}{\pi / K}\right] = 1$$

Subscripts U and L refer to the joints at the two ends of the column section being considered and

$$G = \frac{\Sigma I_c / L_c}{\Sigma I_g / L_g}$$

where

Σ = summation for all members rigidly connected to that joint and lying in the plane in which buckling of the column is being considered

I_c = moment of inertia of the column about the axes perpendicular to the plane of buckling

L_c = unsupported length of a column

I_g = moment of inertia of the girder about the axes perpendicular to the plane of buckling

L_g = unsupported length of a girder

G.4

For column ends supported by, but not rigidly connected to, a footing or foundation, G may be taken as 10 for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1.0. Smaller values may be used if justified by analysis.

G.5

Refinements in girder I_g/L_g may be made when conditions at the far end of any particular girder are known definitely or when a conservative estimate can be made. For the case with no sideway, multiply girder stiffnesses by the following factors:

a) 1.5 if the far end of the girder is hinged; and
b) 2.0 if the far end of the girder is fixed against rotation (i.e., rigidly attached to a support that is itself relatively rigid).

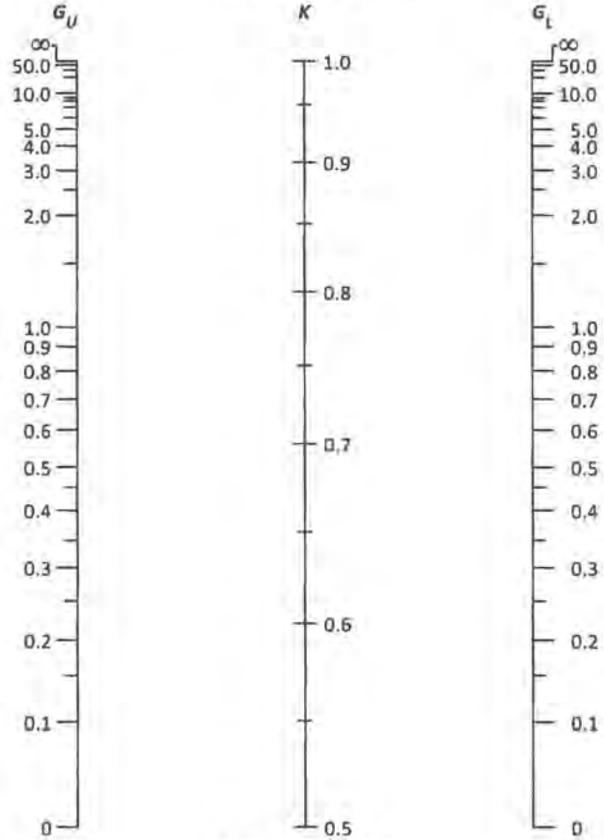
G.6

Having determined G_U and G_L for a column section, the effective length factor, K, is determined at the intersection of the straight line between the appropriate points on the scales for G_U and G_L with the scale for K.

G.7

The nomograph may be used to determine the effective length factors for the in-plane behaviour of compression members of trusses designed as axially loaded members even though the joints are rigid. In this case, there should be no in-plane eccentricities and all members of the truss meeting at the joint should not reach their ultimate load simultaneously. If it cannot be shown that all members at the joint do not reach their ultimate load simultaneously, the effective length factor of the compression members should be taken as 1.0.

Figure G.1
Nomograph for effective lengths column in continuous frames
(See Clause G.2.)



Sidesway prevented

Annex H (informative)
Deflections of composite beams, joists, and trusses due to shrinkage of concrete

Note: *This Annex is an informative (non-mandatory) part of this Standard.*

H.1
Shrinkage-induced deflections result from the following process. Concrete decreases in volume as it cures, at first rapidly and then at a decreasing rate. When restrained, tensile strains and therefore tensile stresses can develop in the concrete. (It can even crack if the tensile strength is reached.)

A curing slab is restrained by the steel shape to which it is connected.

H.2
Figure H.1 shows the shrinkage strains that develop through the depth for a composite beam and the corresponding equilibrium conditions for unshored construction. It is evident that unshored composite members will deflect downward. (Shoring reduces the shrinkage deflection substantially, especially in the early stages when the rate of shrinkage is the greatest.)

H.3
Branson's (1964) method is used in this Standard to determine shrinkage deflections. As illustrated in Figure H.2 a), the first step in the method is to assume temporarily that the shrinkage of the concrete slab is not restrained by connection to the steel beam. The connection between the concrete slab and the beam is accounted for in two additional steps. First, a tensile force is applied to the centroid of the unrestrained slab so that the displacement of the slab under the force is equal to the unrestrained shrinkage displacement [see Figure H.2 b)]. Compatibility is satisfied in this step. Second, equilibrium is satisfied by applying an equal and opposite force to the composite section [see Figure H.2 c)].

The method does not account for the cracking of concrete in tension, the non-linear stress-strain relationship of concrete, and other factors. To account for these factors and match theory with test results, the free shrinkage of the concrete is multiplied by an empirical coefficient.

The method gives reasonable results when an appropriate value is used for the empirical coefficient and suitable values are used for the free shrinkage and modular ratio.

H.4
The shrinkage deflection is directly proportional to the assumed free shrinkage strain. The free shrinkage strain depends on concrete properties such as the water/cement ratio, percentage of fines, entrained air, cement content, and curing conditions. A value of 583×10^{-6} may be used if other data are not available. This value was determined for composite beams supporting 75 mm concrete topping on 75 mm deck (150 mm total thickness) for inside conditions (see Ghali, et al. (2002), Annex A.2).

H.5
The modular ratio is calculated from the age-adjusted effective modulus of concrete, which in turn depends on the aging and creep coefficients. These coefficients may be taken as 0.73 and 2.7, respectively, if other data are not available. These coefficients were determined for the composite beams described in Clause H.4, assuming the age at loading is 7 days (see Ghali, et al. (2002), Annexes

A.7 and A.2, respectively). The shrinkage deflection is not sensitive to the modular ratio because both the transformed moment of inertia of the composite beam and the distance, *y*, vary with it.

H.6

The procedure in this Standard is used for determining the shrinkage deflections of simply supported composite beams, joists, and trusses. For many structural configurations, moments develop at the ends of beams, joists, and trusses as a result of partial or full continuity with adjacent members. It is often appropriate to account for continuity with adjacent members when determining shrinkage deflections.

H.7

Kennedy and Brattland (1992) propose an alternative method to determine shrinkage deflections. The method uses strain compatibility between steel and concrete, and a time-dependent modulus of elasticity of concrete in tension [see Shaker and Kennedy (1991)]. It is iterative because the concrete response is non-linear. It is more difficult to use than the method specified in this Standard; however, the tensile stress-strain relationship of the concrete is satisfied.

H.8

Montgomery et al. (1983) give an example where the shrinkage deflections were excessive. Jent (1989) provides information on shrinkage effects on continuous composite beams.

H.9

Branson, D.E. (1964). *Time-dependent effects on composite concrete beams. Proceedings, American Concrete Institute Journal,* 61, 212–229.

Ghali, A., Favre, R. and Elbadry, M. (2002). *Concrete structures: Stresses and deformations,* 3rd ed. London: Spon Press.

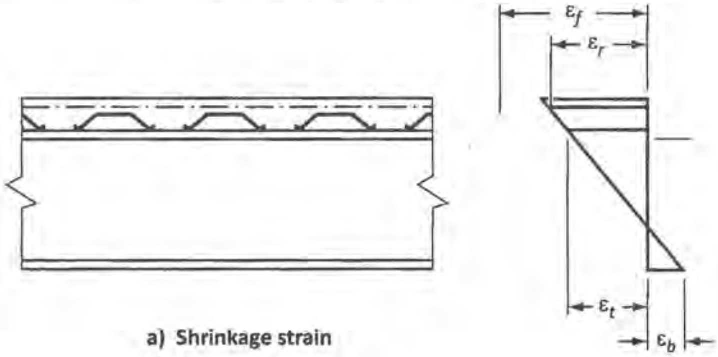
Jent, K.A. (1989). *Effects of shrinkage, creep and applied loads on continuous deck-slab composite beams.* M.Sc. thesis, Queen's University, Kingston, Ontario.

Kennedy, D.J.L. and Brattland, A. (1992). "Shrinkage tests of two full-scale composite trusses". *Canadian Journal of Civil Engineering,* 19 (2), 296–309.

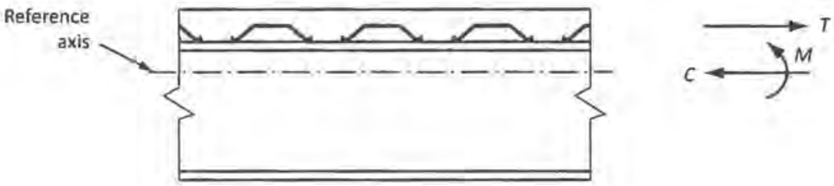
Montgomery, C.J., Kulak, G.L. and Shwartsburd, G. (1983). "Deflection of a composite floor system". *Canadian Journal of Civil Engineering,* 10 (2), 192–204.

Shaker, A.F. and Kennedy, D.J.L. (1991). *The effective modulus of elasticity of concrete in tension.* Structural Engineering Report 172, Department of Civil Engineering, University of Alberta, Edmonton.

Figure H.1
Composite beam subject to shrinkage forces
(See Clause H.2.)



a) Shrinkage strain

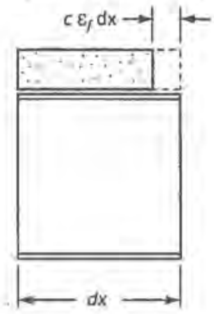


b) Free-body diagram

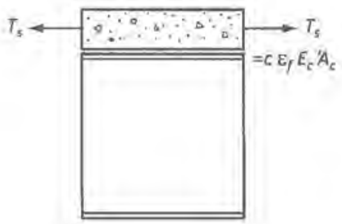
Legend:

ε_f = free shrinkage strain of the concrete
ε_r = resulting restrained shrinkage strain
ε_t = compressive strain at top of steel beam
ε_b = tensile strain at bottom of steel beam
T = tensile force in concrete
C = compressive force in steel beam
M = moment in steel beam required for equilibrium about reference axis

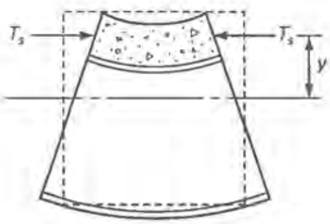
Figure H.2
Composite beam subject to shrinkage forces
(See Clause H.3.)



a) Unrestrained shrinkage of concrete slab



b) Enforce compatibility



c) Satisfy equilibrium

Legend:

c = empirical coefficient used to match theory with test results, which may be taken as 0.5

T_s = tensile force applied at centroid of unrestrained slab

A_c = effective area of concrete slab (for metal deck spanning perpendicular to the beam, the concrete area is taken above the flutes, and for metal deck parallel to the beam, the full concrete area is taken)

y = distance from centroid of effective area of concrete slab to the centroidal axis of the composite steel beam

E_c' = age-adjusted effective modulus of elasticity of concrete

E_f = unrestrained shrinkage strain of the concrete slab

Annex I (informative)
Arbitration procedure for pretensioning connections

Note: *This informative Annex has been written in mandatory language to facilitate adoption by anyone wishing to do so.*

I.1 General

For pretensioned connections, when there is disagreement concerning the results of inspection of bolt pretensioning procedures, the following arbitration procedure shall be used unless an alternative has been specified:

a) The inspector shall use a manual or power torque inspection wrench capable of indicating a selected torque value.

b) Three bolts of the same grade and diameter as those under inspection and representative of the lengths and conditions of those in the structure shall be placed individually in a calibration device that indicates bolt tension. There shall be a washer under the part turned if washers are so used in the structure or, if no washer is used, the material abutting the part turned shall be of the same specification and condition as that in the structure.

c) When the inspection wrench is a manual wrench, each bolt specified in Item b) shall be pretensioned in the calibration device by any convenient means to an initial tension of approximately 15% of the required bolt tension and then to the minimum tension specified for its size in Table 7. Tightening beyond the initial condition shall not produce greater nut rotation than that permitted by Table 8. The inspection wrench shall then be applied to the tightened bolt, and the torque necessary to turn the nut or head an additional 5° shall be determined. The average torque measured in the tests of three bolts shall be taken as the job inspection torque to be used in the manner specified in Item e).

d) When the inspection wrench is a power wrench, it shall first be applied to produce an initial tension of approximately 15% of the required fastener tension and then adjusted so that it will tighten each bolt specified in Item b) to a tension of 5% to 10% greater than the minimum tension specified for its size in Table 7. This setting of the wrench shall be taken as the inspection torque to be used in the manner specified in Item e). Tightening beyond the initial condition shall not produce greater nut rotation than that permitted by Table 8.

e) Bolts represented by the sample prescribed in Item b) that have been tightened in the structure shall be inspected by applying, in the tightening direction, the inspection wrench and its job inspection torque to 10% of the bolts, but not less than two bolts, selected at random in each connection. If no nut or bolt head is turned by this application of the job inspection torque, the connection shall be accepted as properly tightened. If any nut or bolt head is turned by the application of the job inspection torque, this torque shall be applied to all bolts in the connection and all bolts whose nut or head is turned by the job inspection torque shall be tightened and re-inspected. Alternatively, the fabricator or erector may choose to retighten all the bolts in the connection and then resubmit the connection for the specified inspection.

Annex J (normative)
Qualification testing provisions for seismic moment connections and buckling restrained braces

Notes:
1) *Where the physical testing alternative as permitted in Clauses 27.2.5.1 b), 27.4.4.1 c), or 27.7.8.1 is chosen as the basis for design of these connections, this Annex serves as a normative (mandatory) part of the Standard.*
2) *This Annex serves as a normative (mandatory) part of the Standard to describe the qualification testing of buckling restrained braces prescribed in Clause 27.8.6.*

J.1 Seismic moment connections

J.1.1
Clause J.1 specifies testing protocols that aim to demonstrate the deformation and strength characteristics of moment-resisting connections in moment-resisting frames, eccentrically braced frames, and plate walls that permit the frames to achieve specified interstorey drift capacity when the connections are designed using the full-scale physical testing alternative as provided in Clause 27.2.5.1 b), Clause 27.4.4.1 c), or Clause 27.7.8.1.

J.1.2
Extensive physical testing and analytical studies conducted over the last two decades have advanced the knowledge of behaviour of several connection types now used for construction of ductile moment-resisting frames (ANSI/AISC 341 and CISC 2014). However, availability of physical test data for other connection types and configurations, and link-to-column connections used in eccentrically braced frames lags behind. Clause J.1 provides the requirements and guidance for such physical tests.

J.1.3
The test assemblies shall represent the size, detailing, and fabrication of the prototype, in recognition of the effects of size, bracing arrangements, welding details, and welding procedures on the inelastic cyclic behaviour of the connection type. The test loading shall represent both the deformation magnitude and cyclic nature expected in a severe seismic event. These tests shall comply with the requirements provided in Section K2 of ANSI/AISC 341, except that the criteria of acceptance as pertain to interstorey drift capacity shall comply with the appropriate clauses in this Standard. However, the provisions for welds and welding in accordance with CSA W59 and W48, instead of those in AWS D1.1 where referenced in ANSI/AISC 341, might apply. Existing test data for successful tests conducted in accordance with testing protocols as given in publications by the U.S. Applied Technology Council (ATC-24) and U.S. Federal Emergency Management Agency (FEMA350) shall nonetheless remain valid.

J.2 Buckling restrained braces

J.2.1
Clause J.2 specifies testing protocols for full-scale qualification testing of bracing members in buckling restrained braced frames, as prescribed in Clause 27.8.6.

Note: *The cyclic inelastic response of bracing members in buckling restrained braced frames heavily depends on the design, detail and fabrication of the buckling restrained braces. Physical testing is prescribed to demonstrate the cyclic inelastic performance of the members and obtain design values for the maximum tension and compression forces that are expected to develop in the buckling restrained member at maximum anticipated axial* *deformations. This Clause provides the requirements for such physical tests and references to test data. The information given in this Clause may be used for the testing of other bracing members designed to dissipate seismic input energy through nonlinear axial response.*

J.2.2

The test specimens shall represent the size, detailing, and fabrication of the prototypes, in recognition of the effects of cross-section size, shape and orientation of the steel core and the material and method of separation between the steel core and the buckling restraining mechanism on the inelastic cyclic behaviour of the buckling restrained braces. The test loading shall represent both the deformation magnitude and cyclic nature expected in a severe seismic event. The tests shall comply with the requirements provided in Section K3 in Chapter K of ANSI/AISC 341, except that the criteria of acceptance as pertain to deformation capacity shall comply with Clause 27.8.6.

J.3 Bibliography

AISC. (2010). ANSI/AISC 341-10, *Seismic provisions for structural steel buildings,* American Institute of Steel Construction (AISC),Chicago, Illinois.

AISC. (2011). ANSI/AISC 358-10 and ANSI/AISC 358s1-11, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications,* including Supplement No. 1,American Institute of Steel Construction (AISC),Chicago, Illinois.

ATC. (1992). *Guidelines for seismic testing of components of steel structures*. ATC-24. Redwood City, California.

CISC. (2014). *Moment connections for seismic applications*. Canadian Institute of Steel Construction, Markham, Ontario.

FEMA. (2000a). *Recommended seismic design criteria for new steel moment-frame buildings*. Report FEMA350. Washington, D.C.

FEMA. (2000b). *State of art report on connection performance*. Report FEMA355D. Washington, D.C.

Annex K (normative)
Structural design for fire conditions

Note: *This Annex is a normative (mandatory) part of the Standard.*

K.1 General

K.1.1 Scope
This Annex specifies criteria for the design and evaluation of structural steel components, systems and frames for fire conditions. These criteria provide for the determination of the heat input, thermal expansion, and degradation in mechanical properties of materials that cause progressive decreases in strength and stiffness of structural components and systems at elevated temperatures.

K.1.2 Definitions
This Annex uses the following terms in addition to the terms defined in Clause 2:

Active fire protection — building materials and systems that are activated by a fire to mitigate adverse effects or to notify people to take some action to mitigate adverse effects.

Convective heat transfer — the transfer of thermal energy from a point of higher temperature to a point of lower temperature through the motion of an intervening medium.

Design-basis fire — a set of conditions that define the development of a fire and the spread of combustion products throughout a building or portion thereof.

Elevated temperatures — heating conditions experienced by building elements or structures as a result of fire, which are in excess of the anticipated ambient conditions.

Fire — destructive burning, as manifested by one or more of light, flame, heat, or smoke.

Fire endurance — a measure of the elapsed time during which a material or assembly continues to exhibit fire resistance.

Fire resistance — the property of assemblies that prevents or retards the passage of excessive heat, hot gases, or flames under conditions of use and enables them to continue to perform a stipulated function.

Fire resistance rating — the period of time a building element, component, or assembly maintains the ability to contain a fire, continues to perform a given structural function, or both, as determined by test or methods based on tests.

Fire separation — a construction assembly that acts as a barrier against the spread of fire and whose construction is formed of fire-resisting materials and tested in accordance with CAN/ULC-S101, or another approved standard fire resistance test, to demonstrate compliance with requirements prescribed by the regulatory authority.

Flashover — the rapid transition to a state of total surface involvement in a fire of combustible materials within an enclosure.

Heat flux — radiant energy per unit surface area.

Heat release rate — the rate at which thermal energy is generated by a burning material.

Passive fire protection — building materials and systems whose ability to resist the effects of fire does not rely on any outside activating condition or mechanism.

Performance-based design or objective-based design — an engineering approach to structural design that is based on agreed-upon performance goals and objectives, engineering analysis, and quantitative assessment of alternatives against the performance goals and objectives using accepted engineering tools, methodologies, and performance criteria.

Prescriptive design — design methods, e.g., specific technical requirements or deemed-acceptable solutions that document specific compliance with general criteria established by the regulatory authority.

Restrained construction — floor and roof assemblies and individual beams in buildings where the surrounding or supporting structure is capable of resisting substantial thermal expansion throughout the range of anticipated elevated temperatures.

Unrestrained construction — floor and roof assemblies and individual beams in buildings that are assumed to be free to rotate and expand throughout the range of anticipated elevated temperatures.

K.1.3 Performance objectives

Structural components, members, and building frame systems shall be designed so as to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy.

Deformation criteria shall be applied where the means of providing structural fire resistance or the design criteria for fire barriers require consideration of the deformation of the load-carrying structure.

Within the compartment of fire origin, forces and deformations from the design-basis fire shall not cause a breach of horizontal or vertical fire separation.

K.1.4 Design by engineering analysis

The analysis methods specified in Clause K.2 may be used to document the anticipated performance of steel framing when subjected to design-basis fire scenarios. These methods provide evidence of compliance with the performance objectives established in Clause K.1.3.

The analysis methods specified in Clause K.2 may be used to demonstrate an equivalency for an alternative material or method, as permitted by the regulatory authority.

K.1.5 Load combination and required resistance

The required resistance of the structure and its elements shall be determined based on the following gravity load combination specified in *User's Guide — NBC 2010: Structural Commentaries (Part 4)* Commentary A, Paragraph 25 ("Load Combination for Determination of Fire Resistance"):

$D + T_S + (\alpha L \text{ or } 0.25S)$

where

D = specified dead load, as given in Clause 6.2.1

T_S = effects due to expansion, contraction, or deflection caused by temperature changes due to the design-basis fire specified in Clause K.2.2. T_S can be taken equal to zero for statically determinate structures or for structures that have sufficient ductility to allow the redistribution of temperature forces before collapse

α = 1.0 for storage areas, equipment areas, and service rooms and 0.5 for other occupancies

L = specified occupancy live load, as given in Clause 6.2.1

S = specified variable load due to snow, as given in Clause 6.2.1

Notional lateral loads, in accordance with Clause 8.4.1, shall be applied in combination with this gravity load combination.

K.2 Structural design for fire conditions by analysis

K.2.1 General

Structural members, components, and building frames may be designed for elevated temperatures due to fire in accordance with this Clause.

K.2.2 Design-basis fire

K.2.2.1 General

A design-basis fire shall be identified to describe the heating conditions for the structure. These heating conditions shall relate to the fuel commodities and compartment characteristics present in the assumed fire area. The fuel load density based on the occupancy of the space shall be considered when determining the total fuel load. Heating conditions shall be specified in terms of a heat flux or temperature of the upper gas layer created by the fire. The variation of the heating conditions with time shall be determined for the duration of the fire.

When the analysis methods specified in Clause K.2 are used to demonstrate an equivalency as an alternative material or method as permitted by the regulatory authority, the design-basis fire shall be determined in accordance with CAN/ULC-S101.

K.2.2.2 Localized fire

Where the heat release rate from the fire is insufficient to cause flashover, a localized fire exposure shall be assumed. In such cases, the fuel composition, arrangement of the fuel array, and floor area occupied by the fuel shall be used to determine the radiant heat flux from the flame and smoke plume to the structure.

K.2.2.3 Post-flashover compartment fires

Where the heat release rate from the fire is sufficient to cause flashover, a post-flashover compartment fire shall be assumed. The determination of the temperature versus time profile resulting from the fire shall include fuel load, ventilation characteristics to the space (natural and mechanical), compartment dimensions, and thermal characteristics of the compartment boundary.

K.2.2.4 Exterior fires

The exposure of exterior structure to flames projecting from windows or other wall openings as a result of a post-flashover compartment fire shall be considered along with the radiation from the interior fire through the opening. The shape and length of the flame projection shall be used along with the distance between the flame and the exterior steelwork to determine the heat flux to the steel. The method specified in Clause K.2.2.3 shall be used for describing the characteristics of the interior compartment fire.

K.2.2.5 Fire duration

The fire duration in a particular area shall be determined by considering the total combustible mass, i. e., fuel load, available in the space. In the case of a localized fire or post-flashover compartment fire, the time duration shall be determined as the total combustible mass divided by the mass loss rate, except where determined from Clause K.2.2.3.

K.2.2.6 Active fire protection systems

The effects of active fire protection systems shall be considered when describing the design-basis fire.

Where automatic smoke and heat vents are installed in non-sprinklered spaces, the resulting smoke temperature shall be determined from calculation.

K.2.3 Temperatures in structural systems under fire conditions

Temperatures within structural members, components, and frames due to the heating conditions posed by the design-basis fire shall be determined by a heat transfer analysis.

K.2.4 Material properties at elevated temperatures

K.2.4.1 General

Material properties at elevated temperatures shall be determined from test data. In the absence of such data, the material properties specified in Clause K.2.4 may be used. These reduction factors shall not apply to steels with a yield strength in excess of 450 MPa or concretes with specified compression strength in excess of 55 MPa.

K.2.4.2 Thermal elongation

The following thermal elongation requirements shall apply:

a) Thermal expansion of structural and reinforcing steels: for calculations at temperatures above 65 °C, the coefficient of thermal expansion shall be 1.4×10^{-5}/°C.
b) Thermal expansion of normal weight concrete (NWC): for calculations at temperatures above 65 °C, the coefficient of thermal expansion shall be 1.8×10^{-5}/°C.
c) Thermal expansion of lightweight concrete (LWC): for calculations at temperatures above 65 °C, the coefficient of thermal expansion shall be 7.9×10^{-6}/°C.

K.2.4.3 Mechanical properties at elevated temperatures

The deterioration in strength and stiffness of structural members, components, and systems shall be taken into account in the structural analysis of the frame. The values F_{ym}, F_{pm}, F_{um}, E_m, f'_{cm}, E_{cm}, ε_{cu}, F_{ubm}, and F_{sbm} at elevated temperature to be used in structural analysis, expressed as the ratio with respect to the property at ambient temperature (assumed to be 20 °C), shall be as specified in Tables K.1, K.2, and K.3. Interpolation between these values may be used. Table K.1 specifies the reduction factors for the stress-strain relationship for steel at the elevated temperatures shown in Figure K.1.

K.2.5 Structural design

K.2.5.1 General structural integrity

The structural frame shall be capable of providing adequate strength and deformation capacity to withstand, as a system, the structural actions developed during the fire within the prescribed limits of deformation. The structural system shall be designed to sustain local damage, with the structural system as a whole remaining stable.

Continuous load paths shall be provided to transfer all forces from the exposed region to the final point of resistance. The foundation shall be designed to resist the forces and to accommodate the deformations developed during the design-basis fire.

K.2.5.2 Strength requirements and deformation limits

Conformance of the structural system to the requirements of this Annex shall be demonstrated by constructing a mathematical model of the structure based on principles of structural mechanics and evaluating this model for the internal forces and deformations in the members of the structure developed by the temperatures from the design-basis fire.

Individual members shall be provided with adequate strength to resist the shears, axial forces, and moments determined in accordance with this Annex.

Connections shall develop the strength of the connected members or the forces specified in this Clause. Where the means of providing fire resistance necessitates consideration of deformation criteria, the deformation of the structural system, or members thereof, under the design-basis fire shall not exceed the prescribed limits.

K.2.5.3 Methods of analysis

K.2.5.3.1 Advanced methods of analysis

The advanced methods of analysis may be used for the design of steel building structures for fire conditions. The design-basis fire exposure shall be that determined in accordance with Clause K.2.2. The analysis shall include both a thermal response and the mechanical response to the design-basis fire.

The thermal response shall produce a temperature field in each structural element as a result of the design-basis fire and shall incorporate temperature-dependent thermal properties of the structural elements and fire-resistive materials in accordance with Clause K.2.3.

The mechanical response results in forces and deflections in the structural system subjected to the thermal response calculated from the design-basis fire. The mechanical response shall explicitly take into account the deterioration in strength and stiffness with increasing temperature, the effects of thermal expansions, and large deformations. Boundary conditions and connection fixity shall represent the proposed structural design. The material properties shall be as specified in Clause K.2.4.

The resulting analysis shall consider all relevant limit states, e.g., excessive deflections, connection fractures, and overall or local buckling.

K.2.5.3.2 Simple methods of analysis

The simple methods of analysis specified in this Clause are applicable to the evaluation of the performance of individual members at elevated temperatures during exposure to fire.

The support and restraint conditions (forces, moments, and boundary conditions) applicable at normal temperatures may be assumed to remain unchanged throughout the fire exposure.

The thermal response may be modeled using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire specified in Clause K.2.2. The maximum steel temperature, T, obtained from this analysis shall be assumed constant through the member cross-section and shall be used to determine the factored resistances of the members in Items a) to f) as follows:

a) Tension members: the factored resistance of a tension member shall be determined as specified in Clause 13.2, using steel properties as specified in Clause K.2.4, with the temperature equal to the maximum steel temperature.

b) Compression members: the factored resistance of a compression member shall be determined as specified in Clause 13.3 using steel properties as specified in Clause K.2.4; however, for steel temperatures equal to or greater than 200 °C, the factored compressive resistance for flexural buckling shall be determined as follows:

$C_r(T) = (1+\lambda(T)^{2dn})^{-1/dn} A F_y(T)$

where

$C_r(T)$ = the factored compressive resistance at temperature, T

$$\lambda(T) = \frac{KL}{r}\sqrt{\frac{F_y(T)}{\pi^2 E(T)}} = \sqrt{\frac{F_y(T)}{F_e(T)}}$$

d = 0.6

n = as specified in Clause 13.3.1

c) Flexural members: the factored shear and moment resistance of a flexural member shall be as specified in Clauses 13.4 to 13.6 using steel properties specified in Clause K.2.4; however, for steel temperatures equal to or greater than 200 °C, the bending strength for lateral-torsional buckling of laterally unsupported doubly-symmetric members shall be determined as follows:

$$M_r(T) = C_K M_p(T) + (1 - C_K)M_p(T)\left(1 - \left(\frac{C_K M_p(T)}{M_u(T)}\right)^{0.5}\right)^{C_z(T)}$$

where

C_K = 0.12

$M_p(T)$ = the plastic moment at elevated temperatures determined using $F_y(T)$

$M_u(T)$ = the elastic critical load at elevated temperatures, determined as follows:

$$M_u(T) = \frac{\omega_2 \pi}{L}\sqrt{E(T) I_y G(T) J + I_y C_w \left(\frac{\pi E(T)}{L}\right)^2}$$

where

ω_2 = as defined in Clause 13.6

$$C_z(T) = \frac{T + 800}{500} \le 2.4$$

d) Combined axial force and flexure: the factored resistance of a member required to resist both bending moments and an axial tensile or compression forces shall be determined as specified in Clauses 13.8 and 13.9 using steel properties specified in Clause K.2.4 and flexural and axial strengths as specified in Clause K.2.5.3.2 a) to c).

e) Composite floor members: the thermal response of flexural elements supporting a concrete slab may be modelled using a one-dimensional heat transfer equation to calculate the maximum temperature of the bottom flange of the steel section. This temperature shall be taken as constant between the bottom flange to the mid-depth of the web and shall decrease linearly from the mid-depth of the web to the top flange of the steel beam by no more than 25%.
The factored resistance of a composite flexural member shall be determined as specified in Clause 17 using steel properties specified in Clause K.2.4.

f) Other components and connections: the factored resistance of other components and connections shall be as specified in Clause 13. Factored resistances shall be calculated using steel properties specified in Clause K.2.4 at the maximum temperature determined by the design-basis fire.

K.3 Bibliography

European Committee for Standardization

EN 1992-1-2:2004
Eurocode 2: Design of concrete structures — Part 1-2: General rules — Structural fire design

EN 1993-1-2:2005
Eurocode 3: Design of steel structures — Part 1-2: General rules — Structural fire design

EN 1994-1-2:2005
Eurocode 4: Design of composite steel and concrete structures — Part 1-2: General rules — Structural fire design

Other publications

Takagi, J., and Deierlein, G. 2009. *Proposed design equations for CAN/CSA-S16* Annex K *provisions for steel members at high temperatures.* Report prepared for the Canadian Institute of Steel Construction, Markham, ON.

Table K.1
Reduction factors for stress-strain relationship of steel at elevated temperatures (Eurocode 3 and Eurocode 4)

(See Clause K.2.4.3.)

	Reduction factors at temperature, T_{steel}, relative to the value of F_y or E at 20 °C			
Steel temperature, T_{steel}, °C	**Reduction factor (relative to E) for the slope of the linear elastic range, $k_E = E_m/E$**	**Reduction factor (relative to F_y) for proportional limit, $k_p = F_{pm}/F_y$**	**Reduction factor (relative to F_y) for effective yield strength, $k_y = F_{ym}/F_y$**	**Reduction factor (relative to F_y) for effective tensile strength, $k_u = F_{um}/F_y$**
20	1.00	1.00	1.00	1.25
100	1.00	1.00	1.00	1.25
200	0.90	0.807	1.00	1.25
300	0.80	0.613	1.00	1.25
400	0.70	0.420	1.00	1.00
500	0.60	0.360	0.78	0.78
600	0.31	0.180	0.47	0.47
700	0.13	0.075	0.23	0.23
800	0.09	0.050	0.11	0.11
900	0.0675	0.0375	0.06	0.06
1000	0.0450	0.0250	0.04	0.04
1100	0.0225	0.0125	0.02	0.02
1200	0.00	0.00	0.00	0.00

Legend:

E = elastic modulus of steel (200 000 MPa assumed; earthquake loads and effects)

E_m = slope of the linear elastic range for steel at elevated temperature T_{steel}

F_{pm} = proportional limit for steel at elevated temperature T_{steel}

F_{um} = effective tensile strength of steel at elevated temperature T_{steel}

F_y = specified minimum yield stress, yield point, or yield strength

F_{ym} = effective yield strength of steel at elevated temperature T_{steel}

k_E = slope of linear elastic range, relative to slope at 20 °C

k_p = proportional limit, relative to yield strength at 20 °C

k_u = effective tensile strength, relative to yield strength at 20 °C

k_y = effective yield strength, relative to yield strength at 20 °C

Table K.2
Values for the main parameters of the stress-strain relationships of normal weight concrete (NWC) and lightweight concrete (LWC) at elevated temperatures (Eurocode 2 and Eurocode 4)

(See Clause K.2.4.3.)

Concrete temperature, $T_{concrete}$, °C	Reduction factor (relative to f_c') for effective compressive strength, $k_c = f_{cm}'/f_c'$			ε_{cu}, %
	NWC	LWC	E_{cm}/E_c	NWC
20	1.00	1.00	1.00	0.25
100	1.00	1.00	0.92	0.40
200	0.95	1.00	0.75	0.55
300	0.85	1.00	0.59	0.70
400	0.75	0.88	0.43	1.00
500	0.60	0.76	0.26	1.50
600	0.45	0.64	0.10	2.50
700	0.30	0.52	0.083	2.50
800	0.15	0.40	0.067	2.50
900	0.08	0.28	0.050	2.50
1000	0.04	0.16	0.033	2.50
1100	0.01	0.04	0.017	2.50
1200	0.00	0.00	0.00	—

Legend:

E_c = elastic modulus of concrete

E_{cm} = tangent modulus of the stress-strain relationship of the concrete at elevated temperature $T_{concrete}$

f_c' = specified compressive strength of concrete at 28 days

f_{cm}' = effective value for the compressive strength of concrete at elevated temperature $T_{concrete}$

k_c = effective compressive strength relative to compressive strength at 20 °C

ε_{cu} = concrete strain corresponding to f_{cm}'

Note: *For LWC, values of ε_{cu} shall be obtained from tests.*

Strain range	Stress, σ	Tangent modulus
$\varepsilon \le \varepsilon_{pm}$	εE_m	E_m
$\varepsilon_{pm} < \varepsilon < \varepsilon_{ym}$	$F_{pm} - c + (b/a)\,[a^2 - (\varepsilon_{ym} - \varepsilon)^2]^{0.5}$	$\dfrac{b(\varepsilon_{ym} - \varepsilon)}{a\left[a^2 - (\varepsilon_{ym} - \varepsilon)^2\right]^{0.5}}$
$\varepsilon_{ym} \le \varepsilon \le \varepsilon_{tm}$	F_{ym}	0

(Continued)

(Continued)

Strain range	Stress, σ	Tangent modulus
$\varepsilon_{tm} \le \varepsilon \le \varepsilon_{um}$	$F_{ym}\,[1-(\varepsilon-\varepsilon_{tm})/(\varepsilon_{um}-\varepsilon_{tm})]$	—
$\varepsilon = \varepsilon_{um}$	0.00	—

Notes:

1) *Parameters:*
   a) $\varepsilon_{pm} = F_{pm}/E_m$
   b) $\varepsilon_{ym} = 0.02$
   c) $\varepsilon_{tm} = 0.15$
   d) $\varepsilon_{um} = 0.20$

2) *Functions:*
   a) $a^2 = (\varepsilon_{ym}-\varepsilon_{pm})\left(\varepsilon_{ym}-\varepsilon_{pm}+c/E_m\right)$
   b) $b^2 = c(\varepsilon_{ym}-\varepsilon_{pm})E_m + c^2$
   c) $c = \dfrac{\left(F_{ym}-F_{pm}\right)^2}{(\varepsilon_{ym}-\varepsilon_{pm})E_m - 2\left(F_{ym}-F_{pm}\right)}$

Table K.3
Properties of A325M/A325 and A490M/A490 high strength bolts at elevated temperatures
(See Clause K.2.4.3.)

Steel temperature, T_{bolt}, °C	F_{ubm} / F_{ub} or F_{sbm} / F_{sb}
20	1.00
100	0.97
200	0.93
300	0.89
400	0.75
500	0.54
600	0.27
700	0.12
800	0.07
900	0.03
1000	0.03
1100	0.00

Legend:

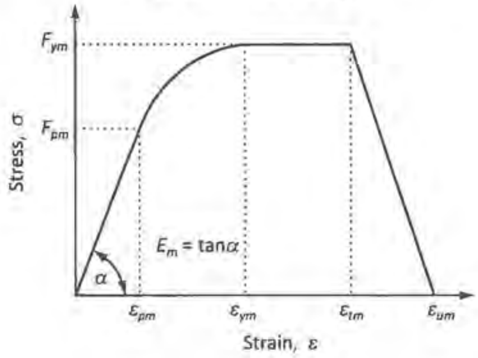
F_{ubm} = effective tensile strength of bolt at elevated temperature

F_{ub} = effective tensile strength of bolt

F_{sbm} = effective shear strength of bolt at elevated temperature

F_{sb} = effective shear strength of bolt

Figure K.1
Stress-strain relationship for steel at elevated temperatures (Eurocode 3)
(See Clause K.2.4.3.)



Legend:

E_m = slope of the linear elastic range

F_{pm} = proportional limit

F_{ym} = effective yield strength

ε_{pm} = strain at proportional limit

ε_{tm} = limiting strain for yield strength

ε_{um} = ultimate strain

ε_{ym} = yield strain

Annex L (informative)
Design to prevent brittle fracture

Note: *This Annex is an informative (non-mandatory) part of this Standard.*

L.1 General

Brittle fracture is a fracture mechanism accompanied by limited or no plastic deformation. Consequently, it is sudden and occurs with little to no warning, which makes it an undesirable failure mode that should be avoided by the adoption of a fracture control plan.

The design guidelines presented in this Annex are applicable to members and structural components subjected to tensile stresses arising from direct tension or bending when the rate of applied loading is high, e.g., dynamic or impact loading. Members and connections that contain notches, fabrication discontinuities, or other stress raisers need particular attention. Structures that are exposed to low temperatures are more susceptible to brittle fracture than those that are not. Although relatively uncommon, brittle fracture can also occur at normal temperatures, when fracture-sensitive details or metals (base or weld) with low notch toughness are subjected to dynamic tensile stresses.

The protected zones of seismically loaded structures should be designed to control brittle fracture.

When plates or heavy rolled sections are subjected to tensile stresses in the through-thickness direction, additional consideration needs to be given to the selection of the steel quality (see Clause L.3).

Statically loaded structures that are subjected to low temperature do not normally require the use of notch-tough steel. In these structures, brittle fracture can normally be avoided by following the design and fabrication criteria provided in this Standard and CSA W59. Special attention should be paid to anchor rods (see ASTM F1554) special loading conditions during construction, use of thick steel plates, fabrication procedures that give rise to high tensile residual stresses, and details that give rise to stress concentrations.

Designers should be aware that the availability of notch-tough steel is somewhat limited (see Clause L.3).

L.2 Material selection

The potential for brittle fracture depends mainly on the following factors (Barsom and Rolfe, 1999):
a) steel strength;
b) material thickness;
c) loading rate;
d) minimum service temperature;
e) material toughness; and
f) type of structural element.

These factors should be considered when selecting steel with appropriate notch toughness (Barsom, 1975; Barsom, 2002). Connection details and the presence of stress raisers also needs to be considered. Required notch toughness is expressed in terms of the test temperature at which the Charpy V-notch energy has a minimum value of 20 J or 27 J or 34 J, as specified in CSA G40.21.

The approach presented in Tables L.1 to L.4 consists of defining the Charpy V-notch energy level and the testing temperature for four different service temperature ranges. Figure L.1 can be used to determine the minimum service temperature appropriate for structural steel exposed to outdoor conditions. The testing temperature can be significantly different from the service temperature to account for the difference in strain rate between the Charpy impact test and the strain rate applied to the structure.

Dynamic loading and impact loading are recognized in Tables L.1 to L.4. Dynamic loading is applicable to intermediate strain rates such as those occurring in structures subjected to seismic ground motions, wave loads, or wind-induced vibration or truck traffic loading on highway bridges. The strain rates for such applications are typically around 10^{-3} sec^{-1}. Impact loading is applicable to high strain rates that occur, for example in explosive, crash conditions, or impact forces when large weights are dropped on structures. The strain rates for impact conditions are around 10 sec^{-1}. The selection of the applicable toughness level should be based on a judicious evaluation of the applicable strain rate.

When specifying steel for a specific application, the engineer should consider the probability of low temperature and extreme loading conditions occurring simultaneously. Care should also be taken not to specify excessively high fracture toughness since specification of higher than necessary fracture toughness can cause delays with sourcing the material.

The consequence of brittle fracture is recognized in the material selection. Fracture-critical members or joints are those for which local failure would cause complete structural collapse with serious consequences to life or very high cost. Primary tension members (tension and bending members or joints) are those for which failure would be restricted to localized areas not resulting in structural collapse. The fracture toughness of secondary framing members need not be considered.

The impact energy for the weld metal needs to be higher than for the base metal because welds usually have discontinuities, stress raisers, and high tensile residual stresses, which make weld metals more susceptible to brittle fracture. Given that the cost of weld metals is small relative to that of the structure, it is good practice to specify high-toughness filler metal to lower the risk of brittle fracture.

Table L.1
Recommended test temperatures and Charpy V-notch impact test values for primary tension members under dynamic loading
(See Clause L.2.)

Steel Grade	Base metal test temperature, °C, for minimum service temperature, T_s, °C				Weld metal test temperature, °C, for minimum service temperature, T_s, °C		
	Minimum average energy, J	$T_s > 0$	0 $T_s > -30$	$-30 > T_s > -52$	Minimum average energy, J	$T_s > -40$	$-30 > T_s > -52$
260 WT	20	20	0	−20	20	−30	−30
300 WT	20	20	0	−20	20	−30	−30
350 WT and AT	27	20	0	−20	27	−30	−30
400 WT and AT	27	20	0	−20	27	−30	−30
480 WT and AT	27	10	−10	−30	27	−30	−40
700 QT	48	0	−20	−35	48	−30	−40

Table L.2
Recommended test temperatures and Charpy V-notch impact test values for primary tension members under impact loading
(See Clause L.2.)

Steel Grade	Base metal test temperature, °C, for minimum service temperature, T_s, °C				Weld metal test temperature, °C, for minimum service temperature, T_s, °C		
	Minimum average energy, J	$T_s > 0$	0 $T_s > -30$	$-30 > T_s > -52$	Minimum average energy, J	$T_s > -40$	$-30 > T_s > -52$
260 WT	20	−30	−50	−70	20	−65	−75
300 WT	20	−25	−45	−65	20	−60	−70
350 WT and AT	27	−20	−40	−60	27	−55	−65
400 WT and AT	27	−20	−40	−60	27	−55	−65
480 WT and AT	27	−20	−40	−60	27	−55	−65
700 QT	48	−10	−30	−40	48	−40	−45

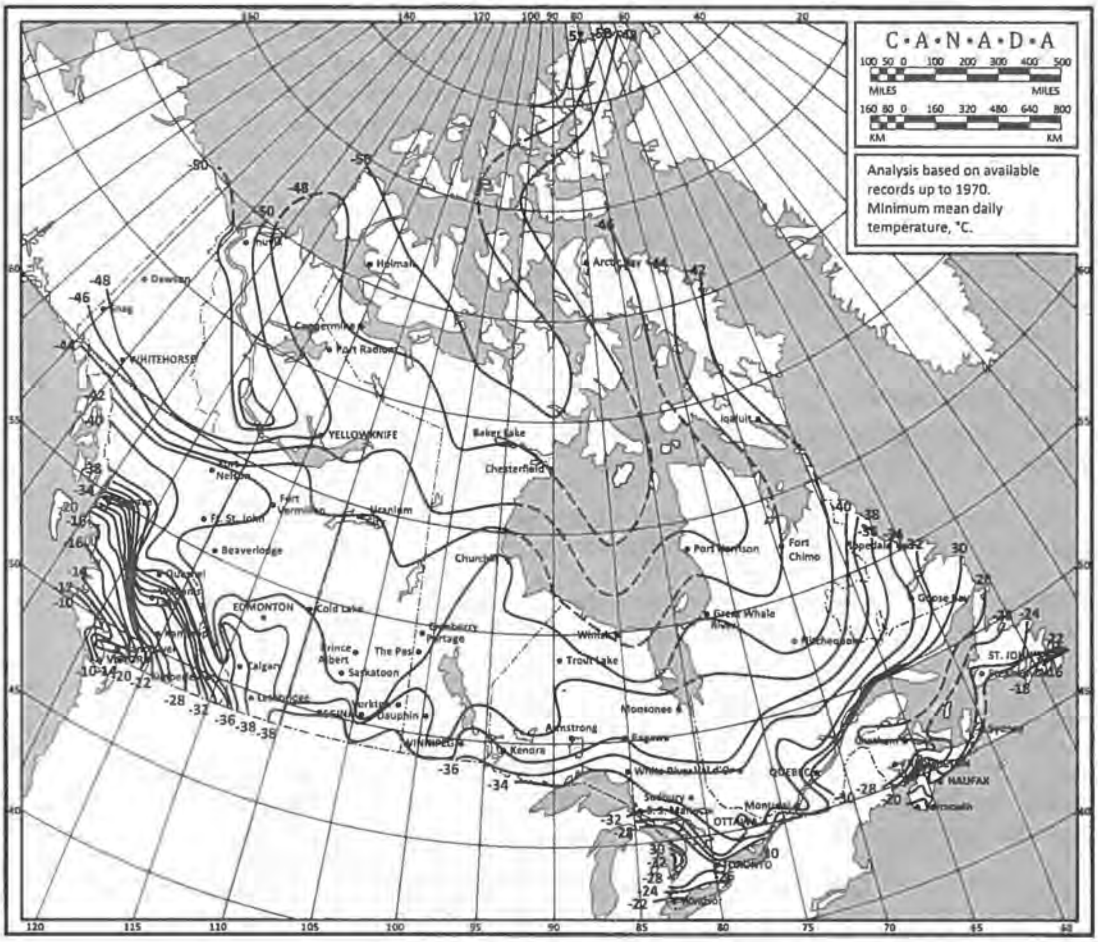
Table L.3
Recommended test temperatures and Charpy V-notch impact test values for fracture critical members under dynamic loading
(See Clause L.2.)

Steel Grade	Base metal test temperature, °C, for minimum service temperature, T_s, °C				Weld metal test temperature, °C, for minimum service temperature, T_s, °C		
	Minimum average energy, J	$T_s > 0$	0 $T_s > -30$	$-30 > T_s > -52$	Minimum average energy, J	$T_s > -40$	$-30 > T_s > -52$
260 WT	20	0	−20	−30	20	−30	−30
300 WT	20	0	−20	−30	20	−30	−30
350 WT and AT	27	0	−20	−30	27	−30	−30
400 WT and AT	27	0	−20	−30	27	−30	−40
480 WT and AT	27	0	−20	−40	27	−40	−50
700 QT	Not permitted						

Table L.4
Recommended test temperatures and Charpy V-notch impact test values for fracture critical members under impact loading
(See Clause L.2.)

Steel Grade	Base metal test temperature, °C, for minimum service temperature, T_s, °C				Weld metal test temperature, oC, for minimum service temperature, T_s, °C		
	Minimum average energy, J	$T_s > 0$	0 $T_s > -30$	$-30 > T_s > -52$	Minimum average energy, J	$T_s > -40$	$-30 > T_s > -52$
260 WT	20	−45	−65	−70	20	−70	−70
300 WT	20	−45	−60	−70	20	−70	−70
350 WT and AT	27	−35	−50	−70	27	−70	−70
400 WT and AT	27	−35	−50	−65	27	−70	−70
480 WT and AT	27	−30	−45	−65	27	−70	−70
700 QT	Not permitted						

Figure L.1
Minimum mean daily temperatures
(See Clause L.2.)



L.3 Steel availability

When selecting materials for structures at risk of brittle fracture, it is recommended to consult with suppliers regarding the availability of notch-tough materials. In general for Grade WT material, plates are more readily available than rolled shapes.

The inherent toughness of Grade W steel is often sufficient to prevent brittle fracture at low temperatures, although the required toughness is not guaranteed. CSA G40.21 type W steel may be substituted for CSA G40.21 type WT steel only when the Charpy impact energy requirements are verified by the submission of test documentation.

L.4 Control of discontinuities

In addition to specifying materials with appropriate toughness, the control of discontinuities is equally important to provide an acceptably low probability of brittle fracture.

Discontinuities can result from deliberate changes in geometry from the design or unintentional such as flaws in welded connections. Although it is impossible to avoid all flaws in welded construction, Welded structures should meet the requirements of CSA W59 for acceptable size of discontinuities.

Connection details should be designed to minimize stress raisers such as sharp corners and abrupt changes of stiffness resulting from changes in cross-section. Thick or high-strength materials are generally more susceptible to cold cracking in the heat-affected zones of welds and in areas of high residual stresses. In such cases, the choice of appropriate welding procedures is as important as the selection of the material.

When selecting construction details, the designer needs to account for the fact that some materials cannot be welded, or can be welded only under strict conditions:

a) Prestressing steels, anchor rods, and high-strength bolts cannot be welded.
b) High-carbon steels can be welded only under specific conditions. For welding of reinforcing steel, see CAN/CSA-G30.18.

L.5 Bibliography

ASTM F1554-07a (2011). *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength*, ASTM International, West Conshohocken, PA.

Barsom, J.M. (2002). "Development of fracture toughness requirements for weld metals in seismic applications", *Journal of Materials in Civil Engineering*, Vol. 14, No. 1, pp. 44-49.

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King, P. (2011). *Review of the Fracture Toughness Requirements for Weld Metal in the Canadian Highway Bridge Design Code, W793D Individual Studies Report*, Department of Materials Science and Engineering, The Ohio State University, May.

Medlock, R.D. (1998). *Qualification of Welding Procedures for Bridges: An Evaluation of the Heat Input Method*, MSE Thesis, Department of Civil Engineering, The University of Texas, Austin, Texas.

Annex M (informative) ***Seismic design of industrial steel structures***

Note: *This informative (non-mandatory) Annex has been written in normative (mandatory) language to facilitate adoption where users of the Standard or regulatory authorities wish to adopt it formally as additional requirements to this Standard.*

M.1 General

M.1.1

This Annex applies to industrial type structures that are expected not to respond to seismic ground motions in a fashion similar to conventional buildings because of non-uniform distribution of mass, strength and stiffness in the building, absence of clearly defined floors, or reduced damping due to limited architectural components. The intended use of these structures are essentially to support equipment and material for an industrial process that may significantly affect the structure seismic response, and do not include the shelter of persons. These provisions do not apply to warehouses or to office buildings for industrial complexes. These provisions do not apply to nuclear facilities.

M.1.2

All requirements in this Standard shall apply except as otherwise specified in this Annex.

M.2 Seismic force resisting systems

M.2.1 System restrictions

The seismic force resisting system shall be chosen from Table M.1.

M.2.2 System redundancy

When $I_E F_a S_a(0.2)$ is greater than 0.45 or $I_E F_a S_a(1.0)$ is greater than 0.30, structures that exceed 40 m in height and that are designed to resist seismic loads based on a ductility-related force modification factor, R_d, greater than 1.5 shall be either

a) configured in such a way that failure of any brace or brace connection, or any rigid beam-to-column joint, does not increase earthquake effects by more than 33% in the remaining members of the seismic force resisting system; or
b) designed to resist gravity loads combined with 1.3 times the seismic loads.

M.3 Analysis

M.3.1 Methods of analysis

Analysis for design earthquake actions shall be carried out in accordance with the Dynamic Analysis Procedure described in the *NBCC*, except that Sentences of 4.1.8.12.(6), 4.1.8.12.(8), 4.1.8.12.(9) and 4.1.8.12.(11) do not apply.

M.3.2 Damping coefficient

The design spectrum S(T) in the *NBCC* shall be multiplied by the damping coefficient $\beta = (0.05/\ \xi)^{0.4}$, where ξ = 0.02 for welded structures and 0.03 for bolted structures.

M.3.3 Effective mass

The effective mass corresponds to the seismic weight as defined in the *NBCC*, including the mass of the operational contents of tanks, vessels, bins, hoppers, piping, and other similar equipment.

When determining vertical earthquake effects, the effective mass must include 100% of the mass resulting from the probable accumulation of equipment and storage of materials.

The design of the supporting tower or pedestal, anchorage, and foundation for seismic overturning shall assume the material stored is a rigid mass positioned at its centre of gravity. The effects of fluid-structure interaction may be considered in determining the forces, effective period, and mass centroids of the system if the following requirements are met:

a) the sloshing period, T_c is greater than $3T$ where T = natural period of the tank with confined liquid (rigid mass) and supporting structure; and
b) the sloshing mechanism (i.e., the percentage of convective mass and centroid) is determined for the specific configuration of the container by detailed fluid-structure interaction analysis or testing.

M.3.4 Number of modes

When the modal response spectrum analysis method is used, the number of modes in each direction of analysis shall be sufficient to accurately represent the seismic dynamic response of the structure. As a minimum, the combined participating mass of all the modes included in the analysis should total at least 95% of the total mass.

M.3.5 Direction of analysis

When $I_EF_aS_a(0.2)$ is greater than 0.35 or $I_EF_vS_a(1.0)$ is greater than 0.30, columns that form part of two or more intersecting seismic force resisting systems shall be designed to resist 100% of the earthquake effects from the seismic loads applied in one direction plus 30% of the earthquake effects from seismic loads applied in the perpendicular direction, with the combination requiring the greater element strength being used in the design.

M.3.6 Vertical earthquake effects

When $I_EF_aS_a(0.2)$ is greater than 0.35 or $I_EF_vS_a(1.0)$ is greater than 0.30, vertical earthquake effects shall be considered for

a) bars and rods supporting hanging equipment, including their supports, and steel beams when the dead load of the equipment represents more than 75% of the total tributary gravity loads;
b) foundations, anchors, and bearings for structures and equipment; and
c) structures that can be affected by vertical seismic motion such as cantilevered structures.

Vertical earthquake effects shall be determined using the modal response spectrum analysis method with 2/3 the design spectrum S(T) in the *NBCC*, as multiplied by the damping coefficient defined in Clause M.2.2, and R_dR_o = 1.3.

M.4 Anchorage

M.4.1 Anchorage strength

Anchor rods shall be designed for earthquake forces corresponding to R_dR_o = 1.3, without exceeding forces corresponding to the probable resistance of the connected components determined using the probable yield stress R_yF_y.

M.4.2 Anchorage detailing

Where anchor rods are not threaded over their full length, they shall

a) be detailed such that $A_{aru}F_y \leq 0.75\ A_{ar}F_u$, where A_{aru} is the cross-section area of the unthreaded portion;
   and

b) have a minimum length of 75 mm of thread is left under the nut.

For anchorage of towers, chimneys, or cantilevered structures designed with R_d equal to or greater than 1.5, or anchorage of fixed base columns of structures designed with R_d equal to or greater than 1.5, the anchor rods shall have a stretch length sufficient to accommodate the expected inelastic elongation, but not less than the larger of 250 mm and 8 times the diameter of the anchor rod.

M.5 Special requirements

M.5.1
Design of columns in braced frames

When two orthogonal braced bays share a column, the column shall be design for the forces resulting from the braces in both orthogonal directions reaching their probable resistances simultaneously.

M.5.2 Tanks and vessels supported by buildings

Tanks and vessels that are supported within buildings or are incidental to the primary function of the tower are considered mechanical equipment and shall be designed in accordance with the *NBCC* Chapter 4.

M.5.3 Welded steel water storage structures

Welded steel elevated water storage structures shall be designed and detailed in accordance with the seismic requirements of AWWA D100 with the height limits imposed in Table M.1.

Table M.1
Seismic force resisting systems

Structure type	Clause*	Restrictions†				
		Cases where $I_E F_a S_a(0.2)$				Cases where $I_E F_v S_a(1.0)$
		< 0.2	≥ 0.2 to < 0.35	≥ 0.35 to ≤ 0.75	> 0.75	> 0.3
Elevated tanks, vessels, bins, or hoppers supported on symmetrically braced legs	27.5	NL	NL	60	60	60
Elevated tanks, vessels, bins or hoppers supported on asymmetrically braced legs	27.6	NL	NL	60	60	60
Elevated tanks, vessels, bins or hoppers supported on a single pedestal or skirt	27.11	NL	NL	60‡	60‡	60‡
Horizontal saddle supported welded steel vessels	27.5	NL	NL	NL	NL	NL
All other distributed mass cantilever structures not covered above including stacks, chimneys, silos and skirt supported vertical vessels	27.11	NL	NL	NL‡	NL‡	NL‡
Trussed towers (free standing or guyed)	27.6	NL	NL	NL	NL	NL
Guyed stacks and chimneys	27.6	NL	NL	NL	NL	NL
Cooling towers	27.5	NL	NL	60	60	60
	27.11	NL	NL	NL‡	NL‡	NL‡
Pole towers	27.11	NL	NL	NL‡	NL‡	NL‡
All other self-supporting structures, tanks or vessels not covered above		NL	NL	NL	NL	NL

* *Structures shall meet the requirements of this Clause, including the applicable factors R_d and R_o.*
† *NP = system is not permittedNL = system is permitted and not limited in height as an SFRS. Numbers in this Table are maximum height limits in m. The most stringent requirement governs.*
‡ *Earthquake effects shall be determined with R_dR_o = 1.0 when the height exceeds 40 m.*

① *Annex N (normative)* ***Design and construction of steel storage racks***

Note: *This Annex is a mandatory part of the Standard.*

N.1 Introduction

N.1.1

Steel storage racks are unique structures in part because they have semi-rigid frames with proprietary connections. They are high-performance structures in that they typically resist forces 20 to 50 times their self-weight, and often more in high seismic zones.

N.1.2

This Annex provides requirements for the design, fabrication, and installation of steel storage racks.

N.1.3

This Annex is to be used in conjunction with CSA A344.

N.1.4

This Standard is to be used to design hot rolled steel members of steel storage racks. Storage racks are not buildings, and Clauses of this Standard that are not pertinent to the design of storage racks are not requirements of this Annex.

Note: *The* National Building Code of Canada (NBCC) *does not define racks as buildings; however, they are seen as structures that are similar to buildings and can be designed as such in accordance with this Annex.*

N.2 General

N.2.1

This Annex shall be limited to free-standing, selective-type storage racks where the principal structural components are upright frames and load support beams.

Note: *Storage racks are typically used to store pallets and loaded by using powered lift equipment.*

The selective-type storage racks shall consist of regularly spaced, braced frames in the cross-aisle direction and parallel, multi-level moment resisting frames in the down-aisle direction.

Note: *There are limited racks produced with moment frames in both the down-aisle and cross-aisle directions. This introduces biaxial bending in the column, which has not been considered in the Annex. In addition, braced frames in the down-aisle direction are outside the scope of this edition of the Annex. Braced rack frames in the down-aisle direction are not precluded, provided that they are properly addressed in the design.*

N.2.2

This Annex shall apply only to racks made from steel, either hot-rolled or cold-formed. It shall not apply to racks of other configurations such as drive-in and drive-through racks, cantilever racks, and portable racks.

N.2.3

Certain parts of this Annex, particularly the structural demand criteria, may be used as guidance when designing rack structures outside the scope of this Annex as there are additional forces on these systems that are not considered in this Annex.

N.2.4

Unless otherwise specified in this Annex, the provisions of CSA S136 shall include the requirements of Appendix B (Provisions Applicable to Canada) of that Standard.

N.3 Definitions

The following definitions shall apply in this Annex:

Note: *This includes a list of terms of typical storage rack components (see Figure N.1). These definitions are consistent with those in CSA A344.*

Anchor — a post-installed mechanical or adhesive fastener used to secure a pallet rack structure to a building structure, e.g., the baseplate to the floor slab.

Back-to-back row — two parallel rows of pallet racks that are joined by one or more levels of row spacers between frames.

Base plate (footplate) — a plate fixed (usually by welding) to the bottom of frame columns to facilitate anchoring to the floor slab and to distribute the weight of the loaded column over a larger area of the floor slab.

Beam (load beam, load support beam, stringer) — a horizontal member, usually arranged in pairs such that the upper horizontal surfaces support pallets placed on them.

Note: *Beams are typically attached to frames by beam connectors welded to each end.*

Beam connector locking device — a device such as a lock pin or safety clip or mechanical fastener approved by the manufacturer used to help resist the accidental dislodgement of beam connectors from frames.

Beam connectors (beam brackets, end plates) — a formed, stamped, or punched part welded to each end of the beams to facilitate their attachment to the frames.

Column — the vertical member of a frame.

Cross-aisle ties — horizontal members that span across an aisle connecting the tops of two opposing frames.

Damping-of-the-load — energy dissipation due to the contents, as distinct from that inherent in the rack structure itself.

Diagonal brace — member welded or bolted diagonally between frame columns to resist cross-aisle forces imposed on frames and to brace the column.

Double posting — doubling of the front and/or rear frame columns to a specific height, usually by welding one column behind the other to create a composite section.

Frame (end frame, upright frame) — an assembly of two vertical members (columns) and braces that is used to support load beams.

Horizontal brace — members welded or bolted horizontally between frame columns to resist cross-aisle forces imposed on frames and to brace the columns.

Material handling equipment — lift trucks of all classes and other types of powered equipment used to directly or indirectly load and unload a pallet rack.

Owner — a person who purchases and/or directs the use of a pallet rack.

Note: *While the interests might not always be the same, in the context of this Annex, the terms "owner", "user", or designates of any of them (such as a consultant) have the same meaning.*

Pallet (skids) — a platform on which goods can be stacked to facilitate unit load transportation or storage on a pallet rack.

Pallet rack (storage rack, racking, racks) — a combination of frames, beams, and accessories used after assembly into a structure to support unit loads whether or not such loads are palletized.

Pallet safety bar — a member supported by and oriented perpendicular to load beams, intended to temporarily support misplaced pallets.

Pallet support bar — a bar similar to a safety bar except a pallet support bar is intended to support the full weight of a loaded pallet in cases where the size, strength, or style of a pallet prohibits its placement directly onto a pair of load beams.

Post guard (column guard, post protector) — a member or device designed to resist accidental impact to the column member.

Product load — a maximum load of pallets and products stored on the racks.

Row spacer — a member used to connect two aligned frames, one behind the other, ensuring that while joined they remain separated by a specific distance.

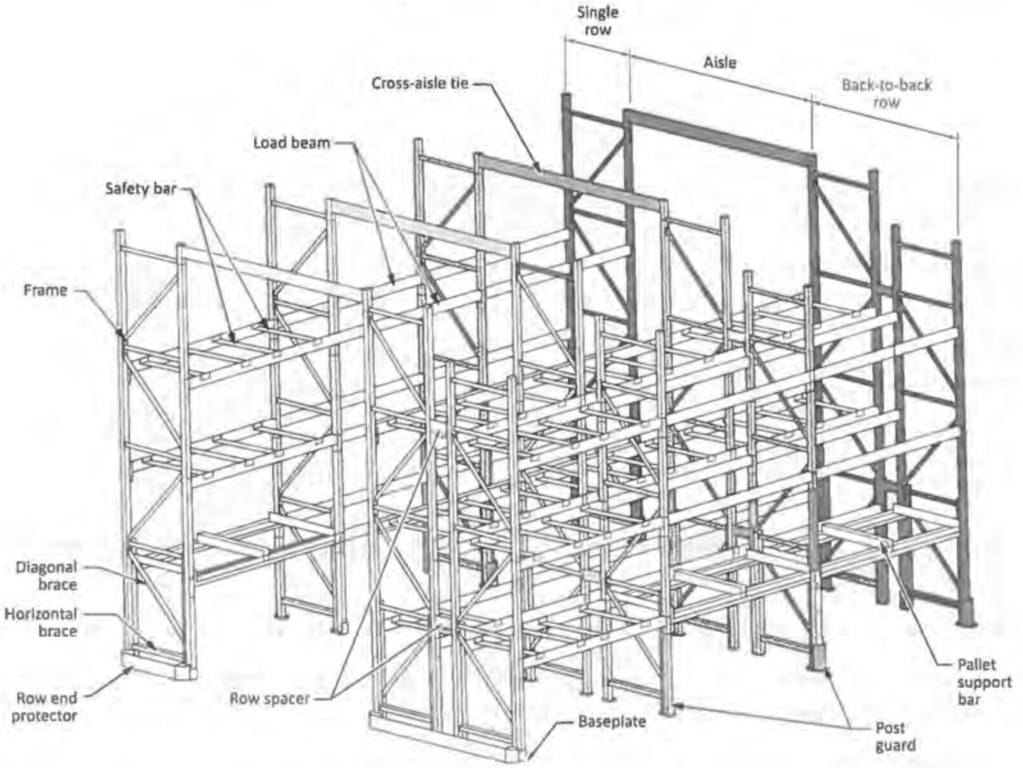
Selective pallet rack — a pallet rack arranged in one row or a series of rows such that every pallet loaded on the rack faces an aisle allowing direct access by material handling equipment.

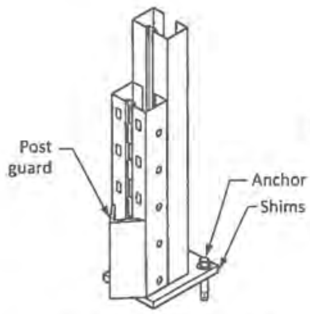
Single row — a row of a pallet rack that is not joined to an adjacent rack row using row spacers.

Unit load — overall size (depth × width × height) of goods, including the pallet, if used.

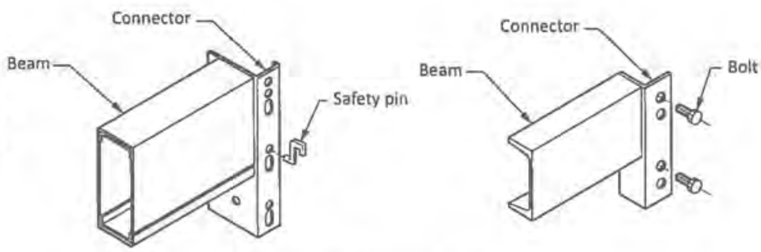
Unit load weight — overall weight of a unit load, including the weight of the pallet or container.

Figure N.1
Illustration of typical rack
(See Clause N.3.)





Baseplate and anchor detail



Beam and connector detail

Note: *This Figure is also contained in CSA A344 and illustrates some key terms to assist readers who might not be familiar with the construction of selective storage rack systems.*

N.4 Steel and welding specifications

N.4.1 Steel specifications

Structural steel quality shall be in accordance with Clause 5 or CSA S136, Section A2.

N.4.2 Welding specifications

All arc welding shall be as specified in CSA W59. All resistance welding shall be as specified in CSA W55.3.

N.4.3 Fabricator and erector qualification

Fabricators and erectors responsible for welding rack structures fabricated or erected under this Annex shall meet the requirements of CSA W47.1 (Division 1, 2, or 3) or CSA W55.3, or both, as applicable.

Note: *Fabricator qualification may be Division 3 for rack structures, similar to previous rack standards, so that welding qualifications would not be misinterpreted by building officials for this specific product.*

N.4.4 Existing racking components

All welded connections on the racking components shall be inspected for the requirements of the engineer responsible for capacity certification for racking components that are repurposed, resold, or reinstalled and where it cannot be demonstrated that the original welded fabrication met the requirements of Clause N.4. The inspection shall ensure they comply with the requirements of CSA W59 or CSA W55.3, as applicable. The inspector shall be an individual who meets the requirements of CSA W178.2 Level 2 or 3 to ensure the visual acceptance criteria of CSA W59 or CSA W55.3, as applicable, have been met.

Additionally, where the resistance of members or welded connections, or both, cannot be confirmed by the original manufacturer and if $I_E F_a S_a(0.2)$ is less than 0.35,

a) the resistance factor of members and welded connections shall be reduced to 67% of that specified in this Standard; and

b) the mechanical properties of the electrode used shall be considered as equivalent to CSA W48 E430XX classification.

Where $I_E F_a S_a(0.2)$ is greater than or equal to 0.35, the existing racking components shall not be used.

N.5 Bolted connections

Bolts shall be in accordance with this Standard with the inclusion of SAE J429 bolts. Unmarked bolts shall be assumed to be ASTM A307 bolts.

Note: *SAE bolts are subject to hydrogen embrittlement, which can be induced by the zinc plating process. These bolts should be manufactured to mitigate hydrogen embrittlement.*

N.6 Product identification

Beams and frames shall bear a permanent identification mark that is traceable to their manufacturer.

Note: *The requirement for permanent identification marks on rack beams and frames is to help both rack manufacturers and rack users identify rack components in the field. CSA A344 cautions that racks must not be assembled using a mixture of components from different manufacturers unless the structure has been reviewed and approved by qualified professionals. Racks could become unstable and collapse when seemingly compatible components from different manufacturers are combined.*

N.7 Design provisions

N.7.1 Design standards

N.7.1.1 Limit states design and structural integrity

Design shall be performed using limit states design principles in accordance with the requirements of Clause 6, including Clause 6.1.2 for structural integrity.

N.7.1.2 Resistance and stiffness of members

Resistance of members shall be determined in accordance with the provisions of this Standard and CSA S136, unless otherwise stated in this Annex. Stiffness of members and their connections shall be determined in accordance with normal engineering provisions and modified where necessary by the provisions of this Annex.

N.7.1.3 Non-calculable by CSA S16 or CSA S136

Where the members/elements are such that the resistance and/or stiffness cannot be determined in accordance with the provisions of Clause N.7.1.2, structural performance shall be established from either of the following:

a) factored resistance or stiffness shall be determined by tests undertaken and evaluated in accordance with Clause N.7.4; or
b) factored resistance or stiffness shall be determined by rational engineering analysis based on appropriate theory, related testing if data are available, and engineering judgment. Specifically, the factored resistance shall be determined from the calculated nominal resistance by applying the resistance factors.

Note: *Inelastic action of storage racks typically occurs in the joints and the behaviour of the rack joints used in the design has to be validated through physical testing.*

N.7.1.4 Members with perforations

Where hot-rolled members have perforations, their resistance shall be evaluated in accordance with Clause N.7.3.

Note: *The cold-formed steel provisions that account for the effects of perforations on the load-carrying capacity of compression members may be applied to hot-rolled members that are typically used as columns in storage racks.*

N.7.2 Design principles

N.7.2.1 Load factors

The load factors shall be those specified by the *NBCC*. The product loads shall be subject to a product load factor of 1.4.

Note: *Product load is the weight of items placed into the storage rack and has a load factor between that of a dead and live load. This factor reflects the fact that the upper bound of pallets or maximum pallet loads can be estimated more accurately relative to, for instance, live loads. At a certain pallet load, the lift truck equipment simply cannot lift the pallet loads into the rack so product loads have a sharp cut-off in the statistical distribution tail.*

N.7.2.2 Load combinations

Load combinations used for combining loads shall follow the companion action approach given in the *NBCC*, Table 4.1.3.2.A.

N.7.3 Design provisions for members with perforations

N.7.3.1 Design

Members with perforations, cold formed or hot formed, shall be designed in accordance with Clause N.7.3 of this Standard and CSA S136.

N.7.3.2 Perforation factor

A perforation factor, Q, shall be established using stub column test results in accordance with Chapter F of CSA S136, as follows:

$$Q = \frac{P_{ult}}{F_y' A_{nm}}$$

where

Q = perforation factor and shall be less than or equal to 1.0

P_{ult} = ultimate compressive strength of stub column by tests

F_y' = actual yield stress of the column material if no cold work of forming effects are to be considered; or the weighted average yield point F_y calculated in accordance with the test methods referred to in Section F3.1 of CSA S136, if cold work of forming affects are to be considered

A_{nm} = net minimum cross-sectional area obtained by passing a plane through the section normal to the axis of the column

Note: *Stub column tests are performed to account for the effects of perforations on the load-carrying capacity of compression members such as rack frame columns. Stub column tests are referenced in Section A7.2(a) of CSA S136 for the determination of strength increase due to cold work of forming. This Annex uses the same tests to account for the effects of perforations on the load-carrying capacity of compression members. Section A7.2(a) of CSA S136 makes reference to Section F3.1(b), which deals primarily with how the compressive yield stress is evaluated. Section F3.1 of CSA S136.1 provides additional details, including the specifications that provide test methods for stub column tests.*

N.7.3.3 Concentrically loaded compression members

N.7.3.3.1

Where the member satisfies the requirements for $\frac{b_{el}}{t}$ given in Clause 11 for a class 1, 2, or 3 section, the axial resistance of the member shall be computed using the provisions of Clause 13 appropriate for the class of section and lateral restraint of the member.

N.7.3.3.2

Where the compression member does not satisfy the requirements of a class 3 section as defined in Clause 11, the factored compressive resistance, P_r, shall be calculated in accordance with Section C4 of CSA S136, as follows:

$$P_r = \phi_c A_e F_n$$

where

ϕ_c = resistance factor for concentrically loaded compression member

A_e = effective area at stress, F_n

F_n = nominal buckling stress

The parameters used to calculate F_n shall be based on the net minimum cross-sectional area.

The effective area, A_e, at stress F_n shall be determined as follows:

$$A_e = A_{nm}\,[1 - (1 - Q)(F_n / F_y)^Q]$$

where

A_e = effective area at stress F_n

A_{nm} = net minimum cross-sectional area obtained by passing a plane through the section normal to the axis of the column

F_n = nominal buckling stress

F_y = yield point used for design, not to exceed specified yield point or established in accordance with Section F3 of CSA S136 or as increased for cold work of forming in Section A7.2, or as reduced for low ductility steels in Section A2.3 of CSA S136

Q = perforation factor and shall be less than or equal to 1.0

N.7.3.4 Laterally supported members in bending

N.7.3.4.1

Where the bending member satisfies the requirements for $\frac{b_{el}}{t}$ given in Clause 11 for a class 1, 2, or 3 section, the bending and shear resistance of the bending member shall be computed using the provisions of Clause 13 appropriate for the class of section and lateral restraint of the member.

N.7.3.4.2

Where the bending member does not satisfy the requirements of a class 3 section as defined in Clause 11, the factored moment resistance, M_r, calculated using procedure I of Section C3.1.1 of CSA S136, shall be determined as follows:

$$M_r = \phi_b S_e F_y [(Q + 1) / 2]$$

where

ϕ_b = resistance factor for bending strength

S_e = elastic section modulus of effective section calculated relative to extreme compression or tension fibre at F_y

F_y = yield point used for design, not to exceed specified yield point or established in accordance with Section F3 of CSA S136 or as increased for cold work of forming in Section A7.2 of CSA S136 or as reduced for low ductility steels in Section A2.3 of CSA S136

Q = perforation factor and shall be less than or equal to 1.0

The calculations in procedure II of Section C3.1.1 of CSA S136 that utilize inelastic reserve capacity shall not be used.

N.7.3.5 Laterally unsupported members in bending

The factored moment resistance, M_r, calculated in accordance with Section C3.1.2.1 (lateral-torsional buckling resistance of open cross section members) of CSA S136, shall be determined as follows:

$$M_r = \phi_b S_c F_c [(Q + 1) / 2]$$

where

ϕ_b = resistance factor for bending strength

S_c = elastic section modulus of effective section calculated relative to extreme compression fibre at F_c

F_c = critical buckling stress

Q = perforation factor and shall be less than or equal to 1.0

Calculation of the lateral-torsional buckling stress, F_c, shall be based on σ_{ex}, σ_{ey}, and σ_t, in accordance with Section C3.1.2.1 of CSA S136, using the full unreduced gross section properties.

N.7.4 Testing

Where the configuration of rack components precludes calculation of performance, the determination shall be made by tests. Testing shall be performed to define the behaviour being investigated. Tests should be done using methods established in RMI/ANSI MH 16.1 and EN 15512. The engineer shall be responsible for selecting the appropriate test procedures, interpreting the results, and using them in conjunction with the calculations to evaluate the storage rack's structural performance, behaviour, and safety.

Note: *Established methods for storage racks are available in the standards referenced in this Clause. In some cases new methods will need development, and the engineer should obtain specialist assistance in the design and performance of the test protocol.*

N.7.5 Certification

N.7.5.1

The design calculations and, where applicable, capacities evaluated with testing shall be approved in writing by an engineer. These records shall be maintained on file in accordance with Clause N.15.

N.7.5.2

The user shall be provided documentation that establishes the allowable capacity, configuration, and use.

N.8 Stability effects and loads

N.8.1 General

The stability effects shall be considered in the design of rack structures in accordance with this Standard, except that notional loads shall be calculated in accordance with Clause N.8.3.

N.8.2 Design loads

Product loads and/or unit load weights shall be those provided by the user or their representative.

N.8.3 Notional and minimum horizontal loads

N.8.3.1

Rack structures and their elements (columns, beams, bracing, connections, etc.) shall be designed to withstand the forces of notional loads combined with factored loads.

Note: *Notional loads are introduced to account for the effect of out-of-plumb on the stability of a framed structure and unique characteristics of rack structures.*

N.8.3.2

At every level, notional loads shall be equal to the notional load coefficient, ϕ_N multiplied by the factored gravity load contributed by that level, where the notional load coefficient is defined as follows:

ϕ_N = (0.003 + erection tolerance as defined in Clause N.13.1)

Erection tolerance shall include any out-of-plumb effects caused by floor slope.

Note: *As an example, if the erection tolerance is equal to 1:240 plus an out-of-plumb of 1% due to floor slope, then ϕ_N would be equal to 0.003 + (1 / 240) + (0.01) = 0.0172.*

N.8.3.3
The notional lateral loads shall be applied at every storage level in both orthogonal directions independently.

N.8.3.4
The individual column-to-beam connections and bracing members and their connections shall be designed to resist the effect of a minimum horizontal load of not less than 1.5% of the total static vertical load, with forces and moments evaluated using a first-order analysis. These loads shall not to be combined with any other lateral loads.

N.8.4 Loads due to attached equipment

N.8.4.1
Storage racks that support or interact with equipment shall be designed to resist the static and dynamic loads imparted to them by the equipment.

N.8.4.2
The equipment manufacturer shall supply the storage rack designer with the magnitude and location of the maximum static and dynamic forces that result from the equipment. The equipment manufacturer shall also supply any applicable impact factors.

N.8.5 Seismic loads
The computation of seismic loads shall be in accordance with Clause N.9.

N.8.6 Special loads

N.8.6.1 Overturning
Overturning shall be considered for the most unfavourable combination of vertical and horizontal loads using a minimum ratio of the restoring moment to the overturning moment (due to the product load and the dead load) of 1.0. Forces resulting from the product loads and the anchorage of the columns to the floor shall be considered in the stability evaluation. Racks that do not have positive anchorage to a floor slab or other resisting element shall have a minimum ratio of restoring to overturning moment ratio of 1.5.

Storage racks with a height-to-depth ratio exceeding 6:1 (height of the topmost beam to the upright frame depth), and subject to lateral impacts due to powered loading equipment, shall be designed to resist a lateral force of 10% of the factored weight of the maximum product load that is to be placed on the beams, in a single loading operation, adjacent to the frame being considered.

The lateral force shall be applied to a single-frame location at the topmost beam position in a direction perpendicular to the aisle and need not be applied concurrently with other horizontal design forces.

Restraints shall be designed to resist the uplift forces when applied to a frame supporting empty beam levels.

When the lateral force is transmitted to adjacent structures, the structures shall be designed to resist their calculated portion of the force.

Note: *Upright frames with high aspect ratios (height-to-depth ratios exceeding 8:1) that are subject to lateral impacts imparted by powered loading equipment require close examination of their stability. The usual and preferred practice is to improve stability by attaching the top of the upright frame to adjacent racks (top tying). Alternatively, anchorage may be designed that fully accounts for the ability of the base plate, anchors, and slab to withstand the anticipated uplift forces.*

N.8.6.2 Single rows

The designer shall consider additional measures appropriate to the unique circumstances of installing single rows.

Note: *Pallet racks are meant for use in typical warehouse environments employing careful, well-trained material handling equipment operators handling stable, common pallet loads. Single rows, even though of equal capacity to back-to-back rows, are more susceptible to overturning or collapse if damaged. Therefore, CSA A344 addresses single rows of rack and encourages the user to consult with the designer, so that the appropriate measures are put in place to lessen the chances of a collapse due to accidental impact. Because there are various methods that can be employed, this Annex does not single out an individual prescriptive requirement for single rows of rack. Additional measures to be considered, but are not limited to, include*

a) *post guarding;*
b) *double-posting the aisle leg of the frame;*
c) *heavy-duty frame bracing;*
d) *heavy-duty base plates and anchors;*
e) *overhead cross-aisle ties; and*
f) *wall connectors (appropriately designed and installed in accordance with Clause N.14).*

N.8.7 Other loads

Racks that are located outside and exposed to wind and snow loads shall be designed in accordance with the *NBCC*, Subsections 4.1.6 and 4.1.7.

N.9 Seismic loads and design

N.9.1 General

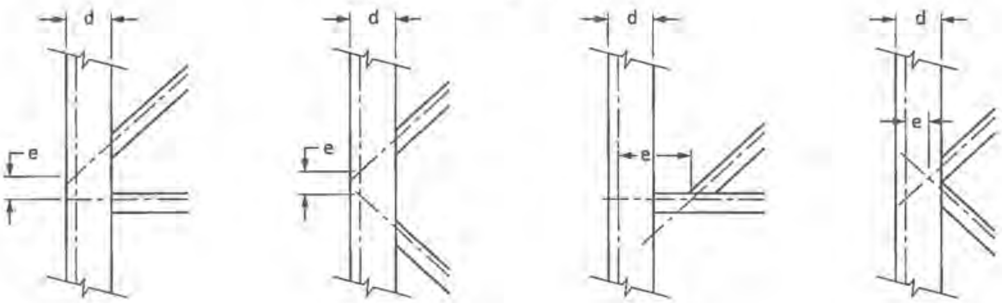
N.9.1.1 Seismic force resisting systems

Resistance to earthquake effects shall be provided by steel concentrically braced frames in the cross-aisle direction and steel moment resisting frames in the down-aisle direction. In the braced frame direction, when $I_E F_a S_a(0.2)$ is less than or equal to 0.5, braces may be absent in any isolated panel of the frame provided that combined axial load and moment have been accounted for as specified in Clause N.9.8.2 c).

Note: *On occasion, certain areas of a rack frame may eliminate a panel of bracing to allow a conveyor through the vierendeel section of the frame. If the level of seismicity does not exceed the specified limit, the absence of a brace in an isolated panel (vierendeel section) for the cross-aisle direction may be permitted if properly addressed in the design.*

Eccentricity of any centroidal work points for concentrically braced frames shall not exceed 200% of the column dimension in the plane of the connection, unless the eccentric moments have been taken into account (see Figure N.2).

Figure N.2
Eccentricities in frame bracing
(See Clause N.9.1.1.)



Legend:

d = overall depth of frame column

e = eccentricity at joint work points

N.9.1.2 Seismic design provisions

N.9.1.2.1

The seismic design of rack structures shall be performed in accordance with Clause 27 or with this Annex.

Note: *Reference to Clause 27 was intended to provide the design engineer with another method of seismic analysis for storage racks constructed of hot-rolled structural steel members. This method precedes all other seismic design options since it is very general and one does not need to go any further in the standard if its provisions are chosen for seismic design.*

N.9.1.2.2

When the seismic design is performed in accordance with Clause 27,

a) the seismic force resisting system shall be one of the systems defined in Clauses 27.3, 27.4, 27.5, 27.6, or 27.11;

b) the seismic force resisting system shall be designed using the force-based seismic design method in Clause N.9.3.2.1 using the factors R_o and R_d defined in Clause 27 for the selected system;

c) the period T_a used to determine earthquake effects may be determined in accordance to Clause N.9.4.5.2;

d) all requirements specified in Clause 27 for the selected system shall be satisfied, including requirements on restrictions, section class for columns of brace frames and moment frames, section class for moment frame beams, and connections; and

e) the seismic weights and gravity loads at every level may be determined as defined in Clause N.9.2.

N.9.1.2.3

When the seismic design is performed in accordance with this Annex,

a) the height shall not exceed 10 m to the topmost beam level when $I_E F_a S_a(0.2)$ is greater than or equal to 0.35;

Note: *For storage racks exceeding 10 m, the structure is to be designed using rational, recognized engineering principles and current engineering practice that demonstrate compliance with the intent of the applicable building code. The design review for these racks needs to be carried out by engineers experienced and knowledgeable in seismic analysis methods and with proper modelling and testing of the actual rack's* *behaviour. The rack design must have a clearly defined, seismic force resisting system that validates a stable, seismic response.*

b) the provisions of Clause 27.1.2 shall apply;
c) the seismic weights and gravity loads at every level shall be determined as defined in Clause N.9.2;
d) earthquake effects shall be determined using the methods described in Clauses N.9.3 to N.9.5, as applicable;
e) stability effects shall be as specified in Clause N.9.6;
f) drift limits as specified in Clause N.9.7 shall be satisfied;
g) special design requirements specified in Clause N.9.8 shall be satisfied; and
h) for the moment frame direction, the performance of the beam-to-column and column base connections shall be demonstrated through a qualification procedure, as specified in Clause N.9.9.

N.9.2 Seismic weight and gravity loads

N.9.2.1 General

Shedding of the load is a performance requirement and not a structural design issue. It is the owner that is ultimately responsible to ensure that loads do not shed because it depends on how the load is restrained on the pallet. The requirements for the retainment of loads during a seismic event is included in CSA A344 as it deals with their maintenance and operation.

N.9.2.2 Seismic weight

N.9.2.2.1 Calculation

The seismic weight shall be calculated by including the dead load of the structure plus the expected loading as specified by the storage rack user, but not less than 100% of the design product load in the cross-aisle direction and not less than 60% of the design product load in the moment frame direction.

Well-substantiated product statistics from the user of the storage rack that account for the facility's loading practices shall support any reduction of the product load.

Notes:

1) *The seismic weight may be reduced by the dynamically active fraction of the load by up to 2/3 of the product load.*
2) *Research has shown that the stored goods do not move entirely in unison with the rack structure and the 2/3 factor accounts for this damping-of-the-load behaviour. If the designer knows that for a particular application the dynamic portion of the load is likely to be greater than 67%, then such a higher magnitude will be used in the determination of the lateral forces.*
3) *The products placed on the storage rack shelves are often less than the capacity for which the individual shelves are designed. In most operating warehouses, these are several open product slots available for storing incoming product. Therefore, the total row seismic mass for computing the down-aisle seismic effects may be reduced by the product of a probabilistic factor to account for the amount of load expected on the rack at the time of an earthquake. Reduction in the cross-aisle direction and for the vertical load is not permitted.*

N.9.2.2.2 Distribution of seismic weight

The most unfavourable loading configuration shall be considered for the seismic analysis. As a minimum, the following cases shall be considered:

a) in the braced frame direction, rack fully loaded and top level fully loaded only; and
b) in the moment frame direction, rack fully loaded.

N.9.2.2.3 Variation in seismic response

Any variation in seismic response due to reduction in the expected product loading at the time of seismic event shall be evaluated separately in each of the two principal directions.

N.9.2.3 Gravity loads

N.9.2.3.1 General

Vertical load effects considered with lateral seismic loads shall be taken equal to 100% the dead load of the structure plus 100% of the design product load.

N.9.2.3.2 Racks subject to snow loading

Where racks are subject to snow load, the seismic weight *W* shall include 25% of the snow load applied to the rack in conjunction with the product load.

N.9.3 Analysis methods

N.9.3.1 Direction of analysis and in-plane torsion

N.9.3.1.1

The analysis may be performed independently in each of the two orthogonal directions.

N.9.3.1.2

In-plane torsional effects need not be considered.

N.9.3.2 Racks mounted at ground level

N.9.3.2.1

Earthquake effects shall be determined using force-based methods in accordance with the *NBCC* and as modified in this Annex, using either

a) the equivalent static force procedure (see Clause N.9.4.5);
b) the linear dynamic analysis method (see Clause N.9.4.6); or
c) the method prescribed for elements of structures, non-structural components, and equipment (see Clause N.9.4.7).

N.9.3.2.2 Displacement-based method

A displacement based-method, as described in Clause N.9.5, may be used in the moment frame direction.

Notes:

1) *Displacement-based design was proposed to address concerns with the use of similar ductility values for buildings and storage rack given that their connections are designed to behave differently during an earthquake. While energy dissipation in a building occurs primarily in the beams, storage racks have fairly weak columns and strong beams and the energy dissipation occurs in the connection in the down-aisle direction. It has been included in the* NBCC, *Structural Commentary J, stipulating that displacement-based design may be used provided they have been proven by testing and analysis.*
2) *A simplified displacement-based procedure in the down-aisle direction has been developed in FEMA 460 using a traditional linear elastic methodology and is approved in RMI/ANSI MH16.1. The reader may also reference Higgins (2007). for additional guidance.*

N.9.3.3 Racks mounted on building floors above grade

Earthquake effects for racks mounted on building floors above grade shall be determined using either

a) the method specified in *NBCC*, Article 4.1.8.18., when the seismic weight of the rack as defined in Clause N.9.2 of this Standard is less than 30% of the seismic weight of the supporting floor; the coefficient A_x shall be computed with h_x equal to the height of the highest point of connection in the building (see Clause N.9.4.7 of this Standard); or

b) the linear dynamic analysis method described where both the rack and the building structures are considered (see Clause N.9.4.6 of this Standard).

Note: *In some cases, racks may be installed above grade where the dynamics of the building will affect the seismic behaviour of the racks. In these cases, the methods for mechanical/electrical components might need to be used as they have amplification factors to account for the type of attachment and the variation of the response of the racks with elevation within the building (A_r and A_x as defined in the* NBCC*). Treating racks as mechanical/electrical components produces conservative forces; therefore, an analysis that fully accounts for the dynamic behaviour of the building and the racks is the recommended method for evaluating storage racks to be installed above grade.*

N.9.4 Force-based methods

N.9.4.1 Seismic force modification factors

The force modification factors shall be those used for conventional construction: R_d = 1.5, R_o = 1.3. In the moment frame direction, R_o shall be taken equal to 1.0 when non-normalized connection test data is used in design and R_d may be taken equal to 2.0 provided that the requirements of Clause N.9.8.1.2 are satisfied.

N.9.4.2 Importance factor

The seismic importance factor, I_E, shall be taken equal to the importance factor applicable to the building in which the rack is located.

Note: *If a building contains hazardous material, it may have a higher I_E factor, such as 1.3 for a high importance category, and this would also be the seismic importance factor used in the seismic design of the storage rack. Designers may also use a higher I_E factor at their discretion for storage racks accessible to the public.*

N.9.4.3 Connection stiffness

In the moment frame direction, the rotational stiffness of the beam-to-column and base connections shall be taken equal to the initial secant stiffness obtained at 60% of the connection's moment capacity, $M_{c,max}$, as determined from the connection qualification procedures specified in Clause N.9.9.

N.9.4.4 Seismic design displacements

The seismic design displacements including inelasticity effects shall be taken as the lateral displacements obtained from the analysis multiplied by R_oR_d / I_E.

N.9.4.5 Equivalent static force procedure

N.9.4.5.1 Limitations

The equivalent static force procedure may be used for rack structures at the ground level if

a) $I_EF_aS_a(0.2)$ is less than 0.35; or

b) the total height h_n is less than 6.0 m and the fundamental period T_a is less than 1.0 s in the braced frame direction and less than 2.0 s in the moment frame direction.

N.9.4.5.2 Periods

The fundamental period of racks shall not be evaluated using the formulas for the fundamental period of building structures in Part 4 of the *NBCC*. When determining earthquake forces, the period T_a in any direction shall be determined from methods of mechanics, except that the so-computed period for determining base shear and earthquake forces shall not exceed 0.15 h_n in the braced frame direction and 0.3 h_n in the moment frame direction. In the moment frame direction, the rotational stiffness of the beam-to-column and base connections shall not be less than the stiffness determined as specified in Clause N.9.4.3.

Note: *Typically, a designer will design the rack for seismic loading assuming the rack is full. Under these circumstances, the rack period is evaluated using a model that has a seismic weight in accordance with Clause N.9.2.1. If the rack is to be evaluated half full, then another dynamic analysis is done to determine the fundamental period with the adjusted seismic weight. If the base shear is computed using the procedures of* NBCC, *Article 4.1.8.18., the calculation of fundamental period for the rack is not required.*

N.9.4.5.3 Vertical distribution of loads

The top load (F_t), as specified in *NBCC* Sentence 4.1.8.11(7), shall be taken equal to zero. When the top of the beam level is less than 300 mm above the floor, its seismic weight may be omitted when determining the vertical distribution of the loads.

Note: *The static distribution of load provides a good distribution of load if the weight on the racks is similar at all levels but is less accurate if the mass distribution is not equal. Racks that have a large mass on the bottom level can get unreasonably high overturning moments if this is not accounted for. Under these circumstances, the rack designer may elect to use dynamic analysis to get a more accurate distribution of the overturning forces acting on the rack.*

a) If the top of the first beam level is less than 300 mm above the floor, the lateral force at the first beam level shall be

$$F_1 = (S(T_a)M_v I_e w_1) / (R_d R_o)$$

and, for beams levels above the first level, the lateral force shall be

$$F_x = \frac{(V - F_1)w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

b) If the top of the first beam level is greater than 300 mm, the lateral force at all beam levels shall be

$$F_x = \frac{V w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

where

V = total design lateral force or shear at the base of the rack

w_i or w_x = the portion of the total seismic weight of the rack at the designated beam level, level i or x

h_i or h_x = the height from the floor to level i or x

N.9.4.5.4 Minimum earthquake load

Clause 4.1.8.11.(2)(c) of the *NBCC* shall not be applied.

N.9.4.6 Linear dynamic analysis method

N.9.4.6.1 Limitations

A linear dynamic analysis method may be used without limitations.

N.9.4.6.2 Method

The response spectrum analysis method shall be used.

N.9.4.6.3 Minimum earthquake load

Sentences 4.1.8.12.(6), 4.1.8.12.(8), and 4.1.8.12.(9) of the *NBCC* shall not be applied.

N.9.4.7 Racks as elements of structures, non-structural components, and equipment

N.9.4.7.1 Limitations

Racks may be designed as elements of structures, non-structural components, and equipment, as defined in Article 4.1.8.18. of the *NBCC*, if the total height of the rack does not exceed 4.0 m.

N.9.4.7.2 Parameters

The parameters equivalent to those outlined in Table 4.1.8.18 of the *NBCC* shall be used except that the coefficient A_x shall be taken equal to 1.0 for racks mounted on grade.

N.9.5 Displacement-based method

N.9.5.1 Limitations

A displacement-based method may be used in the moment frame direction if the total height of the rack does not exceed 7.6 m.

N.9.5.2 Method

When applying the displacement-based method of analysis,

a) the effective properties of the equivalent single-degree-of-freedom system shall be determined from an appropriate model representing the rack structure inelastic first mode response with the effective stiffness of the beam-to-column and column base connections determined at the seismic design displacement;

b) the effective stiffness and energy dissipation capacity of the beam-to-column and column base connections shall be determined from the qualification procedure specified in Clause N.9.9;

c) P-delta effects shall be taken into account;

d) the seismic design displacement shall be determined using the design displacement spectrum specified in Clause N.9.5.3 using the effective period of the fundamental period and the equivalent damping properties of the rack structure; and

e) the structure equivalent viscous damping properties shall be based on the energy dissipation capacity of the beam-to-column and column base connections as specified in Item b). It may also include the inherent damping of the structure up to 3%.

N.9.5.3 Design displacement spectrum

The design spectral displacement values $S_d(T)$ at periods T = 0.0, 0.2, 0.5, 1.0, 2.0, 5.0, and 10.0 s shall be determined using $S_d(T) = 250\ S(T)T^2$ (in millimeters). Values for intermediate values of T shall be determined using linear interpolation.

N.9.5.4 Minimum lateral resistance

At every level, the frame shall have a minimum lateral resistance in the moment frame direction equal to

$$\frac{2\Sigma C_f\Delta}{h_s}$$

where Δ and h_s are defined in Clause N.9.6.2, unless it can be demonstrated through nonlinear dynamic analysis as described in the *NBCC* that the rack has stable seismic response and the drift limits specified in Clause N.9.7 are satisfied. As a minimum, the required analysis shall account for the inelastic cyclic response of the connections as obtained from the qualification procedure specified in Clause N.9.9, including strength degradation, if any, and P-delta effects.

N.9.6 Stability effects

N.9.6.1 Notional loads

Notional lateral loads as specified in Clause N.8.3 shall be applied when using a force-based analysis method. Notional loads need not be considered when determining drifts.

N.9.6.2 P-delta effects

When using force-based methods of analysis, P-delta effects shall be considered for members and connections for which inelastic response is expected by multiplying forces due to lateral load at every level by the factor U_2:

$$U_2 = 1 + \frac{\Sigma C_f\Delta}{R_o V_f h_s}$$

where

Δ = the relative lateral displacement occurring in the level, as obtained from the seismic design displacements

V_f = the total horizontal shear force at the level

h_s = the height of the level

N.9.7 Drift limits

In the moment frame direction, the seismic displacement shall be such that

a) at any level, the drift angle corresponding to the seismic design displacements does not exceed 0.05 radians; and

b) the total rotation imposed on the beam-to-column connections from gravity loads plus the rotation from the amplified seismic displacements does not exceed the rotation capacity of connections, as prescribed in Clause N.9.8.1.2.

Note: *While drift in buildings is limited due to architectural considerations and sensitivity to motion by occupants, racks are allowed greater drifts and yet still be serviceable.*

N.9.8 Special design requirements

N.9.8.1 Connecting design

N.9.8.1.1 Bracing member connections

Connections of the frame bracing shall resist forces due the combination of the gravity loads plus the earthquake loads corresponding to force modification factor of $R_dR_o = 1.3$.

N.9.8.1.2 Moment connections

N.9.8.1.2.1

When using force-based analysis methods, the connection strength may be taken as the connection moment capacity, $M_{c,max}$, as defined in Clause N.9.9.3.

N.9.8.1.2.2

When using force based analysis methods for moment-resisting frames, the total rotation imposed on the beam-to-column connections from gravity loads plus R_d times the seismic design displacements shall not exceed the rotation capacity of the connections, $\theta_{c,max}$, as defined in Clause N.9.9.3.6.

N.9.8.1.2.3

When using displacement-based analysis methods, the total rotation imposed on the beam-to-column connections from gravity loads plus two times the seismic design displacements shall not exceed the rotation capacity of the connections, $\theta_{c,max}$, as defined in Clause N.9.9.3.6.

Note: *For displacement-based analysis methods, beam-to-column connections are tested through to a rotation of two times the anticipated rack deflection, in other words, only half the rotation the connection is capable of undergoing is used as a design limit. The rationale behind this provision is that the designer might underestimate the displacements that actually occur during an earthquake and/or to establish occurrence of strength degradation so therefore the connection is tested for a larger rotation than expected.*

N.9.8.2 Column design

Column members shall be designed to resist

a) in the braced frame direction, the axial loads due the combination of the gravity loads plus the earthquake loads corresponding to a force modification factor of $R_dR_o = 1.3$;

b) in the moment frame direction, the full gravity loads in combination with the bending moments induced at each level by the lesser of the combination of the gravity loads and the earthquake loads corresponding to the values obtained using a force modification of $R_dR_o = 1.0$ or 1.2 times the ultimate flexural capacity of the beam-to-column connections; and

c) when braces are absent in the braced frame direction, as permitted in Clause N.9.1.1, axial load and bending moments from gravity loads plus earthquake loads determined with $R_dR_o = 1.3$.

N.9.8.3 Beam design

In the moment frame direction, beams shall resist a moment equal to 1.2 times the combined bending moments and shears induced at each level by the lesser of the combination of the gravity loads and the earthquake loads corresponding to the values obtained using a force modification of $R_dR_o = 1.0$ or 1.2 times the ultimate flexural capacity of the beam-to-column connections or that will develop at the attainment of the maximum moment capacity of the beam-to-column connections used at their ends.

N.9.9 Qualification procedures

N.9.9.1 General

Qualification procedures ensure that there is no conflict between the design and the acceptance material. A more consistent design approach or method is achieved through a test program that is controlled within the document. The testing protocol for beam-to-column connections is similar to that of the RMI to harmonize rack product testing between the USA and Canada.

Each type of beam-to-column connection and column base connections used in any a new rack structures or racks that are repurposed, resold, or reinstalled shall have physical qualification tests as prescribed in this Clause. Alternative testing requirements may be used when approved by the engineer of record subject to a third-party review.

Rotational strength and stiffness properties of column base connections may be obtained from calculations.

Note: *This protocol provides requirements for qualifying cyclic tests of beam-to-column moment connections in steel storage rack beam-to-column connectors for seismic loads. Testing provides evidence that a beam-to-column connection has the strength, stiffness, and inelastic rotational capacity to satisfy the demands that are being imposed upon them. It is also the purpose of this series of tests to determine the moment-rotation characteristics, or "dynamic spring relationship" of the beam-to-column connections of the various designs and manufacturers.*

N.9.9.2 Test specimens

N.9.9.2.1 General

The test specimen shall replicate, as closely as is practical, the pertinent design, detailing, and construction features, and the material properties of the actual rack structural elements. Material used in each member of connection elements that contribute to the inelastic rotation at yielding is tested to determine its yield stress and yield strength. Material properties shall be determined in accordance with the applicable ASTM A370 test procedures and Section F3 of CSA S136 or CSA G40.20/G40.21, as appropriate. When tensile coupons are taken after the completion of testing, they shall originate from flat portions of the specimen at regions of low bending moment and shear force. Tension-test results from certified mill test reports shall be reported but are not permitted to be used in place of specimen testing for the purposes of this Clause. In addition, consideration shall be given to any variation between the design thickness and yield strength, and the actual thickness and material strengths of the specimens used in the tests.

The testing program shall include tests of at least two specimens of each combination of beam and column and connector size.

N.9.9.2.2 Test specimen beams and columns

The size of the beams and columns used in the test specimen shall be representative of typical full-size storage rack beams and columns. The beam-to-column connectors and the connection details used in the test specimen shall represent the prototype connection details as closely as possible. The bolted portions of the test specimen shall replicate the bolted portions of the prototype connection as closely as possible.

N.9.9.3 Beam-to-column connection tests

N.9.9.3.1 General

The test sub-assemblage shall include a column element and two cantilever beam elements with integral attached beam-to-column connectors (see Figure N.3). For members subject to twisting, such as channel and Z sections, the twist shall be restrained.

N.9.9.3.2 Vertical loads on beams

Prior to the application of any cyclic loading, a constant downward load, P_c, of 4.45 kN shall be applied to each beam segment adjacent to each connector on both sides of the beam-to-column connection simulating the design downward-acting gravity pallet loads that serve to fully engage the beams and their connectors into the columns receiving them (see Figure N.3).

N.9.9.3.3 Displacement protocol

The test specimen shall be subjected to cyclic loading by imposing equal cyclic vertical displacements, Δ, at each end of each beam, in accordance to the following protocol defined based on drift angle, θ:

a) 5 cycles at $\theta = 0.005$;
b) 3 cycles at $\theta = 0.010$;
c) 3 cycles at $\theta = 0.015$;
d) 3 cycles at $\theta = 0.025$;
e) 3 cycles at $\theta = 0.050$;
f) 3 cycles at $\theta = 0.075$;
g) 3 cycles at $\theta = 0.100$;
h) 2 cycles at $\theta = 0.150$; and
i) 2 cycles at $\theta = 0.200$.

Additional cycles shall be performed at increments of $\theta = 0.050$ radians, with two cycles of loading at each increment, up to failure of the specimen.

Notes:

1) *The drift angle, θ, is defined as the vertical displacement, Δ, divided by the distance ℓ (see Figure N.3).*
2) *A loading cycle is defined as starting from zero drift angle to zero drift angle, including one positive and negative peaks at the prescribed drift angle values.*
3) *Other loading sequences may be used when they are demonstrated to be of equivalent or greater severity.*

N.9.9.3.4 Acceptance criteria

Inelastic rotation shall be developed in the test specimen by inelastic action in the same members and connection elements as anticipated in the prototype, i.e., in the beam, in the column, in the panel zone, or within the connection elements.

The test program shall be satisfactory if the connection moment capacity, $M_{c,max}$, and the connection rotation capacity, $\theta_{c,max}$, from the two tests are within 10% of the mean value. Otherwise, the test program shall be redone.

When beam-to-connection test results are used for displacement-based analysis, the effective stiffness and energy dissipated per cycle (EDC) of the two tests shall be within 10% of the mean value. Otherwise, the test program shall be redone. For both parameters, the verification shall be performed for every loading cycle starting from the first loading cycle at a drift angle of 0.03 radians up to and including the loading cycle corresponding to the connection rotation capacity $\theta_{c,max}$.

N.9.9.3.5 Connection moment capacity

The connection moment, M_C, shall be taken as (see Figure N.3):

$$M_c = 0.5(P_L + P_R)L$$

The connection moment capacity, $M_{c,max}$, shall be taken as the maximum value of $M_{c,max,cycle}$ among all loading cycles, where $M_{c,max,cycle}$ is the average of the peak positive and peak negative moments M_c, in absolute values, reached within the same loading cycle [see Figure N.4 a)].

N.9.9.3.6 Connection rotation capacity

The connection rotation, θ_C, is obtained by subtracting from the drift angle θ the drift angle due to elastic flexural deformation of the beams (see Figure N.3):

$$\theta_c = \theta - 0.5(P_L + P_R)L^2 / 3EI_b$$

When significant, the effect of the flexural deformations of the columns on connection rotation shall also be subtracted using the same procedure.

The connection rotation capacity, $\theta_{c,max}$, shall be taken as the peak connection rotation $\theta_{c,peak}$ in the last loading cycle during which $M_{c,peak}$ is equal to or greater than 0.80 times $M_{c,max}$, where $M_{c,peak}$ and $\theta_{c,peak}$ are respectively the average of the positive and negative moments M_c and rotation θ_c, in absolute values, reached at the maximum positive and negative drift angles in the same loading cycle (see Figure N.4).

Presentation of test results shall be properly reduced to actual values by correcting, where appropriate, initial readings.

Note: *Since rotations are measured at the beam end, the raw displacement data is a sum of beam deflection, connector rotation, and column deflection. To obtain the true connector rotation data, the beam deflection value is to be subtracted from the data. Since the column is fixed at both ends, its deflections may be neglected in the data process due to its very small values.*

N.9.9.3.7 Effective stiffness and energy dissipation capacity

For each loading cycle, the effective connection rotation stiffness, $k_{c,eff}$, shall be determined as follows [see Figure N.4 b)]:

$$k_{c,eff} = M_{c,peak} / \theta_{c,peak}$$

For each loading cycle, the energy dissipated per cycle (EDC) shall be taken equal to the area enclosed by the M_c-Δ_c connection moment-rotation hysteretic curve during the complete loading cycle. The test values of P_L and P_R shall be summed for each value of Δ tested.

The following defines the seismic design parameters obtained from beam-to-column connection tests:

a) The average moment M, at each Δ, shall be $(P_L + P_R)\,\ell/2$.

b) The rotational angle θ, at each Δ, shall be Δ/ℓ.

c) M versus θ shall be plotted for each Δ tested θ.

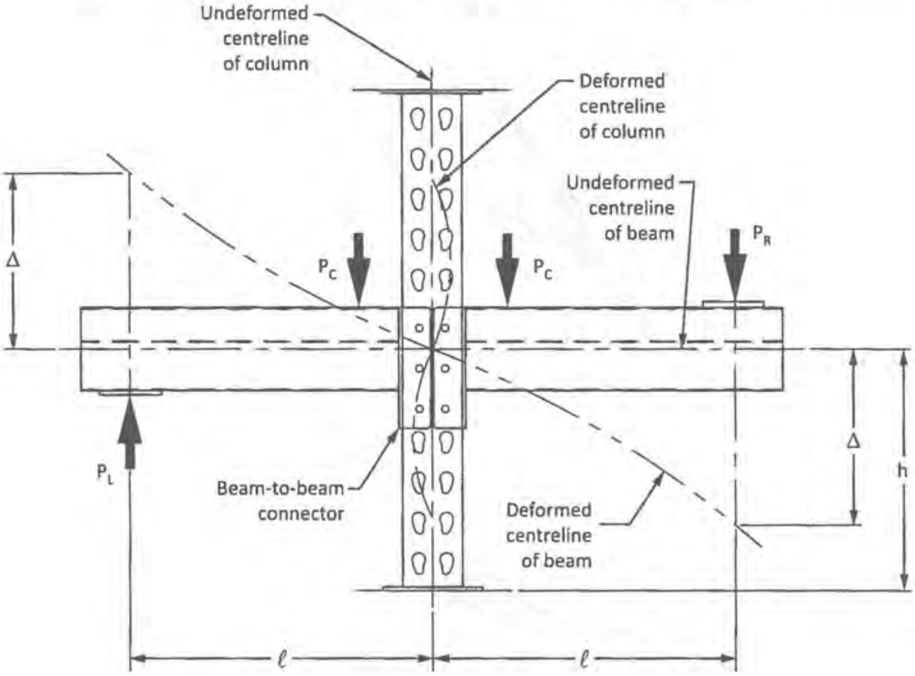
d) The maximum value of M_{max} shall be the maximum moment and the maximum rotational capacity θ_{max} shall be the lowest value of θ where the maximum moment occurs. The value of M_{max} shall be sustained for two cycles.

e) The design moment strength shall be ϕM_{max}, where ϕ is 0.9.

f) The rotational stiffness should be determined based on the calculated moment, M, for the design loads from the analysis using the plot of M versus θ.

   Note: *As an example, for the calculated design moment, M, one would go to the plot for M and determine the corresponding θ. The rotational stiffness would be M/θ. Since the calculated period and design forces depend on the stiffness, the value of M depends on θ. This means determining the appropriate rotational stiffness is an iterative process.*

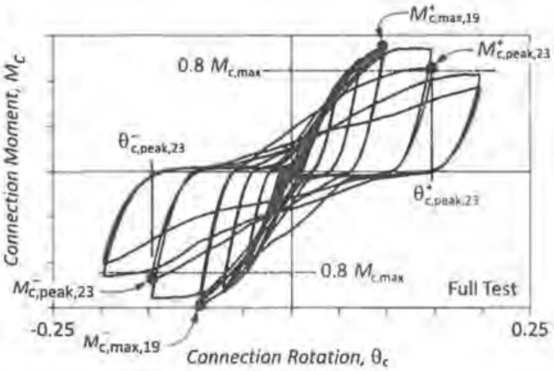
Figure N.3
Beam-to-column test set-up
(See Clauses N.9.9.3.1, N.9.9.3.2, N.9.9.3.3, N.9.9.3.5, and N.9.9.3.6.)



$$\theta = \frac{\Delta}{\ell}$$

Figure N.4
Typical moment-rotation response of a beam-to-column connection: a) under the entire loading protocol (27 cycles); b) in the 13th cycle
(See Clauses N.9.9.3.5, N.9.9.3.6, and N.9.9.3.7.)

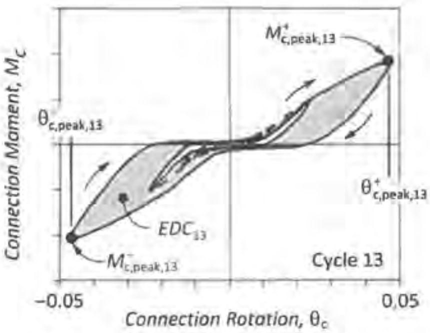
a)



$$M_{c,max} = \frac{M^+_{c,max,21} - M^-_{c,max,21}}{2}$$

$$\theta_{c,max} = \frac{\theta^+_{c,peak,25} - \theta^-_{c,peak,25}}{2}$$

b)



$$M_{c,peak,15} = \frac{M^+_{c,peak,15} - M^-_{c,peak,15}}{2}$$

$$\theta_{c,peak,15} = \frac{\theta^+_{c,peak,15} - \theta^-_{c,peak,15}}{2}$$

$$k_{c,eff,15} = \frac{M_{c,peak,15}}{\theta_{c,peak,15}}$$

Note: *This Figure represents an example of a moment-rotation response of a rack's beam-to-column moment connection that was obtained through the cyclic testing protocol for a specific manufacturer's beam-to-column connector and is not necessarily representative of all rack manufacturer's product. The last ascending loop is often 2 to 3 cycles before failure in the connections hysteretic response.*
The connection rotation capacity, $\theta_{c,max}$, is taken at the peak connection rotation, $\theta_{c,peak}$, in the last loading cycle, during which $M_{c,peak}$ is equal to or greater than 0.80 times $M_{c,max}$. This permits some degradation in the rack connector's moment capacity past the last ascending loop and allows more rotational capacity for ductile connectors that peak early, which is desirable and beneficial.

N.9.9.4 Column base connections

The rotational strength and stiffness of a column base shall be determined with consideration of the strength and stiffness associated to each contributing local and global deformations of the assembly (the base plate, the column, anchor bolts, the supporting floor, and column axial load) up to a base connection rotation corresponding to two times the anticipated rack displacements, including inelastic deformations.

For racks designed using the displacement-based analysis, the effective rotational stiffness, $k_{b,eff}$, and EDC shall be determined from cyclic tests under the drift angle protocol of Clause N.9.9.3.3 up to the loading cycle corresponding to two times the maximum anticipated rack displacements including inelastic deformations.

Base column properties and performance shall be evaluated for a range of column axial loads up to the maximum compressive resistance of the column. The evaluation may be performed independently for each axial load level considered, assuming a constant axial load intensity.

Column base connections shall maintain 80% of the flexural strength assumed in the design at a peak rotation corresponding to two times the anticipated rack displacements, including inelastic deformations.

Notes:

1) *The base fixity of frame columns is influenced by column base plates and their stiffness is typically used in structural analysis models to determine the behaviour and design loads of racks. Given that the magnitude of the axial load in the rack column has a significant effect on the fixity of the base connection, the column base connection test needs to measure the moment-rotation characteristics between the rack column and floor for a range of axial loads up to the maximum compressive resistance of the column. Column base rotation and the corresponding column base moments are to be recorded at constant axial load intensities during test runs.*
2) *Guidance on a test procedure for the fixity of column base that are bolted to the column can be found in EN 15512. Ongoing research is being performed by the Rack Manufacturers Institute in developing a base testing protocol.*

N.10 Beams and front-to-rear supports

N.10.1 Beam calculations

N.10.1.1

If the end-fixity of the beam connection is considered in the beam analysis, the corresponding connection moment shall be taken into account in the design of the upright column.

N.10.1.2

Where the product load is supported on ancillary structures, or the load is supported on a base that delivers concentrated forces to the beam, the moment shall be evaluated accordingly.

Note: *The live load factors provided in this Annex allow for the delivery of pallets by lift trucks with careful operators. Where the delivery of the load does not meet these criteria, the effects of impact are to be evaluated and accounted for in the design of the beams. Where beams are supporting pallets with a construction and vertical spacing that result in a load condition that approximates a uniformly distributed load, the effects of the load on the beam may be simplified by assuming the load to be uniformly distributed.*

N.10.1.3

N.10.1.3.1

Except as noted in Clause N.10.1.3.2, pallet support beam deflection shall not exceed 1/180 of the span measured with respect to the ends of the beam.

N.10.1.3.2

Pallet support beam deflection may exceed the maximum specified in Clause N.10.1.3.1, if approved by the design engineer and if the owner is informed in writing of the actual deflection and accepts it in writing.

N.10.1.3.3
Where the product load is supported on ancillary structures, or the load is supported on a base that delivers concentrated forces to the beam, the deflection shall be evaluated accordingly.

N.10.2 Front-to-rear supports

N.10.2.1
The deflection of pallet safety bar need not be considered.

N.10.2.2
Front-to-rear supports shall be positively restrained against lateral displacement.

Note: *Front-to-rear supports are intended to provide support for product loads (pallets). They are typically installed between the supporting beams and designed as a simply supported member, although continuous members are possible. The front-to-rear supports may be attached to the load support beam or dropped over the beam using bracketry. The end condition is to be designed for the worst load condition.*

N.10.2.3
When specified, pallet safety bars shall be designed with the product load (pallets) placed in the most unfavourable position.

Note: *A typical case to be evaluated for the design of the safety bar is when one edge of the pallet, either the front or rear, is overhanging a load support beam such that the distance between the mid-span of the safety bar and the centreline of the pallet is equal to one-fourth of the depth of the pallet. In this condition, the pallet places a point load on the safety bar flexural member a calculated distance from its simple support.*

N.10.2.4
Pallet support bars shall be designed using the same methods as specified for pallet support beams in Clause N.9.1.

N.11 Overall stability of trussed braced upright frames
Where the trussed braced frame has a high aspect ratio (slenderness about the frame's strong axis), a check shall be done to ensure global stability of the upright frame.

Note: *It is rare that the capacity of an upright frame is governed by the overall stability of the frame's braced plane (as opposed to member stability); nevertheless, a check is needed to ensure this failure mode does not govern capacity. This condition occurs where the upright frame has a large aspect ratio whereby the buckling occurs in a manner such that the buckling length under consideration is the overall height of the frame. RMI/ANSI MH 16.1 (Section 6.4) and EN 15512 (Appendix C) both provide analytical formulae addressing this condition. This Annex does not provide formulae that address this condition because it seldom comes into play and there are sufficient engineering methods available in other standards and textbooks.*

N.12 Connections and bearing plates

N.12.1 General
The strength and stiffness of proprietary connections shall be determined by tests.

N.12.2 Beam–column connections (beam connectors)

N.12.2.1
Beam-column connections shall be designed for the forces and moments resulting from the load in Clause N.8. The engineer shall distribute the forces in accordance with Clause N.8 or any other proven methods that employ a rational load path and satisfy static equilibrium. The non-linear, semi-rigid nature of storage rack connectors shall be considered in the analysis. Without ample proof of rigid behaviour, the designer should not make the rigid connection assumption in analysis.

Note: *The non-linear, semi-rigid nature of moment rotation response of most storage rack connectors makes them and the entire frame refractory to many forms of analyses.*

N.12.2.2
If the moment resistance of the beam connections is less than that required by Clause N.12.2.1, an alternative system shall be provided to resist the full lateral forces in Clause N.8.

N.12.2.3
If components from multiple manufacturers are incorporated and are necessary for adequate structural performance of the system, the behaviour of the specific combination of components shall be considered.

N.12.2.4
The behaviour and strength of the beam-to-column connection shall be evaluated using the test methods outlined in Clause N.10.1.

N.12.3 Beam connector locking device (safety pins)

N.12.3.1
Racks loaded by material handling equipment shall have connection-locking devices capable of preventing the disengagement of the beam connector from the upright column when subjected to an upward vertical force of 4.5 kN per connector.

N.12.3.2
Connection-locking devices shall remain functional when exposed to repeated upward vertical forces as specified in Clause N.12.3.1.

N.12.4 Row spacers

N.12.4.1 General
Adjacent (back-to-back) upright frames shall be connected together with row spacers, unless otherwise approved by the rack designer.

N.12.4.2 Row spacer design
Row spacers shall be designed to transmit the forces in Clause N.8. The size of the row spacer and its connections should be selected so that it can perform its intended functions. Vertical spacing of row spacers should be nominally 2.4 m to 3.6 m. Row spacers should be located at elevations that provide a direct load path between the columns in the frame.

Note: *Row spacers can provide structural redundancy to the brace member of a frame and improve general structural stability.*

N.12.5 Base plate design

N.12.5.1
The base plate shall have provisions for anchorage and designed to support the entire profile of the column placed on it.

N.12.5.2
The base plate shall be designed to transfer the column load to the supporting structure.

Note: *This Annex does not provide design criteria for floors that typically support storage racks. The design of the floor is the responsibility of the user and the building engineer. The effective area of the base plate can be evaluated using the CISC* Handbook of Steel Construction.

N.12.6 Concrete fasteners (anchors)

N.12.6.1
Concrete fasteners shall be used on all rack columns. Where they are required to resist design forces, specific calculations shall be computed in accordance with Clause N.12.6.3. Where the use of concrete fasteners is not feasible, other attachment methods shall be used.

Note: *Racks need to be attached to the supporting structure using a suitable fastener, even if the design forces do not make their use mandatory. Attachment serves to*

a) *resist design forces;*
b) *reduce potential damage to the lower portion of the upright should the column be struck by material handling equipment; and*
c) *maintain the installed geometry of the racks.*

N.12.6.2
The selected concrete fastener length shall allow for proper embedment in the concrete with a combined thickness of the base plate, any necessary shims, and any non-structural toppings.

N.12.6.3
The factored tensile and shear resistance of concrete fasteners shall be determined using the requirements of CSA A23.3, Annex D, in conjunction with material supplied by the anchor manufacturer.

N.12.6.4
Concrete anchorage in existing buildings shall be taken into account in the review of the floor. The resistance, thickness, and anchorage embedment and applied loads shall be compatible with the existing concrete slab. The rack designer shall provide rack column loads and proposed anchorage to the building owner for review by an engineer prior to installing the racks.

N.13 Erection tolerances

N.13.1 Tolerances
The rack erection tolerance on out-of-plumb shall not exceed 1:240. If the rack is deliberately installed on a sloped floor, any out-of-plumb from the floor slope may be added to the erection tolerance. The maximum out-of-plumb erection tolerance shall be considered in the design. If the erection tolerance considered in design differs from 1:240, it shall be shown on the rack design drawings.

N.13.2 Shims

Unless specifically provided for in drawings or documentation, shims shall be the same nominal depth and width as the base plate to be placed upon it and provide full contact between the base plate and the supporting concrete to transfer column loads to the floor.

Where accidental shifting or dislodgement of the shims can occur, measures shall be taken to ensure the shims remain in place.

Shims shall be of a material having equal or greater bearing strength of the floor.
Note: *The use of "finger shims" will allow the shims to be placed around anchor bolts, provide essentially full contact, and keep the shims in place.*

N.14 Interaction with buildings

N.14.1

The connection of storage racks to buildings, or to other structures other than the floor, is not recommended. Where such connections are made, the engineer(s) responsible for the rack, buildings, or other structures shall provide for their interaction. The design of rack-to-building connections is not within the scope of this Annex.
Note: *Storage racks and buildings have inherently different dynamic behaviours. Accordingly, any interconnection of the two will invariably lead to force transfers between them. In seismic events, the transfer is almost invariably from the high drift rack to the low drift building. Given the enormous mass disparity between them, such connections require either careful design or provisions to defeat any force transfer at all by maintaining a proper separation.*

N.14.2

Where connections are not made, storage racks shall be located so that the building and racks do not collide due to the design seismic forces. Deflections shall be computed in accordance with Clause N.9, and separation provided shall be in accordance with the *NBCC*.

N.14.3

Storage racks located at levels above the grade elevation shall be designed to resist design seismic forces that consider the responses of the building and storage racks as a combined structure to seismic ground motion.
Note: *For racks supported on a floor above grade, this Annex permits a parts-and-portions approach to design or a dynamic analysis provided that seismic amplification is accounted for. The parts and portions method takes into account soil type, the seismic risk at the site, the height of the base of the rack in the building, and whether the rack is storing toxic or flammable liquids but it does not differentiate between racks with short or long periods.*

N.15 Documentation

N.15.1

The users of the rack system shall maintain records of the permissible rack configurations readily available on site.

N.15.2

Where tests have been used to substantiate or determine the capacity of a member, component, or assembly, the test reports shall be maintained on file by the rack manufacturer.

N.15.3

The user of the rack system shall be responsible for the posting of permanent capacity plaques that are placed in one or more visible locations to specify the load limitations.

Note: *The user should reference CSA A344 for the details of capacity plaques or drawings to be posted.*

N.16 Use of rack

N.16.1

The rack user shall be responsible to ensure that the racks are configured, maintained, and used in accordance with the documentation provided by the rack manufacturer.

Note: *Rack capacities are based on new, undamaged components. Isolate and discontinue use of damaged components and have them unloaded, replaced, or repaired under the guidance of the rack manufacturer or an engineer.*

N.16.2

When modifications are made to the racks or configurations, they shall be approved by an engineer and the documentation specified in Clause N.15.1 shall be updated accordingly. Any reconfiguration of a rack structure shall comply with this Annex.

Note: *CSA A344 provides guidance regarding the reconfiguration and use of storage racks. Reconfiguration or rearrangement of a rack structure, including the relocation of beams, is not permitted without approval or certified documentation from the rack manufacturer or an engineer as it can create an overload condition.*

PART TWO
CISC COMMENTARY ON CSA S16-14

Preface

This Commentary has been prepared by the Canadian Institute of Steel Construction in order to provide guidance on the intent of various provisions of CSA Standard S16-14, "Design of Steel Structures". This Commentary and the information contained in the references cited provide an extensive background to the development of the Standard and its technical requirements including the changes and new provisions introduced in the 2014 edition. The Preface to the Standard itself outlines the history of its development since the first edition in 1924.

CSA Standard S16-14 has been prepared by the Canadian Standards Association (CSA), an approved standards development organization of the Standards Council of Canada, according to the rules for development of consensus standards. The National Building Code of Canada 2015 has adopted CSA Standard S16-14 by reference.

The Institute gratefully acknowledges the efforts of the various members of the CSA Technical Committee on Steel Structures for their valuable contributions to the Commentary, especially, G. Grondin, R. Tremblay, R.G. Driver, J.A. Packer and A.F. Wong who helped to rewrite a significant portion of this edition. The contributors include many former members of the Committee, in particular, D.J.L. Kennedy who had chaired the committee, served as a key author of the Commentary and provided valuable background information pertaining to many requirements introduced in all previous editions of the Standard prior to S16-09.

The information contained in the Commentary is provided by the Institute. It is not to be considered the opinion of the CSA Committee, nor does it detract from that Committee's responsibility and authority insofar as interpretation and revision of the Standard are concerned. For information on requesting interpretations, see Note (5) to the Preface of CSA S16-14.

The Institute provides this Commentary as a part of its commitment to the education of those interested in the use of steel in construction. Neither the Institute nor the authors of this Commentary assume responsibility for errors or oversights resulting from the use of the information contained herein. Anyone making use of the contents of this Commentary assumes all liability arising from such use. All suggestions for improvements of this Commentary will receive full consideration for future printings.

Introduction

Since the Canadian Standards Association introduced the first limit states design standard for structural steel, S16.1-1974 "Steel Structures for Buildings–Limit States Design" in 1974, the Standard has undergone a number of technical improvements, but its major requirements have remained virtually unchanged. However, with the introduction of the 1989 edition, a number of more significant changes were introduced, in part reflecting the maturing of the Standard but also the acquisition of more detailed information on behaviour of steel structures. Specific seismic design requirements for ductile behaviour were provided, and a new ductile system, eccentrically braced frame, was introduced. The 1994 edition continued this process with the refining of some requirements and the addition of a new lateral load-resisting system, the plate wall. The 2001 edition of the Standard was reorganized in a more logical order. In addition, the Standard underwent a number of technical improvements reflecting the incorporation of research results, including a significant expansion of Clause 27, Seismic Design Requirements.

When CSA S16-09 arrived, 35 years after the birth of its first limit states design version, S16.1-1974 and 25 years after the official withdrawal of its last allowable stress design version, S16-1969, the title of the Standard was shortened to read "Design of Steel Structures". In S16-09, two Annexes were added: Annex K, a normative annex that outlines the requirements for structural design for fire conditions and Annex L, an informative annex that provides information on design against brittle fracture.

Notable changes and new provisions incorporated in CSA S16-14 are: explicit recognition of several ASTM structural steel grades, use of non-matching weld electrodes as permitted in W59-13, an optional approach for design of ductile connections in moment-resisting frames, design rules for modular links in eccentrically braced frames and Annex M, an informative annex on seismic design of industrial steel structures. Some specific changes introduced in CSA S16-14 are highlighted in the Preface of the Standard itself.

Background

To serve their intended purposes, all structures must meet the requirement that the probability of occurrence of various types of collapse or unserviceability be limited to a sufficiently small value. Limit states are those conditions of the structure corresponding to the onset of the various types of collapse or unserviceability. The conditions associated with collapse are the ultimate limit states (ULS); those associated with unserviceability are the serviceability limit states (SLS), and that associated with fatigue is the fatigue limit state (FLS).

In limit states design, the capacity or performance of the structure or its components is checked against the various limit states at certain load levels. For the ultimate limit states of strength and stability, for example, the structure must retain its load-carrying capacity up to factored load levels (at the ultimate limit states), with only an acceptably small probability of being exceeded. (A factored load is the product of a specified load and its load factor.) For serviceability limit states, the performance of the structure at service load levels must be satisfactory. (For most applications, the service loads consist of dead load and one variable load only; thus the service loads are the specified loads. The National Building Code of Canada (NBCC) provides guidance on the use of companion load factors for situations where a companion loads should also be accounted for.) Examples of the serviceability requirement include prevention of damage to non-structural elements and restrictions on deflections, permanent deformations, slip in slip-critical connections, and acceleration under vibratory motion. For the fatigue limit state, the stress ranges for critical elements due to the loads applied to the structure over its useful life must not exceed the prescribed stress ranges.

The loads acting on a structure as well as the resistance of a member can only be defined statistically. When considering the ULS, a load factor (α) is applied to the specified load to take into account the fact that loads have a statistical distribution and that loads higher than those anticipated may exist, and also to take into account approximations in the analysis of the load effects. A resistance factor (ϕ) is applied to the nominal member (or component) strengths, or resistances (R), to take into account that the resistance of the member due to variability of the material properties, dimensions, and workmanship may be different than anticipated, and also to take into account the type of failure and uncertainty in the prediction of the resistance. A major advantage, therefore, of limit states design is that the factors assigned to loads arising from different sources can be related to the uncertainty of their prediction, and the factors assigned to different members and components can be related to their reliability and to the different types of failure. Thus, a greater degree of consistency against failure can be obtained (Kennedy 1974; Allen 1975; Kennedy *et al.* 1976).

For the failure of structural steel members by yielding, the resistance factor is taken to be 0.90 (Kennedy and Gad Aly 1980). To maintain simplicity in design, the resistance formulas for buckling or other types of member failure have been adjusted so that a uniform resistance factor, $\phi = 0.90$, can be used, while providing the necessary safety required in the definition of the resistance factor. Several failure modes and applications justify the use of smaller ϕ-values. For example, smaller values for weld metals and bolts have been adopted in the Standard in order to promote a lower probability of failure for connectors.

Probabilistic studies (Allen 1975) show that consistent probabilities of failure are determined for all dead-to-live load ratios when a dead load factor of 1.25 and a live load factor of 1.50 for use and occupancy loads are used. The NBCC gives load factors for environmental loads such as those due to snow and rain, wind, and earthquakes. As well, importance factors are applied to building structures depending on their use and occupancy (building importance category), with the highest factors applied to post-disaster structures. For certain types of structures, if there is a high degree of uncertainty in the loads, the designer may elect to use larger load factors. However, in situations where the dead load and the live loads are counteractive, it is important that α_D be taken as 0.9 or less, as appropriate, except that when dead load counteracts earthquake effects, α_D is taken as 1.0 or less.

Kennedy (1974) and Allen (1975) provide considerably more information on the type of probabilistic, calibration, and design studies that were performed while developing the limit states standard. The NBCC 2015 contains a more extensive discussion on limit states design. Kennedy and Gad Aly (1980) and Baker and Kennedy (1984) provide information on the statistical determination of the resistance factors (ϕ).

In the Commentary clauses that follow, the numbers and headings refer to the relevant clause numbers and headings of Canadian Standards Association (CSA) Standard S16-14. This will be referred to simply as S16-14 herein and after.

1. SCOPE AND APPLICATION

This Standard applies generally to steel structures and structural steel components in other structures. The analysis, design, detailing, fabrication, and erection requirements contained in the Standard normally provide a satisfactory level of structural integrity for most steel structures.

Clause 1.2 states that requirements for some specific types of structures and members are given in other CSA Standards. Situations where additional requirements may be necessary are given in Clause 1.3. The Structural Commentaries to the National Building Code of Canada provide references to the technical literature on the topic of structural integrity.

Clause 1.3 describes types of structures that may need supplementary rules for design. Crane-supporting structures are included in this list.

Clause 1.4 prohibits the substitution of any other structural steel design standard (e.g. CSA S16-1969 or AISC) for S16-14. The treatment of a number of important technical issues relating to safety, such as notional loads, beam-columns, ductility of members, and connections for earthquake loads, is either not covered or is treated in a manner inconsistent with the intent of S16-14 or the NBCC.

Clause 1.4 permits the designer (subject to approval from the Regulatory Authority) to supplement the formulas given in the Standard by a rational method of design. It is required that the structural reliability provided by the alternative (as measured by the reliability index, for example) be equal to, or greater, than those in the Standard. An example of such a rational method would be the design of stub-girders using the method set out by Chien and Ritchie (1984) based on tests (Bjorhovde and Zimmerman 1980, Kullman and Hosain 1985, Ahmad *et al.* 1990). Since structural design is an inextricable part of the design-construction sequence, substitution of other standards or criteria for fabrication, erection, inspection or any combination thereof, unless specifically directed by S16-14, is prohibited.

2. REFERENCE PUBLICATIONS

The Standards listed are the latest editions at the time of printing. When reference is made to undated publications in specific clauses of this Standard, it is intended that the latest edition and revisions of these publications be used. Two new Standards have been added.

3. DEFINITIONS AND SYMBOLS

In Clauses 3.1 and 3.2, new definitions and symbols have been introduced.

3.3 Units

All coefficients appearing in equations and expressions in this Standard are consistent with forces measured in Newtons and lengths in millimetres. While most coefficients are themselves non-dimensional, in Clause 17.9.10, the coefficient 2.76 has units of megapascals (MPa).

4. STRUCTURAL DOCUMENTS

4.1 General

The title "Structural Documents" reflects the fact that drawings are only part of a broadened range of structural documents that are currently used in the industry.

4.2 Structural Design Documents

4.2.1 Structural steel design documents, by themselves, should show all member designations, axis orientations, and dimensions needed to describe the complete steel structure. It should not be necessary, in order to ascertain information on structural steel components, to refer to documents produced for the use of other trades, as in some situations the fabricator may not be given, or have access to, the documents produced for other trades.

4.2.2 This list gives the minimum information to be included on the structural design documents in a logical order (much of it, no doubt, on Drawing S-1). By serving as a checklist, it will help insure that all the information the fabricator needs is provided and will help resolve disputes before they arise.

4.2.2(l) The development of adequate connections for structural members requires that the design engineer determine the shears, moments, and axial forces resulting from the governing load combinations for which the connection must be designed. For complex combinations, a useful presentation of this information may be to list the maximum value of each (e.g. shear, moment, and axial force), along with the values of the others which coincide with that maximum. The principle is to provide coexistent sets of forces so that free body diagrams can be identified to ensure that governing forces are transmitted through connections and panels.

4.2.2(m) Structural stability, a fundamental consideration of design, extends to the behaviour of elements within a member as well as to the functioning of members in total. Stabilizing components are needed to achieve both the correct local behaviour and the correct overall behaviour anticipated by the design. Therefore, the design engineer must define bracing, stiffeners, and reinforcement that are required to prevent failure due to instability. An example is web reinforcement in moment connections to prevent local instability. It may actually be more economical to use a heavier section and avoid the need for stiffeners or reinforcing detail material. This option should best be considered at the design stage.

4.2.3 The importance of proper recording of revisions on design documents, whether electronic files or paper, is emphasized. Control of documents is addressed in Steel Fabrication Quality Systems Guideline (CISC 2002) and in the CISC Code of Standard Practice, Appendix J, in Part 7 of this Handbook.

4.2.4 Architectural, electrical, and mechanical documents may be used for supplementary information, provided that the requirements in Clauses 4.2.1 and 4.2.2 for structural steel are shown on the structural documents.

4.3 Fabrication and Erection Documents

Although five types of documents are identified in the Standard, many structures which use pre-engineered connections from company or industry sources require only *shop details* and *erection diagrams*.

4.3.1 Connection Design Details

Connection design details, which often take the form of design brief sheets, typically show the configuration and details of nonstandard connections developed for specific situations. They are submitted to the design engineer for review to confirm that the structural intent has been understood and met, and they may be stamped by a professional engineer when appropriate. Drafting technicians use connection design details to prepare shop details.

4.3.2 Shop Details

Shop details frequently take the form of traditional shop drawings and are used to provide the fabrication shop with all the specific information required to produce the member. They are submitted to the design engineer for review to confirm that the structural intent has been understood and met. *Shop details* are not stamped by a professional engineer because they generally do not contain original engineering.

4.3.3 Erection Diagrams

Erection diagrams convey information about the permanent structure that is required by field personnel in order to assemble it. They are submitted to the design engineer for review, but are not stamped by a professional engineer because original engineering is generally not added by the fabricator.

4.3.4 Erection Procedures

Erection procedures outline methods and equipment, such as falsework and temporary guying cables, employed by the steel erector to assemble the structure safely. They may be submitted to the design engineer for review and may be stamped by a professional engineer when appropriate.

4.3.5 Field Work Details

Field work details are drawings which describe modifications required to fabricate members. The work may be done either in the shop or at the job site depending on circumstances. When extra material is involved, *field work details* effectively become *shop details*. They are submitted to the design engineer for review.

5. MATERIAL – STANDARDS AND IDENTIFICATION

The design requirements have been developed on the assumption that the materials and products that will be used are those listed in Clause 5. These materials and products are all covered by standards prepared by the Canadian Standards Association (CSA) or the American Society for Testing and Materials (ASTM).

The standards listed provide controls over manufacture and delivery of the materials, and products that are necessary to ensure that the materials and products will have the characteristics assumed when the design provisions of S16 were prepared. The use of materials and products other than those listed is permitted, provided that approval, based on published specifications, is obtained. In this case, designers should assure themselves that materials and products have the characteristics required to perform satisfactorily in the structure. In particular, ductility is often as important as the strength of the material. Weldability and toughness may also be required in many structures.

The values for yield and tensile strength reported on mill test reports are not to be used for design. Only the specified minimum values published in product standards and specifications may be used. This requirement was implicit in earlier editions of the Standard by definition of the terms F_y and F_u but was made explicit in more recent editions. Furthermore, when tests are done to identify steel, the specified minimum values of the steel, once classified, shall be used as the basis for design.

When, however, sufficient representative tests are done on the steel of an existing structure to be statistically significant, those statistical data on the variation of the material and geometric properties may be combined with that for test/predicted ratios available in the literature to develop appropriate resistance factors. This is by no means equivalent, for example, to substituting a new mean yield stress for a specified minimum value as the new reference value, and the bias coefficient must be established. It could well be that, although a higher mean value of the yield stress is established, the bias coefficient, depending as it does on the reference value, would be less. It would be expected that the coefficient of variation for the material properties in particular, derived for the steel in a single structure, would be less than for steel in general.

In Clause 5.1.3, both CSA and ASTM are referenced standards for structural steel. Because W-shapes are no longer produced by Canadian mills, mill test certificates will more often refer to ASTM A992/A992M or to ASTM A572/A572M. While ASTM A572 Grade 50 is comparable to G40.21 350W, ASTM A992 is a more restrictive version of A572 Grade 50 as it was developed specifically for seismic-resistant structures, but has become the most popular grade of wide-flange products available in North America. ASTM A913/A913M grades have been added to this Clause in S16-14. While Grade 65 (450) products are usually specified, Grade 70 (485) and Grade 50 (345) W-shapes are also produced.

The Standard requires that the design properties for ASTM A500/A500M HSS be determined from a wall thickness equal to 90% of the nominal wall thickness to account for the 10% under-tolerance for thickness permitted in ASTM A500/A500M and the lack of under-mass restriction. This requirement is consistent with the practice adopted in the CISC Handbook of Steel Construction. Grade C is the dominant grade for A500 HSS in Canada. ASTM A1085, a standard introduced in 2013, covers HSS that are produced to conform to a minimum average Charpy V-notch impact value. Other specific requirements include maximum yield stress and minimum corner radius controls. At the preparation time of this Commentary, users are advised to confirm availability prior to specifying A1085 HSS.

In Clause 5.1.7, ASTM Standards for bolts and bolt assemblies are referenced. ASTM F3125, a consolidation and replacement of six standards, A325, A325M, A490, A490M, F1852, and F2280, was published in January 2015. Since the name of each bolt standard becomes a bolt grade in this "umbrella" standard, F3125 (e.g. A490 becomes F3125 Grade A490), a seamless transition is anticipated.

6. DESIGN REQUIREMENTS

This clause clearly distinguishes between those requirements that must be checked using specified loads (the fatigue and serviceability limit states) and those which must be checked using factored loads (the ultimate limit states). Many of the serviceability requirements (deflections, vibrations, etc.) are stipulated qualitatively and guidance, in quantitative form, is provided in Annexes. Thus, the designer is permitted to use the best information available in order to satisfy the serviceability requirements, but is also provided with information that the Technical Committee on Steel Structures considers to be generally suitable, when used with competent engineering judgement.

6.1 General

6.1.2 Structural Integrity

A clause on structural integrity acts as a reminder that measures may be necessary to guard against progressive collapse as a result of a local incident. Being inherently ductile, steel structures have generally had an excellent record of behaviour when subjected to unusual or unexpected loadings. However, connection details are particularly important in achieving this ductile behaviour. Details which rely solely on friction due to gravity to provide nominal lateral force resistance may have little or no resistance to unanticipated lateral loads if subjected to abnormal uplift conditions and should be carefully evaluated for such an eventuality or completely avoided.

6.2 Loads

Dead loads are to include the additional mass of construction materials that will be built into a structure as a result of deflections of supporting members, such as a concrete floor slab placed to a level plane but supported by members that were not cambered and that deflect under the weight of the concrete.

6.3 Requirements Under Specified Loads

6.3.1 Deflection

6.3.1.2 Even though deflections are checked under the actions of specified loads, additional loading may result from ponding of rain on roofs, or the ponding of finishes or concrete, while in the fluid state, on floors or roofs. Such additional loads are to be included in the design of the supporting members under ultimate limit states as required by Clause 7. More information on ponding is available in the National Building Code of Canada (NBCC 2015).

6.3.3 Dynamic Effects

6.3.3.2 Additional information on vibrations of floor systems may be found in Allen (1974), Murray (1975), Allen and Rainer (1976), Rainer (1980), Allen *et al.* (1985), Allen and Murray (1993), Murray *et al.* (1997).

6.7 Requirements Under Fire Conditions

Background information on S16-14 Annex K, Structural Design for Fire Conditions, is available on the CISC Fire Protection webpage:

www.cisc-icca.ca/CommentaryS16AnnexK

6.8 Brittle Fracture

Annex L of S16-14 provides some design information to prevent failure of steel structures by brittle fracture. Annex L identifies the circumstances under which brittle fracture can occur and situations where brittle fracture should be considered as part of the design process. Annex L serves as a non-mandatory guide.

7. FACTORED LOADS AND SAFETY CRITERION

This clause sets forth the fundamental safety criterion (strength and stability) that must be met, namely:

Factored Resistance ≥ Effect of Factored Loads,

or

$$\phi R \geq \sum \alpha_i S_i$$

The factored resistance is given by the product ϕR where ϕ is the resistance factor and R is the nominal member strength, or resistance. The resistance factors of various types of members are given in Clause 13.1.

8. ANALYSIS OF STRUCTURE

Three types of construction are recognized, namely, "rigidly connected and continuous", "simple", and "semi-rigid (or partially restrained)". While semi-rigid construction was developed in the 1930's and 1940's, both in the USA and in the UK, and was previously a successful practice, it is now not in common use in North America.

With semi-rigid connections, because the angles between connected parts change under applied bending moments, the joint behaviour is non-linear and the moment/rotation response must be established by test, although many connection configurations have been tested and their moment/rotation responses have been compiled (Chen *et al.* 2011, Faella *et al.* 2000). Design of a semi-rigidly connected structure must take into account the effect of the "semi-rigid" connection stiffness on the stability of the structure. A second-order analysis is preferred because the non-linearities due to connection response and due to frame drift need to be assessed.

It is assumed that, if the connection has adequate capacity for inelastic rotation when subjected to the first application of factored gravity and lateral loading, under subsequent loading cycles the connection will behave elastically, although it will have a permanent inelastic deformation (Sourochnikoff 1950, Disque 1964). Such an assumption is valid except in joints where load fluctuation would create alternating plasticity in the connection (Popov and Pinkney 1969). With this form of construction, it is also important to consider the possibility of low-cycle, high-strain fatigue.

The use of open-web steel joists as connected members of these frames has been shown to be inadequate (Nixon 1981).

Clause 8 also permits the use of the two general methods of analysis – elastic and plastic analysis. Methods of elastic analysis are familiar to most designers.

8.3.2 Plastic Analysis

The use of plastic analyses at the factored load levels to determine the forces and moments throughout a structure implies that the structure achieves its limiting load capacity when sufficient plastic hinges have developed to transform the frame into a mechanism. As successive plastic hinges form, the load-carrying capacity of the structure increases above that corresponding to the formation of the initial plastic hinge until a mechanism develops. To achieve this, the members in which the hinges form before the mechanism develops must be sufficiently stocky (Class 1 sections) and well braced so that inelastic rotations can occur without loss of moment capacity.

Deflections at the specified load level are, of course, limited in accordance with Clause 6.3.1.1. Plastically designed structures are usually "elastic" at specified load levels, i.e. no plastic hinges have formed. Therefore, the deflections would generally be computed on the basis of an elastic analysis.

8.3.2(a) Material

The plastic method of analysis relies on certain basic assumptions for its validity (ASCE 1971). Therefore, restrictions are imposed to preserve the applicability of the plastic analysis theory. The basic restriction (Clause 8.3.2(a)) that the steel exhibit significant amounts of strain-hardening is required to ensure that satisfactory moment redistribution will occur (Adams and Galambos 1969). This behaviour should exist at the temperatures to which the structure will be subjected in service. Also, although not explicitly stated, plastically designed structures usually entail welded fabrication, and therefore the steel specified should also be weldable. At normal temperatures all the steels referred to in Clause 5.1.3 should be satisfactory except for CSA G40.21, 700 Q and 700 QT steels, for which $F_y > 0.85 F_u$.

A reassessment of the stress-strain data (Dexter and Genticore 1997, Dexter *et al.* 2002) showed that the requirement that the yield strength not exceed 0.80 of the ultimate strength could be relaxed to 0.85 of the latter.

8.3.2(b) Width-to-Thickness Ratios

In order to preclude premature local buckling and thus achieve adequate hinge rotation to ensure sufficient moment redistribution to reach a plastic collapse mechanism, compression elements in regions of plastic moment must have width-to-thickness ratios no greater than those specified for Class 1 (plastic design) sections in Clause 11.2.

8.3.2(c) Lateral Bracing

The lateral bracing requirements are considerably more severe than those for structures designed on the basis of an elastic moment distribution because of the rotation needed at the location of the plastic hinges. Such requirements, as are needed to ensure adequate behaviour in earthquakes, are the basis for these new requirements. These equations were derived for non-cyclic plastic rotations of 3 and 4 times the elastic rotation at first yield following the procedure proposed by Bansal (1971) and summarized in Chapter 10 (Figure 10.27) of Bruneau *et al.* (1998). For traditional plastic design, case (a) is applicable, and to provide for the ductility demands implied for the three types of seismic moment frame categories, cases (a) or (b) are applicable as indicated. Test results on inelastic beams under moment gradient are reported by Lay and Galambos (1967).

Because the final hinge in the failure mechanism does not require rotation capacity, the bracing spacing limitations of this clause do not apply, and the elastic bracing requirements of Clause 13.6(a) may be used.

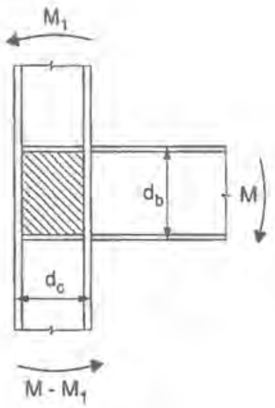
Lateral bracing is required to prevent both lateral movement and twisting at a braced point. Lateral bracing is usually provided by floor beams or purlins that frame into the beam to be braced. These bracing members must have adequate axial strength and axial stiffness to resist the tendency to lateral deflection. These requirements are given in Clause 9.2. Further information on the design of bracing members is given in Lay and Galambos (1966) and Chapter 12 of Ziemian (2010). When the bracing member is connected to the compression flange of the braced member, the brace should possess bending stiffness to resist twisting of the braced member. Some information on the bending stiffness of braces is given in Essa and Kennedy (1995).

A concrete slab in which the compression flange is embedded or to which the compression flange is mechanically connected, as in composite construction, or metal decks welded to the top flange of the beam in the positive moment region, generally provide sufficient restraint to lateral and torsional displacements. When the lateral brace is connected to the tension flange, provision must be made for maintaining the shape of the cross-section and for preventing lateral movement of the compression flange. This can be accomplished with either diagonal struts to the compression flange or adequately designed web stiffeners.

8.3.2(d) Web Crippling

Web stiffeners are required on a member at a point of load application where a plastic hinge would form. Stiffeners are also required at beam-to-column connections where the forces developed in the beam flanges would either cripple the column web or, in the case of tension forces, distort the column flange with incipient weld fracture. The rules for stiffener design are given in Clause 21.3 (Kennedy *et al.* 1998). See ASCE (1971) for further details of stiffeners and Fisher *et al.* (1963) for special requirements pertaining to tapered and curved haunches.

When the shear force is excessive, additional stiffening may be required to limit shear deformations. The capacity of an unreinforced web to resist shear is taken to be that related to an average shear yield stress based on the Huber-Henckey-von Mises criterion of $F_y/\sqrt{3}$. For an effective depth of the web of a rolled shape of about 95% of the section depth, Clause 13.4.2 gives:



WITHIN HATCHED ZONE

$$w_c \geq \frac{M}{0.8\phi\, d_c\, d_b\, F_s}$$

Figure 2-1
Web Thickness at Beam-to-Column Connections

$$V_r = 0.95\phi\, w d F_y / \sqrt{3} = 0.55\phi\, A_w F_y = 0.8\phi A_w F_s$$

At beam-to-column connections, when the shear force exceeds that permitted above, the excess may be carried by providing doubler plates to increase the web thickness or by providing diagonal stiffeners (Figure 2-1). The force in the beam flange that is transferred into the web as a shear is approximately

$$V = M / d_b$$

Equating this to the shear resistance as given in Clause 13.4.2 (where now, $w = w_c$ and $d = d_c$), and solving for the required web thickness,

$$w_c \geq \frac{M}{0.8\phi\, d_c\, d_b\, F_s}$$

If the actual web thickness is less than w_c, the required area of diagonal stiffeners may be obtained by considering the equilibrium of forces at the point where the top flange of the beam frames into the column. Using a lower bound approach, the total force to be transmitted ($V = M/d_b$) is assumed to be taken by the web and the horizontal component of the force in the diagonal stiffener:

$$V = M / d_b = 0.8\phi\, w_c d_c F_s + \phi\, F_y A_s \cos\theta$$

where

A_s = cross-sectional area of diagonal stiffeners

$\theta = \tan^{-1}(d_b/d_c)$

The required stiffener area is therefore

$$A_s = \frac{1}{\cos\theta}\left(\frac{M}{\phi\, F_y\, d_b} - \frac{0.8\, w_c d_c F_s}{F_y}\right)$$

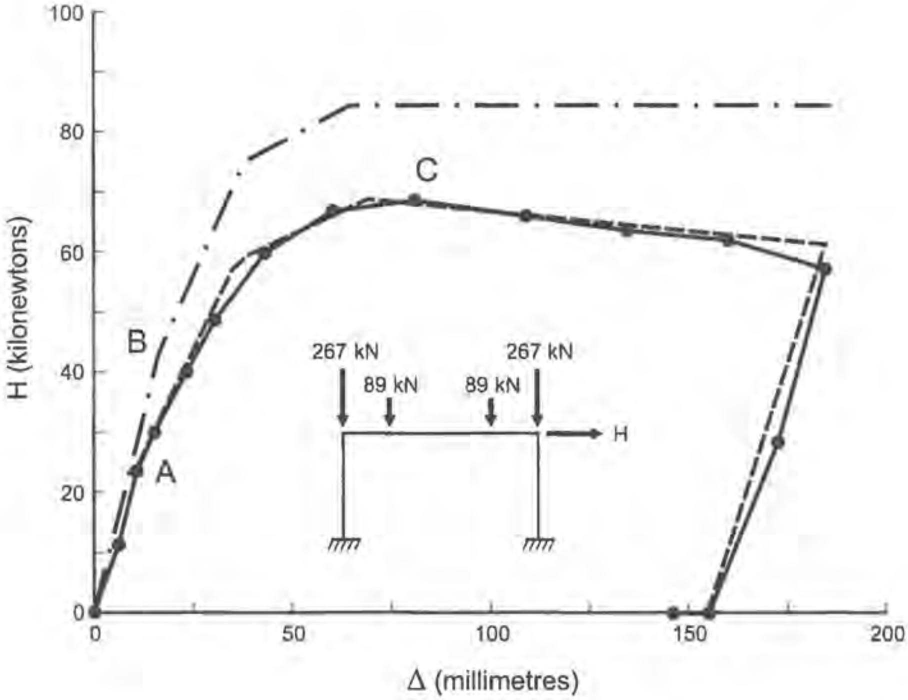


Figure 2-2
Observed and Predicted Load-Deflection Relationships

8.3.2(e) Splices

The bending moment diagram corresponding to the failure mechanism is the result of moment redistribution that occurred during the plastic hinging process. For example, points of inflection in the final bending moment distribution may have been required to resist significant moments to enable the failure mechanism to have developed (Hart and Milek 1965). To ensure that splices have sufficient capacity to enable the structure to reach its ultimate load capacity, a minimum connection requirement of 0.25 M_p is specified in Clause 8.3.2(e). Also, at any splice location, the moments corresponding to various factored loading conditions must be increased by 10% above the computed value. The splice is then designed either for the larger of the moments so increased or for the minimum requirement of 0.25 M_p.

8.3.2(f) Impact and Fatigue

The use of moment redistribution to develop the strength of the structure corresponding to a failure mechanism implies ductile behaviour. Members that may be subjected repeatedly to heavy impact and members that may be subject to fatigue should not be designed on the basis of a plastic analysis because ductile behaviour cannot be anticipated under these conditions. Such members, at least for the present, are best proportioned on the basis of elastic bending moment distribution.

8.3.2(g) Inelastic Deformations

For continuous beams, inelastic deformations may have a negligible effect on the strength of the structure. For other types of structures, in particular multi-storey frames, these secondary effects may have a significant influence on the strength of the structure (ASCE 1971).

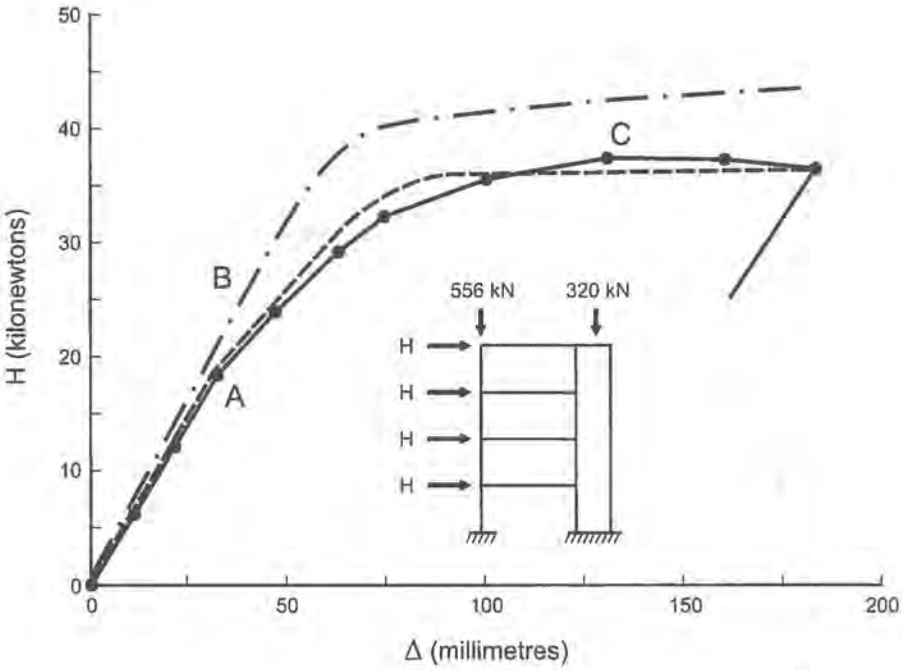


Figure 2-3
Load-Deflection Relationships

In the structure shown inset in Figure 2-2, the secondary effects have reduced the lateral load-carrying capacity (while maintaining the same vertical load) by approximately 25% (ASCE 1971; Adams 1974). The first plastic hinge formed at stage A in this structure, while the ultimate strength (considering moment redistribution) was not attained until stage C. The inelastic deformations between these two stages have reduced the overall strength of the structure. Clause 8.4 requires that the sway effects produced by the vertical loads be accounted for in design. Therefore, Clause 8.3.2(g) requires that, in a structure analyzed on the basis of a plastic moment distribution, the additional effects produced by inelastic sway deformations be accommodated. In most cases the actual strength of the structure can only be predicted by tracing the complete load-deflection relationship for the structure or for selected portions (Beedle *et al.* 1969). Methods are available to perform this type of design. For braced multi-storey frames, however, simpler techniques have also been developed (AISI 1968).

8.4 Stability Effects

Clause 8.4 recognizes that all building structures, whether unbraced or braced, are subjected to sway deformations. The vertical loads acting on the deformed structure produce secondary bending moments in the case of a moment-resisting frame, or additional forces in the vertical bracing system, in the case of a braced frame. These additional moments or forces (the stability effects) reduce the strength of the structure, as shown for a moment-resisting frame in Figure 2-2. In addition, bending moments and deflections, which exceed those predicted by a first-order analysis, are produced at all stages of loading (Adams 1974). Similar effects are produced in structures containing a vertical bracing system, as shown in Figure 2-3 where the steel frame is linked to a shear wall (Adams 1974).

8.4.1 Within the context of elastic analysis, there are essentially two general categories of procedures used to assess the stability of frames, namely, effective length approaches and notional load approaches. In S16.1-M89, the effective length approach in use prior to that time was abandoned because of the complexity involved in getting the approach to yield the correct solution. The notional lateral load approach makes use of the actual column length ($K = 1.0$) and was adopted in 1989. It has been used for the design of beam-columns in Canada since then (MacPhedran and Grondin, 2007).

The concept of notional lateral loads is an internationally recognized technique for transforming a sway buckling problem into a bending strength problem. It accounts for the effect of initial out-of-plumb in the columns and for partial yielding at factored load levels. Following the recommendation of Kennedy (1995), the notional load is applied to all design load combinations. Thus, the factored lateral force to be used in establishing the value of Δ at the various levels of the building is the summation of the applied lateral force and the notional load and the horizontal reaction to prevent sway from gravity loads. Since the notional loads are applied for the only purpose of accounting fully for the P-Δ effects on the overturning moment without the necessity of incorporating the initial out-of-plumb and inelastic effects in the analysis of the structure, they do not need to be considered for shear design. These notional shear forces do not exist when equilibrium of the structure is considered on the structure in its deformed configuration.

The magnitude of the notional lateral load, applied at each storey, is taken as 0.005 times the sum of the factored gravity loads contributed by that storey. While there is variation in international standards regarding the magnitude of the notional load coefficient (Bridge *et al.*, 1997), Clarke and Bridge (1992, 1995) have shown that $0.005\Sigma P$, established conservatively for a flagpole column (Kennedy *et al.*, 1990b), is an appropriate value that results in an adequate prediction of strengths in comparison with "exact" plastic zone analyses (Kanchanalai, 1977). There may be, as stated above, some conservatism in applying this magnitude of notional load to all load combinations in buildings where double-curvature bending of the columns predominates.

The use of the notional lateral load fulfills several important functions. The applied notional loads transform a bifurcation problem of sway buckling into a bending strength problem. Second, because it accounts for the P-Δ moments directly, the use of effective length factors greater than 1.0 is obviated, and its use allows effective lengths equal to the actual length to be used. At best the effective lengths used for sway buckling analyses are based on elastic analyses that are not appropriate for use with beam-column interaction equations that take into account inelastic material behaviour. Third, when equilibrium is formulated including the notional loads, the girders and beams restraining the columns are designed for the increased P-Δ moments that must exist in them for equilibrium just as the columns are. The use of effective lengths only accounts for increased moments in the columns and then only in an approximate manner with assumed elastic behaviour. Thus, although there may be some slight conservatism in using a notional load of $0.005\Sigma P$ compared to a lesser value, this is more than offset by the three advantages enumerated above.

It is noted that the flagpole column is bent in single curvature, whereas many columns in actual structures have some degree of double curvature. Consider now a sway column with complete fixity at both ends. It has very significant double curvature and an effective length of L. The sway buckling strength is now equal to the bending strength of a pin-ended column of the actual length with no notional lateral load because the effective length for buckling is equal to the actual length, L. These two cases show that the notional load required to transform the bifurcation problem of sway buckling into a bending strength problem depends on the end conditions in the actual structure and is greater when the degree of restraint is less. On the average, therefore, the notional load should be less than $0.005\Sigma P$, but Clarke and Bridge (1992, 1995) deem it to be the appropriate value.

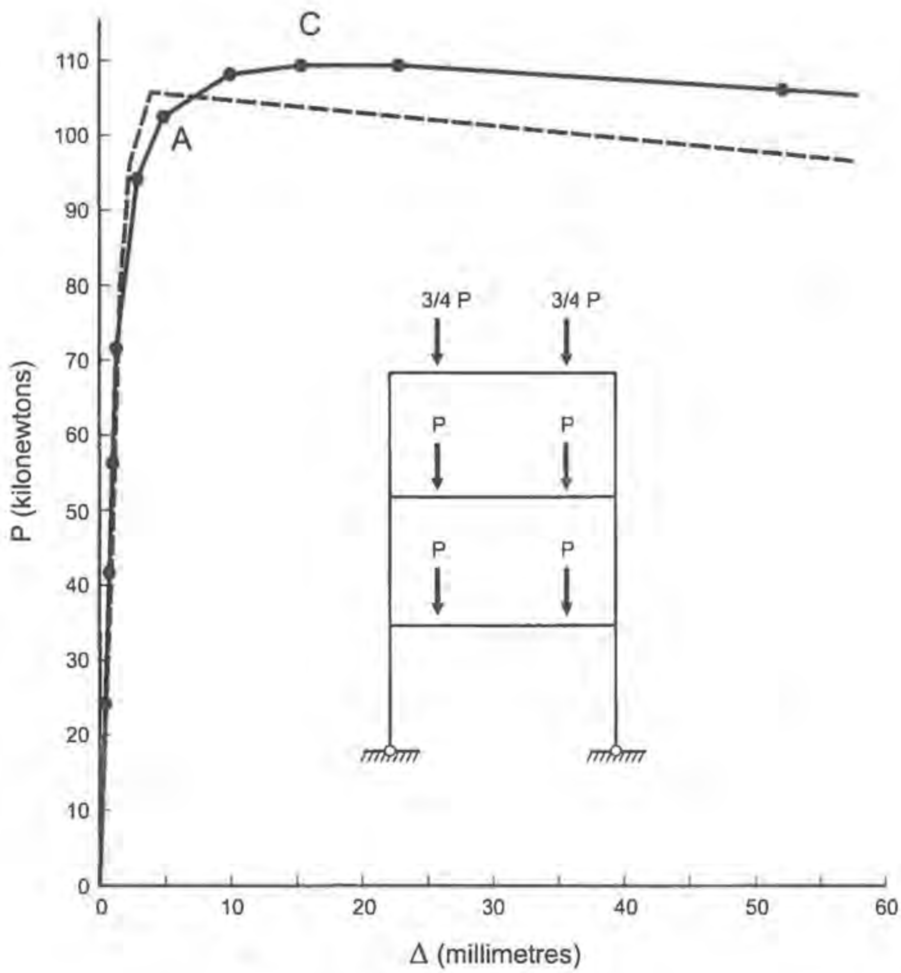
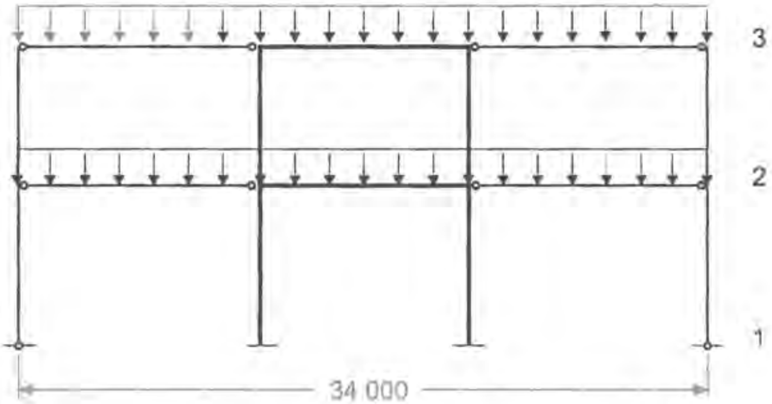


Figure 2-4
Load-Deflection Relationship – Vertical Load Only

The use of the notional lateral load remains of particular importance for structures subject to gravity loads only that may have insignificant lateral deflections and may only fail by elastic or inelastic sway buckling. Figure 2-4 shows a frame subject to vertical loads only. As the loads are increased, the effects of the vertical loads acting on the initial imperfections resulting from fabrication and erection lead to failure through instability, much the same as for the combined load case shown in Figures 2-2 and 2-3. The notional lateral loads of 0.005 times the factored gravity loads acting at each storey, as required by clause 8.4.1, simulate this condition. Figure 2-5 shows, for a frame loaded with gravity loads only, the notional lateral loads that would be used to calculate the translational moments and forces for this load combination.

When either the gravity loads or the structure or both are asymmetric, horizontal reactions at floor levels are obtained when computing M_{fg}, defined as the first-order moment under factored gravity loads determined assuming that there is no lateral translation of the frame as shown in Figure 2-6. These horizontal reactions, when released by applying sway forces in the opposite direction, produce translational effects and must be considered for all valid load combinations, in addition to the notional lateral loads or the actual lateral loads as appropriate.

8.4.2 Since the introduction of S16.1-M89, the designer must account for the sway effects directly. This is done by (1) performing a second-order geometric elastic analysis for the moments



Loads	Specified		Factored Gravity			Notional Lateral Load
	DL	LL	DL	LL	Total	
Level 3	10.8	22.5	13.5	33.75	47.25	0.005(47.25 x 34) = 8.03 kN
Level 2	18.0	18.0	22.5	27.00	49.50	0.005(49.5 x 34) = 8.42 kN

Note: For complete analysis of this frame, see Kennedy, *et al*, 1990.

Figure 2-5
Notional Lateral Loads for a Frame Subject to Gravity Loads

and forces, or (2) accounting for these effects by amplifying the first-order elastic translational moments by the factor U_2. The notional lateral loads (discussed in Clause 8.4.1) must be included in both of the above methods of analysis.

Computer programs are now commonly available to perform second-order elastic analyses based on equilibrium of the deformed structure. With these types of programs, the additional moments or forces generated by the vertical loads acting on the displaced structure (the so-called P-Δ effect) are taken into account directly and this method of analysis is the preferred method in Clause 8.4.2. In addition, most second-order programs also account for the change in column stiffness, caused by their axial loads (Galambos 1968).

The second approach in Clause 8.4.2 is simply to amplify the results of a first-order analysis to include the P-Δ effects. With this "amplification factor method", it is necessary to do two first-order analyses, one for gravity loading and the other for translational loading. From the horizontal displacements produced by the factored lateral loads, the amplification factor U_2 may be established. The factored moments or forces, including the effects of side-sway, may then be computed from:

$$M_f = M_{fg} + U_2\, M_{ft} \quad \text{or from} \quad T_f = T_{fg} + U_2\, T_{ft}$$

where $U_2 = \dfrac{1}{1 - \dfrac{\sum C_f \Delta_f}{\sum V_f h}}$

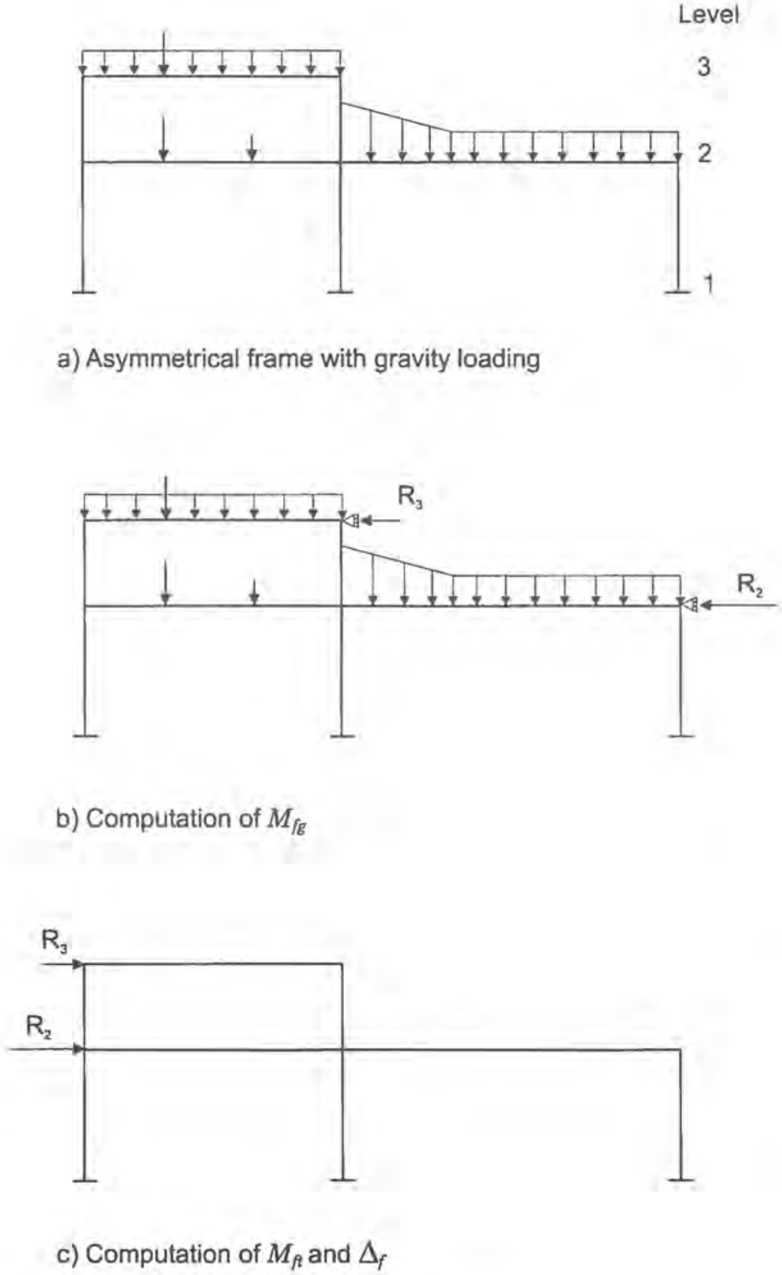
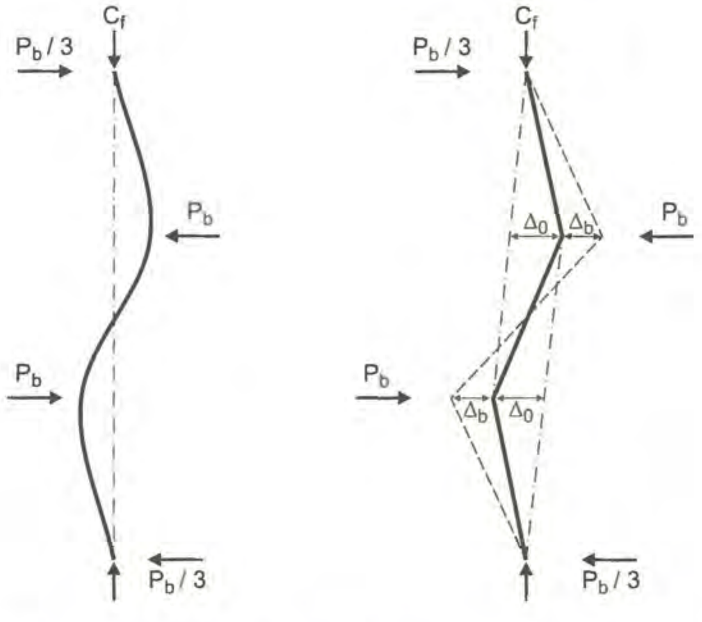


Figure 2-6

Starting with the 2001 Standard, the upper limit of 1.4 on the amplification factor U_2 was removed. The 1.4 limit was removed because the strength predictions for beam-columns compare well with the results of "exact" plastic zone finite element analyses when notional loads are applied to all load combinations. Nevertheless the designer is cautioned against designing structures that have excessive lateral deformations not only for the ultimate limit state of stability but also for serviceability considerations.



The solid line represents the initial misalignment. The dotted line represents the final displaced configuration due to all the forces acting on the system.

Figure 2-7
Δ_0 and Δ_b – Two Braces

9. STABILITY OF STRUCTURES AND MEMBERS

9.1 Stability of Structures

Emphasis continues to be placed on the designer's responsibility to ensure stability of the structure and of the individual members. Clause 8.4 requires the structure as a whole to resist the P-Δ effects.

The stability of the column-girder assembly and the girder web, when a girder is continuous over a column, requires careful assessment. The column, girder web, and the girder flange are all in compression, creating a condition of inherent instability. Stability can be achieved by providing lateral support to the girder-column joint or by properly designed web stiffeners restraining the rotation of the joint. See also the commentary on Clauses 16.5.11.1, 13.6 and the references cited therein.

9.2 Stability of Members

This clause applies equally to columns, the compression chord of joists and trusses, and the compressed portion of beams. For the last, it is only necessary to compute the maximum factored compressive force in that portion. The basic equation for the stiffness of the brace (Winter 1958) is derived on the premise that the brace or braces force the member to buckle into a series of half-sine waves of length, L, the distance between bracing points, with nodes at the bracing points. For this to occur, the braces must provide both strength and stiffness.

Most bracing assemblies in buildings have inherent torsional resistance. Normally, header connections provide sufficient torsional restraint at supports; however, Cheng *et al.* (1988), Cheng and Yura (1988) and Yura (1995) note that beams with deep or extended copes should be given special consideration.

Massey (1962) examined lateral bracing forces for beams, while Zuk (1956) and Lay and Galambos (1966) considered requirements for structures analysed plastically. Ziemian (2010) summarized many of the design requirements for bracing assemblies.

For additional discussion on member bracing, refer to Chapter 12 of Ziemian (2010).

9.2.1 Initial Misalignment at Brace Point

This requirement has changed in this edition of S16. Winter also showed that a critical parameter in designing the bracing is the initial out-of-straightness Δ_0 at the brace point. Based on S16 tolerances (Clause 29.3.3), a value for Δ_0 of no more than 0.001 times the distance between brace points may be used with Winter's model. A common construction technique used to reduce the initial misalignment is to pull the structure within tolerance at brace locations, bringing all column sections into compliance with the aforementioned Clause 29.3.3 for plumbness. Thus, when the structure is pulled into alignment, one brace point at a time, the Δ_0 that results is the erection tolerance.

Figure 2-7 shows the critical values of Δ_0 when two brace points exist.

9.2.2 Displacement of Bracing Systems

Δ_b is the displacement of the member being braced at the brace point perpendicular to the member caused by the force P_b and any other external forces. This deflection may be the result of axial shortening or elongation of the bracing or its flexural displacement depending on whether the bracing resistance is provided axially or by bending. In addition to the brace deformation, the brace connection deformation and the brace support displacement must be included.

The Simplified method of analysis is premised on a displacement Δ_b not greater than Δ_0, and therefore Δ_b shall not exceed Δ_0. When justified, this limit may be exceeded in either of the detailed methods.

In the case of girts bracing columns in the plane of the wall, the girts are the bracing members that deform. There could be deformation in the connections, and the shear deformation of the cladding is the displacement at the brace support.

In the case of a brace angle bracing the lower flange of a beam and connected to the upper flange of a secondary flexural member, the brace angle deforms axially; the connection to the brace angle may deform and the supporting secondary member may deflect flexurally to contribute to the deflection of the brace point perpendicular to the axis of the member.

When braces are supported by a truss system, the deformation of the truss between its points of support (assuming these are on the same line as the support of the members being braced) is the displacement of the brace supports. Figure 2-7 shows values of the displacements Δ_0 and Δ_b for a member braced at two locations.

9.2.4 Twisting and Lateral Displacements

The possibility of twisting of a member at brace points should be investigated and the bracing provided if necessary to prevent this.

The top (tension) flange at a cantilever, if not braced, can deflect laterally more than the bottom flange and therefore bracing of the cantilever end tension flange should be considered.

Torsional bracing can also increase the buckling load of cantilever beams.

The distortional buckling of steel beams in cantilever-suspended-span construction was examined by Albert *et al.* (1992), and Essa and Kennedy (1995) investigated torsional restraint stiffness provided by open-web steel joists. In this type of construction it is essential to analyze potential lateral displacements at the tops of supporting columns, because the beam web is also in vertical compression.

An inflection point cannot be considered a brace point (Ziemian 2010). Header connections normally provide sufficient torsional restraint at supports; however, Cheng *et al.* (1988), Cheng and Yura (1988) and Yura (1995) note that beams with deep or extended copes do not have similar torsional stiffness and should be given special consideration. This also applies for extended shear tab connections. Sherman and Ghorbanpoor (2002) indicate that bracing near the connection will compensate for the low torsional stiffness of the connection, and Thornton and Fortney (2011) give some design examples on calculating the torsional stiffness of an unbraced connection.

Simply supported beams in single curvature typically require only lateral bracing at the compression flange.

9.2.5 Simplified Analysis

The simplified analysis permitting a brace to be designed conservatively for a force equal to $0.02C_f$ has been reintroduced but with the qualification that the resulting deflection Δ_b shall not exceed the initial misalignment Δ_0, which is consistent with Winter's original provisions. As the long history of successful use has shown, this provides a brace of such strength that both the stiffness and strength requirements are generally satisfied.

9.2.6 Detailed Analysis

9.2.6.1 Second-Order Method

To begin this solution manually, a deformed configuration is assumed and bracing forces determined by statics in terms of the deflections. With these calculated forces, the resulting bracing deflections are computed and compared to the deformations initially assumed. The process is repeated until satisfactory convergence is achieved. In checking a design, if the calculated deformations are less than the assumed deformed configuration, the conditions of strength and stiffness are satisfied and there is no need for further calculations, unless further optimization is desired. Alternatively, brace forces and deformations can be obtained from a computerized second-order analysis that accounts for P-Δ effects provided that the structure is modelled with the most critical initial misalignment condition.

Iterations to determine the forces and deflections are performed on the most critical deformed configuration. Typical deformed configurations to be investigated include those shown on Figure 2-7. When hinges are assumed in the braced member at the brace points, a slightly conservative solution is obtained.

The second-order method is useful in checking as-built conditions.

9.2.6.2 Direct Method

The design brace force is given directly in the expression for P_b, where the factor β from Winter (1958) depends on the number of equally spaced braces by assuming that the displacement of the bracing system Δ_b is equal to the initial misalignment Δ_0. The required brace stiffness is $P_b/(\Delta_b + \Delta_0)$.

The initial assumption that Δ_b does not exceed Δ_0 must be confirmed.

By defining the maximum compressive force, C_f, as the maximum compression force in the segments bound by the brace points on either side of the brace point under consideration, the situation where a brace occurs near a point of contraflexure is accounted for. For trusses this applies when there is a significant change in the force in the chord at a panel point.

Consideration shall be made for cantilevered beams and beams bent in double curvature. Yura (1993) gives an amplification factor $C_d = [1 + (M_s/M_l)^2]$ where M_s and M_l refer to the smaller and larger moments, respectively. This yields a maximum value of 2 when $M_s = M_l$.

For loads applied above the shear centre, brace forces may be amplified. Yura (1993) gives a factor $C_d = (1 + 1.2/n)$ where n is the number of braces. Braces counterbalance this effect.

Calibration of the bracing requirements with finite element analyses suggest that in some cases twisting of chords in trusses may result in brace forces 25% higher than those predicted by the direct method.

9.2.8 Accumulation of Forces

When an element in a structure must resist the bracing forces from more than one member, the average maximum out-of-straightness of the members should be used to compute the bracing forces. Provided that member misalignment is independent among members, it can be shown statistically that the average maximum out-of-straightness is a function of the maximum out-of-straightness of one member divided by the square root of the number of members (Kennedy and Neville 1986). The expression given in the standard is a conservative empirical equation that applies the statistical reduction to only 0.80 of the initial misalignment. In the design of such bracing systems it must be recognized that the (axial) displacement of the in-line brace increases from the location where the brace is affixed or restrained to the most remote member, and the force in the in-line brace increases in the opposite direction. Beaulieu and Adams (1980) provide more guidance in selected cases.

In many cases two parallel frames or members are brought into alignment, and whatever misalignment remains is reflected in the initial position of the remaining members. The statistical reduction in the initial misalignment does not apply, and all members have essentially the same Δ_0.

9.2.9 Torsion

Because the shear centre of a monosymmetric or an asymmetric section does not coincide with the centroid, these sections may be loaded so as to (unintentionally) produce torsion and biaxial bending. Both the connections and the members providing reactions should be checked.

10. DESIGN LENGTHS AND SLENDERNESS RATIOS

10.1 Simple Span Flexural Members

For design purposes, it is usually convenient to consider the length of a member as equal to the distance between centres of gravity of supporting members. In most instances the difference resulting from considering a member to be that length rather than its actual length, centre-to-centre of end connections, is small. In some cases, however, there is sufficient difference to merit computing the actual length. Regardless of the length used for design, the actual connection detail may cause an eccentric load, or moment, to act on the supporting member, and this effect must be taken into account.

10.3 Members in Compression

10.3.1 General

The unbraced length and the effective length factors may be different for different axes of buckling. Information about effective lengths is given in Ziemian (2010) and Tall *et al.* (1974).

Further guidance is provided in Annexes F and G of the Standard. The second-to-last sentence of Clause 10.3.1 introduces the concept that effective length factors depend on the potential failure mode – how the member would fail if the forces (and moments) were increased sufficiently – as discussed in subsequent clauses.

10.3.2 Failure Mode Involving Bending In-Plane

When the end moments and forces acting on a beam-column have been determined for the displaced configuration of the structure, that is to say, the sway effects have been included as required by Clause 8.4, the in-plane bending strength of the beam-column can be determined by analyzing a free-body of the member isolated from the remainder of the structure. In-plane displacements between the ends, which contribute to failure, arise from the end-moments and forces acting on the actual length. When the actual member length and the actual (or at least approximate) deflected shape are used, the analysis of the free-body will yield close to the correct member strength. Recourse to effective length factors is neither necessary nor appropriate.

When the actual member length is used together with the interaction expressions of Clause 13.8, the analysis is approximate and the in-plane member bending strength obtained will tend to be conservative. This simply arises because the value of the compressive resistance inherent in the interaction expression by using a length equal to the actual length (a K factor of 1.00) is that corresponding to single curvature buckling. For any other deflected shape, having accounted for sway effects, the compressive resistance is greater because the points of inflection of the deflected member shape are less than the member length apart. Under these circumstances, a better estimate of the strength, as is indeed permitted under Clause 1.4, can be obtained when the compressive resistance is based on the actual distance between points of inflection. Inelastic action of the member in the structure, however, may make this determination onerous. Therefore the relatively simple but sometimes conservative approach given in the Standard which obviates the use of effective length factors is presented as the usual procedure.

10.3.3 Failure Mode Involving Buckling

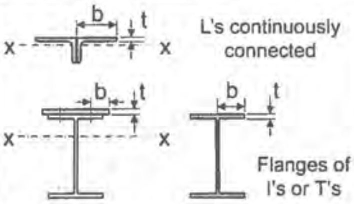
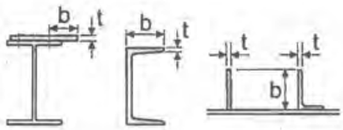
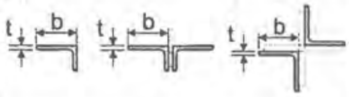
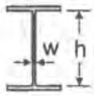
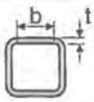
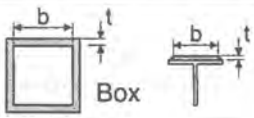
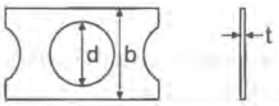
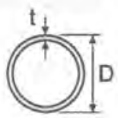
The compressive resistance of an axially loaded column depends on its end restraints, as does the out-of-plane buckling resistance of a beam-column under uniaxial strong-axis bending. The failure is a bifurcation mechanism.

10.4 Slenderness Ratios

The maximum slenderness ratio of 200 for compression members, stipulated as long ago as the 1974 Standard, has been retained in S16-14 for the reason that strength, or resistance, of a compression member becomes quite small as the slenderness ratio increases and the member becomes relatively inefficient.

For considerations of strength, no limiting slenderness ratio is required for a tension member and, indeed, none is applied to wire ropes and cables. However, a slenderness ratio limit of 300 is given with permission to waive this limit under specified conditions. The limit does assist in the handling of members and may help prevent flutter under oscillating loads such as those induced in wind bracing designed for tension loads only. Tension chords of trusses and joists have more stringent slenderness ratios (see commentary on Clauses 15 and 16).

Members whose design is governed by earthquake loadings may be subject to more stringent slenderness ratios, depending on the ductility requirements of the lateral load-resisting system. See Commentary on Clause 27.

Detail	Class 1	Class 2	Class 3
 L's continuously connected; Flanges of I's or T's	$\frac{b_{el}}{t} \le \frac{145}{\sqrt{F_y}}$ †	$\frac{b_{el}}{t} \le \frac{170}{\sqrt{F_y}}$ †	$\frac{b_{el}}{t} \le \frac{200}{\sqrt{F_y}}$ Flanges of I's in minor-axis bending $\frac{b_{el}}{t} \le \frac{340}{\sqrt{F_y}}$
	——	——	Flanges of C's, asymmetric cover plates, plate girder stiffeners $\frac{b_{el}}{t} \le \frac{200}{\sqrt{F_y}}$
	——	——	L's not continuously connected $\frac{b_{el}}{t} \le \frac{250}{\sqrt{F_y}}$
 Stems of T's	$\frac{b_{el}}{t} \le \frac{145}{\sqrt{F_y}}$ †	$\frac{b_{el}}{t} \le \frac{170}{\sqrt{F_y}}$ †	$\frac{b_{el}}{t} \le \frac{340}{\sqrt{F_y}}$
	Bending only $\frac{h}{w} \le \frac{1100}{\sqrt{F_y}}$ Axial compression ——	Bending only $\frac{h}{w} \le \frac{1700}{\sqrt{F_y}}$ Axial compression ——	Bending only $\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}$ Axial compression $\frac{h}{w} \le \frac{670}{\sqrt{F_y}}$
 HSS	$\frac{b_{el}}{t} \le \frac{420}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{670}{\sqrt{F_y}}$
 Box	$\frac{b_{el}}{t} \le \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{670}{\sqrt{F_y}}$
	——	——	$\frac{b_{el}}{t} \le \frac{840}{\sqrt{F_y}}$
	Bending only $\frac{D}{t} \le \frac{13\,000}{F_y}$ Axial compression ——	Bending only $\frac{D}{t} \le \frac{18\,000}{F_y}$ Axial compression ——	Bending only $\frac{D}{t} \le \frac{66\,000}{F_y}$ Axial compression $\frac{D}{t} \le \frac{23\,000}{F_y}$

† Symmetric about plane of bending or including asymmetry effects in analysis

Figure 2-8
Width-to-Thickness Ratios for Compression Elements

11. WIDTH (OR DIAMETER)-TO-THICKNESS – ELEMENTS IN COMPRESSION

Clause 11 emphasizes the distinction between elements in axial compression and elements in flexural compression by placing the maximum width-to-thickness ratios for these elements in Tables 1 and 2 respectively.

Clause 11.1.1 identifies four categories of cross-sections, Class 1 through Class 4, based upon the width-thickness ratios of the elements of the cross-section in compression that are needed to develop the desired flexural behaviour. With the ratios given in Table 2 of Clause 11 for Classes 1, 2, or 3, the respective ultimate limit states will be attained prior to local buckling of the plate elements. These ultimate limit states are: Class 1 – maintenance of the plastic moment capacity (beams), or the plastic moment capacity reduced for the presence of axial load (beam-columns), through sufficient rotation to fulfill the assumption of plastic analysis; Class 2 – attainment of the plastic moment capacity for beams, and the reduced plastic moment capacity for beam-columns, but with no requirement for rotational capacity; Class 3 – attainment of the yield moment for beams, or the yield moment reduced for the presence of axial load for beam-columns. Class 4 – have plate elements that buckle locally before the yield strength is reached.

Elements in Flexural Compression

The requirements given in Figure 2-8 for elements of Class 1, 2, and 3 sections in flexural compression, particularly those for W-shapes, are based on both experimental and theoretical studies. For example, the limits on flanges have both a theoretical basis (Kulak and Grondin 2014; ASCE 1971; Ziemian 2010) and an extensive experimental background (Haaijer and Thurlimann 1958; Lay 1965; Lukey and Adams 1969). For webs in flexural compression the limits $1100/\sqrt{F_y}$, $1700/\sqrt{F_y}$ and $1900/\sqrt{F_y}$ for Class 1, 2 and 3, respectively, when $C_f/\phi C_y = 1.0$ come from both theory and tests on Class 1 sections (Haaijer and Thurlimann 1958) but mostly from test results for Class 2 and 3 sections (Holtz and Kulak 1973 and 1975).

For circular hollow sections in flexure, see Stelco (1973) for the requirements for Class 1 and Class 2 sections and Sherman and Tanavde (1984) for Class 3.

Elements in Axial Compression

The distinction between classes based on moment capacity does not apply to axially loaded members as the plate elements need only reach a strain sufficient for the plate elements to develop the yield stress. This strain is affected by the presence of residual stresses, but there is no applied strain gradient across elements of the cross-section as there is for members subject to flexure. The width-thickness limits for the various plate elements are not dependent on the Class of the section and are only a function of the residual stress pattern and the edge conditions. Thus for webs, from Table 2 in the Standard for each of Classes 1, 2 and 3 when $C_f/\phi C_y = 1.0$, the limit on h/w is the same value of about $670/\sqrt{F_y}$ as given in Table 1. The width-thickness limit for the flanges of axially loaded columns, based on the same argument, is the same as for Class 3 beam flanges, i.e., $200/\sqrt{F_y}$ (Dawe and Kulak 1984). As well the limit on the D/t ratio of $23\,000/F_y$ (Winter 1970) for circular hollow sections in axial compression is the same irrespective of the Class.

Elements in Compression Due to Bending and Axial Load

(a) Major-axis bending and axial compression

In Figure 2-9, the requirements for webs in compression ranging from compression due to pure bending to that due to pure compression are plotted. Because all of the web is in

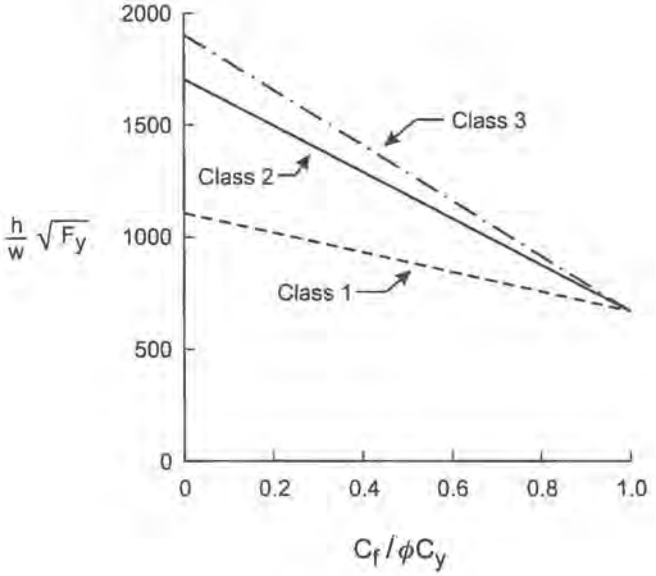


Figure 2-9
Width-to-Thickness Ratios for Webs

compression for columns and only one-half for beams, the depth-to-thickness limits vary as a function of the amount of axial load. The results presented here reflect the research results of Dawe and Kulak (1986).

(b) I-sections in minor-axis bending and axial compression

Since the applied stresses in Class 3 I-sections remain linear-elastic, the web experiences little compressive stress due to bending. Hence, the h/w limits for Class 3 sections in combined major-axis bending and axial compression also apply to Class 3 sections in combined minor-axis bending and axial compression. For Class 1 and Class 2 I-sections, h/w limits that are more stringent than those for major-axis bending and axial compression have been adopted in recognition of the full web in uniform compression.

(c) I-sections in biaxial bending and axial compression

When minor-axis bending stresses dominate in Class 1 and Class 2 I-sections in biaxial bending and axial compression, more stringent h/w limits than those for major-axis bending and axial compression also apply.

Class 4 Sections

Sections used for columns, beams, or beam-columns may be composed of elements whose width-to-thickness ratios exceed those prescribed for Class 3 provided that the resistance equations are adjusted accordingly. These sections, called Class 4, are evaluated according to the rules given in Clause 13.3 or 13.5 as applicable.

12. GROSS AND NET AREAS

12.1 Application

The design and behaviour of tension members is integrally related to the proportioning and detailing of connections. Consequently, Clauses 12, 13.2 and 13.11 are related. Two possible overall failure modes exist: unrestricted plastic flow of the gross section and fracture of a net section. The second of these consists itself of three modes depending on the failure path and the degree of ductility available. The Commentary on Clause 13.2 discusses three specific tensile failure modes and that on Clause 13.11 treats combined tension and shear. Commentaries on failure areas are given here.

12.2 Gross Area

Yielding on the gross area from one end of the member to the other resulting in unrestricted plastic flow can occur before fracture on a net section. The gross area is obtained simply as the sum of the products of the thickness and gross widths of all cross-sectional elements.

12.3 Net Area

This clause defines areas used to determine tension member resistances. The requirements apply to both bolted and welded connections.

12.3.1 General

When each portion of the cross-section of a tension member is connected with sufficient fasteners to transmit the load attributable to that portion, the stress distribution at the connection is reasonably uniform, and the provisions of Clause 12.3.1 apply to the net area calculations. To establish the critical net area, all potential failure paths are examined. When the failure plane includes segments inclined to the applied force, an empirical term, $s^2t/4g$, is added to the net area to correct for the presence of each inclined segment.

In determining the net area by summing the net area of each segment along the critical path, it is assumed, as has been demonstrated (Birkemoe and Gilmor 1978; Ricles and Yura 1983; Hardash and Bjorhovde, 1985), that all segments reach their full capacity simultaneously.

12.3.2 Allowance for Bolt Holes

The 2 mm allowance for bolt holes accounts for distortion or local material damage that may occur in forming the hole by punching. If it is not known at the design stage that the holes will be drilled or sub-punched and reamed, then punched holes should be assumed. The 2 mm allowance also is used with oversize or slotted holes.

12.3.3 Effective Net Area - Shear Lag

When the critical net section fracture path crosses unconnected cross-sectional elements, the directly connected elements tend to reach their ultimate strength before the complete net section strength is reached due to shear lag. When all cross-sectional elements are directly connected, shear lag does not occur and the effective net area is the total net area.

The loss in efficiency due to shear lag can be expressed as a reduction in the net area. Munse and Chesson (1963) suggested that this reduction could be taken as $1-\bar{x}/L$ where $\bar{x}$ is the distance from the shear plane to the centroid of that portion of the cross-section being developed and L is the connected length.

Because the connected length is usually not known at the time of tension member design, reduction factors have been derived for specific cases, as given in Clause 12.3.3.2, based on an extensive examination of the results of over 1000 tests (Kulak *et al.* 1987). The reduction factor depends on the cross-sectional shape and the number of bolts (2, 3 or more) in the direction of the tensile load.

More severe reductions for shear lag are provided for angles connected by one leg based on work by Wu and Kulak (1993), who examined many test results on angles in tension connected with mechanical fasteners.

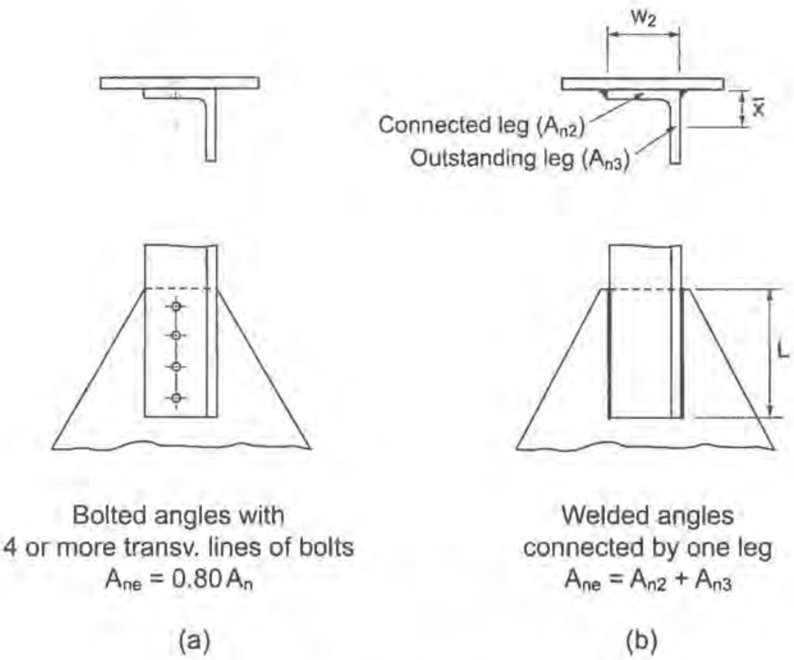


Figure 2-10
Dimensions Used for Shear Lag Calculations

When block tear-out occurs in those elements that are directly connected, shear lag is not a factor. Shear lag need only be considered when the potential failure path under consideration crosses unconnected elements.

12.3.3.3 Similar reductions due to shear lag have been observed in welded connections (Kulak *et al.* 1987) when only welds parallel to the tensile load in the member are used. If the elements of the cross-section are connected by welds transverse to the tensile load, no reduction due to shear lag is necessary. For welded connections with matching electrodes and material of G40.21-300W grade steel, shear lag will be critical for cases where $A_{ne} \leq 0.78 A_g$. For angles, this generally occurs when the length of weld along the toe exceeds the length of weld along the heel.

When the weld length is less than the distance between welds, it is likely that the weld is critical.

Provisions for shear lag in bolted and welded angles are illustrated in Figure 2-10.

12.3.3.4 The shear lag expressions from past specifications have been expanded to specifically address slotted HSS brace end connections. This addition reflects the work of Martinez-Saucedo and Packer (2009) who demonstrated that a non-linear function best described the shear lag effect on a number of different slotted round and rectangular/square HSS connections. This function, designated U for the cross-sectional efficiency such that $A_{ne} = UA_n$, is plotted on Figure 2-11 and makes a smooth transition across the three limit states observed during testing: (1) yielding and necking, (2) net section fracture from shear lag effects, and (3) tube wall tear out from block shear. The new expression from clause 12.3.3.4 is seen to be a reasonably conservative linear approximation of the expression proposed by Martinez-Saucedo and Packer. The term was introduced to emphasize that the eccentricity that should be considered in the shear lag expressions should be measured from the face of the gusset.

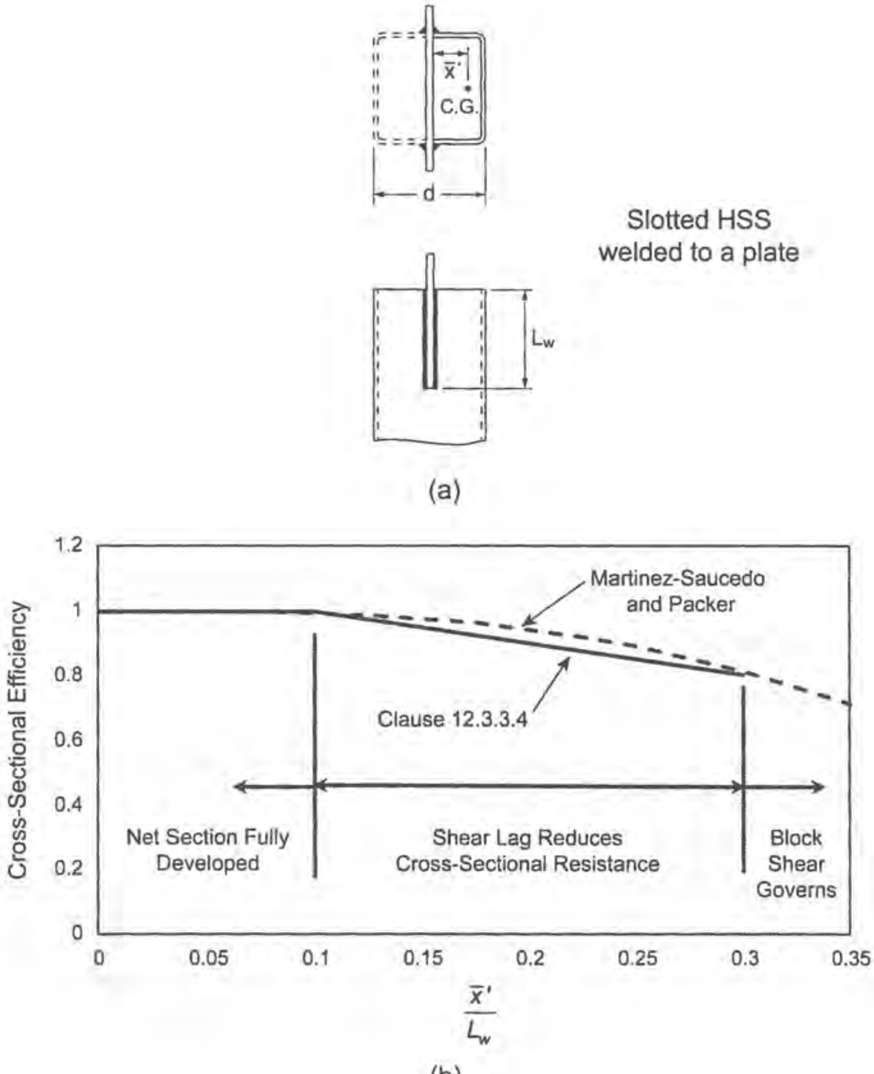


Figure 2-11
Shear Lag Effects on Slotted HSS Brace Ends

12.4 Pin-Connected Members in Tension

The dimensional requirements presented in Figure 2-12 must be met to provide for the proper functioning of the pin.

The pin hole shall be located midway between the edges of the member in the direction normal to the applied force.

The plate shall be of uniform thickness. The width of the plate at the pin hole shall not be less than $2b_e + d$, and the clear end distance, a, beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than $1.33b_e$. The corners beyond the pin hole may be cut at 45° to the axis of the member, provided $c \geq a$ as shown in Figure 2-12.

Pin-connected members are occasionally used as tension members with very large dead loads. Pin-connected members are not recommended when there is sufficient variation in live loading to cause wearing of the pins in the holes.

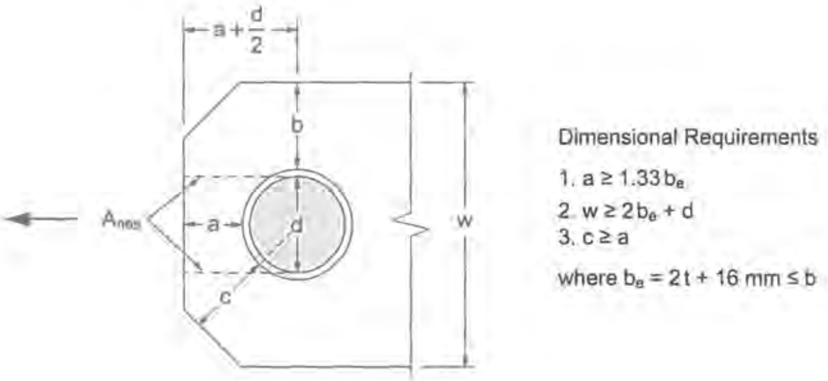


Figure 2-12
Dimensional Requirements for Pin-Connected Members

13. MEMBER AND CONNECTION RESISTANCE

13.1 Resistance Factors

For convenience, all resistance factors are listed in Clause 13.1. The long-used basic value of ϕ of 0.90 for most resistances continues to provide consistent and adequate values of the reliability index when used with the load factors of Clause 7.2. (Kennedy and Gad Aly 1980, Baker and Kennedy 1984, Schmidt and Bartlett 2002). In S16-14, no new resistance factors have been added to the group.

In S16-09, the resistance factor for bolt bearing on steel was increased from 0.67, adopted in a previous edition of the Standard, to 0.80. A recent reliability analysis demonstrated that a resistance factor of 0.80 provides an adequate margin of safety (Stankevicius *et al.* 2009). In earlier versions of S16, the resistance factor $\phi = 0.90$ was used in the equation for rupture of tension members at the net section. This resistance factor was multiplied by a factor of 0.85 to increase the safety index to about 4.0 to 4.5 for this ultimate limit state. Beginning in S16-09, 0.85ϕ has been replaced by $\phi_u = 0.75$, which is just slightly lower than 0.85ϕ. The resistance factor ϕ_c is 0.65, which is consistent with the reinforced concrete design standard A23.3.

13.2 Axial Tension

The two overall potential failure modes for tension members and their connections are yielding of the gross section and fracture of a net section. Fracture of a net section further consists of three possible modes depending on how the elements of the cross-section are connected and how the net sections are loaded. Thus all possible failure modes described must be examined to establish the value that governs the factored tensile resistance. The resistances of two of the fracture modes chiefly involving tension on the net section are presented in Clause 13.2. The failure mode involving a combination of tension and shear, in which a block of material tears out, is referred to as block shear failure and is discussed in the commentary to Clause 13.11.

The appropriate areas to be used in each of the three modes are described in Clause 12.

The first of the three failure modes involves unrestricted plastic flow of the gross section when the yield deformations over the length of the member are excessive. This represents a limit state for which the failure is gradual. A reliability index β of 3.0 is considered acceptable for the tension member and thus the tensile resistance is

$$T_r = \phi A_g F_y; \text{ (with } \phi = 0.90)$$

The second failure mode, involving a combination of tension and shear, in which a block of material tears out – block shear failure – is discussed in the commentary on Clause 13.11.

The third failure mode involves fracture of the member at the net section. The net section area can either be fully effective if all parts of the cross-section are connected, or it can be only partially effective if shear lag is present. Because this fracture occurs with little deformation and no reserve of strength exists beyond rupture, an increased value of β is appropriate for cases of fracture at the net section. In S16-14 (since S16-09) the tensile resistance for this mode is written as:

$$T_r = \phi_u A_{ne} F_u$$

The resistance factor $\phi_u = 0.75$ used for this limit state results in an increased value of β of about 4.5. This philosophy is consistent with the reduced resistance factor used for connectors (bolts, welds, and shear connectors). The net effective area, A_{ne}, accounts for possible shear lag effect. If no shear lag is present, then $A_{ne} = A_n$.

Clause 13.2(b) applies to pin connections, except that more specific requirements apply to eyebars. The equation in (i) provides the gross-section yielding resistance; the equation in (ii) gives the net-section fracture resistance, and the equation in (iii) covers shear rupture or end tear-out.

13.2(b) Pin Connections

The tensile strength requirements for pin-connected members use the same resistance factor ϕ as elsewhere in this Standard for similar limit states. However, the definitions of effective net area for tension and shear, as given in Clause 12.4, are different. The requirements in this Clause have been adapted from ANSI/AISC 360-10. Design of eyebars requires more specific rules.

13.3 Axial Compression

Depending on the type of cross-section, the buckling load of an axially loaded compression member may be governed by flexural buckling, by torsional buckling or by flexural-torsional buckling.

13.3.1 Flexural Buckling of Doubly Symmetric Shapes

Axially loaded compression members with doubly-symmetric cross-sections, such as wide-flange shapes, I-shaped and HSS members that dominate in steel construction, normally reach their ultimate capacity either by yielding or by flexural buckling, the most common buckling mode.

Steel columns are conveniently classified as short, intermediate, or long members, and each category has an associated characteristic type of behaviour. A short column is one that can resist a load equal to the yield load ($C_y = A\,F_y$). A long column fails by elastic buckling. The maximum load depends only on the bending stiffness (EI) and length of the member. Columns in the intermediate range are most common in steel buildings. Failure is characterized by inelastic buckling and is greatly influenced by the magnitude and pattern of residual stresses that are present and the magnitude and shape of the initial imperfections or out-of-straightness. These effects are less severe for both shorter and longer columns. The expressions in this clause account for these effects that are dependent on the cross-section (Bjorhovde 1972).

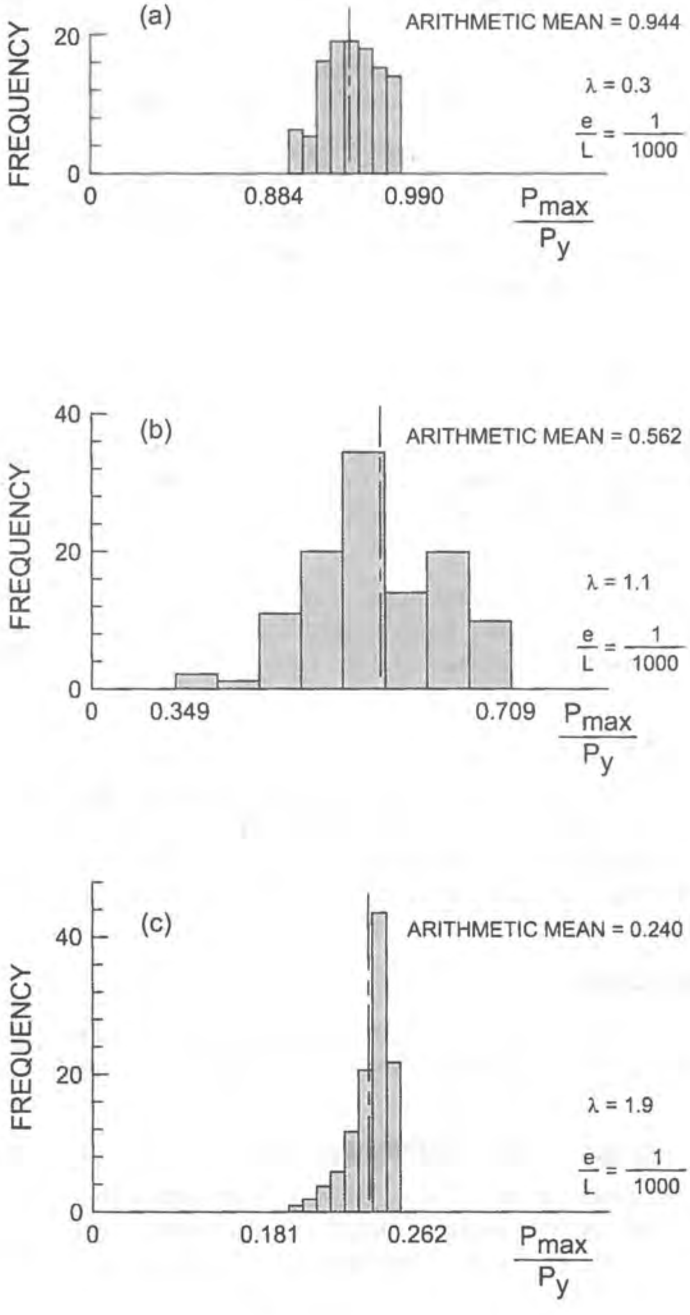


Figure 2-13
Typical Frequency Distribution Histograms for the Maximum Strength of 112 Column Curves (e/L = 1/1000)

Figure 2-13 indicates the variations in strengths for columns of three different values of the slenderness parameter, λ, and with the same out-of-straightness patterns and different residual stress patterns.

The compressive resistance expressions of Clause 13.3.1 are expressed in double exponential form (Loov 1996). With values of the parameter n of 1.34 and 2.24 for the cases shown in Clause 13.3.1, the expressions are always within 3% and generally within 1% of column Curves 2 and 1, respectively, of the Structural Stability Research Council (SSRC) (Ziemian 2010).

Steel shapes, unless explicitly stated, are assigned to SSRC Curve 2 ($n = 1.34$) which is used for hot-rolled, fabricated structural sections and for cold-formed, non-stress-relieved Class C hollow structural sections manufactured according to CSA Standard G40.20 (Bjorhovde and Birkemoe 1979). HSS produced to ASTM A500 grades B and C are cold-formed non-stress relieved, and the use of $n = 1.34$ is therefore appropriate.

Because of a more favourable residual stress pattern and out-of-straightness, hot-formed or cold-formed stress relieved (Class H) hollow structural sections (Kennedy and Gad Aly, 1980) are assigned to SSRC Curve 1 or its equivalent curve here with a value of $n = 2.24$. For the same reasons, doubly-symmetric three-plate members with flange edges oxy-flame-cut are also assigned to the curve with $n = 2.24$ (Chernenko and Kennedy, 1991).

For heavy sections (W310x313 and heavier and W360x347 and heavier, referred to as Groups 4 and 5 sections in earlier versions of CSA Standard G40.20) made of ASTM A7 or A36 steel and welded sections fabricated from universal mill plate, a resistance less than that corresponding to $n = 1.34$ (SSRC Curve 2) is appropriate, and it is recommended that a value of $n = 0.93$, corresponding to Column Curve 3 (Ziemian 2010), be used.

Because column strengths are influenced by the magnitude and distribution of residual stresses, care should be exercised in the use of the expressions in this Standard. For example, adding material such as welded cover plates increases the area and may reduce the slenderness ratio of an existing column, but it may also increase the compressive residual stresses in fibres remote from the centroid of the member, thus detracting from the strength.

13.3.2 Flexural, Torsional or Flexural-Torsional Buckling

Two other modes of buckling, which may occur prior to flexural buckling, are torsional or flexural-torsional buckling.

Torsional buckling with twisting about the shear centre is a possible failure mode for point-symmetric sections, e.g. a cruciform section, and in some circumstances, for doubly-symmetric sections. Flexural-torsional buckling, a combination of torsion and flexure is a possible failure mode for open sections that are singly-symmetric or asymmetric such as T's and angles. Thus, for sections with coincident shear centre and centroid, three potential compressive buckling modes exist (two flexural and one torsional), while for singly symmetric sections two potential compressive buckling modes (one flexural and one flexural-torsional) exist and, for a non-symmetric section, only one mode (flexural-torsional) exists. Closed sections, strong torsionally, also do not fail by flexural-torsional buckling (see Ziemian 2010). For the theory of elastic flexural-torsional buckling see Goodier (1942), Timoshenko and Gere (1961), Vlasov (1959) and Galambos (1968). The equations given here are developed in the latter among others.

As the problem of inelastic flexural-torsional buckling is quite complex and is amenable generally only to inelastic finite element analyses, the approach given here is to compute the elastic buckling stress, F_e, from the equations given for doubly symmetric, singly symmetric or asymmetric sections and then calculate an equivalent slenderness ratio $\lambda = \sqrt{F_y/F_e}$ to be used in the equations of Clause 13.3. This comes from the fact that an elastic buckling curve, when non-dimensionalized by dividing by F_y can be written as $F_e/F_y = 1/\lambda^2$. When the inelastic equations of 13.3 are entered with the equivalent slenderness ratio, an inelastic compressive resistance results.

The equations given here are equivalent to those in CSA Standard S136. There, however, for singly symmetric sections, the x-x axis is taken as the axis of symmetry, because cold-formed channel sections are frequently used.

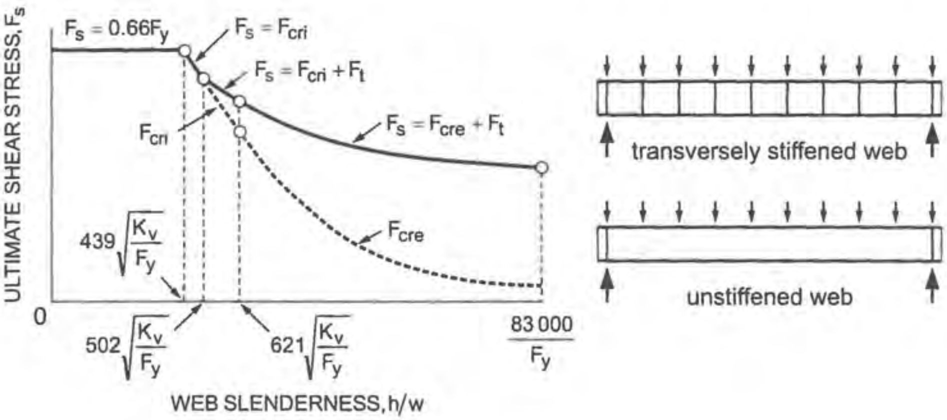
13.3.3 Single-Angle Members in Compression

The design of single angles subjected to axial compression is addressed. The angle is connected by a single leg, which is attached to a gusset plate or the projecting leg of another member by welding or by a bolted connection (at least two bolts), and is not subjected to any transverse loading. The effect of end eccentricity and rotational end restraint, hence, any resulting flexure of the angle, is indirectly accounted for by incorporating the equivalent slenderness expressions provided in this Clause. These expressions have also been adopted by the AISC Specification (2010b). They are essentially equivalent to those specified for equal-leg angles as web members in latticed transmission towers in ASCE 10-97 (ASCE 2000). The slenderness expressions are considered valid for equal-leg angles or unequal-leg angles connected by the longer leg (ratio of long leg / short leg < 1.7). It is assumed that significant restraint about the y-axis, which is perpendicular to the connected leg (note regarding convention: where the longer leg is connected, this axis is defined as the x-axis in the section properties tables for single angles in the CISC Handbook), exists due to the end connections. This causes the angle to flex and buckle primarily about the x-axis, hence, the use of the radius of gyration about the geometric axis parallel to the connected leg, r_x. The expressions for box trusses reflect greater rotational end restraint as compared to that provided by planar trusses. The slenderness expressions are not intended for use in the calculation of compression resistance of single angles used as diagonal braces in a braced frame. The procedure allows for the use of unequal-leg angles attached by the smaller leg provided that the equivalent slenderness is increased by an amount that is a function of the ratio of the longer to the shorter leg lengths. A minimum slenderness limit based on the slenderness about the minor principal axis must be met in all cases.

If the single-angle compression members cannot be evaluated using the equivalent slenderness expressions, then the provisions of Clause 13.3.2 shall be used for design accounting for the effect of end eccentricity and rotational end restraint. In evaluating C_r, the effective length due to end restraint should be considered. The procedure documented by Lutz (1992) to compute an effective radius of gyration for the angle can be implemented.

13.3.5 Members in Compression Subjected to Elastic Local Buckling

Two alternatives are available for approximating the factored compressive resistance of compression members that do not meet the local buckling requirements. The first is based on the notional removal of the width in excess of the limit for plate elements in axial compression to determine a reduced cross-sectional area. This area is used with the specified minimum yield strength and a slenderness based on the gross cross section to determine the factored compressive resistance by Clause 13.3.2 or 13.3.3. In the second alternative, the existing b/t ratio is used to establish the effective yield strength of a section just meeting the Class 3 limits. With this reduced yield strength and the gross cross section properties, Clause 13.3.2 or 13.3.3 establishes the factored resistance. Results by the two methods will not necessarily be the same. It is not necessary to refer to CSA S136 for members in axial compression that are subjected to elastic local buckling.



$$F_{cri} = \frac{290\sqrt{F_y k_v}}{h/w} \qquad k_v = 4 + \frac{5.34}{(a/h)^2} \qquad \text{when } \frac{a}{h} < 1$$

$$F_{cre} = \frac{180\,000 k_v}{(h/w)^2} \qquad k_v = 5.34 + \frac{4}{(a/h)^2} \qquad \text{when } \frac{a}{h} \geq 1$$

$$F_t = k_a\left(0.50F_y - 0.866F_{cri}\right) \text{ when } 502\sqrt{k_v/F_y} < h/w \leq 621\sqrt{k_v/F_y}$$

$$= k_a\left(0.50F_y - 0.866F_{cre}\right) \text{ when } 621\sqrt{k_v/F_y} < h/w$$

Figure 2-14
Ultimate Shear Stress – Webs of Flexural Members

13.4 Shear

13.4.1.1 Elastic Analysis

The expressions for shear strength are given for unstiffened and stiffened plate girders. Unstiffened plate girders and rolled beams are simply special cases for which the shear buckling coefficient, $k_v = 5.34$.

The four ranges of resistance based on Basler (1961) correspond to the following modes of behaviour and are illustrated in Figure 2-14 for stiffened webs:

(a) Full yielding followed by strain-hardening and large deformation. The limiting stress of $0.66\,F_y$ corresponds to shear deformation into the strain-hardening range and is higher than that derived from the von Mises criterion ($0.577\,F_y$), which forms the basis of Clause 13.4.2 for plastic analysis.

(b) A transition curve between strain-hardening and inelastic buckling at full shear yielding. ($F_s = 0.577\,F_y$);

(c) Inelastic buckling, F_{cri}, accompanied by post-buckling strength, F_t, due to tension field action, if the web is stiffened; and,

(d) Elastic buckling, F_{cre}, accompanied by post-buckling strength, F_t, due to tension field action, if the web is stiffened.

In computing the shear resistance, it is assumed that the shear stress is distributed uniformly over the depth of the web. The web area (A_w) is the product of web thickness (w) and

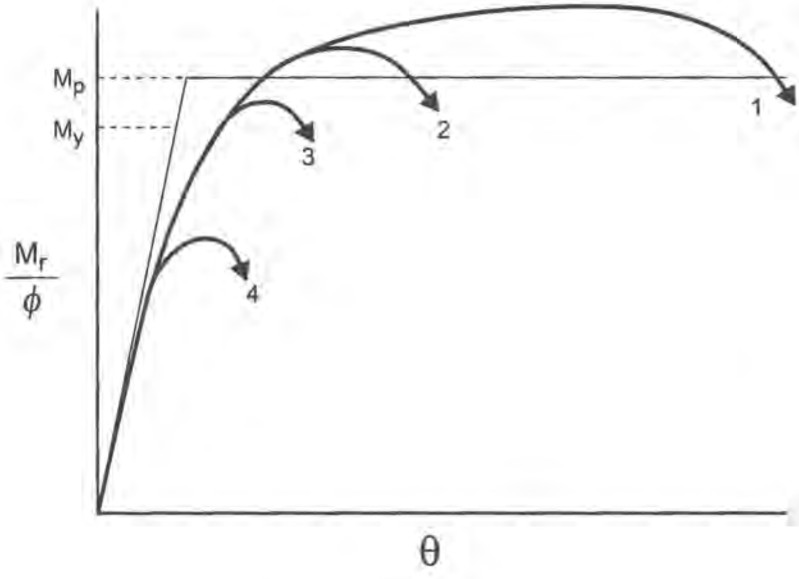


Figure 2-15
Moment-Rotation Curves

web depth (h) except for rolled shapes where it is customary to use the overall beam depth (d) in place of the web depth (h).

In panel zones and locations where strain-hardening develops quickly after the onset of shear yielding, the use of $0.66F_y$ is valid.

13.4.2 Plastic Analysis

For structures analyzed plastically, high shears and moments may occur simultaneously at a hinge location. Yang and Beedle (1951) have shown that, when the maximum shear stress is limited to the von Mises value, the flexural resistance can be maintained at M_p. Taking the effective section depth as 95% of the nominal depth, this Clause gives an approximate shear resistance limited to the von Mises stress. (See Commentary to Clause 8.3.2(d)).

13.4.3 Webs of Flexural Members Not Having Two Flanges

When cross-sections do not have two flanges, the shear stress distribution can no longer be assumed to be uniform. For W-shapes with one flange coped, the elastic shear stress distribution may be determined from $\tau = VQ/It$. Limiting the maximum value to $0.66F_y$ is conservative as it does not allow for any plastification as shear yielding spreads from the most heavily stressed region. For W-shapes with two flanges coped, a parabolic shear stress distribution results from this procedure with a maximum shear stress equal to 1.5 times the average. The maximum shear stress can be based on strain-hardening provided shear buckling does not occur.

13.4.4 Pins

Additional information for pins in combined shear and moment is given in the Canadian Highway Bridge Design Code, CSA S6-14.

13.5 Bending – Laterally Supported Members

The factored moment resistances are consistent with the classification of cross-sections given in Clause 11, as illustrated by moment–rotation curves given in Figure 2-15.

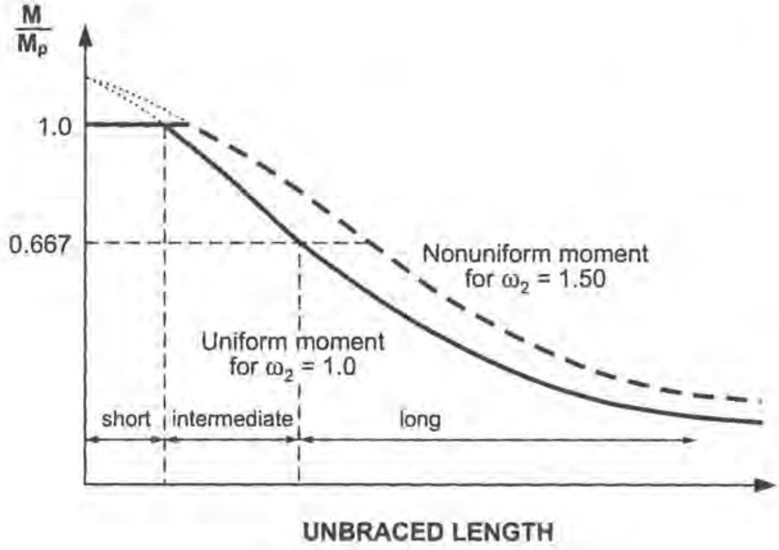


Figure 2-16
Variation of Uniform and Nonuniform Moment Resistances

The fully plastic moment, M_p, attained by Class 1 and 2 sections, implies that all fibres of the section are completely yielded. Any additional resistance that develops due to strain-hardening has been accounted for in the test/predicted ratio statistics used in developing resistance factors (Kennedy and Gad Aly 1980).

The stress distribution for Class 3 sections at the ultimate moment is assumed linear, with a maximum stress equal to the yield stress.

Class 4 sections reach their maximum moment resistance when a flange or web plate element buckles locally. Class 4 sections are divided into three categories.

The first consists of those sections with Class 4 flanges and webs. This type of section is designed to the requirements of CSA Standard S136 using the material properties appropriate to the structural steel specified.

The second category consists of those sections with Class 3 flanges and Class 4 webs. Clause 13.5(c)(ii) logically requires that these sections be designed in accordance with Clause 14.

For the third category with Class 4 flanges and Class 3 webs, a reduced, effective section modulus (Kalyanaraman *et al.* 1977) is used to compute the moment resistance. Alternatively, an effective yield stress established from Class 3 limits may be used to calculate the moment resistance.

13.6 Bending – Laterally Unsupported Members

Laterally unsupported beams may fail by lateral–torsional buckling at applied moments significantly less than the full cross-sectional strength (M_p or M_y). Even when the top flange is laterally supported, under some circumstances – for example, a roof beam subject to uplift – the laterally unsupported bottom flange may be in compression. General information on lateral–torsional buckling is summarized in Chen and Lui (1987).

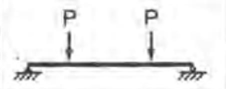
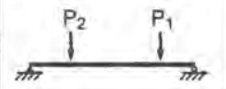
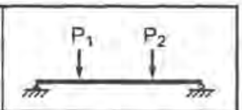
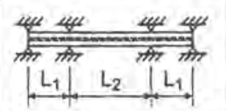
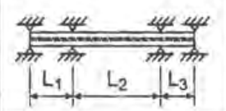
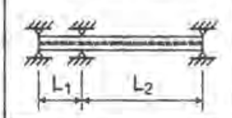
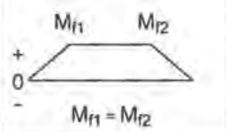
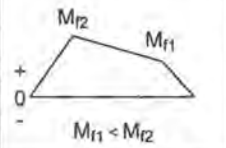
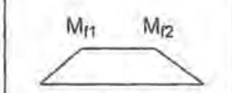
Loading			
Lateral Restraints (Plan view)			
Moment Diagram			
ω_2	1.75 for L_1 1.0 for L_2	1.75 for L_1 $\kappa = \dfrac{-M_{f1}}{M_{f2}}$ for L_2 1.75 for L_3	1.75 for L_1 1.0 for L_2

Figure 2-17
Various Cases of ω_2 for Linear Moment Gradients

Besides cross-sectional properties and aspects related to the loading itself, the lateral–torsional moment resistance depends on the unsupported (unbraced) length. Beams may be considered to be short, intermediate, or long depending on whether the moment resistance developed is the full cross-sectional strength, the inelastic lateral–torsional buckling strength, or the elastic lateral-torsional buckling strength, respectively, as shown in Figure 2-16 for Class 1 and 2 shapes capable of attaining M_p on the cross-section. The curve for Class 3 sections is similar, except that the maximum moment resistance is M_y, while for Class 4 sections, the maximum resistance is limited by local buckling.

The length, L, is generally taken as the distance between lateral supports. When beams are continuous through a series of lateral supports, interaction buckling (Trahair, 1968) occurs, and the segment that tends to buckle laterally first is restrained by the adjoining segments. Nethercot and Trahair (1976a, 1976b), Kirby and Nethercot (1978) and Schmitke and Kennedy (1985) give methods of computing effective lengths under these circumstances. Points of contraflexure for bending about the major axis are not related to lateral-torsional buckling and therefore cannot be considered as points of lateral support (Schmitke and Kennedy 1985).

Without the equivalent moment factor, ω_2, the expression given for M_u is that for a doubly-symmetric beam subject to uniform moment. The factor ω_2 ranges from 1.0 to 2.5 and takes into account the fact that for lateral–torsional buckling a varying moment is less severe than a uniform moment. Also plotted in Figure 2-16 is the moment resistance for a beam for which $\omega_2 = 1.5$. It is seen that in the elastic region ($M_r \leq ⅔\, M_p$) the full value of ω_2 is realized. In the inelastic region, however, the increase in M_r due to non-uniform moments gradually decreases to zero as the moment approaches M_p.

Wong and Driver (2010) and Driver and Wong (2007) demonstrate that the method for calculating ω_2 specified in 2001 and earlier editions of the Standard produces highly erroneous results in some common situations. To address this shortcoming, their general equation for determining ω_2 based on the moments at the quarter-points of the unbraced segment has been introduced into the Standard. This equation uses a similar method to that specified in the AISC Specification (AISC 2010b), except that it employs a square-root format that eliminates the non-conservative results that would otherwise arise in cases where the ends of the unbraced segment are close to rotationally fixed about the major axis. While the upper limit on the value of ω_2 of 2.5 that was associated with the previous method is theoretically no longer required, it was retained to acknowledge the fact that very high lateral-torsional buckling capacities attributable largely to the moment distribution can be highly sensitive to the assumptions about loading and end restraint, and they may not be achievable in practice. Wong and Driver (2010) provide a detailed discussion of aspects that affect the accuracy of equivalent moment factors and they compare numerous methods of determining this factor that have been proposed in the literature and that are being used in design standards around the world.

Due to its simplicity and familiarity to Canadian designers, the method from the 2001 edition of the Standard for determining ω_2 has been retained as an alternative approach for application only to cases where the moment gradient is linear between lateral supports, which is the scenario for which it was derived and therefore produces good results. Figure 2-17 illustrates several cases where this method may still be used. The quarter-point moment method in the Standard also gives excellent results for linear moment gradients.

The expression for M_u assumes that the beam is loaded at the elevation of the shear centre. A downward-acting load that is applied below the shear centre stabilizes the beam, whereas such a load applied above the shear centre destabilizes it. The Standard is now explicit that for the latter case, when the beam is laterally unbraced at the load point and the means of applying the load itself provides neither lateral nor rotational restraint, the reduction in moment capacity must be taken into account. For top-flange loading, a simple and conservative effective length approach (Wong *et al.* 2014) is provided as an alternative to more accurate methods. Since the effective length factor accounts for both the load height and moment distribution effects, ω_2 is set equal to unity. The two effective length factors specified in the Standard are distinguished by the in-plane rotational restraint at the ends of the unbraced beam segment: either simple or restrained. The method does not apply to cantilevers. Detailed discussions on this approach and a graphical method that gives more accurate effective length factors are presented by Wong *et al.* (2014). For other positions of the load, unusual loading cases and other support conditions, Ziemian (2010) may be consulted.

Because laterally unsupported, closed, square and circular sections with $I_x = I_y$ show no tendency to buckle laterally, their moment resistance is established using Clause 13.5 as emphasized in Clause 13.6(c).

For structural systems utilizing cantilever suspended-span construction (Gerber girders), see Albert *et al.* (1992), Essa and Kennedy (1994(a), 1994(b), 1995), and Ziemian (2010) for a rational method of determining the strength of cantilevered beams.

For members bent about both principal axes, it should be remembered that M_{ry} is either M_{yy} or M_{yp} as a function of the class of the section, because there is no reduction for lateral–torsional buckling for weak axis buckling.

The provisions in Clause 13.6(e) were introduced in the 2009 edition of the Standard and address beams that are generally I-shaped and are symmetric about the web's centreline, but which have flanges of unequal sizes, or only one flange (tee sections). In these sections, the shear centre is not coincident with the centroid of the section, and the smaller flange has higher stresses than the larger flange. The effects of these asymmetries are accounted for in the β_x term. The expression provided for β_x is an approximation of the complete expression,

$$\beta_x = \frac{1}{I_x}\int_A y(x^2 + y^2)dA - 2y_o$$

where x and y are coordinates on the cross-section based on an origin located at the geometric centroid, and y_o is the distance in the y-direction from the centroid to the shear centre. An approximate value for the warping torsional constant, C_w, is also provided in this clause. A more thorough treatment of monosymmetric beams can be found in Ziemian (2010).

Sections that have large differences in flange size may experience yielding of the smaller flange under service loads, if designed as Class 1 or Class 2 beams and the factored moment resistance is near ϕM_p. The maximum moment caused by the applied service loads must be less than the smaller M_y value to prevent permanent deformations from occurring during service conditions.

The method of strength determination generally follows the AISC (2010b) methodology, with the exception that the elastic buckling capacity is determined considering the distribution of moments. This approach requires finding two lengths, L_{yr} and L_u, for beams that are in the inelastic buckling regime. L_{yr} is the length at which the elastic buckling moment, M_u, reaches M_{yr} and causes the initiation of yielding; i.e., the extreme fibre reaches 0.70 F_y, and yielding would occur in regions where residual stresses reach 30% of yield. The smaller value of S_x is used to determine M_{yr} (corresponding to yielding of the smaller flange). L_{yr} can be determined by any method, such as iterative approximations, but can be found via a direct solution with the following equation:

$$L_{yr} = \sqrt{\frac{(2P\beta_x + Q) + \sqrt{(2P\beta_x + Q)^2 + 4RP^2}}{2P^2}}$$

where $P = \dfrac{1.4F_y\, S_{x,min}}{\omega_3 \pi^2 E I_y}$, $Q = \dfrac{4GJ}{\pi^2 E I_y}$, and $R = \dfrac{4C_w}{I_y}$.

The other length, L_u, is the length at which the beam can carry its fully braced capacity, either M_p or M_y, depending on its local buckling classification. The value of:

$$1.1 r_t \sqrt{E/F_y}$$

is based on work by White and Jung (2004). The term r_t is the radius of gyration of the tee-shaped area formed by the compression flange, and one-third of the portion of the web in compression as defined by the elastic neutral axis. Inelastic buckling capacity is determined by linear interpolation between M_{yr} and M_p (or M_y), based on the unbraced length of beam, L.

The value of ω_3 for tee sections must be less than or equal to 1.0. This is because reverse curvature in these beams is a worse condition than a uniform moment (Attard and Lawther, 1989), which is different from the case for doubly-symmetric sections. The warping torsional constant for tee sections should be taken as zero.

For monosymmetric sections other than those described above, a rational method must be used.

13.7 Lateral Bracing for Members in Structures Analyzed Plastically

See the Commentary on Clause 8.3.2(c). This clause is consistent with seismic requirements.

13.8 Axial Compression and Bending

This Clause remains unchanged in the 2014 edition. The design for strength and stability of steel frames and beam-columns is based on "second-order analysis" and "notional lateral loads" (Clause 8.4), and "sway stiffness" (Clause 13.8).

(a) A distinction is made between braced and unbraced frames in that the design requirements for beam-columns are different for the two types of frames. The 5/1 stiffness ratio in Clause 13.8 originates from Eurocode 3 where it was stated that if bracing were added to a frame and it reduced the lateral sway deflection by 80% or more, then the bracing was sufficiently effective to consider the frame as being "braced". In the analysis, members such as columns with nominally pinned connections, which do not contribute to the lateral strength and stability of the frame/structure, may be considered to be braced by the frame. The notional loads attributed to such non-contributing columns must be included in the sway analysis of the frame

(b) Cross-sectional strength never governs for prismatic beam-columns in unbraced frames and need not be checked because it will never be smaller than the in-plane strength or the lateral-torsional buckling strength. Parenthetic statements in Clauses 13.8.2(a) and 13.8.3(a) waive this check.

(c) P-δ effects, related to the member deformation between the ends, have been found to be negligible for beam-columns in unbraced frames. This is because the maximum second-order elastic moment, including P-Δ (sway) effects, occurs at the ends of the beam-column. Therefore the factor U_1 is taken as 1.0 in the interaction equation for overall member strength of sway (unbraced) beam-columns in Clauses 13.8.2(b) and (c), and 13.8.3(b) and (c). (The P-δ effects continue to be considered for non-sway beam-columns.)

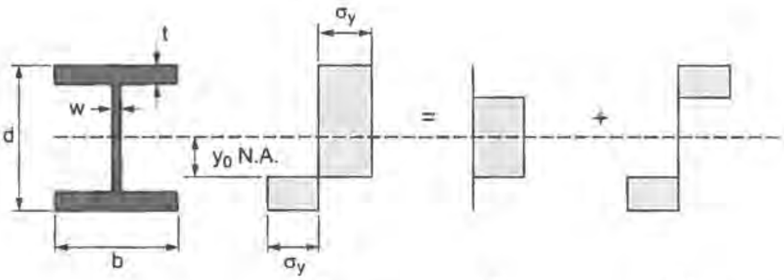
(d) For weak-axis bending, the in-plane strength interaction equation introduced in Clause 13.8.2 with a factor β accounts more accurately for the effect of distributed plasticity on stability, by fitting the plastic-zone strength curves for different values of λ_y more closely. β increases from 0.6 when $\lambda_y = 0$ to 0.85 for values of λ_y greater than 0.625 where the distributed plasticity has a greater effect on the overall weak-axis stiffness.

For a general discussion of all aspects of Clause 13.8 and worked examples, see Essa and Kennedy (2000).

The value each term in the interaction equation takes is prescribed in the three sub-clauses (a), (b), and (c) depending on the particular mode of failure: cross-sectional strength, overall member strength, and lateral-torsional buckling strength, respectively. Clause 13.8.2 is applicable to Class 1 and Class 2 sections of I-shaped members, while Clause 13.8.3 is applicable to all other classes of sections.

The interaction expressions account for the following:

- A laterally supported member fails when it reaches its in-plane moment capacity, reduced for the presence of axial load;
- A laterally unsupported member may fail by lateral-torsional buckling or a combination of weak-axis buckling and lateral buckling;
- A relatively short member can reach its full cross-sectional strength whether it is laterally supported or not;
- When subjected to axial load only, the axial compressive resistance, C_r, depends on the maximum slenderness ratio – below the yield load, the column fails by buckling.



(a) Total stress distribution (b) Stress due to C_f (c) Stress due to M_f

Figure 2-18
Idealized Stress Distribution in Plastified Section of Beam-Column

Column buckling is a bifurcation problem, not a bending strength problem;

- Members bent about the weak axis, or with the same strength about both axes, do not exhibit out-of-plane behaviour,
- A constant moment has the most severe effect on in-plane behaviour. Other moment diagrams can be replaced by equivalent moment diagrams of reduced but uniform intensity;
- A constant moment has the most severe effect on the lateral-torsional buckling behaviour. (See commentary on Clause 13.6). This effect disappears if the member is short enough, in which case, cross-sectional strength controls; and
- Moments may be amplified by axial loads increasing the deflections, the P-δ effect.

Four modes of failure, including local buckling of plate elements, are to be checked in design, as appropriate. They are addressed as follows:

1) Local buckling of an element

Before assessing the member failure modes, the element b/t ratios are checked to confirm the class of the section, the appropriate cross-sectional moment and axial compressive resistances, and to ensure local buckling does not occur prematurely.

2) Strength of the cross-section

The cross-sectional strength of a shape used as a beam-column is not to be exceeded. Clause 13.8.2(a) gives the cross-sectional strength requirements for Class 1 and Class 2 sections of I-shaped members and Clause 13.8.3(a) for all other classes of sections. The cross-sectional strength is also the limiting strength of short members. For prismatic beam-columns in unbraced frames, the cross-sectional strength never governs the design and need not be checked.

The cross-sectional strength of a Class 1 and Class 2 I-shaped section comprising relatively stocky plate elements is derived from the fully plastic stress distribution of the cross-section as shown in Figure 2-18. For uniaxial bending about the x-x axis and the y-y axis, expressions are respectively, using the limit states notation of this Standard:

$$M_{fx} = 1.18\,\phi\,M_{px}\left(1 - \frac{C_f}{\phi A F_y}\right) \leq \phi\,M_{px}$$

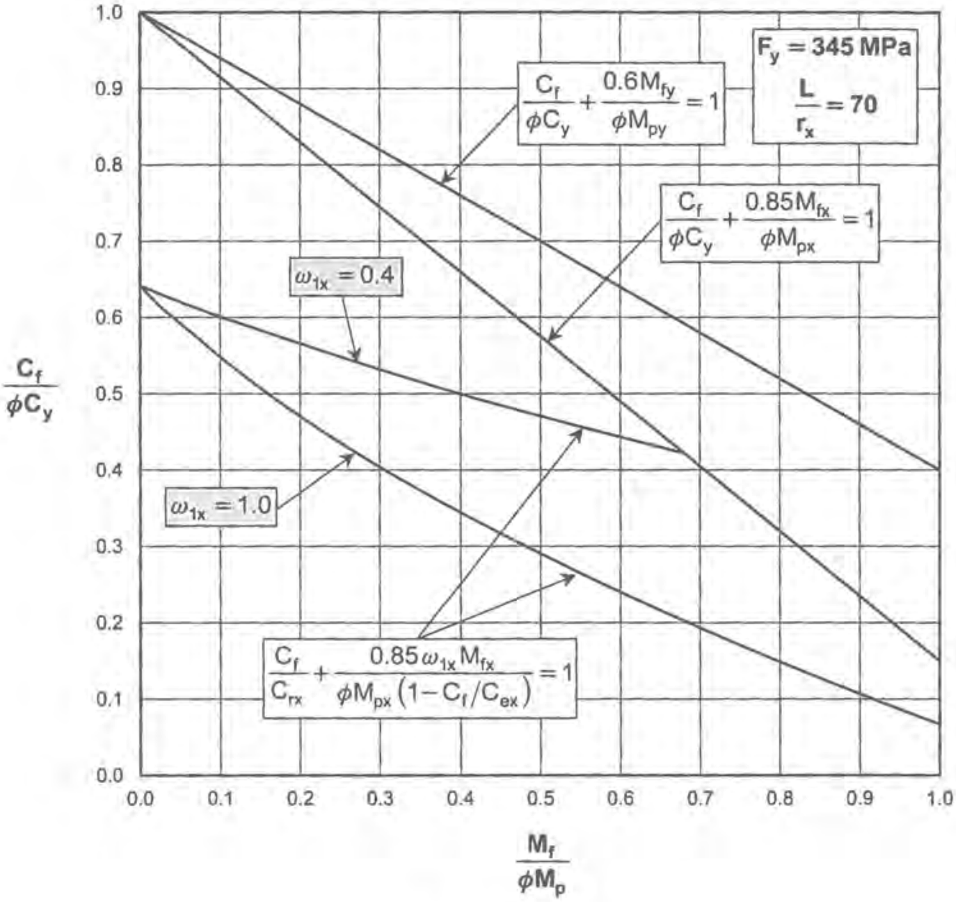


Figure 2-19
Interaction Expressions for Class 1 and Class 2 W-Shapes

$$M_{fy} = 1.67\phi\, M_{py}\left(1 - \frac{C_f}{\phi A F_y}\right) \le \phi\, M_{py}$$

Transposing the terms in the above expressions gives:

$$\frac{C_f}{\phi\, C_y} + 0.85\,\frac{M_{fx}}{\phi\, M_{px}} \le 1.0; \quad \frac{M_{fx}}{\phi\, M_{px}} \le 1.0$$

$$\frac{C_f}{\phi\, C_y} + 0.6\,\frac{M_{fy}}{\phi\, M_{py}} \le 1.0; \quad \frac{M_{fy}}{\phi\, M_{py}} \le 1.0$$

as shown in Figure 2-19. For biaxial bending it is conservative to combine these expressions linearly to give, using the limit states notation of this Standard:

$$\frac{C_f}{\phi\, C_y} + 0.85\,\frac{M_{fx}}{\phi\, M_{px}} + 0.6\,\frac{M_{fy}}{\phi\, M_{py}} \le 1.0; \quad \frac{M_{fx}}{\phi\, M_{px}} + \frac{M_{fy}}{\phi\, M_{py}} \le 1.0$$

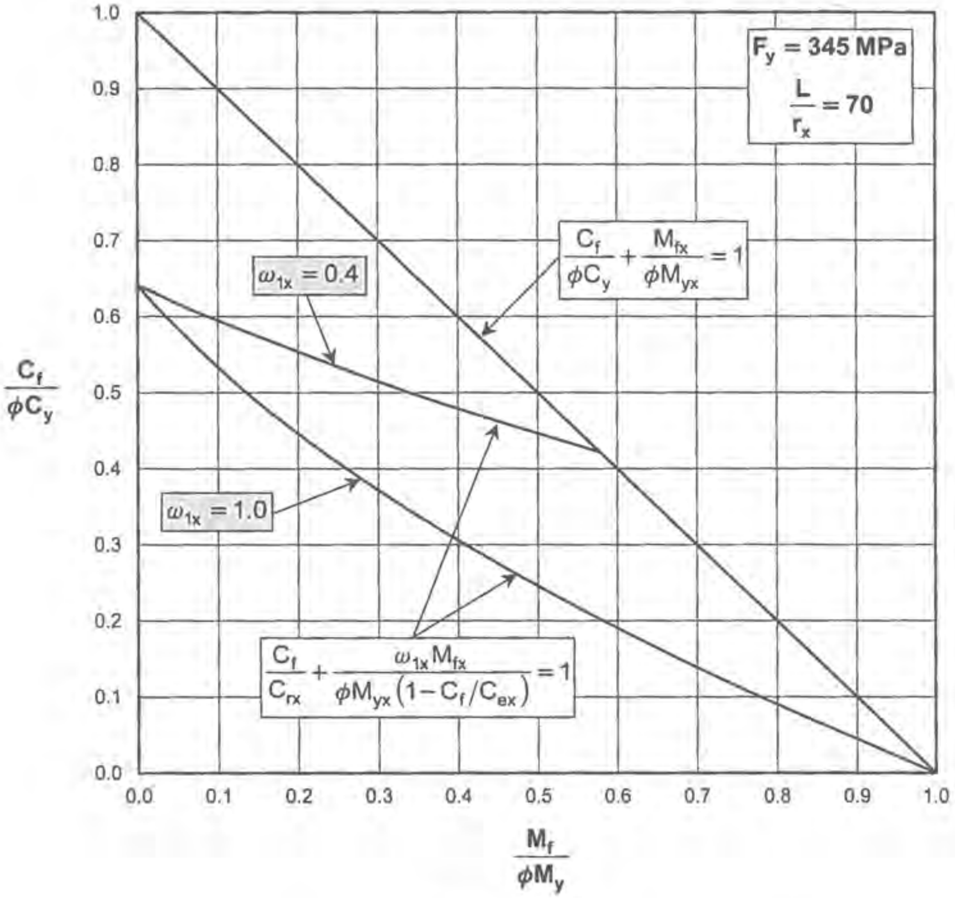


Figure 2-20
Interaction Expressions for Class 3 W-Shapes

This is identical to the two expressions in Clause 13.8.2 when, in the latter in accordance with Clause 13.8.2(a) for cross-sectional strength, U_{1x} and U_{1y} are set equal to 1.0, $C_r = \phi A F_y$ when $\lambda = 0$, $\beta = 0.6$ when $\lambda = 0$, and M_{rx} and M_{ry} are equal to ϕM_{px} and ϕM_{py}, respectively.

For uniaxial bending of sections other than Class 1 and Class 2 I-sections, the appropriate interaction expression is:

$$\frac{C_f}{\phi\, C_y} + \frac{M_{fx}}{M_{rx}} \leq 1.0$$

Extending this linear expression to biaxial bending gives:

$$\frac{C_f}{\phi\, C_y} + \frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0$$

This agrees with Clause 13.8.3(a) when the appropriate values of the factored cross-sectional resistance quantities are used. Thus, for Class 3 sections the factored moment resistances are limited to ϕM_y and for Class 4 sections the resistances, C_r, M_{rx}, and M_{ry}, are based on local buckling.

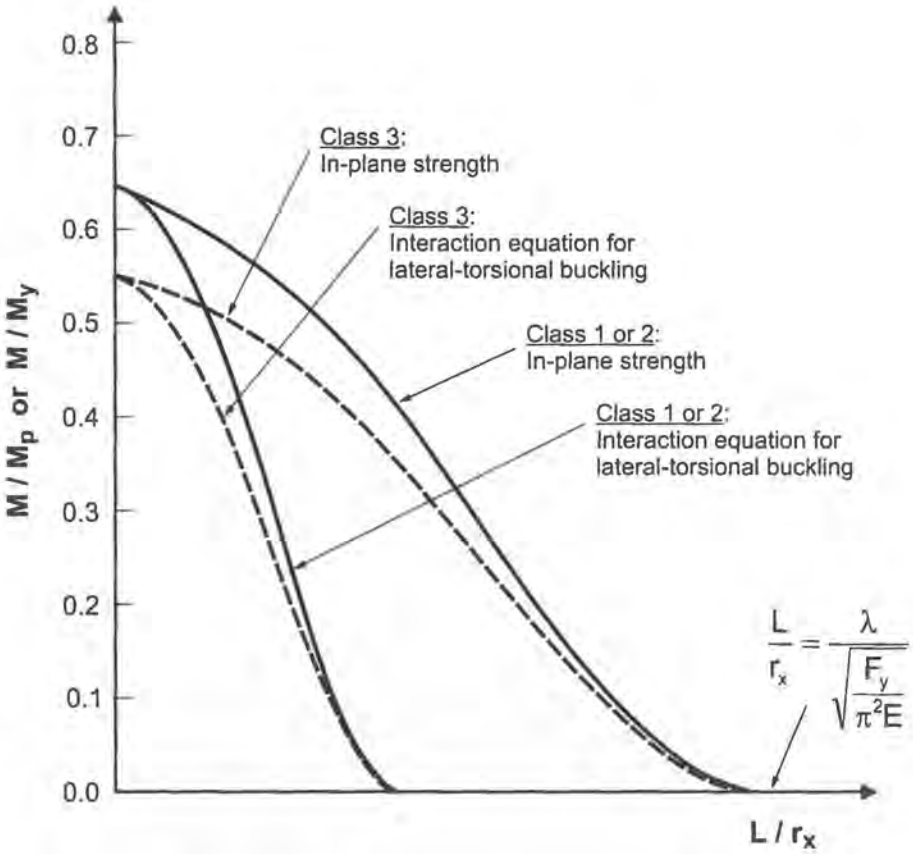


Figure 2-21
Variations of Moment Resistance with Slenderness Ratio

3) Overall member strength

The overall strength (in-plane bending strength) of a member depends on its slenderness. As an actual beam-column has length, the axial compressive resistance, C_r, depends on its slenderness ratio and will be less than or equal to the yield load. For any particular beam-column, this fraction of the yield load can be established and is illustrated in Figure 2-19 for Class 1 or 2 sections, and in Figure 2-20 for Class 3 sections.

In Figure 2-21 the variation in moment resistance in terms of M/M_p as a function of the slenderness L/r_x is plotted schematically as a solid line for a particular laterally supported Class 1 (or Class 2) section subject to a uniform moment about the x-axis and carrying an axial load of $0.35C_y$. An appropriate interaction expression for the in-plane strength of such a Class 1 (or Class 2) I-section is

$$\frac{C_f}{C_{rx}} + 0.85\frac{\omega_1 M_f}{\phi M_p\left(1 - C_f / C_e\right)} \le 1.0$$

which can be deduced from Clause 13.8.2(b) when the terms in that expression are appropriately defined. Note that if the member is short, the expression reduces to that for the cross-sectional strength. The compressive resistance, C_{rx}, is a function of the slenderness ratio L/r_x.

The term:

$$\omega_1 = 0.6 - 0.4\,\kappa \geq 0.4$$

multiplied by the maximum non-uniform moment, M_f, gives an equivalent uniform moment, $\omega_1 M_f$, having the same effect on the in-plane member strength as the non-uniform moment (Ketter 1961).

In order to account for the P-δ effects (the amplification of the moments caused by the axial loads acting on the deformed shape), the equivalent uniform moment, $\omega_1 M_f$, is amplified by the factor:

$$\frac{1}{1-\dfrac{C_f}{C_e}} \quad \text{where: } C_e = \frac{\pi^2 EI}{L^2}$$

The in-plane strength of Class 1 or 2 sections is shown in Figure 2-19 for $F_y = 345$ MPa and $L/r_x = 70$. When $L/r_x = 0$ and $\omega_1 = 1$, the in-plane strength expressions 13.8.2(b) and 13.8.3(b) become the cross-sectional strength expressions 13.8.2(a) and 13.8.3(a), respectively. The curve for Class 3 sections is given in Figure 2-20.

In Figure 2-21, the curve of moment resistance versus slenderness ratio for the in-plane strength of a Class 3 section of equivalent cross-sectional strength to the Class 1 or 2 section is also given. It is similar to that for a Class 1 or 2 section except that, because the cross-sectional strength expression for Class 3 sections does not have the 0.85 factor that is appropriate for Class 1 or 2 and because the Class 3 section can only attain M_y, the curve for Class 3 for zero slenderness ratio reaches only about $0.55\,M_p$ and not $0.65\,M_p$ as for the Class 1 or 2 sections.

For biaxial bending, C_r is based conservatively on the maximum slenderness ratio. It could be argued that for biaxial bending the value used for C_r be interpolated between C_{rx} and C_{ry} on the basis of the proportion of the interaction fractions for bending about two axes. In other words, if a beam-column carries only a small portion of bending about the y-axis, the decrease in C_r from C_{rx} toward C_{ry} should likewise be small.

In Figures 2-19 and 2-20, the in-plane strength interaction expressions are shown for $\omega_1 = 1$. When $\omega_1 < 1$, the limiting strength for low ratios of axial load is the cross-sectional strength expression.

4) Lateral-torsional buckling strength

Building beam-columns are usually laterally unsupported for their full length and, even though they are subject to strong-axis bending moments, failure may occur when the column, after bending about the strong axis, buckles about the weak axis and twists simultaneously. Again this is a buckling or bifurcation problem. For such columns, the lateral-torsional buckling strength is likely to be less than both the cross-sectional strength and the overall member strength.

The curves in Figure 2-21 for a beam-column subject to uniform moment for Class 1 and 3 sections marked "*interaction equation for lateral-torsional buckling*", demonstrate this effect. They are much below those for in-plane strength and would only reach the full cross-sectional strength when the slenderness ratio is zero. The moment resistance is zero for laterally unsupported beam-columns when weak-axis buckling occurs. Thus, for these members the axial compressive resistance is based on L/r_y, and M_{rx} is based on the resistance of a laterally unsupported beam. When subjected to weak-axis bending, members do not exhibit out-of-plane buckling behaviour, and therefore the weak-axis moment resistance is based on the full cross-sectional strength, the plastic moment or yield moment capacity about the weak axis as appropriate for the Class of the section.

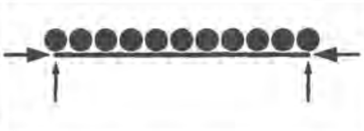
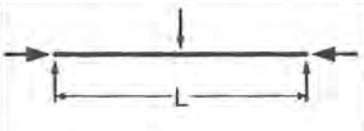
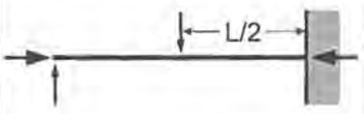
Case	ω_1	Case	ω_1
	1.0		$1-0.2\frac{C_f}{C_e}$
	$1-0.4\frac{C_f}{C_e}$		$1-0.3\frac{C_f}{C_e}$
	$1-0.4\frac{C_f}{C_e}$		$1-0.2\frac{C_f}{C_e}$

Figure 2-22
Values of ω_1 for Special Cases of Laterally Loaded Beam-Columns

In computing M_{rx} from Clause 13.6, the effect of non-uniform moments is included. Therefore, in the interaction expressions when lateral-torsional buckling is being investigated, the factored moment, M_{fx}, must also be a non-uniform moment, and not be replaced by an equivalent lesser moment. It is for this reason that the value of U_{1x} cannot be less than 1.0.

13.8.5 This clause gives generally conservative values of ω_1, the factor by which the maximum value of the non-uniform moment is multiplied to give an equivalent uniform moment having the same effect as the applied non-uniform moment on the overall strength of the member. For further discussion on ω_1, see Ziemian (2010) where it is called C_m. Figure 2-22 gives values of ω_1 for some special cases of transverse bending.

Figures 2-23 and 2-24 give additional guidance for the design of beam-columns subjected to various bending moment effects.

13.9 Axial Tension and Bending

The linear interaction expression of Clause 13.9.1 is a cross-sectional strength check. Conservatively, it does not take into account the fact that the bending resistance for Class 1 and 2 sections does not vary linearly with axial force, for which case a factor of 0.85 multiplying the moment term would appear to be appropriate (see Clause 13.8.2).

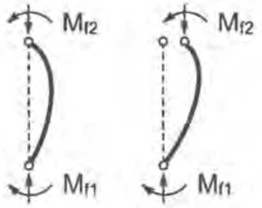
For members subjected predominantly to bending, i.e. when the tensile force is relatively small, failure may still occur by lateral-torsional buckling. The expressions of Clause 13.9.2 result from that of Clause 13.9.1 when a negative sign is assigned to the tension interaction component and when M_r is based on the overall member behaviour taking lateral-torsional buckling into account.

13.10 Load Bearing

The bearing resistance given for accurately cut or fitted parts in contact, Clause 13.10(a), reflects the fact that a triaxial compressive stress state, restricting yielding of the parts in contact,

Conditions **	Design Criteria

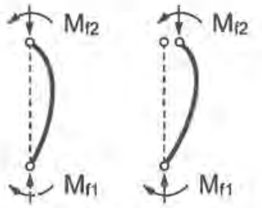
Conditions **



Single curvature bending

$M_{f2} \geq M_{f1}$

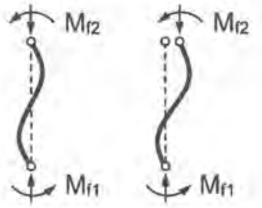
$$\omega_1 = 0.6 + 0.4\frac{M_{f1}}{M_{f2}}$$



Single curvature bending

$M_{f1} = 0$

$\omega_1 = 0.6$



Double curvature bending

$M_{f2} \geq M_{f1}$

$$\omega_1 = 0.6 - 0.4\frac{M_{f1}}{M_{f2}} \geq 0.4$$

Design Criteria

Beam-Colum Design Expressions

$P\Delta$ (frame sway effects), if any, are included in the analysis.

(1) Class 1 and 2 Sections of I-Shapes

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

$$\beta = 0.6 + 0.4\lambda_y \leq 0.85$$

$$\frac{M_{fx}}{M_{rx}^{*}} + \frac{M_{fy}}{M_{ry}^{*}} \leq 1.0$$

(2) All Classes Except Class 1 and 2 Sections of I-Shapes

$$\frac{C_f}{C_r} + \frac{U_{1x}M_{fx}}{M_{rx}} + \frac{U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

Member Strength Checks

(a) Cross-sectional strength (use actual M_f at each location)

$C_r = \phi A F_y$

$M_r = \phi Z F_y$ = (for Class 1 and 2 sections)

$= \phi S F_y$ = (for Class 3 sections)

= See S16-14 Clause 13.5(c) for Class 4 sections

M_{rx}^{*} and M_{ry}^{*} calculated according to Cl. 13.5 or 13.6 as appropriate

$U_1 = \omega_1/(1 - C_f/C_e) \geq 1.0$

(b) Overall member strength (use M_{f2} for M_f)

C_r = Factored compressive resistance (max. slenderness, K = 1), Cl. 13.3, except C_r based on axis of bending for uniaxial bending

M_r = as given for M_r in (a) above

$U_1 = \omega_1/(1 - C_f/C_e)$, except for unbraced frames $U_1 = 1.0$

(c) Lateral-torsional buckling strength (use M_{f2} for M_f)

C_r = Factored compressive resistance (max. slenderness), Cl. 13.3

M_{rx} = value given by Clause 13.6

M_{ry} = as given for M_r in (a) above

For braced frames:

$U_{1x} = \omega_{1x}/(1 - C_f/C_{ex}) \geq 1.0$

$U_{1y} = \omega_{1y}/(1 - C_f/C_{ey})$

For unbraced frames:

$U_{1x} = U_{1y} = 1.0$

C_f = Factored compressive load

C_r = Factored compressive resistance

M_f = Factored bending moment (x-x or y-y axis)

M_r or M_r^{*} = Fact. moment resistance (x-x or y-y axis)

ω_1 = Coefficient used to determine equivalent uniform column bending effect (x-x or -y-y)

U_1 = Factor to account for moment gradient and member curvature second-order effects

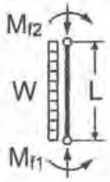
** Moments M_{f1} and M_{f2} may be applied about one or both axes.

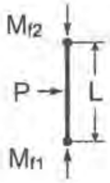
Figure 2-23
Prismatic Beam-Columns – Moments at Ends – No Transverse Loads

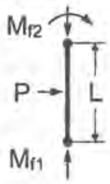
<table>
<tr><th>Conditions **</th><th>Design Criteria</th></tr>
<tr><td>

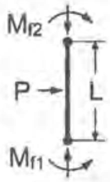

Loaded with UDL
$M_{f1} = M_{f2} = 0$
$M_{f3} = \frac{WL}{8}$ (max.)
</td><td rowspan="6">

Beam-Colum Design Expressions
$P\Delta$ (frame sway effects), if any, are included in the analysis.
(1) Class 1 and 2 Sections of I-Shapes
$$\frac{C_f}{C_r} + \frac{0.85 U_{1x} M_{fx}}{M_{rx}} + \frac{\beta U_{1y} M_{fy}}{M_{ry}} \le 1.0$$
$$\beta = 0.6 + 0.4\lambda_y \le 0.85$$
$$\frac{M_{fx}}{M_{rx}{}^*} + \frac{M_{fy}}{M_{ry}{}^*} \le 1.0$$
(2) All Classes Except Class 1 and 2 Sections of I-Shapes
$$\frac{C_f}{C_r} + \frac{U_{1x} M_{fx}}{M_{rx}} + \frac{U_{1y} M_{fy}}{M_{ry}} \le 1.0$$
Member Strength Checks
(a) Cross-sectional strength (use actual M_f at each location)
$C_r = \phi A F_y$
$M_r = \phi Z F_y$ = (for Class 1 and 2 sections)
$= \phi S F_y$ = (for Class 3 sections)
= See S16-14 Clause 13.5(c) for Class 4 sections
$M_{rx}{}^*$ and $M_{ry}{}^*$ calculated according to Cl. 13.5 or 13.6 as appropriate
$U_1 = \omega_1/(1 - C_f/C_e) \ge 1.0$
(b) Overall member strength (use M_{max} for M_f)
C_r = Factored compressive resistance (max. slenderness, K = 1)
Cl. 13.3, except C_r based on axis of bending for uniaxial bending
M_r = as given for M_r in (a) above
$U_1 = \omega_1/(1 - C_f/C_e)$, except for unbraced frames $U_1 = 1.0$
(c) Lateral-torsional buckling strength (use M_{max} for M_f)
C_r = Factored compressive resistance (max. slenderness), Cl. 13.3
M_{rx} = value given by Clause 13.6
M_{ry} = as given for M_r in (a) above
For braced frames:
$U_{1x} = \omega_{1x}/(1 - C_f/C_{ex}) \ge 1.0$
$U_{1y} = \omega_{1y}/(1 - C_f/C_{ey})$
For unbraced frames:
$U_{1x} = U_{1y} = 1.0$
</td></tr>
<tr><td>


Loaded with UDL
$M_{f1} = 0$
$M_{f2} = \frac{WL}{8}$ (max.)
$M_{f3} = \frac{9WL}{128}$
</td></tr>
<tr><td>


Loaded with UDL
$M_{f1} = M_{f2} = \frac{WL}{12}$ (max.)
$M_{f3} = \frac{WL}{24}$
</td></tr>
<tr><td>


Loaded with PL
$M_{f1} = M_{f2} = 0$
$M_{f3} = \frac{PL}{4}$ (max.)
</td></tr>
<tr><td>


Loaded with PL
$M_{f1} = 0$
$M_{f2} = \frac{3PL}{16}$ (max.)
$M_{f3} = \frac{5PL}{32}$
</td></tr>
<tr><td>


Loaded with PL
$M_{f1} = M_{f2} = \frac{PL}{8}$
$M_{f3} = \frac{PL}{8}$ (max.)
</td></tr>
<tr><td colspan="2">
C_f = Factored compressive load
C_r = Factored compressive resistance
M_f = Factored bending moment (x-x or y-y axis)
M_r or M_r^* = Fact. moment resistance (x-x or y-y axis)
ω_1 = Coefficient used to determine equivalent uniform column bending effect (x-x or -y-y)
U_1 = Factor to account for moment gradient and member curvature second-order effects
</td></tr>
</table>

** Moments M_{f1} and M_{f2} may be applied about one or both axes.

Figure 2-24
Prismatic Beam-Columns with Transverse Loads

generally exists. The value given is based on earlier working stress design standards, which have given satisfactory results.

For a cylindrical roller or rocker, Clause 13.10(b) recognizes that the roller or rocker may rest in a cylindrical groove in the supporting plate. This results in a supporting or contact area larger than that for the case of a flat supporting plate.

In the case of a cylindrical groove in the supporting plate, the maximum shearing stress developed due to a line load of q N/mm, (Seeley and Smith, 1957) is,

$$\tau_{\max} = 0.27\sqrt{\frac{qE}{2\pi\left(1-\nu^2\right)}\left(\frac{R_2-R_1}{R_2R_1}\right)}$$

where ν is Poisson's ratio. From this, the unfactored bearing resistance, qL, is then

$$\frac{B_r}{\phi} = qL = \frac{2\pi L\left(1-\nu^2\right)\tau_{\max}^2}{0.27^2 E}\left(\frac{R_2R_1}{R_2-R_1}\right)$$

Calibrating this resistance to that given in S16-1969 at $F_y = 300$ MPa gives $\tau_{max} = 0.77F_y$, and

$$\frac{B_r}{\phi} = 0.000\,26\left(\frac{R_1}{1-R_1/R_2}\right)LF_y^2$$

For a roller of radius R_1 on a flat plate with $R_2 = \infty$, the "Hertz" solution, as reported by Manniche and Ward-Hall (1975), gives the *allowable* load as

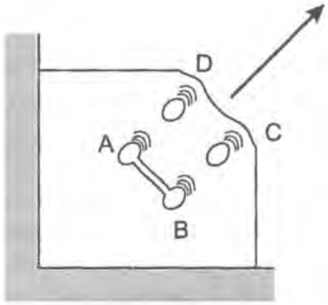
$$2.86DL\frac{\left(2.7F_y\right)^2}{E} = 0.000\,20\,R_1LF_y^2$$

where D is the roller diameter. The above expression indicates that the value of 0.000 26 R_1 obtained by calibration with the existing standard for a yield stress of about 300 MPa is somewhat non-conservative compared to the value of 0.000 21 R_1 proposed by Manniche and Ward-Hall (1975).

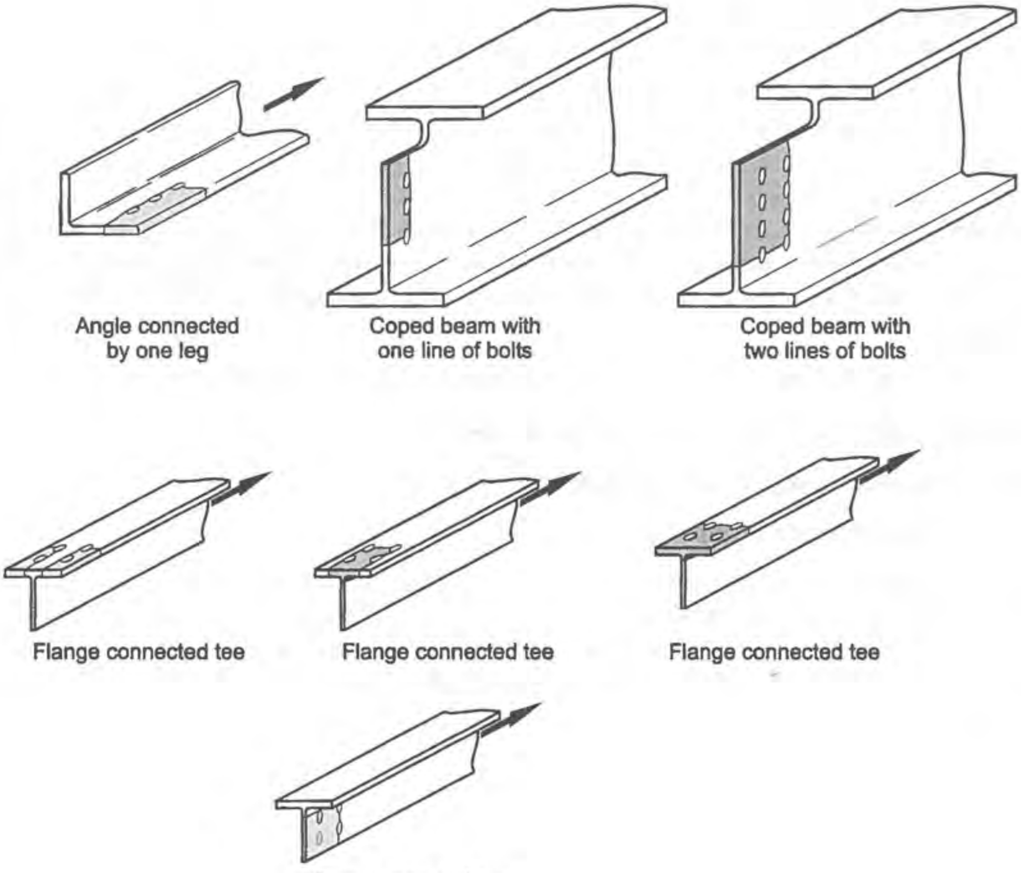
This is confirmed by Kennedy and Kennedy (1987) who reported that at this load no permanent deformation resulted and recommended that this value be used as a serviceability limit. They also reported that the rolling resistance of rollers varied as the fourth power of the unit normal load in kN/mm.

13.11 Block Shear – Tension Member, Beam, and Plate Connections

Tension rupture, which is discussed in Clause 13.2, can also take place in combination with shear through the failure of a block of material in a connection component. The provisions for block shear failure in Clause 13.11 reflect the findings of research by Driver *et al.* (2006), conducted to develop a single unified equation that can be adapted to any block configuration and, in the limit, is consistent with the provisions for pure tensile rupture. An examination of numerous test results on gusset plates, coped beams, angles, and tees indicated that rupture on the tension face occurs before rupture on the shear face of the block of material and, when rupture takes place on the tension face, the shear stress on the gross shear area exceeds the yield strength but is generally less than the ultimate strength. To reflect this limit state, the design equation uses a shear stress equal to the average of the yield and rupture shear strengths on the gross shear area, A_{gv}. The shear term alone also gives the end tear-out capacity for individual bolts or lines of bolts in the direction of the applied force (Cai and Driver, 2010). Due to



(a) Block Shear Failure of Gusset Plate



(b) Block Shear Failure of Angles, Coped Beams, and Tees

Figure 2-25

potentially reduced material ductility, the yield strength is used in the shear term of the design equation for higher strength steels.

The tension component is defined in the unified block shear equation as $U_t A_n F_u$, where U_t is an efficiency factor that accounts for the non-uniformity of the stress distribution on the tension face of the block of material at the limit state. Angles, tees connected by the stem, and coped beams have all shown lower block shear resistances than would be expected if the stress on the tension face were assumed to be uniform. In these cases only one shear face exists, thus resulting in eccentric loading on the block of material that causes the non-uniform tensile stress distribution. Values of U_t vary from 1.0 for cases where no load eccentricity exists on the block of material (e.g. typical gusset plates) to 0.3 for cases with a large eccentricity (coped beams with two lines of bolts). The low efficiency of the tension face in coped beams with two lines of bolts was noted in the work of Franchuk *et al.* (2003). Driver *et al.* (2006) recommended that U_t be taken as 0.9 for angles connected by one leg and stem-connected tees, in combination with an analogous coefficient of 0.9 on the shear term. However, since S16 adopted the unified block shear equation without the shear coefficient, the value of U_t was modified to 0.6 to maintain the same reliability index for the pool of test data available. The simplified approach in the standard could produce non-conservative results for long blocks with a small tension area.

As illustrated in Figure 2-25, the block shear failure of structural tees can take various forms (Epstein and Stamberg, 2002), depending on whether the tee section is flange-connected or stem-connected. The first mode associated with flange-connected tees consists of tension and shear failure confined in the flange only. The other two modes associated with flange-connected tees involve a tension plane in the flange (with or without shear planes in the flange) and a shear plane in the stem. The various possible modes should be investigated. Use of the unified block shear equation for common types of welded connections is discussed by Oosterhof and Driver (2011).

Recommended values of U_t for various connection details are given in Figure 2-26.

13.12 Bolts and Local Connection Resistance

13.12.1 Bolts in Bearing-Type Connections

13.12.1.2 Bolts in Bearing and Shear

In bearing-type connections (Clause 13.12.1.2(a)) excessive deformation in front of the loaded edge of the bolt hole may occur. Tests have shown (Munse 1959; Jones, 1958; de Back and de Jong 1968; Hirano 1970) that the ratio of the bearing stress (B_r/dt) to the ultimate tensile strength of the plate (F_u) is in the same ratio as the end distance of the bolt (e) to its diameter (d). Thus,

$$\frac{B_r}{\phi d t} = \frac{e}{d} F_u$$

or, for n fasteners, $B_r = \phi_{br} t n e F_u$

Because the test results do not provide data for e/d greater than 3, an upper limit of $e = 3d$ is imposed. That is,

$$B_r \leq 3\phi_{br} t d n F_u$$

For the bearing of bolts on steel, the value of ϕ_{br} in Clause 13.12.1.2 is to be taken as 0.80. For the bearing resistance perpendicular to long slotted holes, see Clause 13.12.1.2(b).

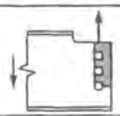
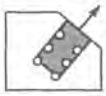
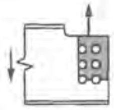
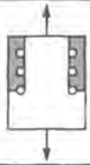
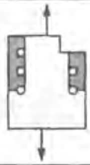
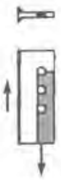
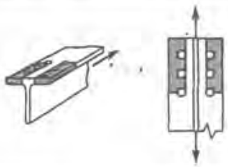
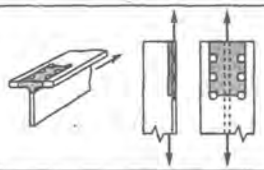
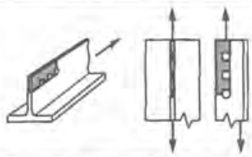
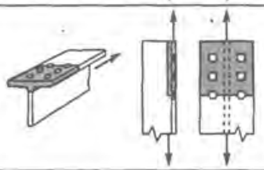
No.	Connections subject to block shear		U_t	No.	Connections subject to block shear		U_t
1		Coped beam with one row of bolts	0.9	7		Gusset plate, symmetrical block and uniform tensile stresses	1.0
2		Coped beam with two rows of bolts	0.3	8		End plate welded to supported beam, bolted to supporting member	0.9
3		Angle in tension connected to one leg	0.6	9		Similar to Case "8" above but with a clipped corner for erection safety	0.9
4		Angle in shear, one leg bolted to supported beam and other leg welded on 3 sides to supporting member	0.6	10		Double angles in shear, one leg bolted to supported beam and other leg bolted (or welded) to supporting member	0.6
5		Single plate (shear tab) bolted to supported beam, welded to supporting member	0.6	11		Similar to Case "10" above but with a clipped leg for erection safety (double-sided connection)	0.6
6a			0.9	12		Tee in shear, stem bolted to supported beam, flanges welded to supporting member	0.6
6b		Flange-connected Tee in tension	1.0	13		Stem-connected Tee in tension	0.6
6c			1.0	CSA S16-14 Clause 13.11: $T_r = \phi_u [U_t A_n F_u + 0.60 A_{gv} (F_y + F_u) / 2]$ $\phi_u = 0.75$			

Figure 2-26
Values of U_t for Block Shear

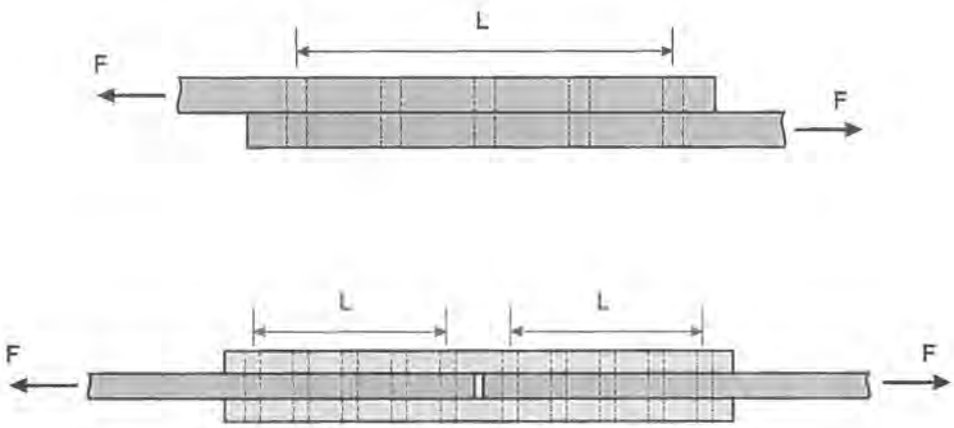


Figure 2-27
Lap Joint Length Definition for Lap and Butt Joints

The note directs designers to Clause 13.11, to investigate any potential for block tear-out when the end distance, *e*, is small and to Clause 22.3.4 for minimum end distances.

Based on extensive testing, it has been established that the shear strength of high-strength bolts is approximately 0.60 times the tensile strength of the bolt material. However, if threads are intercepted by a shear plane, there is less shear area available. The ratio of the area through the threads of a bolt to its shank area is about 0.70 for the usual structural sizes.

In the case of long joints, the load is not shared equally among the bolts with those fasteners towards the ends of the joint carrying the largest portion of the load. The linear reduction in S16-09 has been replaced by a step reduction in bolt capacity when the joint length, L, equals or exceeds 760 mm. This approach has also been adopted in CSA S6-14.

Note that the length L is that in which the load is transferred from one plate to another. For a lap joint with bolts in single shear, this is the total length between the centrelines of the end fasteners. For a butt joint with two lap plates and the bolts in double shear, it is the "half" length (see Figure 2-27).

In this context, "joint length" refers to an axially loaded connection, such as a lap splice, whose length is measured parallel to the direction of applied force. This clause does not apply to a shear connection at the end of a girder web where the load is distributed reasonably uniformly to the fasteners.

13.12.1.3 Bolts in Tension

The ultimate resistance of a single high-strength bolt loaded in tension is equal to the product of its tensile stress area (a value between the gross bolt area and the area at the root of the thread because the failure plane must intercept a thread) and the ultimate tensile strength of the bolt. The tensile stress area is very nearly equal to 0.75 of the gross area of the bolt.

In addition to the applied load, two other tensile forces – prying action and pretensioning – may act on the bolt, and their effects have to be examined. The Standard states, in fact, that the factored tensile force is independent of the pretension but that the tensile prying force shall be added to the external load.

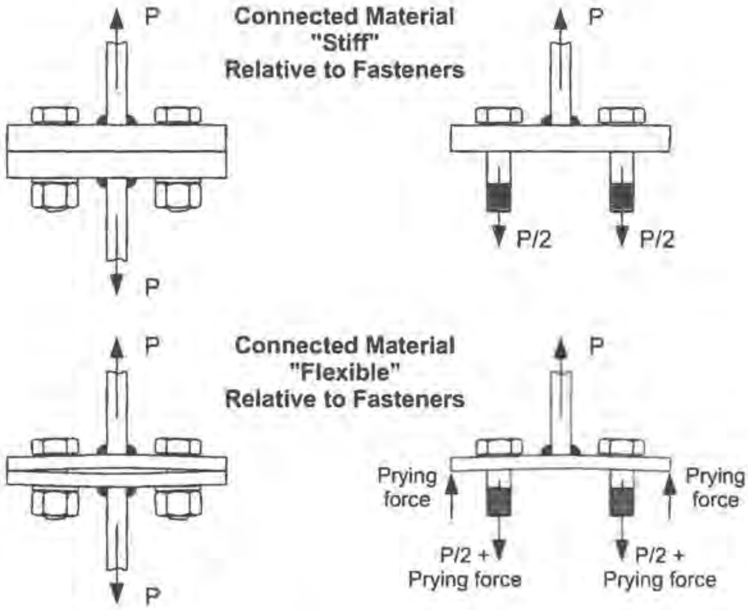


Figure 2-28
Effect of Prying Action on Bolt Tension

Figure 2-28 illustrates qualitatively that the amount of prying action depends on the flexibility of the connected material relative to the bolts. Kulak *et al.* (1987) present a procedure for calculating the prying force depending on the joint geometry, which is presented in Part 3 of this Handbook along with suggested detailing practices to minimize this force.

The statement that the factored tensile force is independent of the pretension derives from Figure 2-29 where, before any external load P is applied, the bolt pretension is balanced by the plate pre-compression. When the external load is applied without distorting the connected material as shown, or equivalently when the connected material is "stiff", as the external force is increased, the bolt force remains almost constant at the bolt pretension, while the contact pressure between the bolted plates decreases. Once the applied force is sufficiently large to separate the plates, the contact pressure goes to zero, and the sum of the bolt forces becomes equal to the applied external force. The level of bolt pretension therefore affects the force at which the bolted plates will separate, but it has no effect on the joint tension capacity.

On the other hand, when the external load is applied through some thickness of material causing it to compress, more bolt elongation is required and there is some increase in the bolt tension. Measurements of actual bolt forces in connections of practical sizes have shown that the increase in the bolt force due to the flexibility of the connection is usually only about 5 to 10%. The Standard neglects this. Figure 2-30 depicts possible variations of the tension on a pretensioned bolt as it is loaded with an external load, P, as pretensions, T_0, decrease and in the presence of a prying force, F.

This Standard requires that high-strength bolts subjected to tensile cyclic loading be fully pretensioned and that the prying force not exceed 30% of the externally applied load. Two options are given to calculate the tensile stress range to compare to the permissible values. The first and most difficult takes into account the prying action, the pretension with possible relaxation due to joint deformations and the applied load. The second assumes the range is that due

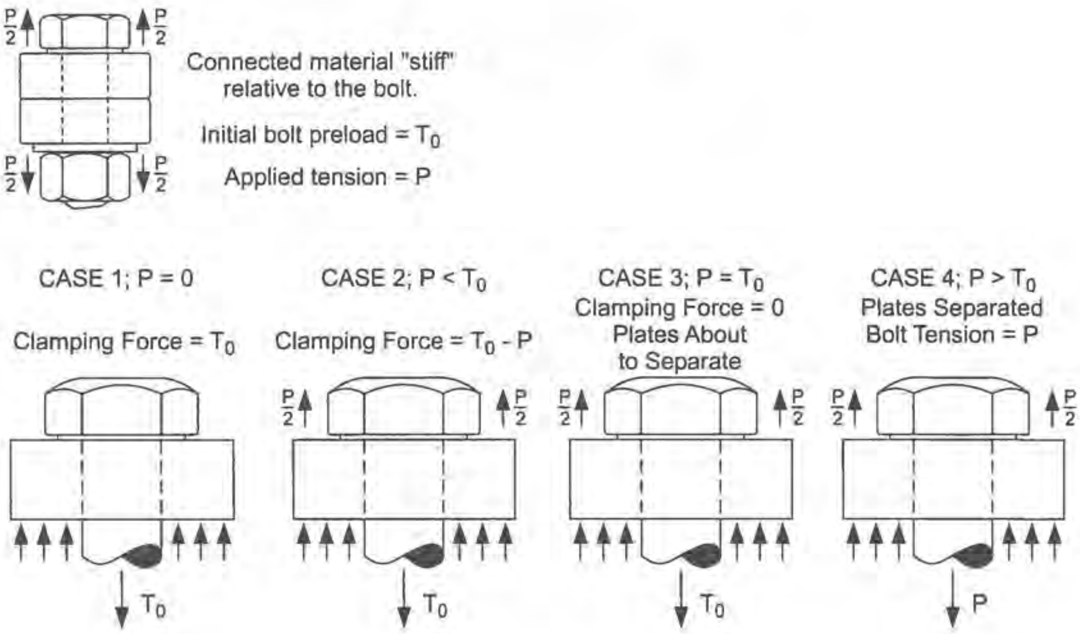


Figure 2-29
Effect of Applied Tension on Tightened High-Strengh Bolts

to the applied loads plus prying action. This is obviously conservative as the pretension reduces the applied load stress range.

13.12.1.4 Bolts in Combined Shear and Tension

The expression for the ultimate strength interaction between tension and shear applied to a fastener has been shown to model empirically the results of tests on single fasteners loaded simultaneously in shear and tension. The values of V_r and T_r are the full resistances in shear and tension, respectively, which would be used in the absence of the other loading. For small components of factored load relative to the resistance in one direction, the resistance in the other direction is reduced only a small amount; e.g. for a factored tension equal to 20 % of the full tensile resistance, the resistance available for shear is only reduced by 2 % of the full value that would be present in the absence of tension.

13.12.2 Bolts in Slip-Critical Connections

13.12.2.2 Shear Connections

Different installation procedures may result in different probabilities of slip; see Kulak *et al.* (1987).

Both the slip coefficient and the initial clamping force have considerable variation about their mean values. The coefficients of friction for coatings can vary as a function of the specific coating constituents and, therefore, values of the mean slip coefficient, k_s, may differ from one coating specification to another. The value of k_s intended for use on a project should be specified.

The clamping force is due to the pretensioning of the bolts to an initial tension, T_i, which is a minimum of 70% of the tensile strength ($0.70 A_s F_u$) where $A_s = 0.75 A_b$. Thus, the clamping force per bolt is

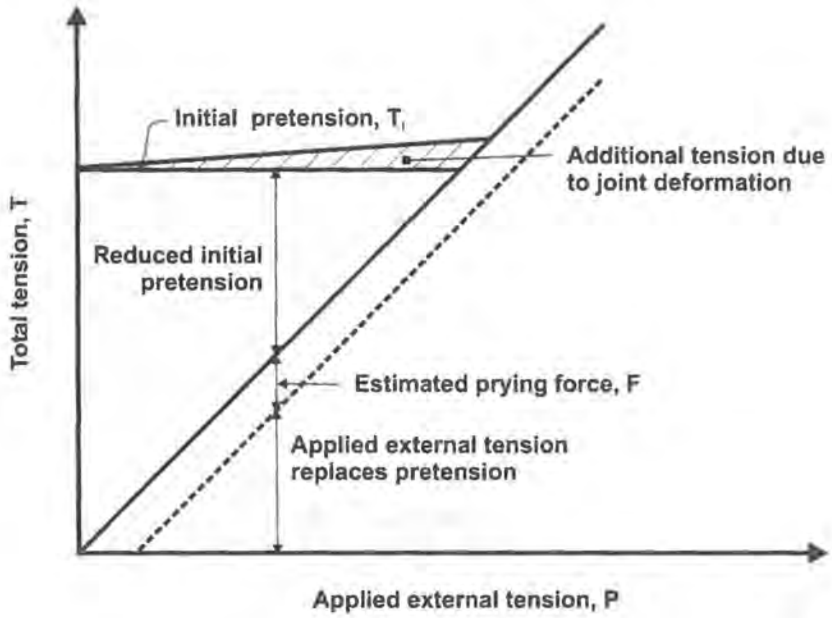


Figure 2-30
Total Tension vs. Applied Tension for a Pretensioned Bolt

$$0.70 \times 0.75 A_b F_u \text{ or } 0.53 A_b F_u$$

The values of the resistance factor, c_s, establish a uniform probability level of slip for the bolt grades and the installation methods. Table 3 of S16-14 gives values of c_s for bolts installed by the three pretensioning methods permitted by the Standard: (a) turn-of-nut method for A325, A325M, A490 and A490M bolts, (b) F1852 and F2280 twist-off type of tension-control bolt assemblies, and (c) use of washer-type direct tension indicators (F959 washers with A325, A325M, A490 and A490M bolts). For methods (b) and (c), smaller c_s values are given as the clamping loads obtained are lower (but still above the minimum required by the Standard) than those obtained by the turn-of-nut procedure. Table 3 also provides the values of k_s for two classes of contact surface. In S16-14, hot-dip galvanized surfaces are grouped under Class A. Values of k_s for some other common surface conditions are given by Kulak *et al.* (1987).

The use of slip-critical connections should be the exception rather than the rule. They are the preferred solution only where cyclic loads or frequent load reversals are present, or where the use of the structure is such that the small one-time slips that may occur cannot be tolerated. See also the Commentary to Clause 22.2.2.

The slip resistance is reduced by a factor of 0.75 for slip-critical connections using long slotted holes to account for the reduced clamping force that otherwise would be present (Kulak *et al.* 1987).

13.12.2.3 The resistance to slip is reduced as tensile load is applied and reaches zero when the parts are on the verge of separation, as no clamping force then remains. The interaction relationship is linear.

The term $1.9/(nA_bF_u)$ is the reciprocal of the initial bolt tension, $0.53\, nA_bF_u$.

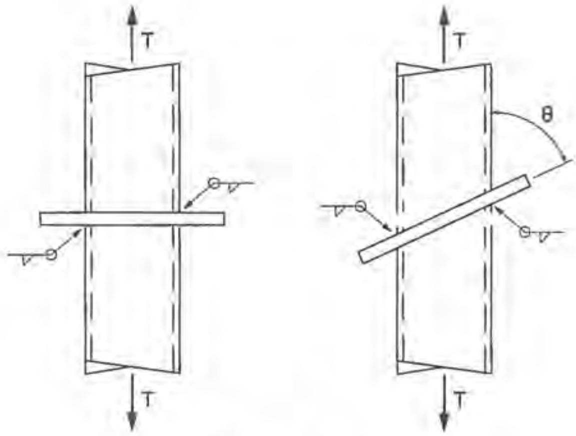


Figure 2-31
Fillet Welds to HSS

13.13 Welds

13.13.1 General

Clause 13.13 covers resistances for welded joints that satisfy matching conditions; provisions and restrictions for use of non-matching electrodes are included in Clause 24. Matching electrodes for various grades of base steel as specified in Table 4 of S16-14 and W59-13 typically are those with ultimate strengths similar to that of the base metal. W59 permits the use of electrodes that are one designation higher than matching (i.e. over-matched), provided certain specific conditions are met and the value of X_u used in the calculation of the weld resistance does not exceed X_u of the matching electrode. When atmospheric corrosion-resistant steel grades are used in the uncoated condition, additional requirements for compatible corrosion resistance or colour are also required for matching electrodes.

A resistance factor of $\phi_w = 0.67$ is used universally in this section, recognizing that a larger value of the reliability index is used for connector resistances.

13.13.2 Shear

In general, the shear resistance of a weld is evaluated on the basis of both the resistance of the weld metal and of the base metal adjacent to the weld. Although the calculations indicate that the resistance of the base metal may govern the capacity of the welded joint, this is seldom the case. Thus, CJPG, PJPG, plug, and slot welds loaded in shear have resistances equal to the lesser of the weld throat or fusion face shear strength. Research on fillet-welded splices (Butler *et al*. 1972; Miazga and Kennedy, 1989; Ng *et al*., 2004a; 2004b; Deng *et al*., 2006; Callele *et al*. 2009) showed that even when fillet welds failed primarily in the fusion zones, the capacity of the weld calculated according to the weld metal capacity only provided a sufficient level of safety with a reliability index of about 4.5. Therefore, for fillet welds oriented at an angle greater than about 45°, where the calculation of the base metal strength indicates that the strength of the base metal would govern the capacity of the joint, the base metal check effectively prevents the designer from taking advantage of the full capacity of the weld. It was concluded by Callele *et al*. (2009) that the tensile strength of the base metal does not represent the actual tensile strength of the material at the fusion face, which is influenced by intermixing of the weld and base metals. Even if over-matched electrodes (see Commentary to Clause 13.13.1) are used, the base metal check is not required for the design of fillet welds, provided the resistance is calculated using the tensile strength of the matching electrode, X_u.

Using the instantaneous centre of rotation concept, the resistance expression in 13.13.2.2 forms the basis of the eccentric load tables given in Part 3 of the CISC Handbook (Butler and Kulak 1971, Butler *et al.* 1972, Miazga and Kennedy 1989, Lesik and Kennedy 1990, Kennedy *et al.* 1990). This ultimate strength analysis, recognizing the true behaviour of the weldments, results in much more consistent strength predictions than the traditional approach (i.e., taking the quantity $1.00 + 0.50\sin^{1.5}\theta$ as 1.0).

In the expression for the shear strength of the weld, the factor 0.67 relates the shear strength of the weld to the weld metal tensile strength, as given by the rated electrode classification number. Lesik and Kennedy (1990) give 0.75 for this factor, based on 126 tests reported in the literature. The coefficient 0.50 in the quantity $1.00 + 0.50\sin^{1.5}\theta$ is for tension-induced shear and is slightly more liberal than the average value of tension- and compression-induced shear of 1.42 reported by Lesik and Kennedy. In addition, the factor 1.50 is the correct value for Clause 13.13.2.2 in which tension is the critical case. The value of 0.50 has also been adopted by AWS and AISC. However, recent experimental research on welded HSS joints (Packer *et al.* 2015) has shown that, in order to yield a reliability index, $\beta = 4.5$, the fillet weld "directional strength enhancement factor" $(1.00 + 0.50\sin^{1.5}\theta)$ for fillet welds to HSS as shown in Figure 2-31 should be used with a ϕ_w value lower than 0.67. For this application, Packer *et al* (2015) recommend setting this factor to unity (i.e. $\theta = 0$) and keeping $\phi_w = 0.67$ for a conservative solution (i.e. $\beta > 4.5$). For further discussion on fillet welds to HSS when the "effective length concept" is used to proportion fillet welds, see the Commentary to Clause 13.13.4.3.

Callele *et al.* (2009) showed that when fillet welds with multiple orientations are contained within the same concentrically loaded joint, the lower ductility of the welds oriented closest to 90° prevents the more ductile welds from reaching their full capacity before failure of the joint takes place. The researchers proposed a simple means of accounting for this phenomenon by reducing the capacity of the more ductile weld segments by up to 15%. This method has been adopted into the Standard using the factor M_w.

Clause 13.13.2.3 provides users of the Standard with an expression to determine the factored resistance of flare bevel groove welds for open-web steel joists based on (a) observed data relating the face width to the effective throat thickness of flare bevel groove welds as reported by Skarborn and Daneff (1998), (b) other data on welds in general from Lesik and Kennedy (1990), and (c) the principles set forth in Galambos and Ravindra (1973). Thus, using $\phi_w = 0.67$ with the effective throat taken as 0.50 of the weld face as selected here leads to a reliability index of 4.25 as determined by Kennedy (2004).

13.13.3 Tension Normal to Axis of Weld

Gagnon and Kennedy (1989) established that the net area tensile resistance, i.e. on a unit area basis, transverse to the axis of a PJPG weld, is the same as for the base metal when matching electrodes are used. The previous conservative practice of assigning shear resistances to these welds was replaced in the 1989 edition with tensile resistances, consistent with the tensile resistance of complete penetration welds equalling the full tensile resistance of the member.

For T-type joints consisting of PJPG weld and a reinforcing fillet weld, Clause 13.13.3.3 provides a conservative estimate of the tensile resistance by taking the vector sum of the individual component resistances of the PJPG and fillet welds.

13.13.4.3 Welds for Hollow Structural Sections

There are two methods currently available for the design of welded connections between square and rectangular HSS (Packer *et al.*, 2010; McFadden *et al.*, 2013):

1) The welds may be designed as "fit-for-purpose" and proportioned to resist the applied forces in the branch. The non-uniform loading around the weld perimeter due to the relative

flexibility of the connecting RHS face requires the use of weld effective lengths. This approach may be appropriate when there is high confidence in the design forces or if the branch forces are particularly low relative to the branch member capacity. Where applicable, this approach may result in smaller weld sizes, providing a more economical design. Weld effective lengths, related to the type of HSS connection and type of loading, have been determined from research by Frater and Packer (1992a, 1992b), Packer and Cassidy (1995), McFadden and Packer (2014) and Tousignant and Packer (2015). An up-to-date summary of weld effective lengths (or weld effective properties) for HSS connections is given in Section K4 of the Specification AISC 360 (AISC 2010b).

However, the fillet weld "directional strength enhancement factor" $(1.00 + 0.50\sin^{1.5}\theta)M_w$, contained in Clause 13.13.2.2, should *not* be applied to fillet welds to HSS when the "effective length concept" is used to proportion fillet welds (McFadden and Packer, 2014; Tousignant and Packer, 2015).

2) The welds may be proportioned to develop the yield strength of the connected branch wall at all locations around the branch. This approach may be appropriate if there is low confidence in the design forces, uncertainty regarding Method (1) above, or if plastic stress redistribution is required in the connection. This method will produce an upper limit for the weld size required and may be excessively conservative in some situations.

13.14 Welds and High-Strength Bolts in Combination

This clause addresses the design of joints in which welds and high-strength bolts are placed in the same shear plane and are expected to share the applied shear force. The provisions are based on the work of Manuel and Kulak (1999) and Kulak and Grondin (2003). The capacity of each connector in this type of shear splice is reflected by its shear strength and shear deformation characteristics. When bolts and welds share the load, the fastener that possesses the least ductility (welds as opposed to bolts or transverse welds as opposed to longitudinal welds) is able to reach its full capacity before the full capacity of the more ductile fastener is fully developed. Therefore, the shear resistance of the joints consists of the full capacity of the least ductile fastener plus a fraction of the capacity of the more ductile fastener. The resistance of the joint is calculated based on the progression of failure from the least ductile fastener to the most ductile fastener. Consequently, the capacity of a typical joint that combines transverse and longitudinal welds and bolts could be limited by (i) the load at which the transverse weld fractures, (ii) the load at which the longitudinal welds fracture, or (iii) the load at which the bolts fracture.

When considering case (i), tests by Manuel and Kulak have shown that the ductility of transverse welds is insufficient to mobilize a significant portion of the bolt shear strength, but sufficient to mobilize about 85% of the strength of the longitudinal welds. Case (ii) considers that the transverse weld, if present, has already fractured. In this case, the longitudinal welds are sufficiently ductile to mobilize a significant portion of the bolt shear strength. The work of Manuel and Kulak showed that the portion of the bolt shear strength that is mobilized by the time the longitudinal welds have fractured depends on the bearing conditions of the bolts at the time that the welds are added to the joint. They made a distinction between the case where the bolts are in full bearing in the direction of the applied load (positive bearing) and the case where the bolts are in bearing in the direction opposite to the applied load (negative bearing).

The results of later tests presented by Kulak and Grondin showed that joints where the bearing conditions are varied randomly could develop at least 50% of the shear strength of the bolts by the time the longitudinal welds fracture. Case (iii) considers the situation where both the transverse and longitudinal welds have fractured. At this point, only the bolts are able to resist the applied load. It should be noted that in cases (i) and (ii) a contribution from the slip resistance can be accounted for when the bolts have been pretensioned in accordance with Clause

23.7. However, in case (iii) no slip resistance is accounted for since the shear deformation in the bolts at the time that their full strength has been mobilized is sufficient to have released their pretension.

Equation (a) of Clause 13.14 considers case (i) described above. For this case, only the welds contribute to the shear resistance plus 25% of the slip resistance if the bolts are pretensioned. The strength of the welds is calculated using Clause 13.13.2.2 with $\theta = 90°$ for the transverse weld segment and $\theta = 0°$ for the longitudinal weld segment. The factor 0.85 is the value of M_w when longitudinal and transverse welds are combined in the same shear plane. Equation (b) considers case (ii) where the transverse weld has already fractured, and only the longitudinal welds and the bolts are left to carry the load. By the time the ductility of the longitudinal welds has been exhausted, 50% of the shear capacity of the bolts would be mobilized. Equation (c) considers case (iii) where only the bolts are left in the joint. At this stage the limit state is fracture of the bolts, and the strength of the joint is limited to the shear resistance of the bolts or the bearing resistance of the plates against the bolts.

It should be noted that in all the cases tested experimentally, the bolt resistance was always governed by bolt shear rather than plate bearing. Since the bearing resistance usually requires more deformation to develop than the shear resistance, it is possible that the contribution from the bolts may be less than 50% when plate bearing governs the bolt resistance. When bearing governs, the designer may want to use less than 50% of the bolt shear resistance. For typical examples of joint strength calculations, see Kulak and Grondin (2003).

14. BEAMS AND GIRDERS

14.1 Proportioning

Lilley and Carpenter (1940) have shown that reductions of flange area up to 15% can be disregarded in determining the effective moment of inertia, due to the limited inelastic behaviour near the holes.

14.2 Flanges

The theoretical cut-off point is the location where the moment resistance of the beam without cover plates equals the factored moment (Figure 2-32). The distance a' increases as shear lag becomes more significant, as is the case when the weld size is smaller, or when there is no weld across the end of the plate. Theoretical and experimental studies of girders with welded cover plates (ASCE 1967) show that the cover plate load can be developed within length a'. Clause 14.2.4 limits the length of a' for welded cover plates and may therefore necessitate an increase in weld size or an extension of the cover plate so that the force at a distance a' from its end equals that which the terminal welds will support.

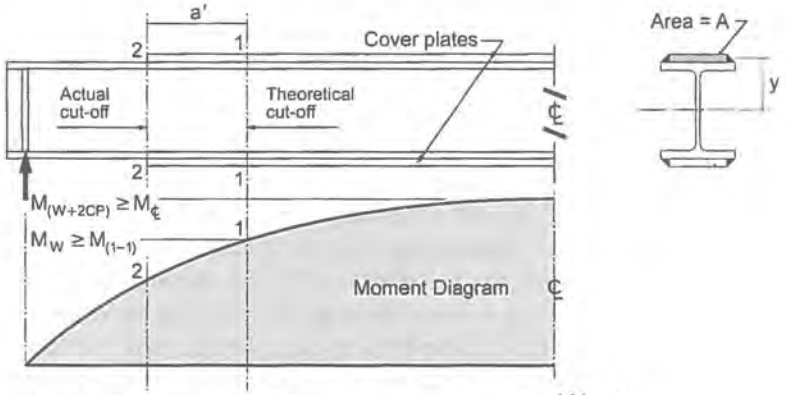
14.3 Webs

14.3.1 Maximum Slenderness

This limit prevents the web from buckling under the action of the vertical components of the flange force arising as a result of the curvature of the girder (Kulak and Grondin 2014).

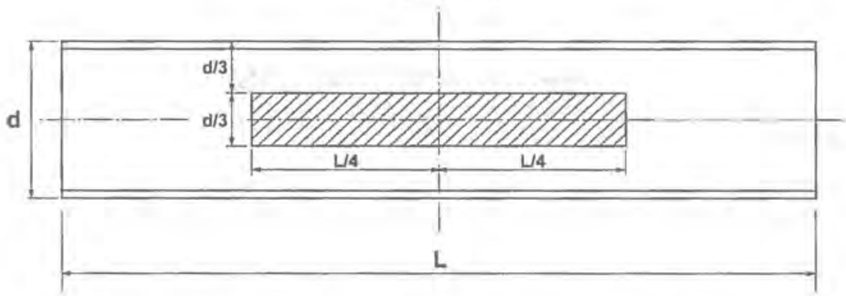
14.3.2 Web Crippling and Yielding

Loads and reactions acting perpendicular to a flange and over a short length along the flange will cause in-plane compressive stresses in the web. The ultimate strength of the unstiffened web subjected to such edge loading may be governed by either yielding of the web or crippling of the web (a localized out-of-plane buckling of the web adjacent to the loaded flange).



Force to be developed in cover plate within distance **a'** is: $\dfrac{A M_{(1-1)} y}{I_{(W+2CP)}}$

Figure 2-32
Cover Plate Development



Unreinforced circular holes may be placed anywhere within the hatched zone without affecting the strength of the beam for design purposes, provided:

1. Beam supports uniformly distributed load.
2. Beam section has an axis of symmetry in plane of bending.
3. Spacing of holes meets the requirements shown below.

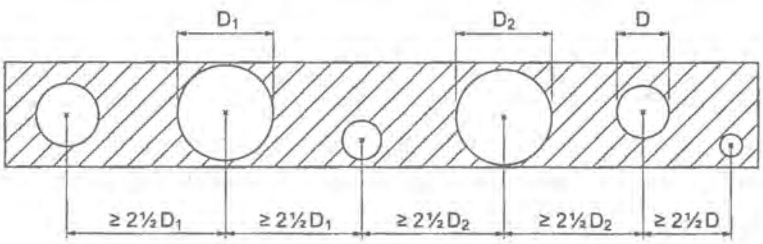


Figure 2-33
Unreinforced Circular Web Openings in Beams

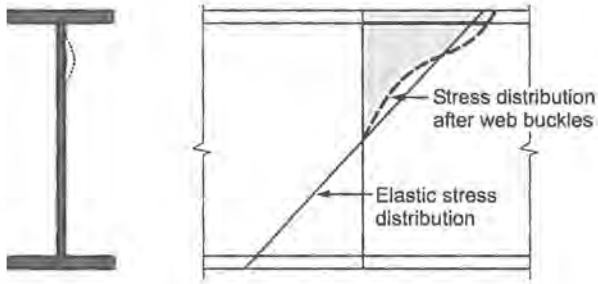


Figure 2-34
Approximate Stress Distribution in Girders with Buckled Web

If the web is relatively stocky, yielding will occur prior to crippling, and expressions 14.3.2(a)(i) and 14.3.2(b)(i) govern web resistances for interior loads and end reactions, respectively.

Relatively thin webs cripple before yielding, and the strength of the web is governed by expressions 14.3.2(a)(ii) and 14.3.2(b)(ii) for interior loads and end reactions, respectively.

The equations presented in the Standard are based on the work of Kennedy *et al.* (1998). These equations are much simplified relative to the 1994 Standard and correlate well with a set of 31 full-scale tests by Benichou (1994) and others at Carleton University. In the expression for web crippling, the contribution of the flange is neglected. It is argued that, at interior load points, the normal stress in the flanges of efficiently designed girders would approach the yield stress at factored loads. Consequently the flanges would not have significant plastic hinge capacity in developing a plastic hinge mechanism in the resistance of transverse loads.

For unstiffened portions of webs, when concentrated compressive loads are applied opposite one another to both flanges, the compressive resistance of the web acting as a column should also be investigated. (See also Clause 21.3.)

Care should be taken in assessing the bearing length under yielding or deforming supports such as girders cantilevering over columns.

14.3.3 Openings

The conditions under which unreinforced circular openings may be used are based on Redwood and McCutcheon (1968) and are illustrated in Figure 2-33.

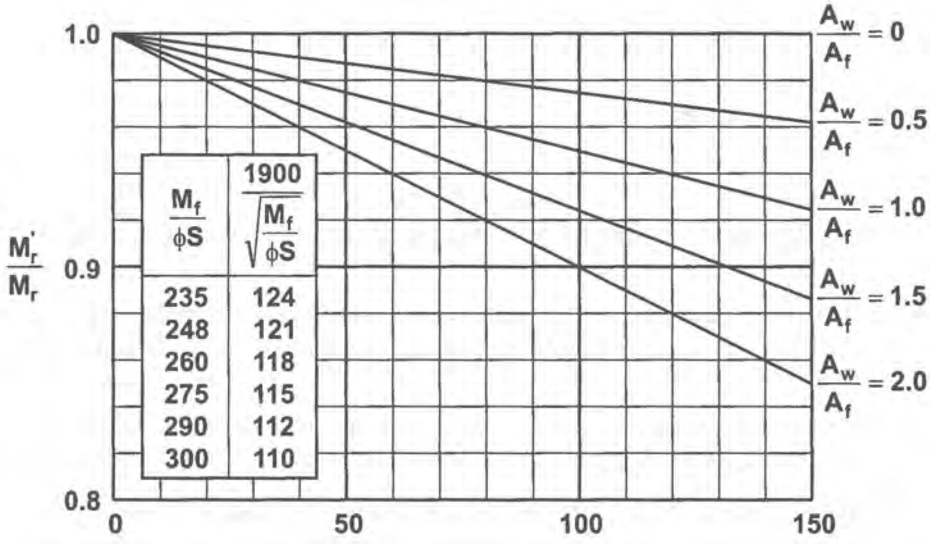
Elastic and plastic analyses to determine the effect of openings in a member are given in Bower *et al.* (1971) and Redwood (1971, 1972, 1973), respectively. See Part 5 of the Handbook for worked examples.

A combination of vertical and horizontal intersecting stiffeners (particularly on both sides of a web) is seldom justified and quite expensive to fabricate. Generally, horizontal stiffeners alone are adequate. When both vertical and horizontal stiffeners are necessary, the horizontal stiffeners should be on one side of the web, and vertical stiffeners on the other, in order to achieve economy.

14.3.4 Effect of Thin Webs on Moment Resistance

A plate girder with Class 3 flanges and Class 4 webs has a maximum moment resistance less than ϕM_y because the Class 4 web may buckle before extreme fibre yielding due to the compressive bending stresses. The reduction in moment resistance is based on Basler and

$$\frac{M'_r}{M_r} = 1.0 - 0.000\,5\frac{A_w}{A_f}\left[\frac{h}{w} - \frac{1900}{\sqrt{M_f/(\phi S)}}\right]$$



$\frac{M_f}{\phi S}$	$\frac{1900}{\sqrt{M_f/\phi S}}$
235	124
248	121
260	118
275	115
290	112
300	110

$$\frac{h}{w} - \frac{1900}{\sqrt{M_f/(\phi S)}}$$

Figure 2-35
Reduced Moment Resistance in Girders with Thin Webs

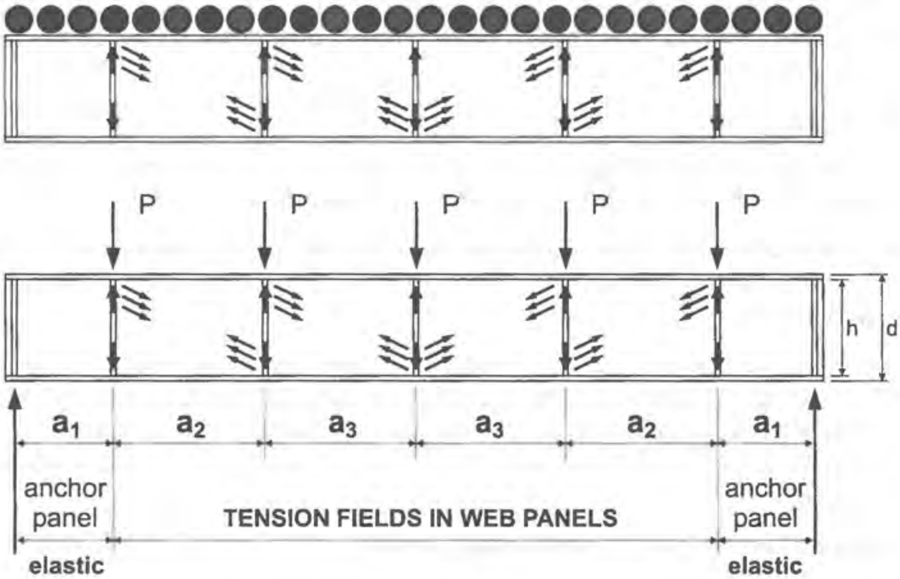


Figure 2-36
Action of a Thin-Web Plate Girder Under Load

Thurlimann (1961). Figure 2-34 shows an approximate stress distribution in a girder with a buckled web. The reduction in moment resistance is generally small, as shown in Figure 2-35.

The limit of $1900/\sqrt{F_y}$ for the slenderness of a Class 3 web is replaced in this clause by $1900/\sqrt{M_f/(\phi S)}$ to account for the possibility that the factored moment may be less than $M_r = \phi S F_y$, thereby reducing the propensity for web buckling.

In some circumstances, a plate girder may be subjected to an axial compressive force in addition to the bending moment (e.g. rafters in a heavy industrial gable frame, beams in a braced frame). The constant 1900 is then multiplied by the factor $(1.0 - 0.65\,C_f/\phi C_y)$ to account for the increased tendency for the web to buckle. The compressive stresses due to the axial load add to the compressive stress due to bending, thus increasing the depth of web in compression (see also commentary to Clause 11).

14.4 Bearing Stiffeners

The inclusion of a portion of the web in the column section resisting the direct load, and the assumption of an effective length of 0.75 times the stiffener length, are approximations to the behaviour of the web under edge loading that have proved satisfactory in many years of use.

14.4.2 In S16-14, the limit for width-to-thickness ratio for bearing stiffeners is explicitly stated. This limit corresponds to the Class 3 limit for plate elements supported along one edge and therefore applies to plate stiffeners for single web girders. For other types of stiffeners and stiffeners with other edge support conditions, Class 3 limits appropriate for each respective stiffener type and support condition may apply.

Where the Class 3 limit is exceeded because the plate width exceeds what is needed to satisfy other requirements in this Clause, the effective area method in accordance with Clause 13.3.5(a) may be used.

14.5 Intermediate Transverse Stiffeners

14.5.1 Figure 2-36 illustrates the action of a thin girder web under load. Tension fields are developed in the interior panels but cannot develop in the unanchored end panels, for which the maximum shear stress is, therefore, either the elastic or inelastic critical plate buckling stress in shear.

14.5.2 The limits on stiffener spacing are based on practical considerations. When $a/h > 3$, the tension field contribution is reduced. For slender webs ($h/w > 150$) the maximum stiffener spacing is reduced for ease in fabrication and handling.

14.5.3 Clause 14.5.3 requires that intermediate transverse stiffeners have both a minimum moment of inertia and a minimum area. The former provides the required stiffness when web panels are behaving in an elastic manner; the latter ensures that the stiffener can sustain the compression, to which it is subjected, when the web panel develops a tension field. Because stiffeners subject to compression act as columns, stiffeners placed only on one side of the web are loaded eccentrically and are less efficient. The stiffener factor (D) in the formula for stiffener area accounts for the lowered efficiency of stiffeners furnished singly, rather than in pairs.

14.5.4 The minimum shear to be transferred between the stiffener and the web is based on Basler (1961c).

14.5.5 The requirement of attaching single intermediate stiffeners to the compression flange is to prevent tipping of the flange under loading.

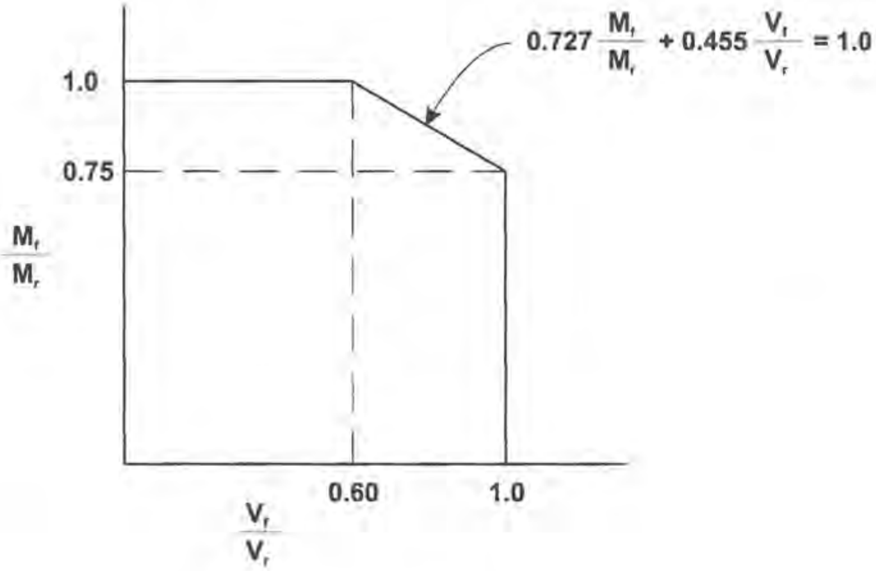


Figure 2-37
Combined Shear and Moment Interaction Expression

14.6 Combined Shear and Moment

This requirement recognizes the limit state of the web yielding by the combined action of flexural stress and the post-buckling components of the tension field development in the web near the flange (Basler, 1961b).

Figure 2-37 illustrates the interaction expression provided in Clause 14.6. When Clause 14.3.4 applies, M_r' replaces M_r in the interaction expression.

14.7 Rotational Restraint at Points of Support

A severe stability problem may exist when a beam or girder is continuous over the top of a column. The compression flange of the beam tends to buckle sideways and simultaneously, the beam-column junction tends to buckle sideways because of the compression in the column. Three mechanisms exist for providing lateral restraint: direct-acting bracing, such as provided by bottom chord extensions of joists, beam web stiffeners welded to the bottom flange, or the distortional stiffness of the web. In the latter two cases, the connection of the beam flange to the column cap plate must have strength and stiffness (Chien, 1989). The restraint offered by the distortion of the web requires very careful assessment. See also the commentaries on Clause 13.6 and Clause 9.2.

14.8 Copes

Flanges are coped to permit beams to be connected to girder webs with simple connections while maintaining the tops of the flanges at the same elevation. Long copes may seriously affect the lateral-torsional buckling resistance of a beam (Cheng and Yura, 1986). The reduced shear and moment resistance at the coped cross-section should be examined. See the Commentary on Clause 9.2.4.

14.10 Torsion

In many cases, beams are not subject to torsion because of the restraint provided by slabs, bracing or other framing members. The torsional resistance of open sections having two flanges consists of the St. Venant torsional resistance and the warping torsional resistance.

Information on moment–torque interaction diagrams for I-shaped members for use in design is given in Driver and Kennedy (1989), Bremault *et al.* (2008) and Estabrooks and Grondin (2008). Serviceability criteria will often govern the design of a beam subject to torsion. The maximum stress due to bending and warping at the specified load level shall be limited to the yield strength to guard against inelastic deformation. For inelastic torsion of steel I-beams, see Pi and Trahair (1995). For elastic analyses, see Seaburg and Carter (1997), and Brockenbrough and Johnston (1974). For methods of predicting the angle of twist in a wide-flange shape beam, see Englekirk (1994).

15. TRUSSES

15.1 Analysis

A "pure" truss is a triangulated system with pinned joints and with loads applied only at the joints. This being the case, the members of the truss are axially loaded "two-force" members acting either in tension or compression. Such trusses are now seldom made, and the members meeting at a joint are likely welded or bolted together, and not infrequently the chords are continuous through several joints. Under these circumstances, when the truss is loaded and the members change length, the geometry of the triangles (including the angles) changes, resulting in rotations of the joints, and end moments develop in the members, causing single or double in-plane curvatures. These deformation moments are called secondary moments, as they are not due to the primary loading but solely due to the deformation of the truss with rigid joints. Moreover, because the truss members are much stiffer axially than they are flexurally, several researchers (Parcel and Murer 1934, Aziz 1972) have shown that, for steel trusses with rigid welded or bolted joints, after initial elastic behaviour the extreme fibres of the members begin to yield under the axial and bending strains. With further axial straining, the moment that can coexist decreases and approaches zero, as shown schematically in Figure 2-38, when all the strains in a member (though not uniform) are either in compression or tension. Thus the truss with sufficient ductility, even with rigid joints, behaves as though its members were pin-ended.

Primary moments are moments that can be induced in truss members due to loadings or due to connection geometry. Sometimes, for example, a top chord is used to support a roof deck directly and the transverse loads between joints bend the chord and induce end moments at the panel points, which are distributed among the members meeting at a joint with some moments carried over to other joints. Thus, there are primary moments distributed throughout the truss. A common procedure is to analyze such a truss as a pin-jointed assemblage and to add to the forces so found the moments due to the transverse loadings.

Primary moments are also induced when the centroidal axes of the members meeting at a joint do not intersect at a common point, causing a rotation of the joint. These can be analyzed as for the other primary moments, taking the truss members as axially loaded members with the bending moments added. If the trusses with primary moments are analyzed using, say, an elastic plane frame analysis, then the stress resultants found will include the axial forces in the members and both the primary and secondary moments. Because the secondary moments for ductile trusses are of little or no consequence, trusses proportioned on this basis will be stronger than they need be.

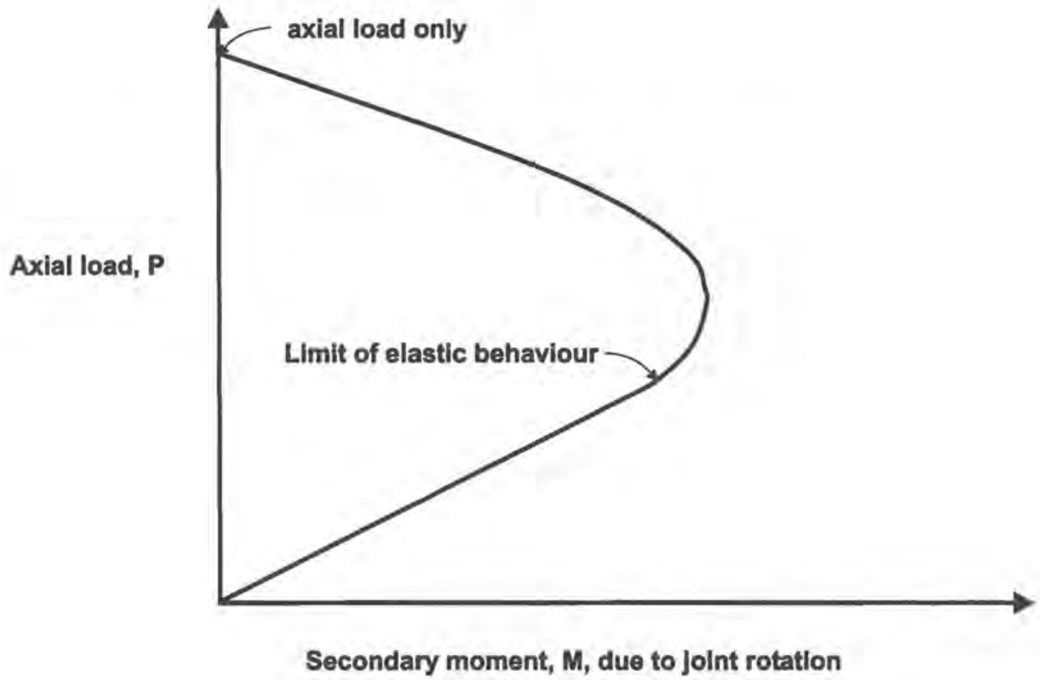


Figure 2-38
Axial Load – Secondary Moment Interaction Diagram
for a Rigid-Jointed Ductile Steel Truss

15.1.1 Simplified Method

The "Simplified Method" of analysis based on pin-connected truss members predicts closely the failure load of the tests, even with large rigid connections, provided there is sufficient ductility at the connections, so that redistribution of forces and moments may take place at the joints as the failure load is approached. Thus the sections must be at least Class 3. Bending effects of transverse loads applied between joints are simply treated as additional load actions to be carried. Out-of-plane buckling of compression members is conservatively not allowed. Alternatively, though not stated, the reduced strength of the truss because of this failure mode could be taken into account.

15.1.2 Detailed Method

This Clause lists the type of trusses for which the assumption of pin connections is not considered valid. Joint fixity must be considered, and the members must be designed for the combination of axial load and bending.

15.2.1 Effective Lengths of Compression Members

The potential failure modes of compression members in trusses are either in-plane bending or buckling modes. The effective length factors are, therefore, either taken to be equal to one or are based on the restraint at the ends. Thus, the following situations arise for in-plane and out-of-plane behaviour.

(a) In-plane behaviour

A compression member with bolted or welded end connections and with in-plane joint eccentricities acts in-plane as a beam-column with axial forces and end moments that can be established. It can be isolated from the structure and is designed as a beam-column based on its actual length, that is, with an effective length factor of 1.0.

A compression member with bolted or welded end connections and without in-plane joint eccentricities, designed as an axially loaded member, has end restraints, provided that all members meeting at the two end joints do not reach their ultimate loads (yielding in tension or buckling in compression) simultaneously. The effective length factor depends on the degree of restraint. This typically occurs for trusses in which some members are oversize, for example, trusses with constant size chords. All members do not fail simultaneously, and the effective length factors may be less than one.

If, however, all members reach their ultimate loads simultaneously and none restrain others, the effective length factor should be taken as 1.0.

(b) Out-of-plane behaviour

Unless members out-of-plane of the truss exist at the end joints under consideration, the restraint to out-of-plane buckling is small and should be neglected. Provided no out-of-plane displacement of the members' ends occurs, an effective length factor of 1.0 is therefore appropriate. It should be noted that Clause 13.3.3 provides a modified slenderness-ratio method, which accounts for the end eccentricity and fixity, for single-angle members that comply with the conditions stated in that Clause.

15.2.2 Joint Eccentricities

When the centroidal axes of the truss members do not intersect at a common point, the Standard requires that the bending moment due to the joint eccentricities be considered in the design.

15.2.3 Stability

Lateral bracing, which provides stability to the compression chords of trusses, must have stiffness and strength to satisfy the requirements of Clause 9.2. Braces must be properly attached to the member being braced, and their ends must be fastened to rigid supports.

15.2.5 Web Members

It has been observed, on occasion, in tests of standardized trusses and joists that the first compression web member fails first, even though the truss deformations may be quite significant. In these cases, certain chords and webs had been designed to S16 requirements to reach their factored loads more or less simultaneously. Because the tension chord, after yielding in the panel where the bending moment is a maximum, continues to carry load into the strain-hardening range, it overloads itself and the truss. The first compression web member with no such reserve then fails by buckling. By reducing the resistance factors for this member and its connections to 85%, more ductile modes of failure are encouraged at little extra cost. This requirement is also applied to joists in Clause 16.5.7.

In tests of trusses where the bottom chord bears on a reaction, severe bending deformations have been observed near the connections of the end compression diagonal because of the geometric distortion of the truss as deflections increase. The Standard requires that the stresses arising from these bending moments be included in the design of the end diagonal. Thus, the analysis of trusses with the bottom chord bearing must be carried out using the Detailed Method.

15.2.6 Compression Chord Supports

A frequently used rule to provide full support (Winter 1960) is for a brace to have a capacity in the order of 2% of the force in the main compression member.

15.2.7 Maximum Slenderness Ratio of Tension Chords

The slenderness ratio of tension chords is limited to 240 simply to facilitate handling during erection. The exceptions to this are noted in the clause.

16. OPEN-WEB STEEL JOISTS

16.1 Scope

Open-web steel joists (OWSJ or joists), as described in Clause 16.2, are generally proprietary products whose design, manufacture, transport, and erection are covered by the requirements of Clause 16. The Standard clarifies the information to be provided by the building designer (user-purchaser) and the joist manufacturer (joist designer-fabricator).

16.2 General

The distinction between a standard and a non-standard OWSJ no longer exists, as OWSJs are designed specifically for each situation by the joist manufacturer.

This clause lists functions that joists may fulfil other than the simple support systems for floors or roofs. These include continuous joists, cantilever joists, joists in lateral-load-resisting systems and support for bracing members.

16.3 Materials

The use of yield strength levels reported on mill test certificates for the purposes of design is prohibited here as throughout the Standard. This practice could significantly lower the margin of safety because any deviation from the specified value has already been accounted for statistically in the bias value – the ratio of the mean strength to the specified minimum value. Thus, all design rules have been, and are, based on the use of the specified minimum yield point or yield strength. For structural members cold-formed to shape, the increase in yield strength due to cold forming, as given in CSA Standard S136, may be taken into account provided that the increase is based on the specified minimum values in the relevant structural steel material standard.

16.4 Design Documents

16.4.1 Building Structural Design Documents

The Standard recognizes that the building designer may not be the joist designer; therefore, the building structural design documents are required to provide specific information for the design of the joists. The information to be supplied includes a note that any drilling, cutting or welding has to be approved by the building designer.

Uplift and downward wind effects, as well as balanced, unbalanced, non-uniform and concentrated loads, are to be shown by the building designer. Figure 2-39 shows a sample joist schedule that could be used to record all gravity loads on joists and any in-plane wind load acting normal to the top chord. Prior to the introduction of National Building Code of Canada 2005, the significance of downward wind effects on roof members depended primarily on the wind-to-snow load ratio. The adoption of load combinations in companion action format in NBCC 2005 eliminated the application of the combination (reduction) factor when wind acts in combination with variable gravity loads. This change resulted in the addition of downward

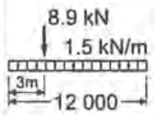
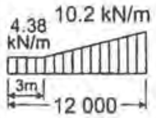
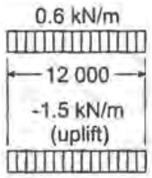
Mark	Depth (mm)	Spacing (mm)	Specified Dead Load	Specified Live Load	Specified Snow Load	Specified Wind Load	Remarks
J1	600	1 300	4.0 kPa	2.4 kPa			$\Delta_{live} \le \frac{span}{320}$ Suggested I_{eff} for vibration = ______
J2	700	2 000	 8.9 kN 1.5 kN/m 3m 12 000		 4.38 kN/m 10.2 kN/m 3m 12 000	 0.6 kN/m 12 000 -1.5 kN/m (uplift)	$\Delta \le \frac{span}{240}$

Figure 2-39
Joist Schedule

wind effects to snow or live load regardless of wind-to-snow load ratios. The NBCC (2015) requires the internal suction in combination with any external downward wind pressure to be included in the total downward wind effect.

All heavy concentrated loads such as those resulting from partitions, large pipes, mechanical, and other equipment to be supported by OWSJs, should be shown on the structural design documents. Small concentrated loads may be allowed for in the uniform dead load.

The building designer should specify the building Importance Category as defined in the NBCC (2015). Alternatively, the NBCC Importance Factors, I_S, I_W and I_E, as appropriate, and the importance factor for live load (see Clause 6.2.2) when not equal to 1.0, should be specified.

Options, such as attachments for deck when used as a diaphragm, special camber and any other special requirements should also be provided. Where vibration of a floor system is a consideration, it is recommended that the building designer give a suggested effective composite moment of inertia, I_{eff} (Murray *et al.* 1997). Because the depth of joists supplied among different joist manufacturers may vary slightly from nominal values, the depth, when it is critical, should be specified.

When sprayed fire protection is contemplated, reduce clearance by thickness of sprayed fire protection material.

Although steel joist manufacturers may indicate the maximum clear openings for ducts, etc. which can be accommodated through the web openings of each depth of their OWSJs, building designers should, in general, show on the building design drawings the size, location and elevation of openings required through the OWSJs (Figure 2-40). Large ducts may be accommodated by special design. Ducts which require open panels and corresponding reinforcement of the joist should, where possible, be located within the middle half of the joist to minimize shear effects. This information is required prior to the time of tendering to permit appropriate costing.

Specific joist designations from a manufacturer's catalog or from the AISC and Steel Joist Institute of the U.S.A. are not appropriate and should not be specified.

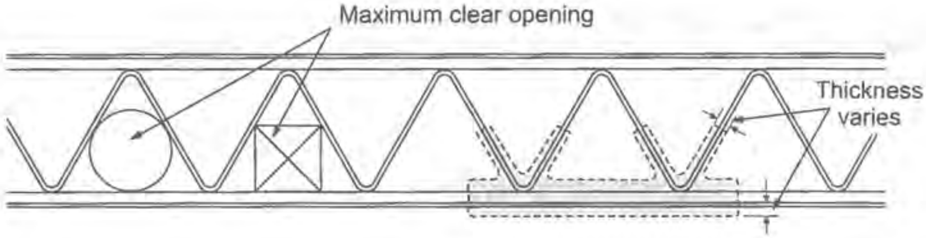


Figure 2-40
Sizes of Openings for Electrical and Mechanical Equipment

16.4.2 Joist Design Documents

The design information of a joist manufacturer may come in varying forms such as: design sheets, computer printout, and tables. Not all joist manufacturers make "traditional" detail drawings.

16.5.1 Loading for Open-Web Steel Joists

Maximum factored moments and shears are established either from the loading conditions in the design documents or from the loading conditions listed in Clause 16.5.1.

These loading conditions are consistent with Section 4.1 and Table 4.1.3.2.A of the National Building Code of Canada (2015). In particular, as required by the National Building Code of Canada, roofs and the joists supporting them may be subject to uplift loads due to wind.

16.5.2 Design Assumptions

The loads may be replaced by statically equivalent loads applied at the panel points for the purpose of determining axial forces in all members. It is assumed that any moments induced in the joist chord by direct loading do not influence the magnitude of the axial forces in the members. Tests on trusses (Aziz 1972) have shown that the secondary moments induced at rigid joints due to joint rotations do not affect the ultimate axial forces determined by a pin-jointed truss analysis.

16.5.5 Bottom Chord

A minimum radius of gyration is specified for bottom chord members, when in tension, to provide a minimum stiffness for handling and erection.

Under certain loading conditions, net compression forces may occur in segments of bottom chords and must be considered. Bracing of the chord, for compression, may be provided by regular bridging only if the bridging meets requirements of Clause 9.2. As a minimum, lines of bracing are specifically required near the ends of bottom chords in tension in order to enhance stability when the wind causes a net uplift.

Bottom chord bracing may be required for continuous and cantilever joists as shown in Figure 2-41.

In those cases, where the bottom chord has little or no net compression, bracing is not required for cantilever joists. However, it is generally considered good practice to install a line of bridging at the first bottom chord panel point as shown in Figure 2-41.

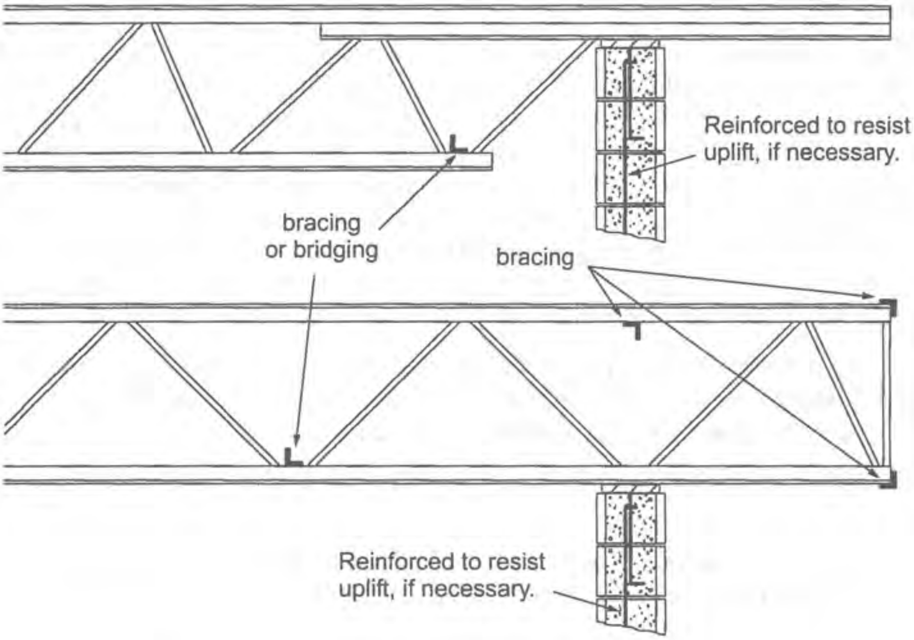


Figure 2-41
Bracing and Bridging of Cantilever Joists

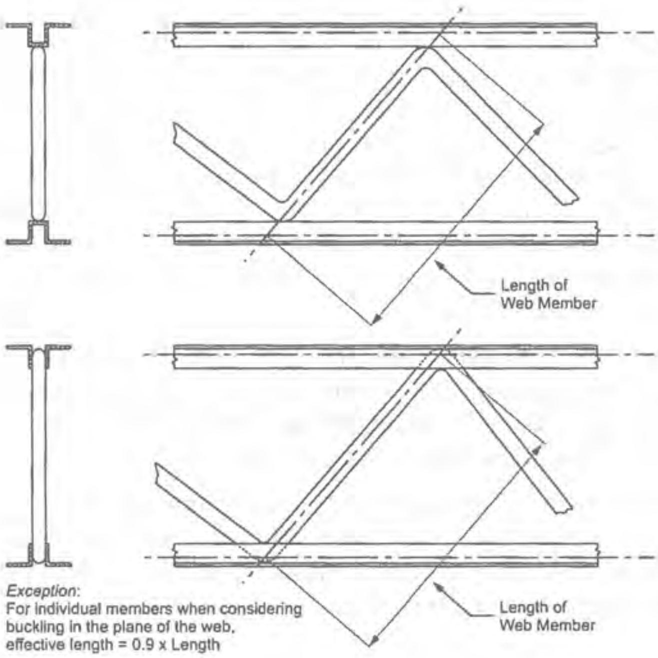


Figure 2-42
Length of Joist Web Members

16.5.6 Top Chord

When the conditions set out in Clause 16.5.6.1 are fulfilled, only axial force need be considered when the panel length is less than 610 mm (Kennedy and Rowan 1964). In these cases, the stiffness of the floor or roof structure tends to help transfer loads to the panel points of the joist, thus offsetting the reduction in chord capacity due to local bending. When the panel length exceeds 610 mm, axial force and bending moment need to be considered. When calculating bending moments in the end panel, it is customary to assume the end of the chord to be pinned, even though the joist bearing is welded to its support. The stiffening effect of supported deck or of the web is to be neglected when determining the appropriate width-thickness ratio (Clause 16.5.4.1) of the compression top chord.

The requirement in Clause 16.5.6.5, that the flat width of the chord component be at least 5 mm larger than the nominal dimension of the weld, should be considered an absolute minimum. Increasing the dimension may improve workmanship. See Clauses 16.8.5.1 and 16.8.5.2 regarding workmanship requirements when laying and attaching deck to joists.

16.5.6.6

S16-14 stipulates this minimum thickness of joist top chord when the deck is connected to it by mechanical fasteners. Joist top chords that are too thin do not work well with pins or screws.

16.5.7 Webs

The length of web members for purposes of design are shown in Figure 2-42. With the exception of web members made of individual members, the effective length factor is always taken as 1.0. For individual members this factor is 0.9 for buckling in the plane of the web (see Clause G7 of Annex G), but is 1.0 for buckling perpendicular to the plane of the web.

It has been observed, on occasion, in the testing of joists that with critical chords and webs designed to reach their factored loads more or less simultaneously using the S16 requirements, that the first compression web member fails first, even though the joist deformations may be quite significant. This appears to happen because the tension chord, after yielding in the panel where the joist bending moment is a maximum, continues to carry load into the strain-hardening range. It overloads itself and the joist. The first compression web member with no such reserve fails by buckling. By reducing the resistance factors for this member and its connections to 85%, more ductile modes of failure are encouraged at little extra cost. This requirement is also applied to trusses in Clause 15.2.5.

Vertical web members of modified Warren geometry are required to resist load applied at the panel point plus a bracing force to preclude in-plane buckling of the compression chord. A frequently used rule to provide full support (Winter 1960) is for a brace to have a capacity in the order of 2% of the force in the main compression member.

Web members in tension are not required to meet a limiting slenderness ratio. This is significant when flats are used as tension members. However, attention should be paid to those loading cases where the possibility of shear reversal along the length of the joist exists. Under these circumstances, it is likely that some diagonals generally near mid-span may have to resist compression forces.

16.5.8 Spacers and Battens

Spacers and battens must be an integral part of the joist, and the steel deck is not to be considered to act as spacers or battens (see Clause 16.5.6.2(c)).

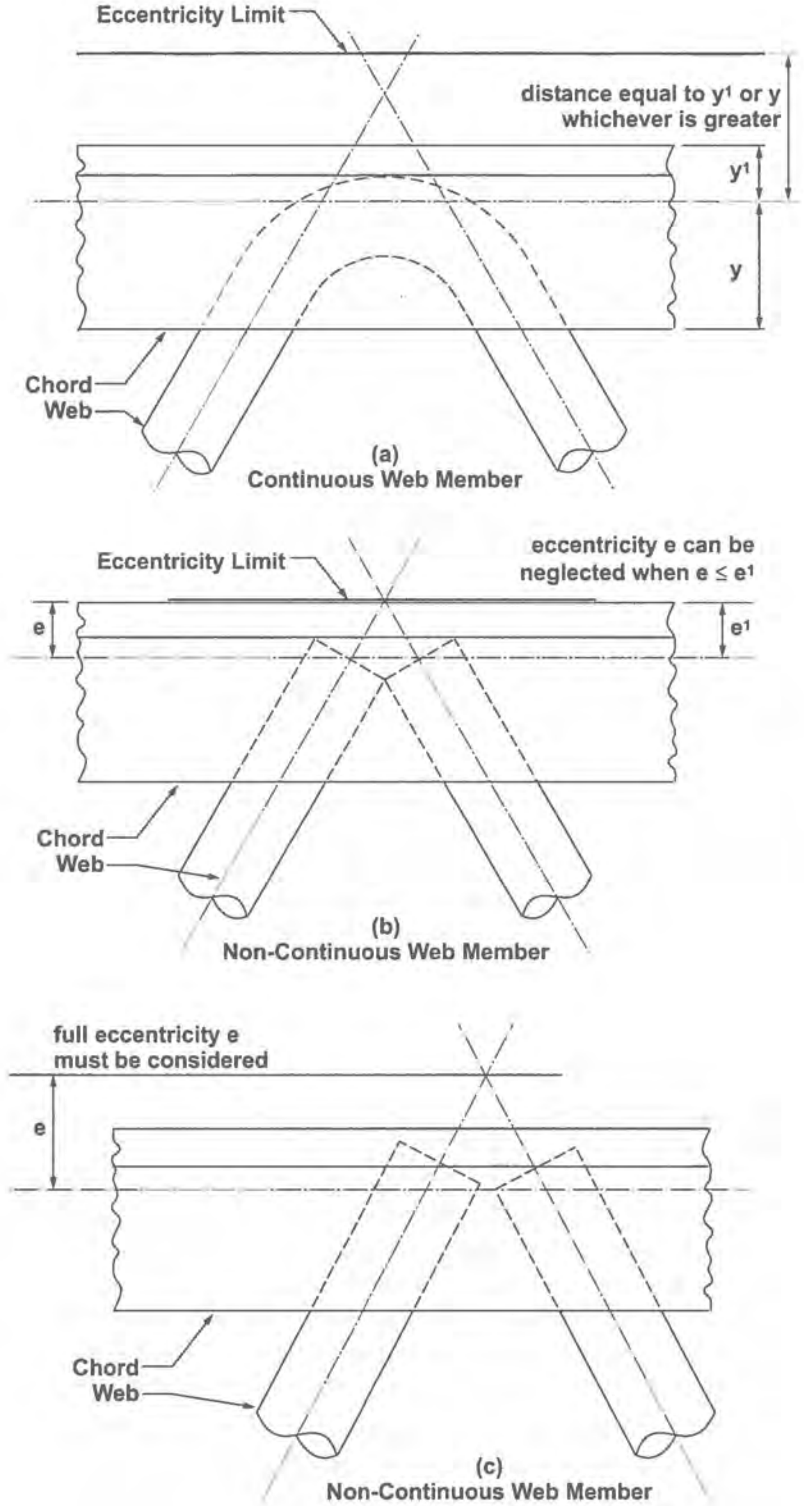


Figure 2-43
Eccentricity Limits at Panel Points of Joists

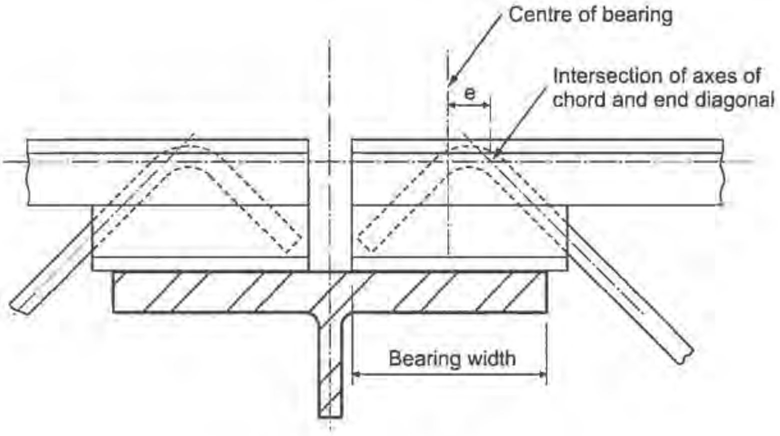


Figure 2-44
Joist End Bearing Eccentricity

16.5.9 Connections and Splices

Although splices are permitted at any point in chord or web members, the splices must be capable of carrying the factored loads without exceeding the factored resistances of the members. Butt-welded splices are permitted, provided they develop the factored tensile resistance of the member.

As a general rule, the gravity axes of members should meet at a common point within a joint. However, when this is not practical, eccentricities may be neglected if they do not exceed those described in Clause 16.5.9.4; see Figure 2-43. Kaliandasani *et al.* (1977) have shown that the effect of small eccentricities is of minor consequence, except for eccentricities at the end bearing and the intersection of the end diagonal and bottom chord. (See also Clause 16.5.10.4.)

16.5.10 Bearings

16.5.10.1 As required by Clause 16.4.1(c), the factored bearing resistance of the supporting material or the size of the bearing plates must be given on the building design drawings.

16.5.10.2 It is likely that the centre of bearing will be eccentric with respect to the intersection of the axes of the chord and the end diagonal, as shown in Figure 2-44. Because the location of the centre of bearing is dependent on the field support conditions and their construction tolerances, it may be wise to assume a maximum eccentricity when designing the bearing detail. In lieu of specific information, a reasonable assumption is to use a minimum eccentricity of one half the minimum bearing on a steel support of 65 mm. When detailing joists, care must be taken to provide clearance between the end diagonal and the supporting member or wall. See Figure 2-45. A maximum clearance of 25 mm is suggested to minimize eccentricities. One solution, to obtain proper bearing, is to increase the depth of the bearing shoe.

For spandrel beams and other beams on which joists frame from one side only, good practice suggests that the centre of the bearing shoe be located within the middle third of the flange of the supporting beam (Figure 2-46(a)). As the depth of bearing shoes vary, the building designer should check with the joist manufacturer in setting "top of steel" elevations. By using a deep shoe, interference between the support and the end diagonal will be avoided, as shown in Figure 2-46(b).

If the support is found to be improperly located, such that the span of the joist is increased, the resulting eccentricity may be greater than that assumed. Increasing the length of the bearing

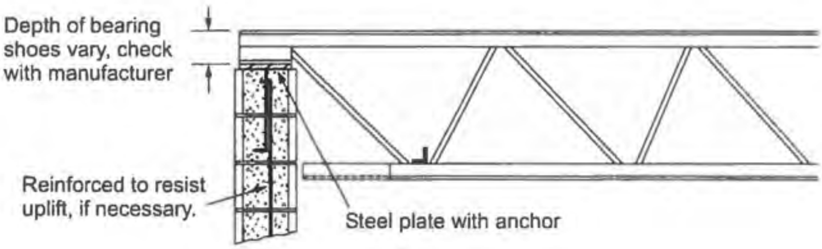
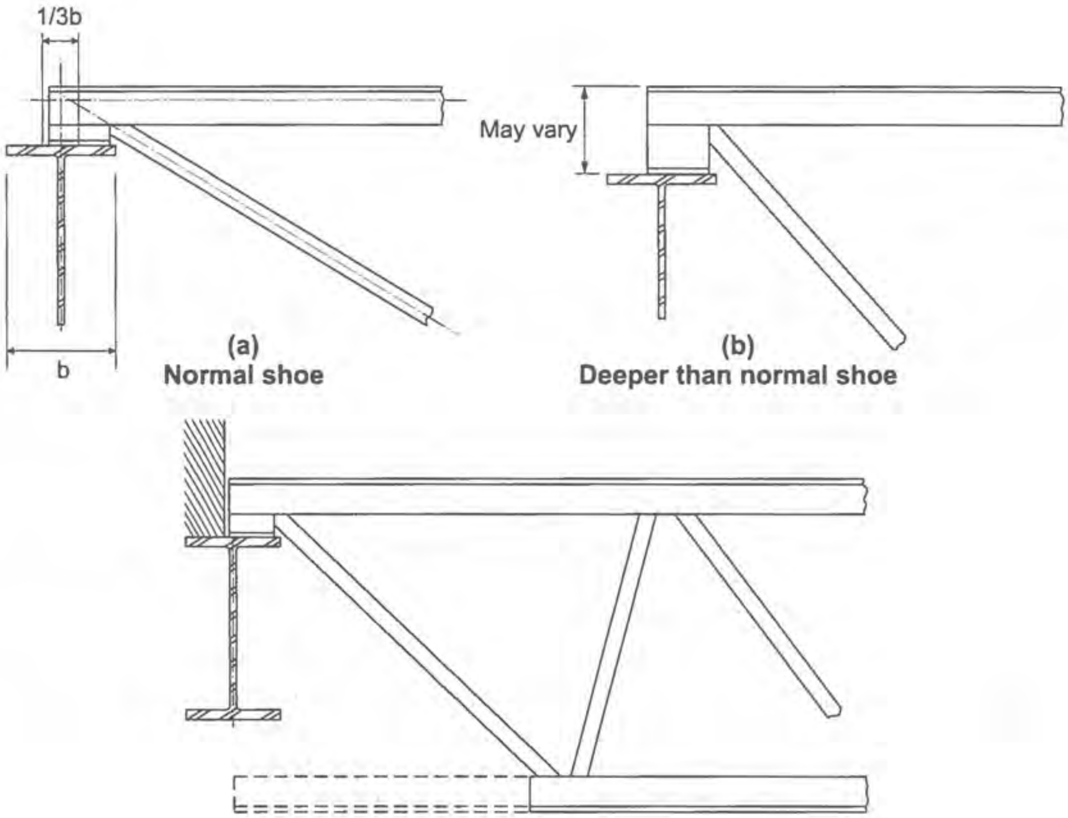


Figure 2-45
Joists Bearing on Steel Plate Anchored to Concrete and Masonry



(a)
Normal shoe

(b)
Deeper than normal shoe

(c)
See Clause 16.5.10.3 when bearing is less than 65 mm

Figure 2-46
Joists Bearing on Steel

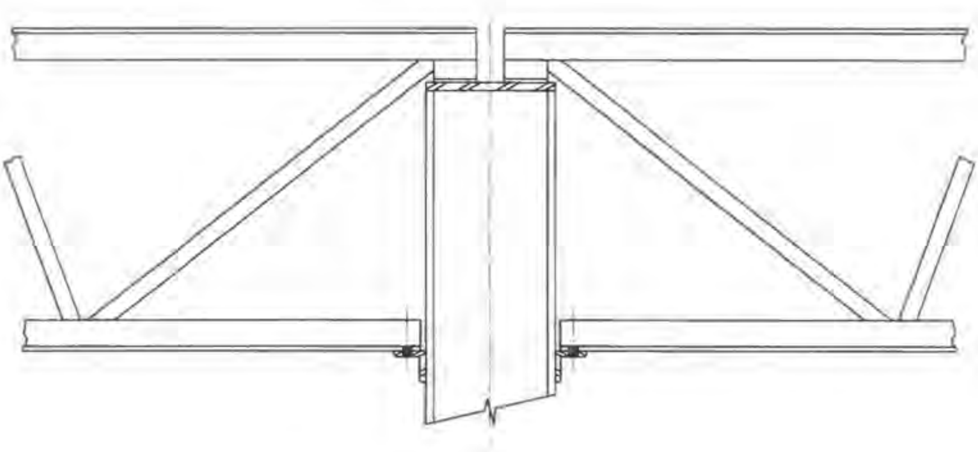


Figure 2-47
Tie Joists

shoe to obtain proper bearing may create the more serious problem of increasing the amount of eccentricity.

16.5.11 Anchorage

16.5.11.1 When a joist is subject to net uplift, not only must the anchorage be sufficient to transmit the net uplift to the supporting structure, but the supporting structure must be capable of resisting that force.

The anchorage of joist ends to supporting steel beams provide both lateral restraint and torsional restraint to the top flange of the supporting steel beam (Albert *et al.* 1992). When the supporting beam is simply supported, the restraint provided to the compression flange likely means that the full cross-sectional bending resistance can be realized.

In cantilever-suspended span construction, the restraint provided by the joists is applied to the tension flange in negative moment regions and is, therefore, less effective in restraining the bottom (compression) flange from buckling.

Albert *et al.* (1992) and Essa and Kennedy (1993) show that, while the increase in moment resistance due to lateral restraint is substantial, in cantilever-suspended span construction, the further increase when torsional restraint is considered is even greater. The torsional restraint develops when the compression flange tends to buckle sideways, distorting the web and twisting the top flange that is restrained by bending of the joists about the strong axis. The anchorage must therefore be capable of transmitting the moment that develops. For welds, a pair of 5 mm fillet welds 50 mm long coupled with the bearing of the joist seat would develop a factored moment resistance of about 1.8 kN·m

16.5.11.2 The function of tie joists is to assist in the erection and plumbing of the steel frame. Either the top or bottom chord is connected by bolting and, after plumbing the columns, the other chord is usually welded (Figure 2-47). In most buildings, tie joists remain as installed with both top and bottom chords connected; however, current practices vary throughout Canada with, in some cases, the bottom chord connections to the columns being made with slotted holes. Shrivastava *et al.* (1979) studied the behaviour of tie joist connections and concluded that they may be insufficient to carry lateral loads which could result from rigid bolting.

The designation tie joist is not intended to be used for joists participating in frame action.

Table 2-1
Camber for Joists

Camber (mm)			
Span	**Nominal Camber**	**Minimum Camber**	**Maximum Camber**
Up to 6 000	12 +	4	20
7 000	14	6	22
8 000	16	8	24
9 000	18	10	26
10 000	20	11	29
11 000	22	13	31
12 000	24	15	33
13 000	26	17	35
14 000	28	18	38
15 000	30	20	40
16 000	32	22	42

16.5.11.3 When joists are used as part of a frame to brace columns, or to resist lateral forces on the finished structure, the appropriate moments and forces are to be shown on the building design drawings to enable the joists and the joist-to-column connections to be designed by the joist manufacturer.

In cantilever-suspended span roof framing, joists may also be used to provide stability for girders passing over columns. See also the commentary on Clauses 16.5.11.1 and 13.6.

16.5.12 Deflection

The method of computing deflections is based on truss action, taking into account the axial deformation of all components rather than the former approximate method of using a moment of inertia equal to that of the truss chords and adding an allowance for the "shear" deformation of the web members.

16.5.13 Camber

The nominal camber based on Clause 16.5.13 is taken to vary linearly with the span and is tabulated in Table 2-1, rounded to the nearest millimetre. Manufacturing tolerances are covered in Clause 16.10.9. The maximum difference in camber of 20 mm for joists of the same span, set to limit the difference between two adjacent joists, is reached at a span of 16 000 mm.

16.5.14 Vibration

Annex E of S16-14, Guide for Floor Vibrations, contains recommendations for floors supported on steel joists. By increasing the floor thickness (mass), both the frequency and the peak acceleration are reduced, thus reducing the annoyance more efficiently than by increasing the moment of inertia (I_x) of the joists. For this reason, the building designer should weigh, at the building design stage, the options in the Guide for Floor Vibrations to achieve the best performance.

16.5.15 Welding

This clause makes reference to Clause 24, which requires that open-web steel joist fabricators be certified by the Canadian Welding Bureau to CSA W47.1 for arc-welded joists, to CSA W55.3 for resistance welded joists, or to both.

This clause further requires that fabricators have welding procedures specific to the fabrication of joists in place; this may include items such as weld sequence, length and profile unique to the joist fabrication. The development and qualification of welding procedures is a mandatory requirement of all fabricators who are certified to the requirements of CSA W47.1 or CSA W55.3.

16.6 Stability During Construction

A distinction is made between bridging, put in to meet the slenderness ratio requirements for top and bottom chords, and the temporary support required by Clause 16.6 to hold joists against movement during construction. Permanent bridging, of course, can be used for both purposes.

16.7 Bridging

Figures 2-48, 2-49 and 2-50 provide illustrations of bridging and details of bridging connections.

16.7.7 Anchorage of Bridging

Ends of bridging lines may be anchored to the adjacent steel frame, or adjacent concrete or masonry walls, as shown in Figure 2-51.

Where attachment to the adjacent steel frame or walls is not practicable, diagonal and horizontal bridging shall be provided in combination between adjacent joists near the ends of bridging lines as shown in Figure 2-52. Joists bearing on the bottom chord will require bridging at the ends of the top chord.

16.7.9 Spacing of Bridging

Either horizontal or diagonal bridging is acceptable, although horizontal bridging is generally recommended for shorter spans, up to about 15 m, and is usually attached by welding. Diagonal bridging is recommended for longer spans and is usually attached by bolting. Bridging need not be attached at panel points and may be fastened at any point along the length of the joists. When horizontal bridging is used, bridging lines will not necessarily appear in pairs as the requirements for support of tension chords are not the same as those for compression chords. Because the ends of joists are anchored, the supports may be assumed to be equivalent to bridging lines.

16.8 Decking

16.8.1 Decking to Provide Lateral Support

When the decking complies with Clause 16.8 and is sufficiently rigid to provide lateral support to the top (compression) chord, the top chord bridging may be removed when it is no longer required. Bottom (tension) chord bridging is permanently required to limit the unsupported length of the chord to $240r$, as defined in Clause 16.7.9.

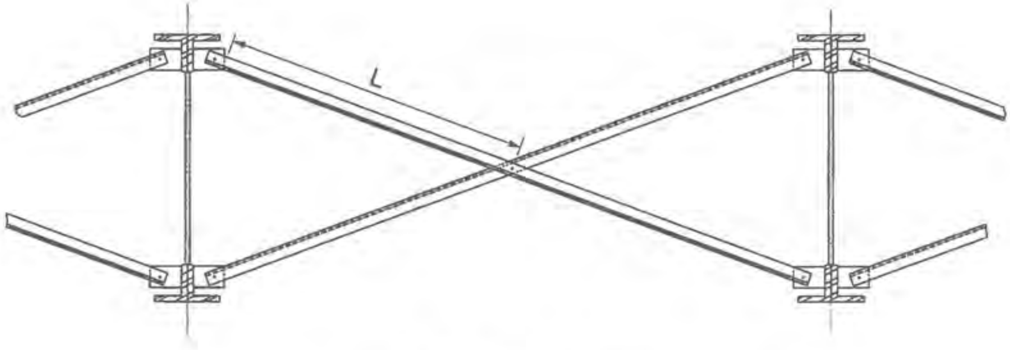


Figure 2-48
Diagonal Bridging of Joists

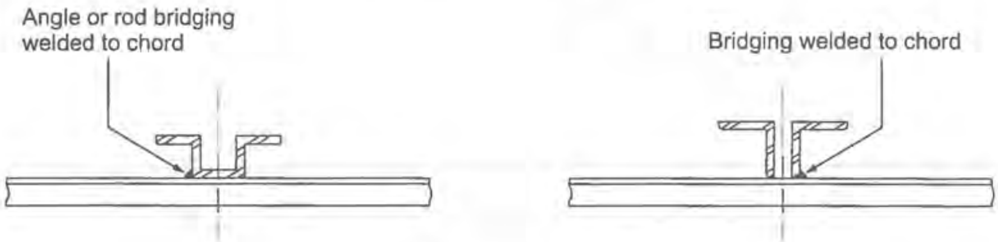


Figure 2-49
Horizontal Bridging Connections to the Joist's Top Chord

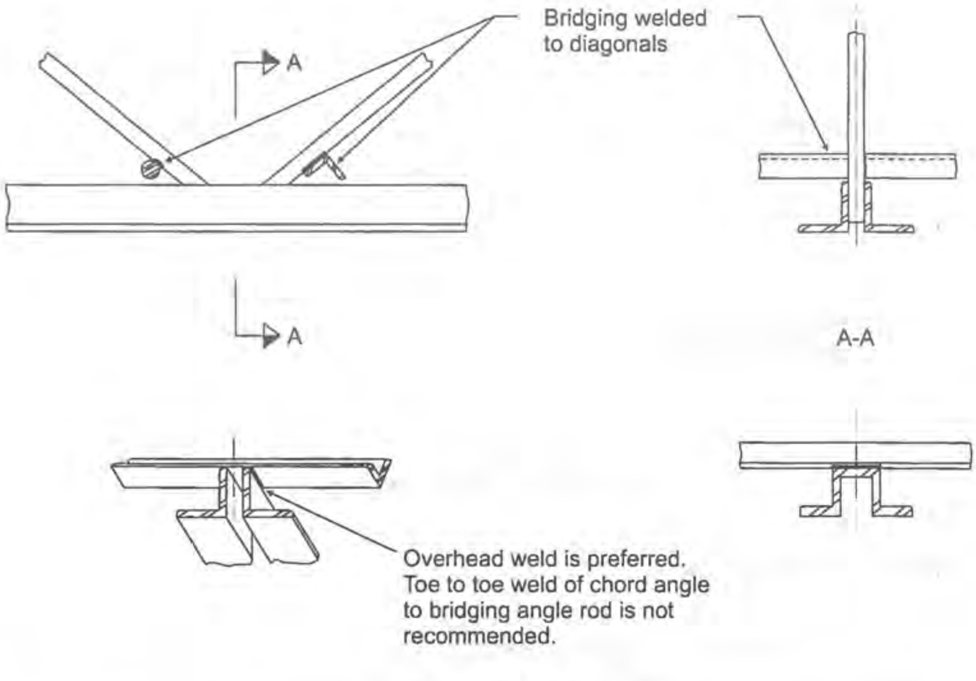


Figure 2-50
Horizontal Bridging Connections to the Joist's Bottom Chord

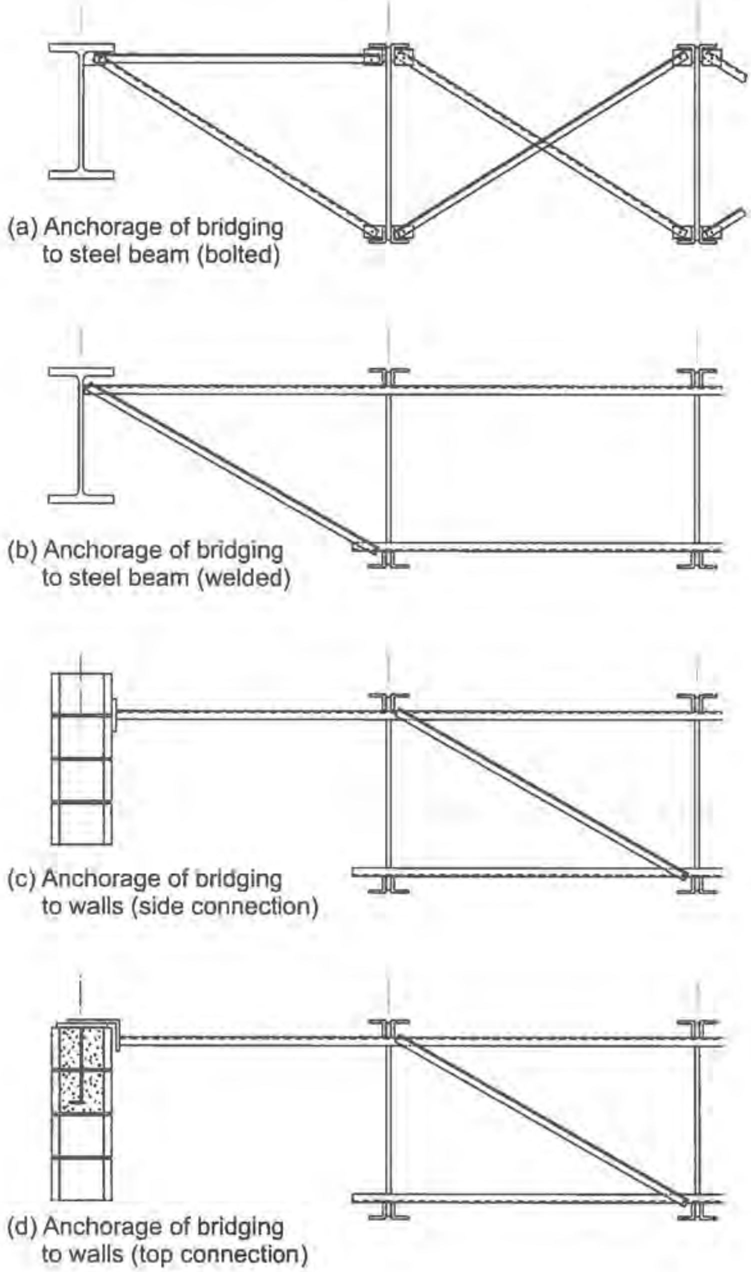
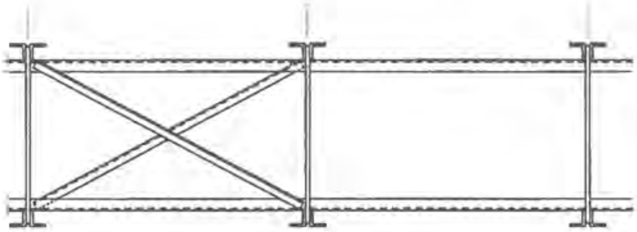


Figure 2-51
Anchorage of Joist Bridging

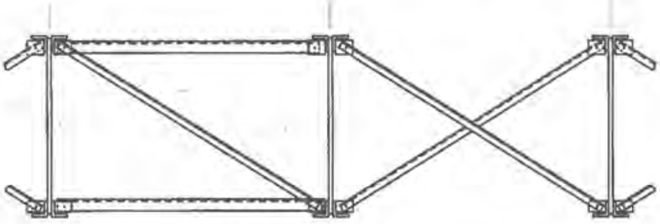
16.8.5 Installation of Steel Deck

16.8.5.1 Workmanship is of concern when decking is to be attached by arc-spot welding to top chords of joists. When the joist location is marked on the deck as the deck is positioned, the welders will be more likely to position the arc-spot welds correctly.

16.8.5.2 Arc-spot welds for attaching the deck to joists are structural welds and require proper welding procedures.



(a) diagonal bridging with horizontal bridging



(b) horizontal bridging with diagonal bridging

Figure 2-52
Bracing of Joist Bridging

16.9 Shop Coating

Interiors of buildings conditioned for human comfort are generally assumed to be of a non-corrosive environment and therefore do not require corrosion protection.

Joists normally receive one coat of paint suitable for a production line application, usually by dipping a bundle of joists into a tank. This paint is generally adequate for three months of exposure, which should be ample time to enclose or paint the joists.

Special coatings and paints that require special surface preparations are expensive, because these have to be applied individually to each joist by spraying or other means. For joists comprised of cold-formed members, surface preparations that were meant to remove mill scale from hot-rolled members are not appropriate.

16.10 Manufacturing Tolerances

Figure 2-53 illustrates many of the manufacturing tolerance requirements.

16.11 Inspection and Quality Control

16.11.3 Quality Control

When testing forms part of the manufacturer's normal quality control program, the test may follow steps 1 to 4 of the loading procedure given in Part 5 of Steel Joist Facts (CISC 1980).

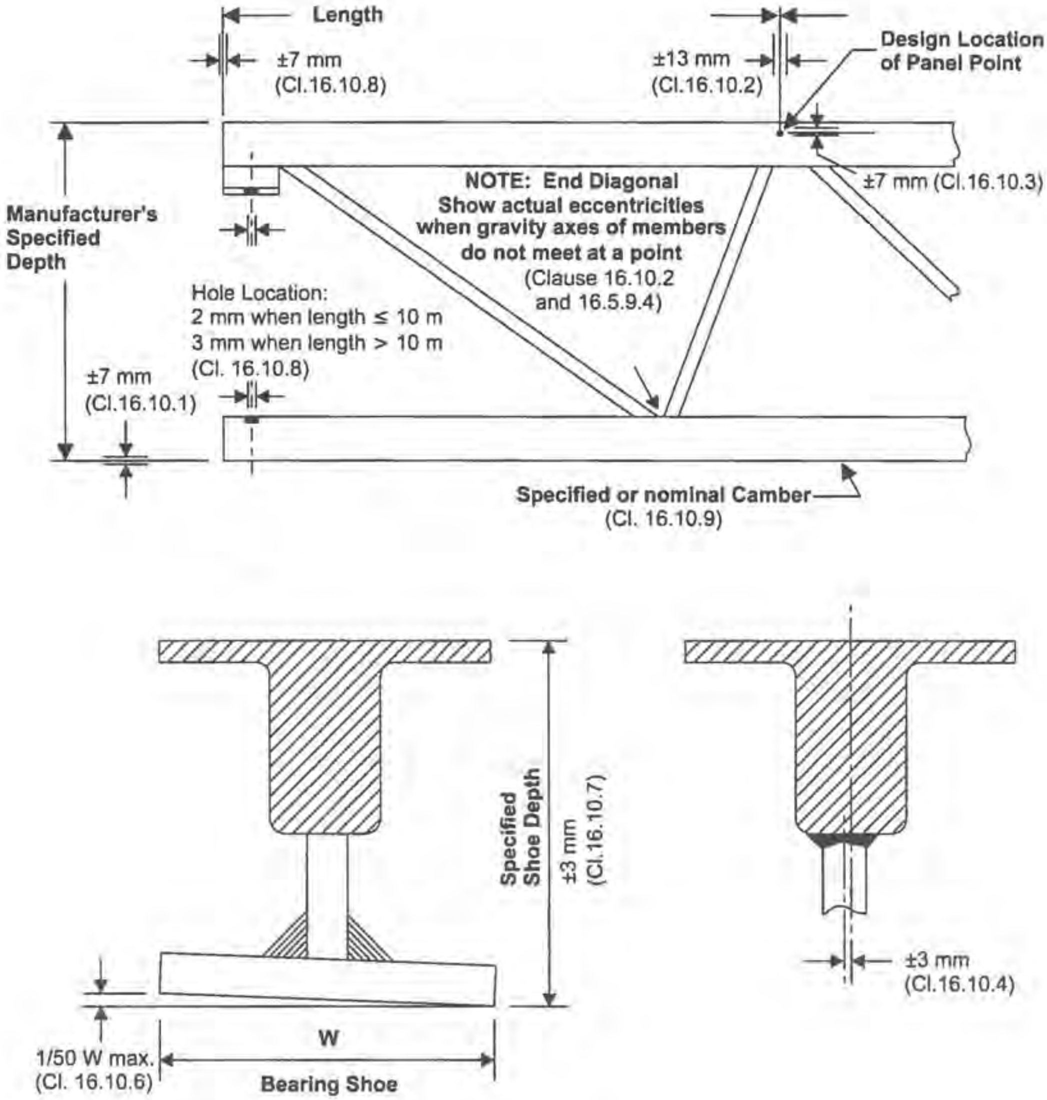


Figure 2-53
Joist Manufacturing Tolerances

16.12 Handling and Erection

16.12.2 Erection Tolerances

Figure 2-54 illustrates many of the erection tolerance requirements. The provisions of Clause 16.12.2.5 aim to control the differential deflection between any three adjacent joists to smooth the supported deck's profile.

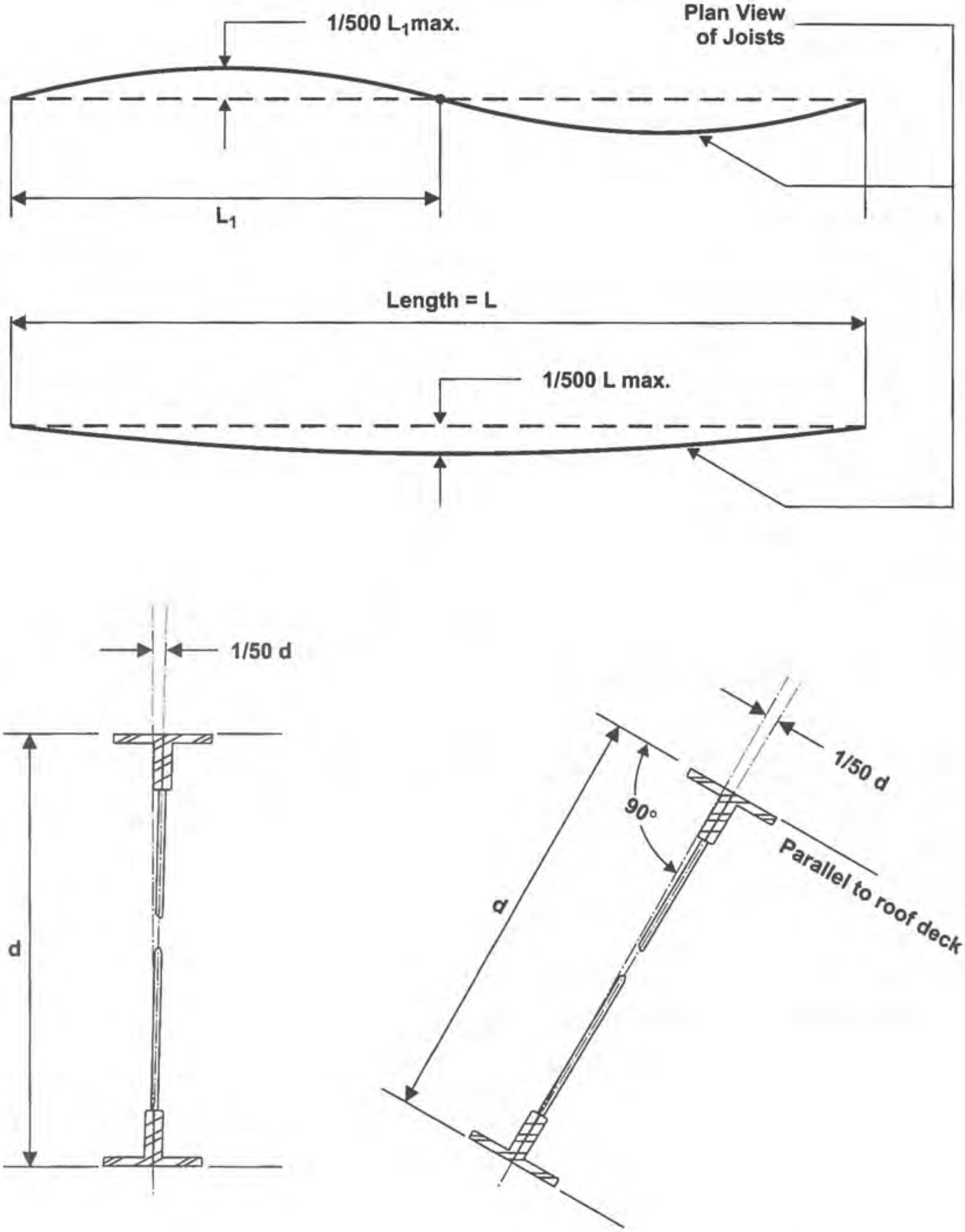


Figure 2-54
Joist Erection Tolerances

17. COMPOSITE BEAMS, TRUSSES AND JOISTS

17.2 Definitions

Definitions particular to this Clause are given here. Figure 2-55 illustrates various cases of effective slab and cover slab thickness.

17.3 General

17.3.1 Deflections

The moment of inertia is reduced from the transformed value to account for the increased flexibility resulting from partial shear connection, p, and for interfacial slip, similar to that coefficient proposed by Grant *et al.* (1977). The factor 0.85 accounts for the loss in stiffness due to interfacial slip, even with full shear connection. To include the effect of shear deformation of the web systems of joists and trusses, the moment of inertia I_s is reduced by 15% unless a detailed analysis is used.

The increase of the elastic deflection of 15% for creep is an arbitrary but reasonable value.

Annex H of the Standard gives a detailed discussion of shrinkage deflections. There it is emphasized that appropriate values of the shrinkage strain and age-adjusted effective modulus of concrete, which in turn depends on the aging and creep coefficients, should be used in calculating these deflections. Values of shrinkage strain, and aging and creep coefficients, for conditions not anticipated in the annex may be obtained from Ghali *et al* (2002). CSA S6 Canadian Highway Bridge Design Code (2014) also contains detailed procedures for evaluating the shrinkage and creep of concrete.

Reference should be made to Ghali *et al* (2002) for a more complete discussion of the procedure proposed in the standard for evaluating shrinkage deflections. An alternative method is presented by Kennedy and Brattland (1992), with further information provided by Maurer and Kennedy (1994) on interfacial slip and, for composite joists or trusses, increases in the flexibility of the system.

17.3.2 Vertical Shear

Clauses 17.3.2 and 17.3.3 follow from the assumption that the concrete slab does not carry any vertical shear.

17.4 Design Effective Width of Concrete

Although the effective width rules were formulated on the basis of elastic conditions (Robinson and Wallace 1973, Adekola 1968), the differences at ultimate load do not significantly affect the moment resistance of the composite beam (Elkelish and Robinson 1986, Hagood *et al.* 1968, Johnson 1975, Heins and Fan 1976).

17.5 Slab Reinforcement

17.5.2 The effectiveness of the minimum requirement of two 15M bars at the ends of beams supporting ribbed slabs perpendicular to the beam proposed by Ritchie and Chien (1980) has been verified experimentally by Jent (1989).

17.5.3 The longitudinal shear forces generated by interconnecting concrete slabs to steel sections, trusses, or joists by means of shear connectors may cause longitudinal cracking of the slab directly over the steel. This effect is independent of any flexural cracking that may occur due to the slab spanning continuously over supports, although the two effects may combine. Longitudinal shear cracking is more apt to start from the underside of the solid slab, whereas flexural cracking is more apt to start at the top surface of the slab. Investigations by Johnson

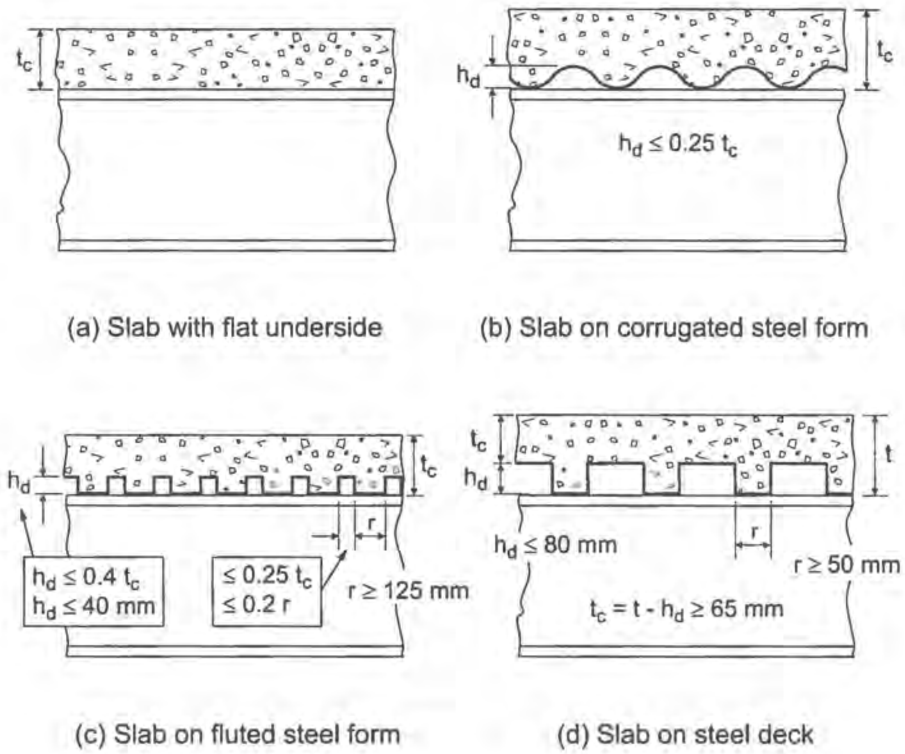


Figure 2-55
Effective Slab Thickness for Composite Beams

(1970), El-Ghazzi *et al.* (1976), and Davies (1969) have shown that a minimum area of transverse reinforcing steel is required to improve the longitudinal shear capacity of a composite beam slab. The minimum reinforcement ratio is the same as that specified in CSA Standard A23.3 (CSA 2014) for temperature and shrinkage reinforcement in reinforced concrete slabs.

17.5.4 For the reasons given in Clause 17.5.3, a minimum transverse reinforcement ratio of 0.002 is also specified for composite beams with ribbed slab when the ribs are parallel to the beam span. This ratio is reduced to 0.001 when the ribs are perpendicular to the beam span, because the steel deck provides a measure of transverse reinforcement. Reinforcement of the cover slab may also be necessary for flexure, fire resistance, shrinkage, or temperature effects.

17.6 Interconnection

When unpainted sections, trusses, or joists are totally encased in concrete as specified, effective interconnection is obtained, and no shear connectors are required.

The total sheet thickness and the total amount of zinc coating are limited in order to achieve sound welds.

Tests have shown that a shear connector is not fully effective if welded to a support which is too thin or flexible (Gobel 1968). For this reason, the stud diameter is limited to 2.5 times the thickness of the part to which it is welded.

17.7 Shear Connectors

The factored resistance of end-welded studs in a solid slab is different from that in a ribbed slab, which depends upon the deck ribs' orientation and size.

For end-welded studs in a solid slab, the values given in Clause 17.7.2.2 are based on work by Olgaard *et al.* (1971) in both normal and light-density solid concrete slabs. The limiting value of $\phi_{sc}A_{sc}F_u$ represents the tensile strength of the stud, as the stud eventually bends over and finally fails in tension.

In previous editions of the Standard, Clause 17.7.2.3(a) gave the same factored shear resistance for studs in ribbed slabs, with ribs parallel to the beam, as in solid slabs, provided that the rib flute is wide enough (Johnson 1975). Hosain and Wu (2002) have shown that this may not always be the case. In S16-09, equation (a) was revised to account for the lower capacity observed through push-out and full-size beam tests. When the flutes are narrow, however, the factored shear resistance of the stud is reduced. The equation in Clause 17.7.2.3(b) of S16-09 gives a more consistent prediction of push-out test results (Hosain and Pashan 2002). A limit is placed on equation (b), such that the shear resistance for $w_d / h_d < 1.5$ does not exceed that obtained with the revised equation (a) at $w_d / h_d = 1.5$.

The provisions for ribbed slabs with ribs perpendicular to the beam are based on work by Jayas and Hosain (1988 and 1989). Push-out tests, as well as full-size beam tests, indicated that failure in this type of composite beam would likely occur due to concrete pull-out. The equations of Clause 17.7.2.4, similar to those suggested by Hawkins and Mitchell (1984), provide better correlation with test results than those using the reduction factor method adopted by AISC (2010b). Figure 2-56 gives diagrams of the pullout surface area. Pullout areas for specific deck profiles and studs are given in Part 5 of the Handbook.

In order to minimize localized stresses in concrete, the lateral spacing centre-to-centre of studs used in pairs should be not less than four stud diameters. The minimum longitudinal spacing of connectors, in both solid slabs and ribbed slabs with ribs parallel to the beam, is based on Olgaard *et al.* (1971). The maximum spacing limits specified for mechanical ties in Clause 17.8 is applicable to headed studs, as they function in this capacity.

Further information on end-welded studs is found in Johnson (1970), Chien and Ritchie (1984), and Robinson (1988).

17.7.3 The shear value of channel connectors is based on Slutter and Driscoll (1965).

17.9 Design of Composite Beams with Shear Connectors

In order to minimize eccentricities before and after composite action, in composite joists and trusses, the web members should be positioned such that the lines of action intersect at a point halfway between the mid-depth of the cover slab and the centroid of the steel top chord.

17.9.1 A minimum flat width for the top chord of $1.4d + 20$ mm is stipulated to facilitate placement of the shear studs.

17.9.3 The factored moment resistance of a composite flexural member is based on the ultimate capacity of the cross-section (Robinson 1969, Vincent 1969, Hansell and Viest 1971, Robinson and Wallace 1973, Tall *et al.* 1974) where the following assumptions are made:

- Concrete in tension is neglected;
- Only the lower chord of a steel joist or truss is considered effective when computing the moment resistance;
- The internal couple consists of equal tension and compression forces;

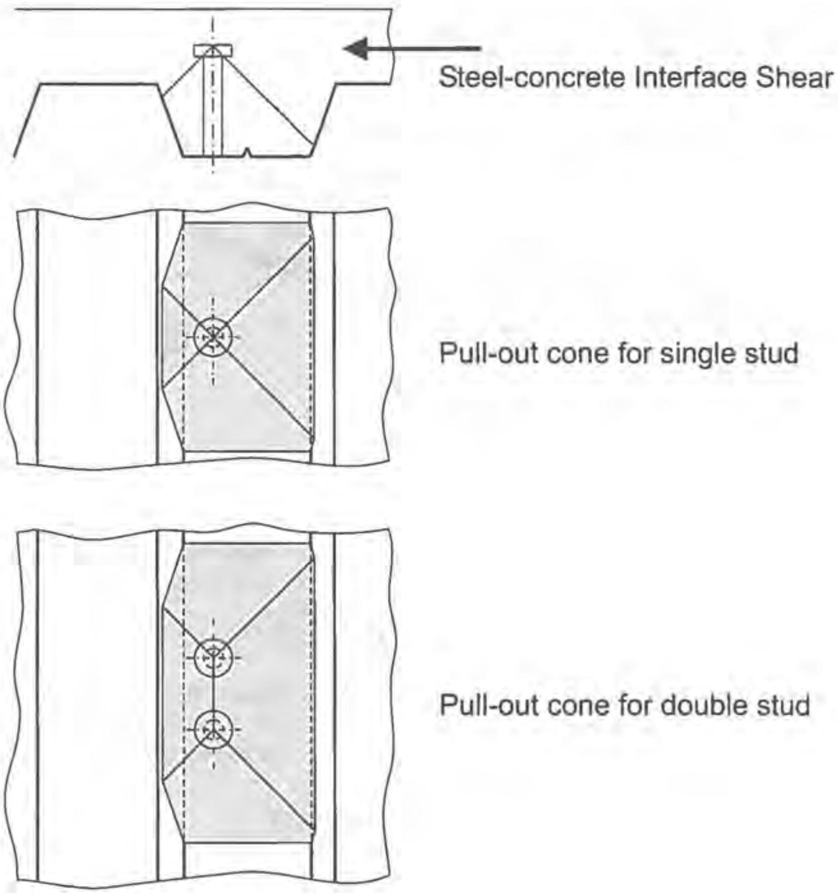


Figure 2-56
Pullout Surface Area with Ribbed Metal Deck

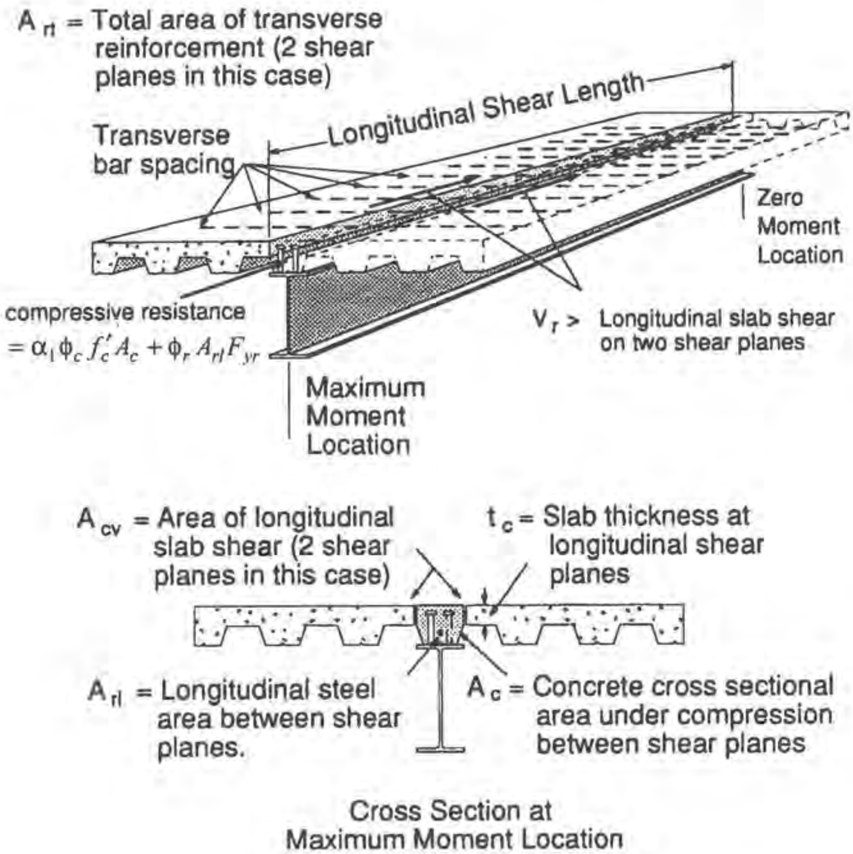
- The forces are obtained as the product of a limit states stress (ϕF_y for steel and $\alpha_1 \phi_c f'_c$ for concrete) times the respective effective areas; and,
- To take into account the greater variability of concrete elements strengths, the resistance factor is taken as 0.65 for concrete as compared to 0.90 for steel.

Three design cases are considered:

- Case 1 representing full shear connection with the plastic neutral axis in the slab;
- Case 2 representing full shear connection with the plastic neutral axis in the steel section; and,
- Case 3 representing partial shear connection for which the plastic neutral axis is always in the steel section.

Only Case 1 is permitted when joists or trusses are used to prevent buckling of the top chord and overloading of the shear connectors. For Case 3, the depth of the concrete in compression is determined by the expression for "a" (Robinson 1969).

Since the release of S16-09, these assumptions have been modified somewhat such that the resistance factor for concrete and the ratio of average stress in the rectangular compression block to the specified concrete strength are consistent with the CSA Standard A23.3-04 (and



$$V_u = \sum q_r - \alpha_1 \phi_c f'_c A_c - \phi_r A_{rl} F_{yr}$$

$$V_r = (0.80\phi_r A_{rt} F_{yr} + 2.76\phi_c A_{cv}) \leq 0.50\phi_c f'_c A_{cv}$$

Figure 2-57
Potential Longitudinal Shear Planes

A23.3-14). Accordingly, $\phi_c = 0.65$ is used in place of 0.60, and $\alpha_1 \phi_c f'_c$ is given as the concrete strength in place of $0.85\,\phi_c f'_c$. Reasoning for these changes to the concrete resistance factor and strength is described by Bartlett (2007).

17.9.4 Robinson (1988) and Jayas and Hosain (1989) show that a lower limit of 40% of full shear connection is acceptable for strength calculations. Below this value, the interfacial slip is such that integral composite action cannot be assured. A lower limit of 25% of full shear connection is used for deflection designs, as deflections are computed at specified load levels. This latter provision is used where the flexural strength is based on the bare steel beam, but the increased stiffness due to the concrete is considered for deflection calculations.

17.9.5 Between the point of zero and maximum moment, a horizontal force associated with the internal resisting couple must be transmitted across the steel-concrete interface.

17.9.8 Uniform spacing of shear connectors is generally satisfactory because the flexibility of the connectors provides a redistribution of the interface shear among them. However, to ensure that sufficient moment capacity is achieved at points of concentrated load, the second provision of this clause is invoked. As the moment capacity of the steel section does not depend on shear connectors, this capacity is subtracted from both M_f and M_{f1}.

17.9.9 To justify composite action in the end panel of joists and trusses, sufficient shear studs must be provided above the seat or along a top chord extension, to transfer the horizontal shear from the slab to the steel section, otherwise the steel top chord acting alone must resist all the forces.

17.9.10 Longitudinal Shear

In order to develop the compressive force in the portion of the concrete slab outside the potential shear plane shown in Figure 2-57, net shear forces, totalling V_u, must be developed on these planes. The expressions for shear resistance are based on Mattock (1974). Values for semi-low-density and low-density concrete are given by Mattock *et al.* (1976) and Chien and Ritchie (1984).

17.10 Design of Composite Beams Without Shear Connectors

This conservative approach assumes that the composite section is about 10% stronger than the bare steel member, although the moment resistance computed according to Clause 17.10.2 typically gives a larger value.

17.11 Unshored Beams

This provision guards against permanent deformations under specified loads by limiting the total stress in the bottom fibre of the steel section. This limit has been shown (Kemp and Trinchero 1992) to be conservative. The ultimate strength of the composite beam, which exhibits its ductile behaviour, is not affected by the stress state at the specified load level.

18. COMPOSITE COLUMNS

This clause includes, in addition to the concrete-filled hollow structural sections, partially encased composite columns acting in compression in Clause 18.3, and rolled steel shapes encased in concrete in Clause 18.4. The latter parallels the requirements of CSA Standard A23.3, but using a column curve consistent with those used throughout this Standard. Thus the designer has in this Standard three types of steel-concrete columns from which to choose.

The design rules apply to specific research which should be consulted in conjunction with the requirements of this Clause.

18.1 Resistance Prior to Composite Action

For some of the systems described here, the designer should be aware that the steel component may be designed to carry some of the loads before the concrete has gained strength.

18.2 Concrete-Filled Hollow Structural Sections

18.2.1 General

18.2.1.2 Axial Load on Concrete

Kennedy and MacGregor (1984) showed that direct bearing of the load on the concrete was not necessary for either axially loaded columns or beam-columns. When loads are applied to the steel shell, pinching between the steel and concrete quickly transfers loads to the concrete core. The Standard conservatively retains the requirement of direct bearing for the uppermost level but not for intermediate levels of multi-storey columns.

18.2.1.3 Composite Action in Bending

CIDECT (1970), Knowles and Park (1970), Wakabayashi (1977), Stelco (1981), Budijgnto (1983) and Bergmann *et al.* (1995) have demonstrated that the compression resistance of composite columns, consisting of hollow structural sections (HSS) completely filled with concrete, arises from both the steel and the concrete core. Obviously the full composite bending resistance at the ends of such members can only be realized when the connections are able to transfer the loads to the composite beam-column.

18.2.2 Compressive Resistance

The expressions for compressive resistance introduced in S16-01 give a better fit to test results than those found in the preceding standard. The contributions of the concrete core and the hollow steel section are simply superposed. Both the steel and concrete contributions to the compressive resistances are decreased as a function of the slenderness parameter, λ, of the composite section that is considered in turn to depend on the elastic flexural stiffness of the steel section and a flexural stiffness of the concrete that is modified to account for creep under sustained loads. The same double exponential form of column curve, as used for other compressive resistances in the standard, is used for consistency. The value of the exponent "n" in the expression is taken as 1.80 to get the best fit with experimental results.

The triaxial load effect on the concrete due to the confining effect of the walls of circular HSS is based on work by Virdi and Dowling (1976). The triaxial effects increase the failure load of the concrete ($\tau' > 1.0$) and decrease the capacity of the steel section ($\tau < 1.0$), because the steel is in a biaxial stress state.

In S16-09, the standard introduced a value of $\phi_c = 0.65$ in place of the earlier 0.60, and $\alpha_1 \phi_c f'_c$ in place of $0.85\,\phi_c f'_c$. Reasoning for these changes to the concrete resistance factor and strength is described by Bartlett (2007). In S16-14 the lower limit for α_1 is changed to 0.73, which is the lowest value possible, considering that the scope only covers concrete strengths up to 80 MPa for axially loaded columns.

18.2.3 Bending Resistance

Lu and Kennedy (1994) show, for rectangular hollow sections with measured flange b/t ratios up to $700/\sqrt{F_y}$, that fully plastic stress blocks are developed in the steel and in the concrete. Their proposed model, based on such stress blocks with the steel stress level taken equal to the yield value, F_y, and the concrete stress level taken equal to the concrete strength, f'_c, at the time of testing, agreed excellently with test results. The two components support each other. The steel restrains or confines the concrete, increasing its compressive resistance to the full value rather than 0.85 of it, as used in reinforced concrete theory, while the concrete prevents inward buckling of the steel wall, thus increasing the steel strain at which local buckling occurs. Therefore, sections not even meeting the requirements of Class 3 sections in bending develop fully plastic stress blocks.

Geometric expressions are given to determine the factored compressive forces in the steel and concrete with rectangular stress blocks when in equilibrium, for both rectangular and circular hollow structural sections.

In S16-09, the standard introduced a value of $\phi_c = 0.65$ in place of the earlier 0.60, and $\alpha_1 \phi_c f'_c$ in place of $0.85\,\phi_c f'_c$. Reasoning for these changes to the concrete resistance factor and strength is described by Bartlett (2007).

Figure 2-58
Partially Encased Composite Columns

18.2.4 Axial Compression and Bending

This clause is analogous to the expression in Clause 13.8.3 for I-shaped beam-columns. Extending the analogy, the cross-sectional resistance and in-plane strength should be checked and, if applicable, the lateral-torsional buckling strength should be checked for rectangular sections bent about their strong axis. Because of the very large torsional resistance of closed shapes, the latter is very unlikely to be a factor. With expressions introduced in S16-01 for circular hollow sections filled with concrete, the lower bound solution for such sections given in former Standards is no longer required.

In S16-09, the standard introduced a value of $\phi_c = 0.65$ in place of the earlier 0.60, and $\alpha_1 \phi_c f'_c$ in place of $0.85\,\phi_c f'_c$. Reasoning for these changes to the concrete resistance factor and strength is described by Bartlett (2007).

18.3 Partially Encased Composite Columns

As stated in the note to this Clause, these columns are a patented structural component. By CSA regulations, in the interests of promoting new technology, they are referenced in this Standard on the understanding with the patent holder that any patent rights will be made available either as a free license or on reasonable terms and conditions.

The basic concept is to provide a steel H-shape of relatively thin plates but with sufficient strength to carry gravity loads during construction until the concrete cast around the shape reaches sufficient strength to carry the remaining dead loads and all live and environmental loads, while working compositely with the steel section. It is envisaged that the columns could be used in multi-storey applications, with the concrete of the about-to-be encased steel shapes cast with the next higher floor that the columns support. Figure 2-58 shows an elevation of the column. The steel links between the column flanges restrain the flanges from buckling locally and at the same time provide limited confinement to the concrete.

18.3.1 General

The scope Clause lays out in detail the limits on geometry and strength of the component elements and materials – the steel section, the steel reinforcement and the concrete – that must be satisfied. These derive from the limits of the extensive series of tests, including full-scale tests, which were carried out at Lehigh University, University of Toronto, McGill University and École Polytechnique (Tremblay *et al.* 2000), and the University of Alberta to confirm and quantify the performance of the columns in all respects. While extensive, the limits are sufficiently broad in scope to design columns of different cross-sections and slenderness limits to carry a wide range of loadings. Based on experimental and numerical research by Prickett and Driver (2006) and Begum *et al.*(2013) on partially encased composite columns with high-strength concrete, the upper limit on concrete strength has been increased from 40 MPa to 70 MPa. The method for determining the bending resistance is provided in Clause 18.3.3, and the interaction expression for combined axial compression and bending in Clause 18.3.4.

18.3.2 Compressive Resistance

The expression for the compressive resistance (Tremblay *et al.* 2000) is of the same double exponential format used for both steel and other composite columns throughout this Standard. The exponent "n", 1.34, is the least value stipulated in the Standard. For both the steel section and the steel reinforcement, specified minimum yield strengths are used as the reference strengths and, for the concrete, 0.95 α_1 of the specified 28-day strength is used, as this value gave a better fit to the test data than the 0.85 factor commonly used. The resistance factors for the three components are consistent with the remainder of the Standard. In S16-09, the standard introduced a value of $\phi_c = 0.65$ in place of 0.60 (Bartlett 2007).

18.3.5 Special Reinforcement for Seismic Zones

Details are provided for longitudinal and transverse bars to be used where the specified one-second spectral acceleration ratio, $I_E F_v S_a(1.0)$, is greater than 0.30, in order to provide satisfactory performance compatible with that of reinforced concrete buildings designed for such seismic categories.

18.4 Encased Composite Columns

18.4.1 General

This Clause is provided because such columns may be found in a steel building structure. This Clause provides the designer with all the information needed to design this composite component as well as all other components in the building. The scope limits the doubly symmetric steel columns encased in concrete to which this clause applies to those given in CSA Standard A23.3.

18.4.2 Compressive Resistance

The factored compressive resistance is of the exact same form as that given in Clause 18.3 for concrete-filled hollow structural sections. In this regard, it differs in form from the resistance given in CSA Standard A23.3 but matches the factored compressive resistance of the latter closely for all slenderness ratios. The contributions of the concrete, structural steel shape and reinforcing steel to the strength are simply superposed. Both the steel and concrete contributions to the compressive resistances are decreased as a function of the slenderness parameter, λ, of the composite section that is considered in turn to depend on the elastic flexural stiffness of the steel section and a flexural stiffness of the concrete that is modified to account for creep under sustained loads. The same double exponential form of column curve, as used for other compressive resistances in the Standard, is used for consistency. In S16-09, the standard introduced a value of $\phi_c = 0.65$ in place of the former 0.60, and $\alpha_1 \phi_c f'_c$ in place of $0.85 \phi_c f'_c$. Reasoning for these changes to the concrete resistance factor and strength is described by Bartlett (2007).

18.4.4, 18.4.5 and 18.4.6 In the unusual case with multiple steel shapes enclosed in the concrete, the steel shapes are to meet the requirements of Clause 19 for built-up shapes until the concrete reaches $0.75 f'_c$. Alternatively, the load on the steel shapes could be limited to the sum of their independent resistances, having due regard as to how the loads are applied.

18.4.5 This clause emphasizes that there must be direct transfer of any load considered to be carried by the concrete.

18.4.6 To determine the bending resistance of encased composite columns, the designer is referred to Ziemian (2010). In S16-09, the standard introduced a value of $\phi_c = 0.65$ in place of the former 0.60, and $\alpha_1 \phi_c f'_c$ in place of $0.85 \phi_c f'_c$. Reasoning for these changes to the concrete resistance factor and strength is described by Bartlett (2007).

19. BUILT-UP MEMBERS

The term built-up member refers to any structural member assembled from two or more components. Such members may be used to resist compression, tension or bending, and the requirements for fastening together the various components vary accordingly.

The diagrams of Figures 2-59 and 2-60 illustrate the main provisions of Clause 19.

Many of the provisions are based on long-established practice and have proven satisfactory. In Clause 19.2.3, it is emphasized that the buckling could occur for outside components. In Clause 19.2.10, because it has been established that the tension diagonal of a crossed tension-compression pair supports the latter (see Commentary to Clause 27.5.3.1), the effective buckling length of the compression lacing can be taken as 0.50 of its total length.

Tension members are stitched together sufficiently to work in unison and to minimize vibration. For exposed members, components in contact should be fitted tightly together to minimize corrosion problems (Brockenbrough 1983).

When a built-up column buckles, shear is introduced in lacing bars (Clause 19.2.9) and battens and their connections (Clause 19.2.17), in addition to any transverse shears (Bleich, 1952).

Further discussion on columns with lacing and battens is given in Ziemian (2010).

For compression members composed of two or more rolled shapes connected at intervals, Clause 19.2.4 requires the use of an equivalent slenderness ratio, increased to take into account the flexibility of the interconnector. This increase is applied to the axis of buckling where the buckling mode of the member involves relative deformation that produces shear forces (see Clause 19.2.6) in the interconnectors between the individual shapes (Duan and Chen, 1988).

The requirements for starred angles are based on work by Temple *et al.* (1986), who showed that with fewer interconnectors the buckling strength was reduced.

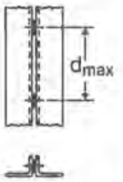
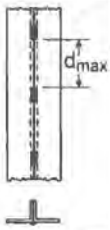
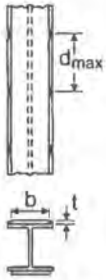
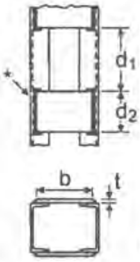
Tension Members	Requirements	Tension Members	Requirements
	TWO ROLLED SHAPES NOT IN CONTACT d_{max} = 300 × Least radius of gyration of one component		TWO ROLLED SHAPES IN CONTACT d_{max} = 600 mm d_{max} may be increased when justified
	SHAPE AND PLATE IN CONTACT d_{max} = 36t or 450 mm whichever is lesser		BATTENS $b \leq 60t \quad d_2 \geq \frac{2b}{3}$ d_{max} = 300 × Least radius of gyration of one component * For intermittent welds or fasteners, max. longitudinal pitch = 150 mm

Figure 2-59
Built-up Tension Member Details

20. PLATE WALLS

20.1 General

Early research at the University of Alberta (Kulak 1991, Driver *et al.* 1997, 1998(a), and 1998(b)) demonstrated that the plate wall system is an attractive alternative for resisting lateral wind and seismic loads. The system has the advantage that it is stiff enough to minimize displacements under extreme loading conditions and has a high degree of redundancy. The system can be used for both new construction and the upgrading of existing structures.

Figure 2-61 shows a typical plate wall. The walls considered by Clause 20 imply thin, unstiffened infill plates. Under lateral loads, it is assumed that the buckling strength of the infill plate is negligible, but tension field action develops to resist lateral shears.

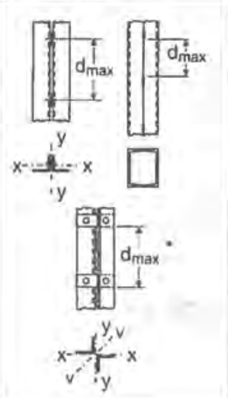
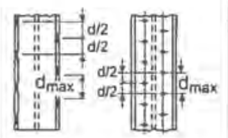
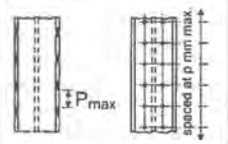
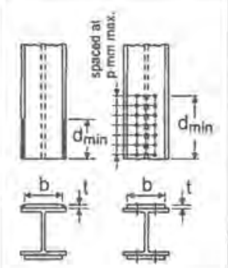
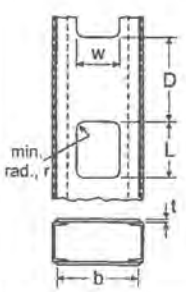
A brief overview of steel-plate shear wall research, along with a comprehensive list of relevant references, can be found in Chapter 6 of Ziemian (2010). Moghimi and Driver (2013) have provided recommendations specifically for designing plate walls economically when a high degree of ductility is not required (e.g. low seismic zones) by using modular construction and shear connections in the boundary frame.

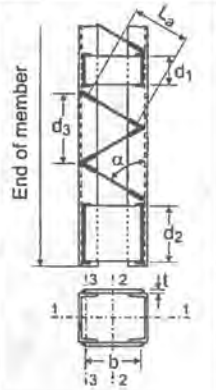
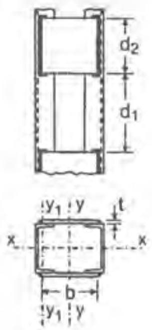
20.2 Seismic Applications

The provisions of Clause 20 must be met for all plate walls. Additional requirements specifically for seismic applications are laid out in Clause 27.

20.3 Analysis

Thorburn *et al.* (1983) demonstrated that the strip model shown in Figure 2-63 predicts the development of tension field action in plate walls subjected to lateral loads. The forces and moments in a plate wall may be estimated by extending the strip model over all storeys using

Compression Members	Requirements
	ROLLED SHAPES $d_{max} = \left(\frac{KL}{r}\right) r_{min}$ KL/r = slenderness of member as a whole r_{min} = least radius of gyration of one component * For starred angles, 1/3 points Cl. 19.2.5. See Clause 19.2.4 for design slenderness and effective slenderness ratios 19.2.4
	STAGGERED FASTENERS OR WELDS $d_{max} = \frac{525t}{\sqrt{F_y}}$ or 450 mm t = outside plate thickness 19.2.3
	FASTENERS OR WELDS NOT STAGGERED $P_{max} = \frac{330t}{\sqrt{F_y}}$ or 300 mm t = outside plate thickness 19.2.3
	ENDS OF BUILT-UP COLUMNS Welded connection: $d_{min} = b$ Bolted connection: $d_{min} = 1.5b$ $P_{max} = 4 \times$ diameter of fastener 19.2.2
	PERFORATED COVER PLATES $b \le \frac{840t}{\sqrt{F_y}}$ $L \le 2W$ $D \ge b$ $r \ge 40$ mm Note: For bolted fabrication, $b \ge 400$ mm is preferred. 19.2.15

Compression Members	Requirements
	LACING AND TIE PLATES $b \le 60t$ $d_1 \ge b/2$ $\quad d_2 \ge b$ $d_3 \le \left(\frac{KL}{r_1}\right) r_3 \le \left(\frac{KL}{r_2}\right) r_3$ $L_a \le 140 \times$ radius of gyration of lacing member $\alpha \ge 45°$ KL = Effective length of member with respect to appropriate axis 19.2.9 – 13
	BATTENS $b \le 60t$ $d_2 \ge b$ $\left(\frac{KL}{r}\right)_e = \sqrt{\left(\frac{KL}{r}\right)_y^2 + \left(\frac{KL}{r}\right)_{y1}^2}$ 19.2.4 and 19.2.16

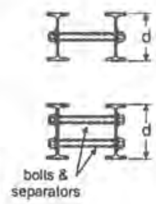
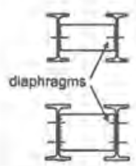
Beams and Grillages	Requirements
	NON-LOAD-SHARING BEAMS Not less than one bolt: $d < 300$ mm Two or more bolts: $d \ge 300$ mm Centres of separator groups ≤ 1500 mm 19.4
	LOAD-SHARING BEAMS Diaphragm shall have sufficient stiffness to distribute required loads. Centres of diaphragms ≤ 1500 mm 19.4

Figure 2-60
Built-up Compression Member Details

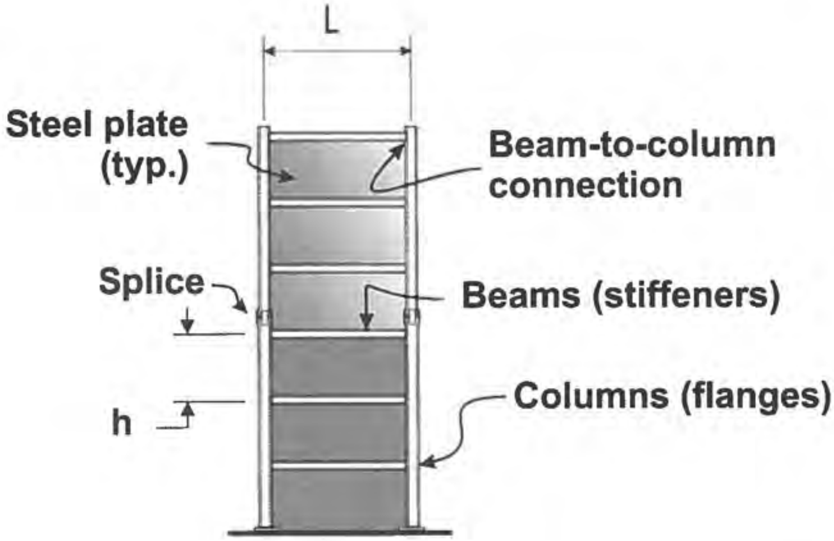


Figure 2-61
Typical Plate Wall

Figure 2-62
Plate Wall Diagonal Tension Brace Model

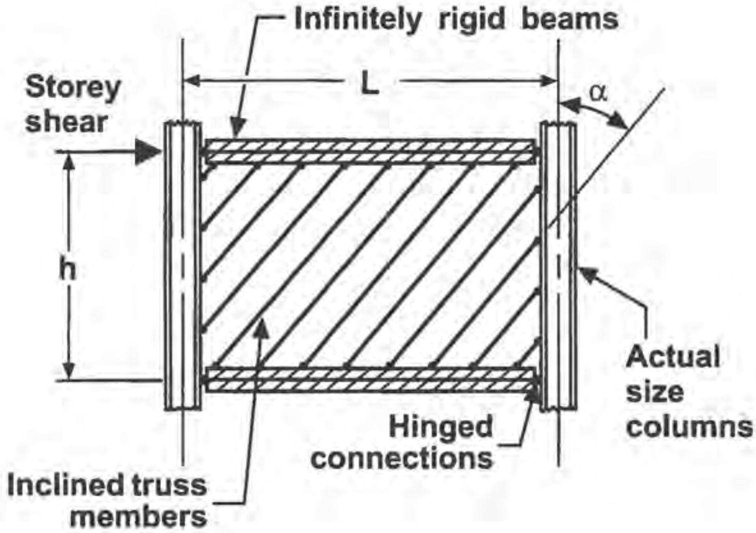


Figure 2-63
Strip Model for a Plate Wall

a plane frame structural analysis program. Ten strips per panel have been found to be sufficient in most cases. The continuity of connections between beams and columns, and the actual sizes of the beams, are accounted for in the analysis. When the entire plate wall is modelled, the average angle of inclination may be used for the complete wall, as stated in Clause 20.4.2. The analysis determines the tensile forces in the infill plates from tension field action, the forces imposed by the infill plate on the boundary beams and columns, and the forces and moments in the boundary beams and columns. Shishkin *et al.* (2009) discuss means of optimizing the strip model in terms of both accuracy and modelling efficiency.

For preliminary design, the overall behaviour of a plate wall can be approximated in a plane frame analysis as a vertical truss by representing each infill panel by a single diagonal tension brace (see Figure 2-62). Thorburn *et al.* (1983) express the equivalent area, A, of the diagonal tension brace as

$$A = \frac{wL\sin^2 2\alpha}{2\sin\theta\sin 2\theta}$$

The beams and columns are taken to have their actual cross-sectional properties in the analysis. When plate walls with moment frames are used, this model also determines the beam and column moments that develop as a result of frame action.

20.4 Angle of Inclination

Shishkin *et al.* (2009) demonstrate that, when using the strip model to analyze plate walls of typical proportions, the overall behaviour of the walls is relatively insensitive to the angle of inclination of the strips. They showed that selecting an angle of 40° from the vertical provides accurate, yet conservative, results over a wide range of wall configurations.

For cases that fall outside of the limits investigated by Shishkin *et al.* (2009), an expression developed by Timler and Kulak (1983) for the angle of inclination of the tension field strips is provided. This expression was determined by minimizing the work in one panel owing to the tension field action in the infill plate, flexure and axial forces in the boundary columns, and the axial force in one boundary beam per panel.

The expression was derived assuming:

- The storey shear is approximately the same in the panels above and below the storey under consideration.
- The beams are attached to the columns with pin-ended connections.
- The columns are continuous.
- The storey heights are approximately equal.

When these assumptions are not met, see Appendix A of Timler and Kulak (1983) to apply the least work derivation to other cases.

20.5 Limits on Column and Beam Flexibilities

In order for the tension field to develop relatively uniformly in the infill plate at each storey, the columns of the plate wall must be sufficiently stiff. Based on the work of Kuhn *et al.* (1952), the column flexibility parameter, ω_h, as given in Clause 20.5.1, shall not exceed 2.5.

The uniformity of the tension fields in the top and bottom panels of the plate wall depend on the stiffnesses of both the adjacent columns and the top or bottom beam, as appropriate. Dastfan and Driver (2008) developed a boundary member flexibility parameter, ω_L, to characterize the boundary stiffness for these extreme panels. The value of ω_L shall not exceed 2.5 at the top of the wall and 2.0 at the bottom, reflecting the relative importance of the behaviour of the bottom panel on the overall performance of the plate wall. The lower limit on ω_L of $0.84\,\omega_h$ is to prevent obtaining a negative beam stiffness. The derivation and application of both ω_L and ω_h are discussed by Dastfan and Driver (2008, 2009).

20.7 Beams and 20.8 Columns

Under high lateral loads, plastic hinges tend to develop in the beams and columns of plate walls. To avoid premature failure, beams shall be Class 1 or Class 2 sections, and columns Class 1 sections.

20.9 Anchorage of Infill Plates

These requirements ensure that the top and bottom infill plates are anchored to members that are sufficiently stiff to develop relatively uniform tension fields, and that the forces developed at the base of the wall are transferred properly into the substructure.

20.10 Infill Plate Connections

The infill plate is to be connected to the surrounding frame – and spliced, if required – to resist the factored ultimate tensile strength of the plate in order to ensure a ductile failure mode. These connections may be either welded or bolted.

21. CONNECTIONS

21.3 Restrained Members

When the compressive or tensile force transmitted by a beam flange to a column (approximated by the factored moment divided by the depth of the beam) exceeds the factored web bearing or flange tensile resistance of the column, stiffeners are required to develop the load in excess of the bearing or tensile resistance.

Taking the length of the column web resisting the compressive force as the thickness of the beam flange plus ten times the thickness of the column flange as in Clause 14.3.2(a)(i) results in the first equation given in Clause 21.3 for the bearing resistance of columns with Class 1 and 2 webs. For members with Class 3 and 4 webs, the bearing resistance of the web is limited by its buckling strength. The expression for the factored bearing resistance is conservatively based on the critical buckling stress of a plate with simply-supported edges:

$$\sigma_{cr} = k\frac{\pi^2 E}{12\left(1-\nu^2\right)\left(h_c / w_c\right)^2} = \frac{723\,000}{\left(h_c / w_c\right)^2} \quad \text{when } k_{min} = 4$$

The number 640 000, given in Clause 21.3(a), reflects a further reduction for the effect of possible residual stresses.

Although not stated, the bearing resistance computed from the second equation should not exceed the first. In both expressions, if the compression flange is applied at the end of a column, the loaded length should be reduced to $t_b + 4t_c$, and the resistance factor should be reduced to ϕ_{be}.

Graham *et al.* (1959) also show, based on a yield line analysis, that the column flange bending resistance, when subject to a tensile load from the beam flange, can be taken conservatively to be $7t_c^2F_{yc}$. Tests have shown that connections proportioned in accordance with this equation have carried the plastic moment of the beam satisfactorily.

When moment connections are made between beams and columns with relatively thick flanges (greater than about 50 mm), prudent fabrication practice suggests that the column flanges be inspected (such as radiographically) in the region surrounding the proposed weld locations to detect and thereby avoid any possible laminations that might be detrimental to the through-thickness behaviour of the column flange. Dexter and Melendrez (2000) reported on the results of recent studies on this topic.

Huang *et al.* (1973) demonstrated that beam-column connections designed such that the web was connected only for the shear force were capable of reaching the plastic capacity of the beam even though in some tests the webs were connected with bolts based on bearing-type connections in round or slotted holes. The slips that occurred were not detrimental to the static ultimate load capacity. For joints in zones of high seismicity, see Commentary on Clause 27.

Bolted extended end-plate-type connections are also commonly used for beam-to-column moment connections. Murray (2003) presents equations for the bearing and tensile resistances of the column flange opposite the flanges of the beam, for use with extended end-plate-type connections. AISC (2013) and Carter (1999) have adopted the design equations presented by Murray (2003). Note that the equation used in calculating the tensile resistance of the column flange is based on research using only ASTM A36 material. For this reason, if columns with higher yield strengths are used, it is recommended (conservatively) that the column yield strength be limited to 250 MPa (36 ksi) for calculating the tensile resistance of the column flange. Detailed design procedures for other limit states (other possible failure modes) for this type of connection are presented in Murray (1990) and AISC (2013). Prying action should also be checked on the end plate connection and the column flange opposite the tension flange of the beam. Clause 22.2.2(e) requires that bolts subject to tensile forces be pretensioned.

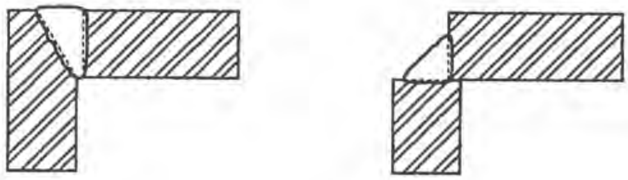
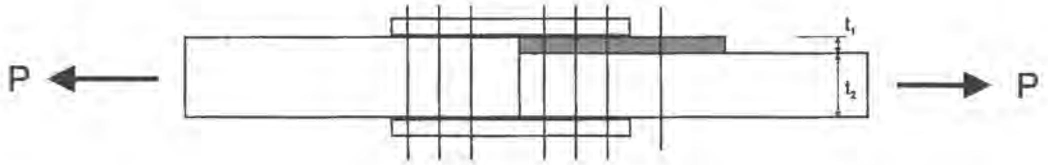


Figure 2-64
Details to Minimize Lamellar Tearing



$$\text{Load on Filler} = P\frac{w_1 t_1}{w_1 t_1 + w_2 t_2}$$

Figure 2-65
Load on Filler Plate

21.4 Connections of Tension or Compression Members

Obviously, the end connections must transmit the factored loads. In order to guard against providing a connection inconsistent with the member it connects, when the member size has been selected for some criterion other than strength, the designer may choose to provide a minimum connection with a capacity higher than the design load.

The requirement for a connection at least equal to 50% of the member's capacity was withdrawn in S16-01 as it was often misapplied, resulting in grossly oversized connections.

21.5 Bearing Joints In Compression Members

When determining the requirement for fasteners or welds to hold all parts securely in place, the stability of the structure shall be considered for all possible load conditions in accordance with the requirements of Clause 6.1.

21.6 Lamellar Tearing

In cases where shrinkage results as a consequence of welding under highly restrained conditions, very large tensile strains may be set up. If these are transferred across the through-thickness direction of rolled structural members or plates, lamellar tearing may result. Thornton (1973) and AISC (1973) give methods of minimizing lamellar tearing. Figure 2-64 illustrates one such case.

21.7 Placement of Fasteners and Welds

Gibson and Wake (1942) have shown that, except for cases of repeated loads, end welds on tension angles and other similar members need not be placed so as to balance the forces about the neutral axis of the member.

21.8 Fillers

The intent of this clause is to ensure that the total load transferred through a connection will be transferred uniformly over the combined cross section of the filler plate and the connected material, in order to avoid bending in the bolt shank. In order to do this, the filler plate should be connected for a load equivalent to the total load multiplied by the ratio of the filler plate thickness to the combined thickness of the filler plate and the connected material (see Figure 2-65). However, in slip-critical joints, tests with fillers up to 25 mm (1 inch) in thickness and with surface conditions comparable to other joint components show that the fillers act integrally with the remainder of the joint, and they need not be developed before the splice material (Kulak *et al.* 2001).

21.10 Fasteners and Welds in Combination

21.10.1 Requirements for the design of joints that combine welds and high-strength bolts placed in the same shear plane are covered in Clause 13.14.

21.11 High-Strength Bolts (in Slip-Critical Joints) and Rivets in Combination

Hot-driven rivets have a clamping force comparable to that of the pretensioned bolts, albeit somewhat more variable.

21.12 Connected Elements Under Combined Tension and Shear Stresses

The new clause 21.12 in the 2014 edition of the standard addresses the state of combined shear and tensile normal stresses in a plane of a connected element. This combined stress state occurs in many common connecting elements, including gusset plates, shear tabs, and beam webs welded to end plates. The research reported by Guravich and Dawe (2006) suggests that the presence of the state of combined stresses does impact the connecting element's strength, but this only occurs after a certain threshold has been reached, i.e. for certain combinations, normal and shear stresses may be considered in isolation. The new clause recognizes this fact and suggests that the tensile stress at full yield can occur simultaneously with a shear stress within 75% of the ultimate shear stress capacity ($0.75 \times 0.66 \approx 0.5$). This clause covers the strength limit state only, and stability of the connecting elements under combined stresses should also be considered.

22. DESIGN AND DETAILING OF BOLTED CONNECTIONS

Note: This Clause primarily applies to high-strength bolts. Although the Standard permits the use of A307 bolts in certain applications, some of the requirements in this Clause do not apply to them.

22.1 General

The behaviour of a joint depends both on how the bolts are loaded and installed. In the 1984 edition, for the first time, the use of snug-tightened high-strength bolts was permitted. Their use has proved successful. As there are four basic types of connections, three with bolts in shear and one with bolts in tension, it is absolutely essential that the design documents specify the type of connections used.

Kulak *et al.* (2001) showed that the ultimate shear and bearing resistances of a bolted connection are not dependent on the pretension in the bolt. As the number of situations (Clause 22.2.2) where pretensioning is required is limited, the norm for building construction is to use snug-tightened bearing-type connections. Departures from the norm are only to be made with due consideration. Few joints in building construction are subject to frequent load reversal nor are there many situations where a one-time slip into bearing cannot be tolerated.

High-strength bolts must be pretensioned when they are subject to shear in slip-critical connections, tension, seismic forces in applications as required in Clause 27.1, or any combination thereof. High-strength bolts subject to shear in bearing-type connections may either be pretensioned or snug-tightened. Only A325 and A490 bolts, the tension-control bolt assemblies (F1852 and F2280), and the metric series (A325M and A490M) may be used in joints requiring pretensioned high-strength bolts.

As a result of normal fabrication practice, minor misalignment of bolt holes may occur in connections with two or more bolts. Such misalignment, if anything, has a beneficial effect (Kulak *et al.* 2001) resulting in a stiffer joint, improved slip resistance and decreased rigid body motion.

A comprehensive summary of bolt requirements is given by Kulak *et al.* (2001).

22.2.1 Use of Snug-Tightened High-Strength Bolts

Snug-tightened bolts may be used, except for the specific cases given in Clause 22.2.2 where the use of pretensioned high-strength bolts is required. Bolts that are not pretensioned must be installed to a snug-tightened condition. These may be A307, A325 or A490 bolts. Because the ultimate limit states of shear through the bolt and bearing on the plate material are not significantly affected by the level of pretension (Kulak *et al.* 2001), it is only logical to permit bolts of higher strength than the A307 bolt to also be installed snug-tight in similar connections. This was recognized, in part, as early as the 1984 edition.

22.2.2 Use of Pretensioned High-Strength Bolts

(a) Pretensioning of the bolts provides the clamping force in slip-critical connections and hence the slip resistance at the specified load level appropriate to the condition of the faying surfaces.

(b) Pretensioning of the bolts provides energy dissipation under cyclic earthquake loading in connections proportioned for seismic applications that trigger the requirements in Clause 27.1, although these connections are proportioned as bearing-type connections for the ultimate limit state. The contact surfaces should be Class A or better for such joints.

(c) See (d).

(d) Pretensioning in both these connections ensures that the bolts don't work loose and, if necessary, ensures adequate fatigue behaviour.

(e) An example of such a connection is a tee-hanger connection. Pretensioning reduces the prying action and the stress range.

(f) In connections with oversize or slotted holes, pretensioning prevents gross movement within the joint. See also Clause 22.3.5.2 to determine for which cases slip-critical connections are required.

For the usual building structure, full wind loads and earthquake loads are too infrequent to warrant design for fatigue, as the number of stress cycles are less than the lower limits given in Clause 26.3.5. Therefore, slip-critical connections are not normally required in buildings for wind or seismic load combinations. However, connections of a member subject to flutter, where the number of cycles is likely higher, is an exception. Popov and Stephen (1972) observed that the bolted web connections of welded-bolted moment connections slipped early in the cyclic process.

Slip-critical connections are required in connections involving oversized holes, certain slotted holes, fatigue loading, or crane runways and bridges. In assessing whether or not the joint slip is detrimental at service level loads, Popov and Stephen (1972) and Kulak *et al.* (2001) have shown that, in joints with standard holes, the average slip is much less than a millimetre. Bolts of joints in statically loaded structures are most likely in direct bearing after removal of the drift pins, due to the self weight of the member, and are thus incapable of further slip (RCSC 2014).

22.2.5 Fastener Components

A325M, A490M, A325 and A490 bolts are produced by quenching and tempering (ASTM 2013, 2012, 2010, 2012, respectively). A325 bolts are not as strong as A490 bolts but have greater ductility. For this reason and reasons of availability, the use of A490 bolts is subject to restrictions as discussed subsequently. ASTM F1852 and F2280 bolts, commonly known as "tension control" or "twist-off" bolts, have mechanical and chemical properties equivalent to A325 and A490 bolts, respectively (ASTM 2011, 2012).

The normal bolt assembly consists of an A325, A490, F1852 or F2280 bolt, with a heavy hex head, restricted thread length, coarse threads, and a heavy hex nut. F1852 and F2280 bolt assemblies consist of a bolt with a splined end which typically has a button head. Alternatives to the normal bolt assemblies are available which differ in various aspects and, in some cases, may offer one or more advantages. Their use is permissible under the conditions set forth in Clause 22.2.5.4.

At the time of preparation of this Commentary, availability of A325M and A490M bolts requires an order of unusually large quantity and a long lead time.

Galvanized A325 bolts are permitted; however, metallic coated A490 bolts are not permitted, as they are especially susceptible to stress corrosion and hydrogen stress cracking (Kulak *et al.* 2001). The rotation requirement of this clause provides a means of testing the galvanized assembly for proper fit and for proper thread lubrication. Installation of F1852 and F2280 assemblies is dependent upon consistent friction properties of the bolt threads and the nut. Therefore, these assemblies should not be hot-dip galvanized. F2280 assemblies should not be electroplated.

22.3.5 Bolt Holes

Details on the sizes and types of holes (standard, oversize, or slotted) permitted for bearing-type and slip-critical connections are given. While the Standard permits several hole-making methods, punching and drilling are the most common. Incremental punching is sometimes used in fabricating slotted holes – especially long slots. Thermal cutting of holes, such as cutting the edges of a slot between two punched holes, is acceptable within the requirements of Clause 28.4.3.

Clause 22.3.5.1 allows selected Imperial bolts in metric holes without restriction.

A hardened washer, when required in Clause 22.3.5.2, is intended to cover the hole (or bridge the slot) if it occurs in an outer ply.

For the use of pretensioned large-diameter A490, A490M and F2280 bolts in oversize or slotted holes, specific requirements for the use of hardened washers apply. See Clause 23.4.2(d).

23. INSTALLATION AND INSPECTION OF BOLTED JOINTS

Note: This Clause primarily applies to high-strength bolts. Although the Standard permits the use of A307 bolts in certain applications, some of the requirements in this Clause do not apply to them.

Bolts required to be pretensioned must be tightened to tensions of at least 70% of their specified minimum tensile strength. All other bolts need only be snug-tightened.

Except when galvanized, A325 bolts may be reused once or twice, providing that proper control on the number of reuses can be established (Kulak *et al.* 2001; RCSC 2014). A490, F1852 and F2280 bolts should not be reused. The level of pretension attained in bolts of Grade A490 decreases significantly when the bolts are re-used.

23.1 Connection Fit-up

The simple phrase "connections in firm contact when assembled" describes the snug-tightened condition.

23.2 Surface Conditions for Slip-Critical Connections

The treatment of the faying surfaces within the plies of slip-critical joints is to be consistent with the mean slip coefficient chosen for design (Clause 13.12). For clean mill scale, the surfaces must be free of substances which would reduce the slip coefficient. For other coatings, the surface preparation, coating application, and curing should be similar to those used in the tests to obtain the slip coefficient. The Society for Protective Coatings (SSPC) provides specifications for cleaning and coating of steel structures. Kulak *et al.* (2001) provide information on slip for various surface conditions and coating types.

23.4 Use of Washers

Clauses 23.4.1 and 23.4.2 list the circumstances when ASTM F436 hardened washers are required under the turned element and with pretensioned bolts. It follows that these washers are not required in A325 bolt installations except for oversize or slotted holes in pretensioned connections.

The use of an 8-mm hardened washer for large-diameter A490 bolts in accordance with Clause 23.4.2(d) aims to distribute the high clamping forces of these bolts. Alternatively, the hole may be covered with a 10-mm mild steel plate washer with a standard hardened washer under the head or the nut. The contract documents should specify any specific requirements.

The requirements for bevelled washers with ASTM A490 bolts are more stringent than for A325 bolts because of the somewhat reduced ductility of the former.

23.5 Storage of Fastener Components for Pretensioned Bolt Assemblies

This clause emphasizes that proper storage of fastener components is particularly critical for ASTM F1852 and F2280 assemblies, because the torque at which the splined end is sheared off the bolt shank depends on the friction characteristics between the bolt threads and the nut, which therefore must be maintained at the as-manufactured condition, so that the relationship between the twist-off torque and bolt pretension is what was intended and is expected to be.

23.7 Pretensioned High-Strength Bolt Assemblies

For all pretensioned high-strength bolt installations, it is critical that inspection for bolt pretension be done while the bolt tightening is in progress. Verification that the installation techniques described in this clause have been followed will provide adequate assurance that the required bolt tensions are being attained.

23.7.1 Installation Procedure

The pretensioning procedures included in this Standard have been proven (Kulak and Birkemoe 1993, Kulak and Undershute 1998) to provide bolt tensions required by this clause. Torque-tension relationships are highly variable and dependent upon many factors including installation procedures, bolt finish, and bolt and nut thread conditions. For this reason, it is not possible to establish a standard bolt torque value that corresponds to the required bolt pretension values. Clause 23.8 describes the proper, simple inspection procedures for bolted connections.

23.7.2 Turn-of-Nut Method

Any installation procedure used for pretensioning high-strength bolts involves elongating the bolt to produce the desired tension. Although the shank of the bolt probably remains elastic, the threaded portion behaves plastically. Because the bolt as a whole is tightened into the inelastic range (the flat portion of the load-deformation curve), the exact location of "snug-tight" is not critical (Kulak *et al.* 2001). The turn-of-nut method is a strain or deformation control method, and even a considerable change in deformation results in little change in load. Thus, application of the specified amount of nut rotation results in pretensions that are not greatly variable. They are also greater than those prescribed in Table 7, which occur about where inelastic action begins. Although there is a reasonable margin against twist-off, the tolerance of $\pm 30^{\circ}$ or $\pm 1/12$ of a turn on nut rotation prescribed in the footnote to Table 8 is good practice, particularly when galvanized A325 bolts or black A490 bolts are used.

23.7.3 and 23.7.4 Use of ASTM F959, F1852 and F2280 Bolting Systems

The Standard permits the use of F1852, F2280 and F959 bolting systems. These systems are proprietary in nature, relying on a discernible physical change in a part of the bolt system indicating that the minimum bolt tension has been achieved. Systems that rely on irreversible deformations or fracture of a part serve only to indicate that, during installation, a force or torque sufficient to deform or fracture the part had been reached. Even with such a system, reliable results are dependent on strict adherence to the installation procedures for snugging of the joint and patterned tightening operations as given in Clause 23.7.1 and to the storage fastener requirements of Clause 23.5 for F1852 and F2280 bolts.

23.8 Inspection Procedures

Bolts, nuts, and washers are normally received with a light residual coating of oil. This coating is not detrimental; in fact it is desirable and should not be removed. This is especially important for F1852 and F2280 bolts, since these bolts depend on the lubricant to achieve the desired level of pretension. Galvanized bolts and/or nuts may be coated with a special lubricant to facilitate tightening. Obviously, this should not be removed.

The inspection procedures used depend on whether the bolts are specified to be snug-tightened or pretensioned. In all cases, by Clause 23.8.1, the inspector shall observe that the procedure for the installation of the bolts conforms with the requirements of this Standard.

When snug-tightening is specified, the tightening is deemed satisfactory when all of the connected elements are in full contact. Galling of the turned element may be evident. Inadvertent pretensioning of snug-tightened bolts is normally not a cause for concern.

When pretensioning is specified, the tightening is deemed satisfactory when all of the elements are in full contact, and observation of the sides of the turned elements shows that they have been slightly galled by the wrench. This is all that is required.

When bolts are tightened by the turn-of-nut method and when there is rotation of the part not turned by the wrench, the outer face of the nut may be match-marked with the bolt point before final tightening, thus affording the inspector visual means of noting nut rotation. Such marks may be made with crayon or paint by the wrench operator after the bolts have been snugged.

Should disagreement arise concerning the results of inspection of bolt tension of bolts specified to be pretensioned, arbitration procedures as given in Annex I are to be followed. The use of inspection torque values other than those established according to the requirements of Annex I is invalid because of the variability of the torque-tension relationship. The inspection procedure given in Annex I is the same as that recommended by the Research Council on Structural Connections (RCSC 2014) and places its emphasis on the need to observe the installation for the proper tightening procedures, rather than using the arbitration procedures which in fact are less reliable.

Regardless of the installation procedure or the type of bolt-washer-nut assembly used, it is important to have all of the plies drawn up tight before starting the specific tightening procedure. This is particularly so for stiff joints that require pattern tightening.

24. WELDING

24.1 and 24.2 Arc and Resistance Welding

Consistent with CSA policy that the requirements of one standard are not repeated in another, the user of this Standard is referred to CSA Standards W59 and W55.3 for the requirements for arc and resistance welding (e.g. weld quality, welding procedure and practice, etc.), respectively, but with two distinct exceptions. The distinction is made that, for arc welds with matching electrodes, the factored resistances for static loadings and the fatigue resistance for fatigue loadings are obtained from Clauses 13.13 and 26 of this Standard, respectively. (Much of the research into weld strengths and formulation of weld resistances has been done by members of this Standard committee.)

Designers' attention is drawn to the fact that, in the U.S.A., cracking has been noted after welding of column web stiffener or of doubler plates on heavily rotarized W-shapes in the fillet regions. This is attributed to the loss of ductility due to cold working.

W59 permits the use of intermittent fillet welds in the compression zone, irrespective of whether fatigue is a consideration.

24.3 Fabricator and Erector Qualification

The intent of Clause 24.3 is simply that the responsibility for structural welding shall lie with the fabricators and erectors certified by the Canadian Welding Bureau to the requirements of CSA W47.1 and/or CSA W55.3, as stated specifically in the clause. Such certification should ensure that the fabricators and erectors have the capability to make structural welds of the quality assumed by S16-14.

There is a specific requirement that fabricators and erectors meet the requirements of CSA W47.1 in Division 1 or Division 2 for arc welding. This will ensure that the fabricator or erector has a suitably qualified welding engineer either on staff or on retainer. However, the clause does permit work to be sublet to a Division 3 fabricator or erector (i.e. organizations without a welding engineer on staff or on retainer), provided the Division 1 or 2 fabricator or erector retains responsibility for the work.

25. COLUMN BASES AND ANCHOR RODS

The clauses on column bases and anchor rods have been combined, as the two are likely found together as components of the same foundation unit. The designer is referred to appropriate clauses of CSA A23.3 (CSA 2014) for the various resistances of the reinforced concrete elements.

In general, the use of base plates bearing directly on grout is preferred to the use of levelling plates interposed between the base plate and the grout. The latter condition may lead to uneven bearing.

Typically, anchor rods – formerly referred to as anchor bolts – are threaded rods that are either supplied in accordance with ASTM F1554 or fabricated from a steel bar of A36 or G40.21-300W steel. The expressions for the tensile, shear, and combined shear and tensile resistance of anchor rods are similar to those for high-strength bolts. The basic elliptical interaction diagram is used for combined shear and tension. For tension and bending, the factored moment resistance is limited to the factored yield moment, because the ductility of the steel used may be limited. For anchor rods in tension, the designer should specify a material with fracture toughness appropriate for the minimum service temperature. Pretensioning of anchor rods is usually not recommended, as there is a tendency for relaxation and a possibility of stress corrosion. Pretensioning requires special attention.

All anchor rod resistances, whether for tension, shear, bearing or moment, or for use in interaction equations, are those given in this Clause.

26. FATIGUE

Clause 26 provides the requirements for the design of members and connections subjected to cyclic loading and susceptible to the formation and growth of cracks during the design life of the structure. The phenomenon of formation and growth of cracks under cyclic loading is called fatigue. The fatigue limit state, which is the limiting case of the slow propagation of a crack within a structural element, can result from either live load effects directly or as the consequence of local distortion within the structure due indirectly to live load effects. These two cases are referred to as live-load-induced fatigue and distortion-induced fatigue, respectively. The limit state of fatigue is checked at load levels expected to occur many times during the life of the structure – loads that are considered to be repetitive. In the event that more than 20 000 stress cycles take place, the loaded members, connections, and fastening elements shall be proportioned so that the probability of fatigue failure is acceptably small. In such cases, the design shall be based on the best available information on the fatigue characteristics of the materials and components to be used. In the absence of more specific information, which is subject to the approval of the owner, the requirements of Clause 26 in its entirety provide guidance in proportioning members and parts. The fatigue design loads are taken to be the specified loads. In addition, Clause 26.1 requires that all members and connections in the structure meet the ultimate limit state requirements, i.e. that factored resistances be at least equal to the effect of factored static loads – load levels that occur very seldom, perhaps only a few times in the life of the structure, but which the structure must nevertheless be able to withstand in order to achieve the required level of safety.

A substantial amount of experimental data, developed on steel beams since 1967 under the sponsorship of the National Co-operative Highway Research Program (NCHRP 1970, 1974; Fisher 1974) of the U.S.A., has shown that the most important factors governing fatigue resistance are the stress range, the type of detail, and the number of cycles. Steel grade and fracture toughness do not have a significant effect on the fatigue resistance.

The provisions of this clause are those commonly used in North American design standards, except for the long-life region of behaviour. The North American fatigue design approach for most civil engineering structures is to base the fatigue life calculation on a nominal stress range (a stress range calculated using basic strength of materials approach, which does not account for stress concentration) and to account for stress concentration in the detail of interest by selecting the appropriate fatigue category varying from Category A, the most favorable detail with no stress concentration, to Category E1, the least desirable detail. Experience has shown that fatigue considerations for details of Category A through B1 rarely govern. Nevertheless, these are included for completeness.

While fatigue is generally not a design consideration for buildings such as those for commercial or residential occupancies, industrial buildings may have many members, such as crane girders, for which fatigue is a concern. Other instances where fatigue is likely a consideration are amusement rides, wave guides, sign support structures, and beams supporting reciprocating machinery. When members and connections are subjected to fatigue loading, Clause 26 requires that they be designed, detailed, and fabricated to minimize stress concentrations and abrupt changes in cross-section. Consideration should also be given to the service conditions, which may change the condition of stress concentration, namely, the fatigue category after the structure has been placed in service. For example, a detail with no significant stress concentration can become one with high stress concentration if the member is exposed to a corrosive environment. Therefore, the designer must consider the possibility of changing stress conditions during the service life of the structure.

Fatigue crack growth is referred to either as load-induced or as distortion-induced. Load-induced stresses are those corresponding to the design loads normally considered by a first-order analysis where the effect of deformations on force effects are not considered. Distortion-induced stresses are those resulting from the relative movement of connected parts of an assemblage in such a way that large localized strains are produced. Because this phenomenon is difficult to include with any level of accuracy in the design calculations, distortion-induced fatigue is best avoided by using recognized details to obviate potential problems. An accurate assessment of distortion-induced stresses requires detailed modeling of the interactions between all structural and non-structural elements of the structure.

26.3 Live-Load-Induced Fatigue

26.3.1 Calculation of Stress Range

The stress range is the algebraic difference between the maximum stress and the minimum stress at a given location due to the passage of the live load. When calculating the applied stress range, the effect of any load eccentricity must be accounted for. Although minor eccentricities are usually ignored at the ultimate limit state because it is expected that yielding of the member at the ultimate limit state will reduce this effect, the member remains elastic at the fatigue limit state. Since the effect of stress concentration is not included in the stress range calculation, its effect must be incorporated by selecting the appropriate fatigue category as described for usual structural details illustrated in Figure 2 and described in Table 9 of the Standard.

Because fatigue cracks grow only if there is a net tensile stress from the live load, it is not necessary to investigate fatigue at locations where the applied stresses are always in compression and at locations where the maximum tensile live load stress is less than the compressive dead load stress.

26.3.2 Design Criteria

The criterion expressed by the relationship $F_{sr} \geq f_{sr}$ simply states that the fatigue resistance (or allowable stress range) of a given detail for the design number of load cycles, nN, over the design life of the structure must be equal to or exceed the calculated stress range. The allowable stress range may be calculated from the equation $F_{sr} = (\gamma / nN)^{1/3} \geq F_{srt}$ when the fatigue resistance is greater than the constant amplitude threshold stress range, or from $F_{sr} = (\gamma' / nN)^{1/5} \leq F_{srt}$ when the fatigue resistance is less than the constant amplitude threshold stress range. Each fatigue curve represents the mean fatigue life minus two standard deviations from a series of constant-amplitude fatigue tests on details representative of the category. The fatigue life constants, γ and γ', for the appropriate detail are obtained from Table 10 of the Standard. Also shown in Table 10 is the constant-amplitude threshold stress range F_{srt}, represented in Figure 1 of the Standard by the horizontal dashed lines. For constant-amplitude stress ranges below F_{srt}, crack growth does not occur, i.e. the fatigue life of the detail is infinite. For variable-amplitude fatigue loading, the slope of the fatigue curves below F_{srt} is reduced to 1/5 because it is expected that some of the applied stress ranges will still lie above F_{srt}, even if the average stress range is smaller than F_{srt}. The number of cycles at which the slope of the fatigue curves changes from 1/3 to 1/5 is designated as nN' and can be either calculated from $(\gamma/nN')^{1/3} = (\gamma'/nN')^{1/5}$ or obtained from Table 10.

26.3.3 Cumulative Fatigue Damage

In reality, fatigue loading is rarely at constant amplitude; it is usually of variable stress amplitude, which results in variable numbers of stress ranges of different magnitudes. The cumulative fatigue damage that results from variable-amplitude loading can be evaluated using the linear damage theory known as the Palmgren-Miner rule. Over the design life of the structure, the number of cycles for each identified stress range is estimated, and the fraction of the fatigue life expended by these cycles of loading is obtained by dividing the number of cycles at a given stress range by the fatigue life for that stress range as found from Table 10 or Figure 1 of the Standard. The sum of these fractions so determined, including those for the long-life region of behaviour where the slope of the S–N curve is 1/5, shall not exceed unity as given in this Clause. Chapter 11 of Kulak and Grondin (2014) provides more detailed information on fatigue.

26.3.4 Fatigue Constants and Detail Categories

The fatigue constants defining the eight fatigue curves illustrated in Figure 1 are given in Table 10. Selection of the appropriate fatigue category is carried out with the assistance of Figure 2 and Table 9. Also added to Table 9 are high-strength bolts under tensile cyclic loading.

26.3.5 Limited Number of Cycles

This clause gives a limit on the number of cycles below which no special consideration other than good detailing is necessary for fatigue. The limit is the greater of 20 000 cycles and the fatigue life of the detail.

26.4 Distortion-Induced Fatigue

Secondary stresses due to deformations and out-of-plane movements are not normally calculated in the design process but can be a source of fatigue failures when proper detailing practices are not followed (Fisher 1978 and 1984). Crane girders, their attachments, and supports require careful design and attention to details to minimize fatigue cracks (Griggs 1976).

If the web of a plate girder without longitudinal stiffeners is sufficiently slender, fatigue cracks may develop at the web-to-flange juncture due to a lateral bending of the web. Tests on girders with a web of h/w ratio greater than $3150/\sqrt{F_y}$ have shown a significant reduction in fatigue resistance to the out-of-plane movement of the web when subjected to in-plane bending (Toprac and Natarajan 1971).

26.5 High-Strength Bolts

High-strength bolts loaded in shear are not susceptible to fatigue failure. However, this is not the case when bolts are placed in direct tension. Pretensioned high-strength bolts in joints that are nominally loaded in tension experience little, if any, increase in axial stress under service loads (Kulak *et al.* 1987). For this reason, bolts that are subjected to cyclic tension shall be pretensioned using the procedure outlined in Clause 23.7 of the Standard. In addition, the prying action shall be kept at a relatively small fraction of the total bolt force. The Research Council on Structural Connections (2014) limits the prying action in joints subjected to cyclic tension to a maximum of 30 percent of the externally applied force.

27. SEISMIC DESIGN

Specific seismic design requirements are given in this clause. While the requirements represent the best available knowledge, designers should be alert to new information leading to improved design procedures.

The NBCC assigns ductility-related force modification factors, R_d, and overstrength-related force modification factors, R_o, (i.e. load reduction factors) to various structural systems in relation to their capacity to dissipate energy by undergoing inelastic deformations and to the minimum level of overstrength which can be counted on for each particular seismic-force-resisting system. The greater the ability of the structure to dissipate energy, the higher is the assigned value of R_d. Values of R_d greater than 1.0 can be justified only if the structure has the ability to undergo inelastic deformations without loss of resistance. The product of R_d and R_o is used as a divisor to reduce the magnitude of the design seismic force.

The objective of Clause 27 is to provide details that will exhibit ductility consistent with the values of R_d and R_o assumed in the analysis. The Clause applies to all steel structures in Canada for which seismic energy dissipation capability is required through ductile inelastic response, i.e. all structures for which $R_d \geq 2.0$. Clause 27 defines the requirements for nine classes of structures with $R_d \geq 2.0$:

- Ductile moment-resisting frames (Type D, with $R_d = 5.0$ and $R_o = 1.5$)
- Moderately ductile moment-resisting frames (Type MD, with $R_d = 3.5$ and $R_o = 1.5$)
- Limited-ductility moment-resisting frames (Type LD with $R_d = 2.0$ and $R_o = 1.3$)
- Moderately ductile concentrically braced frames (Type MD, with $R_d = 3.0$ and $R_o = 1.3$)
- Limited-ductility concentrically braced frames (Type LD, with $R_d = 2.0$ and $R_o = 1.3$)
- Ductile eccentrically braced frames (Type D, with $R_d = 4.0$ and $R_o = 1.5$)
- Ductile buckling-restrained braced frames (Type D, with $R_d = 4.0$ and $R_o = 1.2$)
- Ductile plate walls (Type D, with $R_d = 5.0$ and $R_o = 1.6$)
- Limited-ductility plate walls (Type LD, with $R_d = 2.0$ and $R_o = 1.5$).

In addition, other special framing systems are permitted under Clause 27.12.

In each structural system, certain structural elements are designed to dissipate energy by inelastic straining; other members and connections in the frame must be designed to respond elastically to the loads induced by the yielding elements. Generally, the dissipating elements in moment frames are the beams, in concentrically braced frames the braces, in eccentrically braced frames the links, and in plate walls the wall infill plates. Other elements may also contribute, but to a much lesser extent, for example the connection panel zone in moment-resisting frames, the gusset plates in concentrically braced frames, the outer beam segments in eccentrically braced frames, and beams and columns in steel plate walls.

Properly detailed moment-resisting frames can exhibit very ductile behaviour. Three categories of moment-resisting frames are recognized: first, ductile moment-resisting, or Type D frames, in which members and connections are selected and braced to ensure that severe inelastic straining can take place; second, moderately ductile moment-resisting frames, or Type MD, in which the member details can satisfy the lower inelastic straining demand in structures proportioned to resist the greater design loads, while at the same time, connections are adequate to accommodate the associated forces and deformations. For both systems, beam-to-column connections are required to be designed and detailed in accordance with the CISC Moment Connections for Seismic Applications (CISC 2014), or their performance has to be demonstrated, by means of physical testing, as satisfying minimum criteria under the action of cyclic load as described in Annex J. The third system, Type LD for limited ductility, undergoes still less inelastic demand consistent with the higher design loads and can in general make use of traditional connection detailing, combined with special requirements associated with welding, etc.

Concentrically braced frames are those in which the centre-lines of diagonal braces, beams, and columns are approximately concurrent with little or no joint eccentricity. Inelastic straining must take place in bracing members subjected principally to axial load. Compression members dissipate energy by inelastic bending after buckling, and in subsequent straightening after load reversal. Cyclic local buckling can lead to early fracture, and consequently width-to-thickness limits are restricted for braces. These frames usually have limited redundancy and are prone to concentration of inelastic response in one or a few storeys where energy dissipation is localized. Emphasis in these categories is placed on the presence of braces with similar tensile strength in opposite directions, such that the reduction in storey shear resistance is minimized in the event of brace buckling in a storey.

Two categories of concentrically braced frames are considered, those with moderate ductility (MD) and limited ductility (LD). Both permit several bracing configurations. Compared with past editions of the Standard, the provisions maintain strict limits on width-thickness ratios; overall slenderness limits of braces are relaxed, and changes have been made to the requirements for connection design forces. However, height limitations apply. Since S16-09, bracing configurations with braces intersecting columns at one or more elevations between horizontal diaphragms have been permitted for Type LD braced frames, provided that the columns can accommodate the bending demand due to buckling and yielding of braces within the storey and that horizontal struts are introduced to ensure a continuous load path between tension-acting braces. In S16-2014, the use of this framing configuration has been extended to include Type MD braced frames, and height limits have been extended so that multi-tiered solutions can be obtained from a wider choice.

Ductile eccentrically braced frames are those in which diagonal braces, at least at one end, intersect the beam instead of the beam and column intersection or, in the case of chevron bracing, the two braces do not intersect the beam at a common point. These configurations create eccentric beam links that are designed to dissipate energy. The Standard gives provisions for frames with links in the beams. Beams can be W-shapes or built-up rectangular tubular sections. Lateral bracing at the link ends can be omitted when the latter is used. Provisions for modular links that can be replaced after a severe earthquake have been introduced in CSA S16-14.

The ductile buckling-restrained braced frame system was introduced in CSA S16-09. The braces include a core element with reduced cross-section segment where yielding is expected to develop in both compression and tension. The core is prevented from buckling by means of a lateral restraining mechanism. The system is expected to offer a higher ductility ($R_d = 4.0$) compared to Type MD and Type LD concentrically braced frames. Typically, brace details vary depending on the suppliers, but the inelastic cyclic performance must be demonstrated by means of sub-assemblage and individual qualification cyclic physical testing. The brace compressive and tensile resistances established in these tests must be used in the capacity design process.

Plate walls are formed by thin infill wall plates framed by beams and columns. These highly redundant and stiff systems dissipate energy by yielding of the infill plate and, often, yielding of the framing members. The good seismic performance anticipated is reflected in their respective applicable values of R_d and R_o. Two categories are defined, Types D and LD. In S16-09, new design requirements were included for beams and columns of Type D plate walls. The Standard also permits the introduction of uniformly distributed circular perforations in the infill plates, to avoid excessive lateral overstrength without resorting to using plates that are too thin for practical construction. Corner openings can also be introduced in the wall plates to facilitate the passage of electrical and mechanical equipment. In general, the design and detailing requirements specified for Type D walls also apply to Type LD walls, except that beams and columns need not be rigidly connected for Type LD walls.

In all systems, because the behaviour of connections will often be critical for good performance under severe earthquake loading, the engineer's responsibility for a seismically critical structure includes not only the provision of connection design loads but also the specification of connection type and details.

Structures for which $R_d = 1.5$ have been assumed in the past to have sufficient inherent energy dissipation capacity arising from traditional design and fabrication practices, so that no additional requirements were necessary. However, since energy dissipation properties can only be mobilized if brittle failure is avoided, minimum requirements are prescribed in Clause 27.11 to achieve this for structures subjected to higher seismic demand. The NBCC (since its 2010 edition), permits the use of structures with $R_d = 1.5$ for buildings taller than 15 m when used for occupancies other than assembly occupancy. Special requirements are given in Clause 27.11 for these taller structures. In addition, other special framing systems are permitted under Clause 27.12.

27.1 General

The expression "$I_E F_a S_a(0.2)$", adopted in the NBCC 2010, is referred to as the "specified short-period spectral acceleration ratio" in Clause 27, whereas the expression "$I_E F_v S_a(1.0)$" is referred to as the "specified one-second spectral acceleration ratio".

27.1.1 A distinction is made between the "seismic-force-resisting system" (SFRS) and the "vertical seismic-force-resisting system". The latter corresponds essentially to the vertical bracing, wall or frame system that takes the form of one or more of the systems described in Clause 27. The SFRS is the whole structural system resisting lateral loads, including the foundations, anchorage to foundations, the vertical seismic-force-resisting system, collector elements, and roof and floor diaphragms. In some cases, members specifically designed for gravity loading only may be relied upon for a contribution to a reserve lateral resistance following storey yielding, and in this case some provisions of Clause 27 apply also to these members (see Clause 27.5.5.2).

27.1.2 This Clause sets out the principles of capacity design and states that the ductile energy-dissipating elements must be clearly identified and detailed along the lateral load path, and that a proper strength hierarchy must be provided in the seismic-force-resisting system to constrain inelastic response to these ductile elements. The energy-dissipating elements must be designed to sustain several reversed cycles of inelastic loading with minimum strength and stiffness deterioration. Other elements must be designed to remain essentially elastic for the duration of the seismic ground motion. Anchor rods must transfer the loads to the foundation.

The maximum anticipated seismic loads imposed on the non-dissipating elements can be determined by hand calculations, static incremental (push-over) analysis or nonlinear dynamic time-history analysis. The inelastic behaviour under cyclic loading of the dissipating elements, including yielding, strain hardening or strength degradation, must be accounted for in the calculations and numerical models. Non-dissipative elements can be assumed to behave elastically in numerical models. A number of site-representative ground motions are necessary in nonlinear dynamic analysis, and maximum response loads in the members are to be determined. Such analyses may be of particular value for tall buildings, especially those beyond the height limits imposed by some other provisions of Clause 27. Other applications may be justified in cases where the requirements of capacity design are known to lead to conservative design loads (e.g. moment-resisting frames proportioned for stiffness and wind effects, plate walls or eccentrically braced frames with long links (Han 1998)).

In cases where the energy-dissipating elements have been oversized, a limit has been placed on the maximum forces that the non-dissipating elements must resist by setting the maximum anticipated seismic load equal to that corresponding to $R_d\, R_o = 1.3$. This maximum load corresponds to the elastic seismic load level determined using $R_d = 1.0$, while it is also recognized that the non-dissipating elements generally possess an overstrength level that justifies $R_o = 1.3$. Connections designed for seismic loads corresponding to $R_d\, R_o = 1.3$ must exhibit a ductile governing failure mode, such as yielding in tension or bolt bearing (Tremblay *et al.* 2009). Otherwise, the limit on seismic loads must be increased to loads corresponding to $R_d\, R_o = 1.0$. In computing the forces on the structure corresponding to $R_d R_o = 1.3$, the upper limit of $V = (2/3)S(0.2)I_E\, W/(R_d\, R_o)$ given in the NBCC applies, provided that the seismic-force-resisting system has an R_d equal to or greater than 1.5. In this case, the upper limit becomes $(2/3)S(0.2)I_E W/1.3$. Also, where foundation "rocking" is accounted for in accordance with NBCC, design forces for the SFRS may be limited to values associated with maximum forces that can develop with foundation rocking. Foundation rocking, however, induces larger storey drifts that must be accounted for in the design.

27.1.3 The vertical seismic force-resisting systems described in Clause 27 are expected to exhibit proper performance when non-structural elements such as walls or interior partitions are separated from the structural elements under earthquake deflections. If this cannot be achieved, the effects of the interaction must be accounted for in the analysis and the design.

27.1.4 Gravity load-carrying elements such as columns and beam-to-column connection elements must be able to support the companion gravity loads while undergoing the large deformations expected during earthquakes. For example, a simple beam end connection in the displaced configuration should resist shear due to the companion gravity loads.

Under a severe ground motion, columns in multi-storey structures will be subjected to shear forces and bending moments due to variations in storey drifts that will develop along the structure height. Splices in the columns that are not part of the seismic force-resisting systems must be designed to resist shear forces associated with this response. This provision applies in both orthogonal directions. Requirements for splices in columns that are part of the seismic-resisting systems are given in Clauses 27.2 to 27.11.

27.1.5 This Clause applies principally to the materials used in the yielding elements and connections of the seismic force-resisting system. Limits on the yield stress and the provisions of Clause 8.3.2(a) ensure adequate post-yield behaviour of the material. Use of other materials would require demonstration that the energy-dissipating elements can sustain the very high post-yield strains needed to achieve the performance assumed in design. Because of the dynamic loading, toughness requirements are specified for buildings with specified short-period spectral acceleration ratios, $I_E F_a S_a(0.2)$, greater than 0.55 for thick plates and shapes in energy-dissipating elements, and in welded members anywhere in the seismic force-resisting system. Weld metal in primary connections is also subject to toughness requirements when $I_E F_a S_a(0.2)$ is greater than 0.35. Temperatures for Charpy V-notch testing are specified in the Standard.

In S16-14, welds that are expected to sustain high demand under seismic loading are designated as demand-critical welds, and additional notch-toughness requirements are specified for them. The requirements adopted in S16-14 are consistent with those in ANSI/AISC 341-10 (2010) and AWS D1.8 (2009). Weld metals used for demand-critical welds in clad and heated structures, where the service temperatures seldom drop below +10°C, are required to meet a minimum average Charpy V-notch impact test value of 54 J at +20°C. The 10-degree temperature difference between test and service temperatures accounts for the severity in strain rate of the impact test, etc. For structures exposed to lower service temperatures, the point-in-time service temperature is taken to be 10°C above the 2.5% January design temperature specified in the NBCC. The Standard also permits the test temperature to be 10 degrees warmer to account for the strain rate difference, etc. More stringent test conditions must be considered when more critical service temperatures are expected. For example, for a cold storage structure whose service temperature is lower than the above-mentioned point-in-time temperature, the minimum test temperature should be lower, i.e. 10 degrees above its service temperature.

Lamellar tearing represents a brittle and undesirable failure mode. Welded T-joints and corner-joints must be designed and detailed to minimize the probability of this failure mode in accordance with CSA W59.

27.1.6 The requirements for bolted connections ensure that friction plays a role in load transfer and that too rapid a slip into bearing is avoided. For joints designed as bearing-type and in which bolts are pretensioned, this friction exists if (i) Class A surfaces or better are provided, or (ii) the slip resistance equivalent to (i) is provided by increasing the number of bolts, bolt size, bolt strength, or any combination thereof.

If beam-to-column connections are demonstrated by means of physical testing to meet the various deformation requirements for different categories of moment-resisting frames and eccentrically braced frames, then the requirements of this clause can be waived.

27.1.7 In order to ensure the desired hierarchy of yielding, the relative strengths of dissipating and non-dissipating structural elements must be known. This requires knowledge of the actual, or probable, yield stresses. The specified minimum yield stress must be used when computing the resistance of the non-dissipating elements, whereas the probable yield stress is used in estimating the loads arising from yielding elements. The probable yield stress may be obtained from coupon tests on the same heats of the materials used in the construction or, since the material will not in general be available at the time of design, may be estimated by use of the factor R_y given in this clause. The imposed minimum value of 385 MPa implies a high R_y value for lower-yield steels in common use until quite recently and is due in part to the use of multi-grade material in recent years, and also to the uncertainty of the actual yields achieved in earlier grades.

For W-shapes, similar ratios between expected and nominal yield strengths are observed for the flanges and the web and, hence, the same R_y value can be used for the entire cross section. Surveys by Schmidt and Bartlett (2002) and by Liu *et al.* (2007) showed that HSS exhibit higher characteristic-to-nominal yield strength ratios compared to W-shapes. Furthermore, the ratio for HSS generally increases when the perimeter-to-wall thickness ratio is decreased, i.e. larger ratios for more compact sections such as those required for the energy-dissipating elements. A higher R_y value elevating the product $R_y F_y$ to 460 MPa is therefore specified for HSS in CSA S16. This value corresponds to the mean yield strength value of the data collected by Schmidt and Bartlett (2002). CSA S16 does not provide any requirements for ASTM A53 pipes used as energy-dissipating elements such as bracing members. If this material is used, appropriate R_y values should be considered (see AISC 2010a).

The error in using the minimum specified value rather than the probable value when calculating width-thickness limits is acceptably small. However, a minimum value of F_y is set at 350 MPa for use in this calculation due to the common use of multi-grade steels in recent years. A reduced value of 300 MPa is permitted to be used to verify the width-to-thickness ratios of angles when the specified yield strength is equal to or less than 300 MPa.

27.1.8 In the computation of second-order effects, a linear amplification is given following the procedure outlined in the Structural Commentaries to the National Building Code of Canada. This method differs from that given in Clause 8.4.2 since the displacements, under which this provision ensures that the prescribed lateral resistance can be developed, result from the anticipated inelastic seismic deformations. Notional loads and P-Δ effects must be considered for the design of the energy-dissipating elements. They need not be considered for the design of the non-dissipating elements (e.g. beams and columns in concentrically braced steel frames) as the lateral load effects on these elements are limited by the capacity of the dissipating elements. In case the dissipating elements are overstrong and the seismic loads corresponding to $R_d R_o$ = 1.3 (or 1.0, as applicable) are used to size the non-dissipating elements, notional loads and P-Δ effects must be included in the analysis.

27.1.9 Regions where large inelastic strains are expected to occur in the SFRS are designated as protected zones. Protected zones include plastic hinging regions in moment frames, links of ductile eccentrically braced frames, braces in concentrically braced steel frames, etc. They are defined in the Clauses applicable to the designated system. Within these zones, discontinuity, rapid change in cross-section or material embrittlement caused by welding, cutting or penetration at the fabrication plant or the construction site may lead to premature fracture under cyclic inelastic response. Hence, unless engineered or part of test assemblies satisfying the specified performance, welded, bolted, screwed or shot-in attachments for perimeter edge angles, exterior facades, partitions, ductwork, piping or other construction shall not be placed within protected zones. For instance, welded shear studs and decking attachments that penetrate the beam flange shall not be placed on the beam flanges within the protected zone, unless approved by the Designer. Decking arc-spot welds required to secure decking are, however, permitted. Fabrication or erection operations that cause discontinuities are also prohibited in protected zones. Discontinuities accidentally created within protected zones, such as tack welds, erection aids, air-arc gouging and thermal cutting shall be repaired as required by the Designer. Guidance on acceptable repair methods can be found in CSA-W59.

The extent of the protected zones must be identified on the design documents. The information can be conveyed to the construction site by means of coating and labels on both faces with large lettering pertaining to the restriction on attachments and penetrations. Where the protected zones are subsequently covered by fire protection material, provision for visible labels after the application of fire protection should be considered.

27.2 Type D (Ductile) Moment-Resisting Frames, R_d = 5.0, R_o = 1.5

27.2.1 General

27.2.1.1 Type D moment-resisting steel frames have traditionally been designed to develop inelastic deformations at beam-to-column joints, either by plastic hinging in the beams or columns, or by inelastic shear deformations in the panel zone of H-shaped columns (bent about the strong axis). However, numerous welded moment frames have suffered connection fractures as a result of the 1994 Northridge and 1995 Kobe earthquakes, calling for a comprehensive review of that design practice. Extensive revisions to Clause 27.2 were introduced in the 2001 edition of S16 based on the research findings and engineering consensus reached following these two earthquakes (FEMA 1995, 1997, 2000).

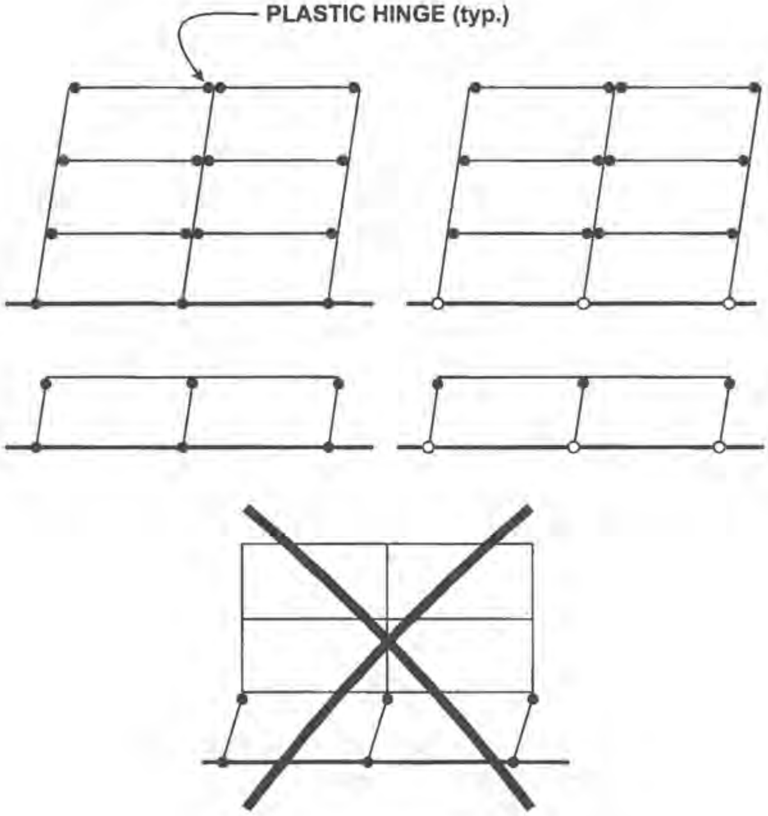


Figure 2-66
Desirable Beam-Sway Collapse Mechanism
and Undesirable Column-Sway Mechanism

The current design philosophy requires that plastic hinges develop at predetermined locations within the frame, such as in beams away from the face of the columns. This is possible either by locally strengthening the beams near the columns (by haunches, cover plates or other methods), by locally weakening the beams at selected plastic hinge locations some distance from the columns, or by using special detailing that ensures ductile response. Annex J references documents giving specific details that will achieve the necessary ductility. Other systems are permissible if demonstrated by physical tests to be capable of providing the performance specified in later Clauses.

Whenever column bases are designed to have a flexural resistance, plastic hinges are necessary to permit development of the preferred plastic collapse mechanism (see Figure 2-66 – Desirable beam-sway collapse mechanism, and undesirable column-sway mechanism). In multi-storey applications, column plastic hinging is otherwise undesirable, as it may lead to formation of a storey plastic mechanism with undue ductility demands compared to other storeys. However, column plastic hinging is permitted at the top of a column stack (usually under a roof beam), as this behaviour is not expected to result in excessive localized damage. This hinging scenario can represent an appropriate solution when deep beams or trusses are used for the roof. Special requirements (see 27.2.3) must be satisfied when column hinging is expected.

Although the panel zone provides excellent ability to absorb energy by means of cyclic plastic shearing deformations (Popov *et al.* 1986), large inelastic deformations there result in large curvatures in the column flanges. For joints in which the beams are welded to the columns, these curvatures may precipitate cracking of the beam weld at that location. Panel zone yielding without considerable concurrent beam yielding is generally not desirable for these connections, and the current provisions limit this behaviour except when using a connection detail for which panel zone yielding has been found appropriate by testing. Note that optimization of panel zone and beam yielding is difficult, given the inherent statistical variability in the steel strength of beams and columns.

27.2.1.3 In evaluating the relative strengths of the structural components at the joint, an estimate should be made of the contribution of the slab. Clause 27.2.8 specifies that studs are not permitted in beam plastic hinge regions. Thus the contribution of the slab can be neglected if specific construction details are provided that prevent the slab bearing on the columns. In the absence of such details, under positive bending moment, the ultimate compressive resistance of the concrete can reach values of $1.3f'_c$.

27.2.2 Beams

In moment frames, beams are nearly always bent in reverse curvature between columns unless one end is pinned. The lateral bracing requirements here assume that the seismic moment at one end of the beam is M_p, and that zero seismic moment exists at the other end; to these the gravity load moments must be added.

Lateral bracing of beams near the plastic hinge location should be provided according to the configuration, strength, and stiffness considered in the tests referenced in the commentary on Clause 27.2.5. Attachments in the area of anticipated plastic behaviour are in general proscribed (see Clause 27.2.8).

27.2.3 Columns (Including Beam-Columns)

27.2.3.1 The width-thickness requirements for columns that develop plastic hinging follow from Clause 27.2.1.2. The axial load in the column is also restricted because the rapid deterioration of beam-column flexural strength (when high axial loads are acting) limits the ductility.

When columns are expected to develop plastic hinging, structural elements adjacent to the column plastic hinges must be able to resist the full plastic moment of the columns. For example, at the base of a column, the intended performance would not be achieved if anchor rods yield instead of the column itself. Due to anchor rod elongation, column base fixity would be lost after a few cycles, resulting in a considerable reduction in base shear resistance and storey stiffness, and the ensuing risk of an undesirable localized storey-collapse mechanism at the first level.

27.2.3.2 Columns may accumulate forces from several yielding elements, and these must be considered.

The equation presented in this clause is intended to minimize plastic hinging in columns and promote plastic hinging of beams. Hence, it does not apply to columns in cases where plastic hinging is expected near the top of the columns. This equation cannot ensure that individual columns will not yield at some time during earthquake response, because of the shifting of column inflection points during dynamic response (Bondy 1996), but the extent of this yielding should not be detrimental. This requirement is in addition to the requirements of Clause 13.8.

For the equation presented to be statically correct, equilibrium requires that the moment at the intersection of the beam and column centrelines should be determined by projecting the sum of the nominal column plastic moment from the top and bottom of the beam moment connection (Figure 2-67 – Free-body diagrams to calculate V_h at the plastic hinge location, and moment at face and centre of column). However, this may be conservative for connections

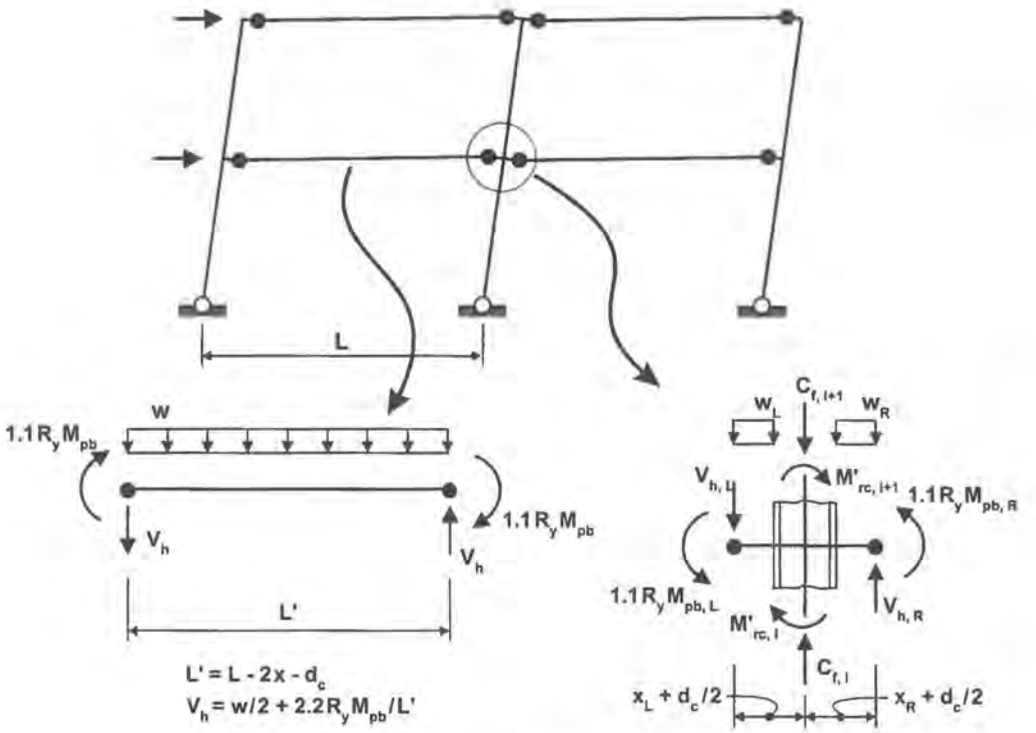


Figure 2-67
Type D Moment-Resisting Frame – Free-Body Diagram

having deep panel zones and/or haunches, and current North American practice permits that the sum of moments at the top and bottom of the panel zone be substituted for the statically correct value.

In addition to this requirement, columns that are expected not to develop plastic hinging must satisfy the requirements of Clause 13.8 under the forces induced by plastic hinging of the beams. In this case, the sum of the beam moments at the centreline of the joints, as defined in Clause 27.2.3.2, can be distributed above and below the joints in the same proportion as the moments obtained from an elastic analysis under the factored seismic loads plus gravity loads. Meeting the requirements of Clause 13.8 may be more critical than Clause 27.2.3.2, particularly for slender columns.

27.2.3.3 The moments in a column when the structure is responding inelastically will not, in general, be known. Conservative estimates of the moment at a splice should be made, based on the possible bending strengths at each end of the column. Because partial-joint-penetration groove welded splices are not ductile under tensile loading (Popov and Steven 1977; Bruneau *et al.* 1987), splices are designed more conservatively, and half-penetration welds on flanges are required as a minimum.

27.2.4 Column Joint Panel Zone

27.2.4.1 Shear force demands on joint panel zones are determined from beam forces acting at the column faces and column forces acting at the levels of the top and bottom beam flanges.

27.2.4.2 The column panel zone has a shear strength greater than the von Mises shear yield value on the web due to: (i) considerable strain-hardening in shear, and (ii) flexural resistance of the column flanges during panel yielding in shear (Krawinkler and Popov 1982). This strength is assumed to be attained at a shear distortion equal to four times the yield shear distortion. This amount of panel zone yielding may be tolerable, provided that plastic hinging first develops in the beams.

Yielding in the panel zone is perceived by some as beneficial, since it reduces the inelastic demand on the beams and provides sharing of energy dissipation. However, some concerns remain for beam welded connections because of the impact of plastic shear distortions and localized column flange bending on the integrity of the beam flange welds. A consensus opinion has not yet been reached. An upper limit of 0.2 is therefore placed on the term $3b_c t_c^2/d_c d_b w'$ to ensure that the panel zone strength is not reached prior to development of the plastic moment enhanced by strain-hardening in the adjacent beams.

The stronger panel zone option is usually more economical. In this case, the von Mises yield criterion ($0.58F_y$ on the entire web of the column) is adopted. The panel zone remains elastic, and special detailing of the panel zone described in the first part of this clause is not warranted. Note that the 0.55 in the shear strength equations is obtained by taking the depth of column web equal to $0.95d_c$.

27.2.4.3 Requirements are provided to ensure stable response of the panel zone. Doubler plates can be used to achieve the required shear strength for the panel zone. Special detailing requirements must then be satisfied to ensure that the shear capacity of the doubler plates can be mobilized and proper load paths exist between the beams, columns, doubler plates and continuity plates, when present.

27.2.5 Beam-to-Column Joints and Connections

27.2.5.1 Extensive research was initiated following the Northridge earthquake to identify the reasons that led to the numerous observed beam-to-column connection fractures and to formulate new connection design requirements. The result of this large research endeavour is a database of connection types that have been experimentally proven able to provide satisfactory seismic performance, with specific information regarding configurations, details, quality control, and other requirements. Minimum performance criteria under reversed cyclic loading are specified in the Standard. The designer must either:

(a) Use connections designed and detailed in accordance with the CISC Moment Connections for Seismic Applications (CISC 2014), or

(b) Use test results in compliance with this Clause. A protocol for such testing is referenced in Annex J.

The 2014 edition of the CISC guide for moment connections provides design and detailing provisions for four different connections: reduced beam section (RBS), bolted unstiffened end-plate (BUEP), bolted stiffened end-plate (BSEP), and bolted flange plate (BFP) connections. The latter has been added to the previous edition of the guide. Provisions are now also given for built-up column shapes, and the ranges of acceptable beam and column shapes have been adjusted to reflect new available data. Additional information on pre-qualified beam-to-column connections can be found in AISC (2011).

27.2.5.2 The beam web connection shall have a resistance adequate to carry shears induced by yielding at the beam-to-column joint.

27.2.6 Bracing

Bracing of both top and bottom beam flanges as well as column flanges shall be considered. If no transverse beams exist at a level, the column must be designed to provide restraint to yielding beam flanges in the manner indicated in (d).

27.2.7 Fasteners

Consideration should be given to the fact that plastic hinge locations will not be predicted by an elastic analysis of the frame.

27.2.8 Protected Zones

Clause 27.2.8 describes the zones that must be protected in ductile moment-resisting frames. This clause should be applied in conjunction with Clause 27.1.9 where limitations applicable to protected zones are defined. In moment-resisting frames, protected zones include segments along the beams and columns where plastic hinges are expected to occur. Limitations on cross-section changes in beam plastic hinges are also specified.

27.3 Type MD (Moderately Ductile) Moment-Resisting Frames, $R_d = 3.5$, $R_o = 1.5$

The ductility-related force modification factor of 3.5 is sufficiently large for Type MD moment-resisting frames to develop large cyclic inelastic deformations during earthquakes. For that reason, and because larger structural members will result from the larger design forces considered, most requirements of Clause 27.2 are applicable. However, beam-to-column joints need only be able to develop a minimum drift angle rotation of 0.03 radians (compared with 0.04 for comparably designed Type D moment-resisting frames). This reduced deformation requirement may be useful when a tested connection fails to reach the 0.04 requirement. A greater advantage of this clause may, however, consist in adopting the relaxed provisions of 27.3(a) and (b) in combination with the higher design load. Practical applications of this system include moment frames in moderate seismicity regions where added frame stiffness is required to satisfy $U_2 \leq 1.4$ (Clause 27.1.8) or wind effects.

27.4 Type LD (Limited-Ductility) Moment-Resisting Frames, $R_d = 2.0$, $R_o = 1.3$

This system can accept limited yielding in beams, columns or joints. Panel zone design follows Clause 27.2.4.2, and thus only limited yielding is expected. These frames are subject to restrictions on height and seismic demand level. They are restricted to 60 metres and 30 metres in height for regions of moderate and high seismicity, respectively. In addition, the strong-column/weak-beam design concept applies to buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) greater than 0.55 and buildings taller than 60 metres (permitted in low seismicity regions only). However, probable plastic beam moments without strain hardening effects ($R_y M_{pb}$) may be considered in the strong-column design, because of limited beam yielding in this system as compared to that in the more ductile categories. To accommodate yielding, sections must be Class 2 or better.

It is anticipated that in many cases joint details will conform to traditional forms of construction used for moment-resisting frames. Clause 27.4.4.2 provides design and detailing requirements for joints with beams welded directly to the flanges of I-shaped columns. Connections that either can accommodate an interstorey drift angle of 0.02 radians, following tests as discussed in Annex J, or are in compliance with CISC (2014) may also be used.

Practical applications of this system include moment-resisting frames in moderate seismicity regions where added frame stiffness is required to satisfy drift limits, $U_2 \leq 1.4$ (Clause 27.1.8) or wind effects, and certain low-rise buildings in higher seismicity areas.

27.5 Type MD (Moderately Ductile) Concentrically Braced Frames, $R_d = 3.0$, $R_o = 1.3$

27.5.1 General

Type MD concentrically braced frames are designed to dissipate energy essentially by yielding of the bracing members. Energy dissipation occurs under brace elongation, inelastic buckling of the braces, and inelastic bending when the braces are subsequently straightened. In low-rise V-brace or chevron-brace frames, energy can also be dissipated through limited bending of the beams at the brace intersection point.

27.5.2 Bracing Systems

27.5.2.1 General

Three bracing configurations are explicitly provided for in the braced frame category, and a maximum building height is specified for each. Multi-storey concentrically braced frames have limited capability of distributing vertically the inelastic demand after buckling and yielding of the braces have developed at a given level. Lateral overstrength resulting from the inherent difference in capacity between tension and compression braces acting in pairs serves to prevent the concentration of inelastic demand (Lacerte and Tremblay 2006). The continuity of the columns which, when provided as specified in Clause 27.5.5.2, provides sufficient reserve strength and stiffness, also helps mitigate the formation of a weak storey response and dynamic instability under severe earthquakes (MacRae *et al.* 2004, Chen *et al.* 2008).

The tendency to instability is more pronounced in tall frames in which the inelastic demand tends to concentrate in the bottom floors, which are the first affected by the ground motion, or in the upper levels due to higher mode effects. Thus, a maximum height is specified for each of the three concentric bracing configurations explicitly provided for in Clauses 27.5 and 27.6.

The provisions of Clauses 27.5 and 27.6 are based on the results of frame behavioural studies using inelastic time-history analysis (Tremblay 2000, Tremblay and Robert 2001, Marino and Nakashima 2006). The buildings studied were regular in form with uniform storey height varying between 3.5 and 4 m. Frames with heights up to 80% of the height limits as specified in NBCC can be expected to perform satisfactorily with no further inelastic analysis needed. Those within the height range of 80% to 100% of the NBCC limits are required to be designed for additional seismic forces as stipulated in Clauses 27.5.2 and 27.6.2 for each respective braced frame configuration. The additional seismic forces need not be considered for determining deflections. Taller buildings, notably those with significantly greater storey heights or other systems (e.g. bracing combined with moment-resisting beam-to-column connections), may require further study, and such systems can be investigated using inelastic time-history analysis. Alternatively, it would be necessary to demonstrate that each storey possesses a reserve of strength and stiffness at the drifts expected under the inelastic response, to prevent a concentration of inelastic actions.

Judgement must also be exercised when the geometry of the frame deviates significantly from the uniform configuration considered in the referenced studies. For instance, industrial buildings or hangars in which the bracing system in any one level includes a stack of two or more bracing panels may be prone to concentration of the inelastic demand in a few bracing members. Such configuration is only permitted for Type LD braced steel frames, and special requirements apply, as described in Clause 27.6.6.

Knee bracing and K-bracing are excluded from the Type MD braced frame category, because plastic hinging that will develop within the clear length of the columns may lead to their instability. Braced frames consisting of more than one X-bracing panel are permitted for Type LD braced frames, as described in the subsequent section.

27.5.2.2 Proportioning

In order to achieve symmetric inelastic response, the storey shear resistance in opposite directions should remain equal or nearly the same under the design earthquake. Because the capacity of a concentrically braced frame after buckling of the braces is mainly governed by its tension braces, the requirement is based on the storey shear resistance provided by the tension-acting braces in each direction. In order to avoid excessive torsional response in the inelastic response, this requirement must be met in each vertical plane of braces and in both orthogonal directions.

27.5.2.3 Tension-Compression Bracing

In tension-compression bracing systems, braces in each vertical plane are designed to resist their share of factored tensile and compressive forces based on the analysis. These braces typically act in pairs as is the case of single-storey X-bracing, two-storey (split) X-bracing, chevron bracing, or V-bracing configuration. Tension-compression bracing also includes configurations consisting of an odd number of braces, provided that they satisfy Clause 27.5.2.2 in every plane of bracing at every level. Compared with the tension-only system, the stockier braces in this system provide greater post-buckling capacity and stiffness. This, combined with the stiffness provided by continuous columns, has been shown to provide stability in frames up to about 32 metres in height (Tremblay 2000, Tremblay and Poncet 2007, Izvernari *et al.* 2007). Therefore, Moderately Ductile tension-compression frames that are within 40 metres in height, as permitted in NBCC, but exceed 32 metres, should be designed for higher forces as required in this clause.

27.5.2.4 Chevron Bracing

The commentary to Clause 27.5.2.3 also applies to this Clause. Chevron bracing, in which the braces (which may be either both above the beam or both below it) meet within the central region of the beam, is permitted in the Type MD concentrically braced frame category, provided that the beams in the bracing bents remain essentially elastic after buckling of the bracing members has occurred. Braces in frames with such strong beams can develop their full yield capacity in tension, and the structure exhibits a more stable hysteretic response than when weaker beams are employed. Frames with weaker beams typically experience rapid and significant deterioration of their storey shear resistance and stiffness after buckling of the braces (Remennikov and Walpole 1998a; Tremblay and Robert 2000, 2001). When the tension brace yields in tension, the compression brace at the same level only develops its post-buckling resistance, C'_u, as defined in Clause 27.5.3.4. This case is illustrated in Figure 2-69. When braces are connected to the beam from above, the expected brace compression resistance of the brace, C_u, must also be considered; this condition may be more critical when there is an extremely high gravity load and the beam plastic bending produces a downward displacement at the plastic hinge. For both cases, the beams must be checked as beam-columns resisting the bending moments and axial forces due to gravity loading and these brace loads without the vertical support provided by the braces. Beam-to-column connections must be sized for the same loading conditions.

Limited yielding in the beams does not adversely affect the response of low-rise chevron braced frames, and the brace tension load to be used in the design of the beams in frames up to 4 storeys has been reduced for such frames (Tremblay and Robert 2000, 2001). In such a case, plastic hinging will likely develop in the beams, and the beam connections should then be designed for shear forces associated with the probable bending resistance of the beams.

In both designs, the beams must be adequately laterally restrained at the brace connection point to resist out-of-plane components of the axial load acting in the beams and the braces.

27.5.2.5 Tension-Only Bracing

Designing the braces to resist, in tension, 100% of the lateral loads acting in each direction can lead to a more economical design when lateral loads are low or moderate, or when long braces are used. Tension-only bracing is not permitted in V-or chevron bracing. Although the contribution of these braces when acting in compression is ignored in resisting design lateral loads, the braces must meet the slenderness limit and detailing requirements in Clause 27.5.3, and the compression loads they can deliver must be accounted for in the design of connections, beams, and columns (see Clauses 27.5.4 and 27.5.5). Because the braces are generally less stocky as compared to tension-compression braces, this system exhibits less energy dissipation capacity, and larger inelastic deformations are therefore expected. Every column in the building is required to be fully continuous in order to resist in bending the concentration of inelastic demand in a single storey. It has been shown that frames up to about 16 metres in height perform satisfactorily (Tremblay 2000). Moderately Ductile tension-only braced frames that are within 20 metres in height, as permitted in NBCC, but exceed 16 metres, should be designed for higher forces as required in this clause. However, other bracing systems may prove to be more economical for frames taller than 3 storeys in height, because erection safety usually dictates field splices for column tiers spanning more than 3 storeys.

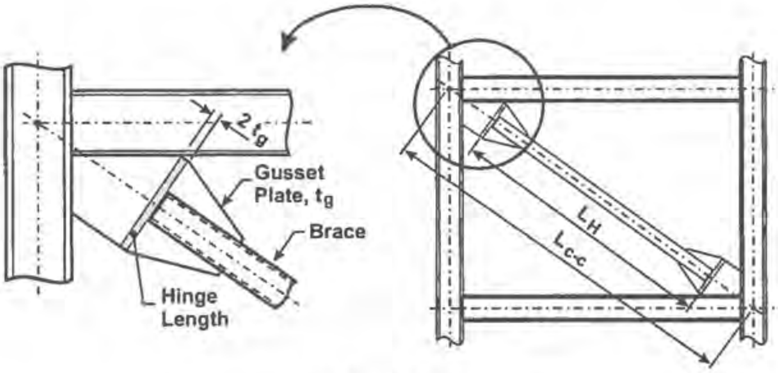
27.5.3 Diagonal Bracing Members

In X-bracing, one of the braces is usually built from two segments and inter-connected at the brace intersection. By selecting both brace segments from the same heat of steel, concentration of yielding in the weaker segment and the potential for premature brace fracture can be avoided.

In most cases, including tension-only systems, the post-buckling capacity of braces is necessary to ensure stability, and therefore, in all these systems, the slenderness limits specified in this clause apply to braces in all Type MD concentrically braced frames, including tension-only systems.

27.5.3.1 The energy dissipation capacity of bracing members under cyclic inelastic loading increases when the effective slenderness ratio, KL/r, is decreased (Jain *et al.* 1980, Popov and Black 1980, Tremblay *et al.* 2003, Lee and Bruneau 2005), and maximum brace slenderness has traditionally been specified to control the dynamic response of braced frames. Bracing systems with slender braces designed to act both in tension and compression have, however, significant lateral overstrength due to the difference that exists between the compressive and tensile capacities of the braces. This overstrength permits the maintenance of a stable inelastic response under severe earthquakes, and for this reason it is possible to allow a brace slenderness limit of 200 for Type MD frames. This limit still provides a minimum energy dissipation capacity that allows the use of tension-only braces in low-rise structures. Past test programs showed that rectangular and circular HSS bracing members with low slenderness ratios can develop premature fracture at the plastic hinge region (Fell *et al* 2009, Tang and Goel 1989, Tremblay *et al.* 2002, Tremblay *et al.* 2008); a minimum effective slenderness ratio is specified to preclude this undesirable failure mode.

When determining the brace slenderness, the actual support conditions of the braces must be accounted for in determining KL. As discussed later (see Clause 27.5.4.3), the brace end connection detail with a single gusset plate and free hinge zone in the gusset, shown in Figure 2-68(a), has gained wide acceptance in practice. When using this detail, the brace effective length KL for out-of-plane buckling can be taken equal to the length between the hinge locations, L_H. Tests on double-angle braces using that detail have shown that a K factor of 0.5 can be



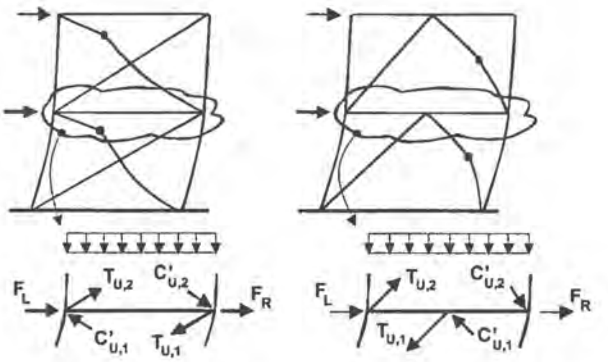
(a) Single Brace

(b) X-bracing

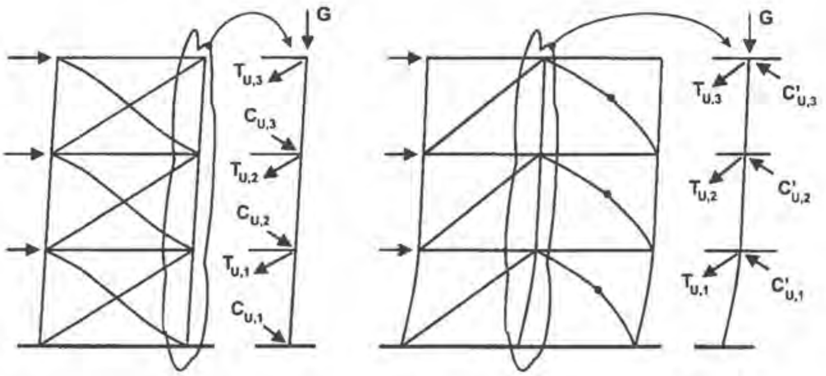
Figure 2-68
Out-of-Plane Buckling of a Brace with Gusset Plates Detailed to Accommodate End Inelastic Rotation

applied to evaluate the brace slenderness for in-plane buckling (Astaneh-Asl and Goel 1984). For X-bracing, when the brace end connections are detailed with single vertical gussets, K can be taken equal to 0.4 and 0.5 for in-plane and out-of-plane buckling, respectively, with L taken as the length between the anticipated plastic hinge locations at the ends of the bracing members (El-Tayem and Goel 1986, Sabelli and Hohbach 1999, Tremblay *et al.* 2003). Caution must be exercised when one of the braces is interrupted at the brace connection point of X-bracing, as this can reduce the stiffness of the tension brace supporting the compression brace and/or lead to local instability of the connecting elements (Kim and Goel 1996, Davaran 2001, Doravan and Hoveidae 2009). These effects can be minimized by reducing the length of the connection or by ensuring minimum continuity at the brace intersection. Additional information on brace effective length can be found in Ziemian (2010).

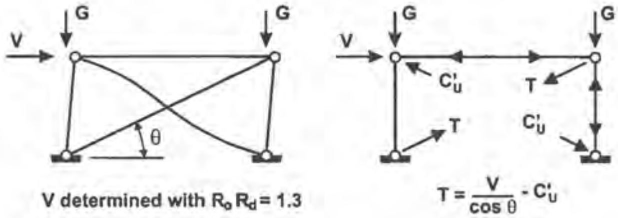
27.5.3.2 Several cycles of inelastic bending are anticipated at hinge location(s) along the bracing members, and limits are imposed on the width-to-thickness ratios of the braces to prevent premature fracture of these members. Physical testing has shown that HSS bracing members exhibit limited fracture life, and relatively more stringent limits are specified for these sections (Fell *et al.* 1989, Lee and Goel 1987, Liu 1987, Sherman 1996, Tang and Goel 1989). Relaxation of width-to-thickness limits is permitted when lower inelastic demand is expected in the braces, such as when slender bracing members are used (buckling becomes essentially elastic) or when the structure is located in a region of low seismicity (Tremblay 2001). The inelastic demand is also less critical in the vertical legs of double-angle bracing members buckling about their plane of symmetry, and less stringent requirements are specified for this case.



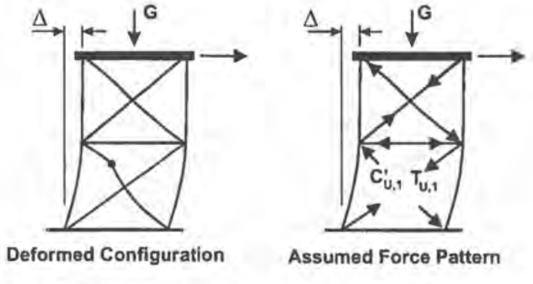
(a) Beams in X-Bracing and Chevron Bracing



(b) Exterior and Interior Columns



(c) Tension Brace Connection – Beam and Column Forces at $R_d R_o$ = 1.3



(d) Columns and Struts When Braces Meet Columns Between Floors

Figure 2-69
Brace Axial Loads for the Design of Members and Connections

27.5.3.3 Buckling of the individual elements of built-up bracing members under earthquake loading may result in high localized inelastic deformations which can lead to premature fracture of the braces (Aslani and Goel 1991). Individual buckling is therefore precluded by limiting the slenderness of the individual components. When buckling of the braces induces shear in the stitch fasteners, these fasteners are expected to transfer in shear the full yield capacity of the smaller brace component upon subsequent straightening of the braces, and the stitch connections must be designed accordingly (Astaneh-Asl and Goel 1985).

Braces with bolt holes at the location of the plastic hinges have exhibited early fracture at the net section, and bolted stitches must be avoided in these regions (Astaneh-Asl and Goel 1984). In determining the governing overall slenderness of the bracing members and the location of plastic hinges, attention must be paid to the actual end fixity and support conditions of the bracing members (see also Clause 27.5.3.1). Plastic hinges in the bracing members will develop approximately at half the distance between supports, i.e., at one quarter and three quarters of the brace length in X-bracing, as well as near the brace end connections if such connections do not permit rotation to develop upon buckling.

27.5.3.4 Probable Brace Resistances

In previous editions of CSA S16, brace expected strength values to be used in capacity design were specified in Clauses related to brace connection design. Recognizing that brace capacities are also used for the design of beams, columns, and connections other than brace connections, etc., a separate clause was introduced in S16-09 to define clearly the expected strength values of braces in tension and compression. A realistic estimate of the expected compressive strength of a brace, C_u, is obtained by multiplying its compressive resistance by 1.2. In this calculation, the probable yield stress of the steel should be used, and the resistance factor does not apply. In tension, the maximum anticipated brace force T_u corresponds to the probable yield tensile strength. The compressive resistance of a brace reduces when the brace is subjected to cyclic inelastic axial loading (Lee and Bruneau 2005), and this post-buckling brace compression resistance can lead to more critical loading conditions for members or connections, such as beams of chevron bracing or interior columns. Figure 2-69(a) shows examples where compression-acting braces in the buckled state (C'_u) produce maximum axial compression in the beam of an X-bracing and maximum bending moment in the beam of a chevron bracing. For the frame in Figure 2-69(b), the exterior columns should be designed for the condition at the brace's probable compressive resistance (C_u), whereas the buckled brace condition (C'_u) should be assumed for the interior. In S16, C'_u is taken as $0.2\,A\,R_y\,F_y$, which corresponds to the value observed in tests at a ductility of 3.0. Tests suggest that higher values can be used for bracing members with very low slenderness, i.e. with λ less than 0.4 (Remennikov and Walpole 1998b, Tremblay *et al.* 2002).

In some cases, braces can be oversized to meet other design criteria such as drift, width-to-thickness ratio, or slenderness limits. For such cases, the brace loads need not exceed the forces induced by a storey shear calculated with $R_d R_o = 1.3$, as specified in Clause 27.1.2. The possibility of brace buckling under that storey shear must be considered in the calculations; the forces in the compression braces are then limited to the probable buckling or post-buckling strength, whichever is more critical, and the load redistribution from the compression braces to the tension braces due to brace buckling must be accounted for when evaluating the forces acting in the tension braces. This is illustrated in Figure 2-69(c) where maximum tension in the tension brace, maximum compression in the beam, and maximum compression in the right-hand-side columns are obtained when the compression brace carries a load C'_u.

27.5.4 Brace Connections

27.5.4.1 Eccentricities in brace connections can lead to damage under cyclic loading and should therefore be kept to a minimum in ductile braced frames.

27.5.4.2 Brace connections must be designed to resist brace axial loads that correspond to the probable buckling strength and tensile yielding strength of the braces. Actual brace end restraint conditions and the presence of intermediate supports must also be taken into account when evaluating the buckling strength of the braces (see Clause 27.5.3.1).

In view of the uncertainty associated with the amplitude of the seismic ground motions and their effects on building structures, connections designed for the upper brace force limit corresponding to $R_d R_o = 1.3$ must be detailed for a ductile mode of behaviour. Details that may be considered to achieve ductile failure modes include gusset plates proportioned for ductility (Cheng and Grondin, 1999), connections that rely on yielding of elements or in which bearing failure of bolts governs (Tremblay *et al.* 2009) in preference to net section fracture or bolt shear failure. Otherwise, the limit on seismic loads must be increased to loads corresponding to $R_d R_o = 1.0$.

The brace tension load can be limited by beam yielding in chevron bracing in which the beams are not designed to carry the full tensile yield load of the braces. In such a case, the brace tension connection load at any level is determined assuming the beam yields while the compression brace still carries 1.2 times its probable nominal compressive strength.

The net section resistance of braces may be based on the probable tensile strength of the brace material, since the load level corresponds to the probable yield stress of the brace. Furthermore, since the principal geometrical parameter of the net and gross sections is identical, the resistance factor may be taken as 1.0. Based on coupon test data assembled by Schmidt (2000), this can be achieved by multiplying the factored net section resistance of the brace by R_y/ϕ, with R_y not exceeding 1.2 for HSS and 1.1 for other shapes. This factor cannot be applied to the factored resistance of other components of the connections such as net section reinforcement plates, gusset plates, bolts, or welds. Information on net section reinforcement for slotted HSS members can be found in Yang and Mahin (2005), and Haddad and Tremblay (2006). Alternative solutions have recently been proposed for HSS brace connections including the modified hidden gap connection by Martinez-Saucedo *et al.* (2008) and structural cast connectors (de Oliveira *et al.* 2008).

27.5.4.3 Buckling of the braces will induce a rotational demand at the brace ends, and the connections must be detailed to avoid any premature fracture at this location. Proper detailing must be provided to allow this rotation to develop in the brace connections or through controlled plastic hinging in the bracing members away from the connections. Note that this ductile rotational behaviour must be allowed for, either in or out of the plane of the frame, depending on the governing effective brace slenderness. If a single gusset plate connection is used, the latter case can be achieved by leaving a clear distance equal to two times the thickness of the gusset at the end of the bracing member (or the connecting elements), as illustrated in Figure 2-68, in order to allow the formation of a hinge in the gusset plate along a line perpendicular to the brace member's longitudinal axis (Astaneh-Asl and Goel 1985). Tearing of the gusset plate will rapidly develop if this geometry is not carefully met. If a plastic hinge is to develop in the bracing member, the connection must have a factored flexural resistance about the anticipated buckling axis equal to $1.1 R_y M_p$ of the bracing member. The Commentary to Clause 27.5.4.2 concerning the factor R_y/ϕ applies also here, except that R_y is not limited to 1.1 when both load and resistance are directly related to the yield stress.

27.5.5 Columns, Beams, and Connections Other than Brace Connections

This clause provides specific requirements for columns, beams, and connections other than brace connections. For brace connection requirements, refer to Clause 27.5.4 and this commentary.

27.5.5.1 Columns, beams, and other connections in the lateral-load-resisting system must be designed to carry the gravity loads together with the effects due to the brace forces that are expected to develop under the design earthquake. Member forces under this condition can be obtained by replacing the bracing members by the brace forces specified in Clause 27.5.3.4. As illustrated in Figure 2-69, in a given storey, it should be assumed that yielding in the tension braces develops simultaneously with either the probable compressive or post-buckling strength of the compression braces, depending upon which case produces the more critical condition for the element being designed. For tension-only systems, the compressive resistance of the braces should not be ignored. In any case, the brace forces need not exceed those associated with a storey shear corresponding to $R_d R_o = 1.3$ (including load redistribution due to brace buckling).

In multi-storey structures, the likelihood of having all the bracing members reaching their full capacity at the same time diminishes as the number of storeys above the level under consideration becomes large. In X-bracing (or split-X bracing), this can be accounted for in determining axial forces in columns by using statistical combinations of the brace-induced loads that have been proposed in the literature (Redwood and Channagiri 1991, Lacerte and Tremblay 2006, Richards 2009). When the axial force in a column is due to brace buckling only, as in chevron bracing with the braces framing below the beams, this reduction is less important, and all braces must be considered as buckling simultaneously (Tremblay and Robert 2001).

When calculating axial loads in beams, attention should be paid to the lateral load path at the level under consideration.

27.5.5.2 Columns in multi-storey structures are most often continuous over two or more storeys, and the flexural stiffness and strength of these columns contribute to reduce the concentration of inelastic demand in a given storey along the height of the building. This behaviour is now explicitly accounted for in this clause, and the columns must therefore be made continuous to prevent a soft-storey formation unless another system is provided (Tremblay 2000, Tremblay 2003). It should be noted that all columns in the frame, and not only those in the vertical bracing system, are to be treated in this way. In addition, the bending moments that are expected to develop in the columns must be accounted for in design. Non-linear dynamic analyses have shown that these moments reach approximately 20% of the plastic moment of the columns, both for gravity columns and columns in bracing bents. It is permitted to splice columns for axial and shear forces only. In order to maintain structural integrity, every splice in the building must be designed for a shear force assuming double curvature in the columns.

Gravity columns possess some reserve capacity due to the reduced factored gravity loads assumed to be present during the design earthquake, and the bending moments are therefore ignored in their design. Class 3 sections are specified, however, to avoid brittle failure in case inelastic rotation occurs over a short period of time during the earthquake. More stringent provisions are prescribed for columns in braced bays, in view of their primary role in resisting lateral loads and the large axial forces they must sustain due to seismic loading. Class 1 or 2 sections are required, and columns must be designed as beam-columns assuming a moment equal to 0.20 times their plastic moment. In this check, columns must be assumed to be bent in single curvature ($\kappa = -1.0$).

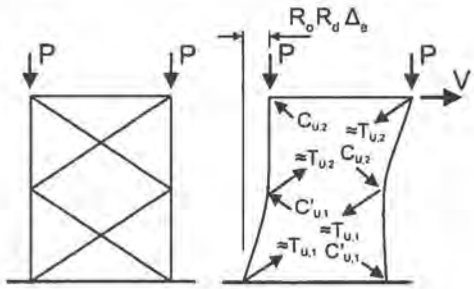
27.5.5.3 See Commentary to Clause 27.2.3.3

27.5.6 Columns with Braces Intersecting Between Horizontal Diaphragms

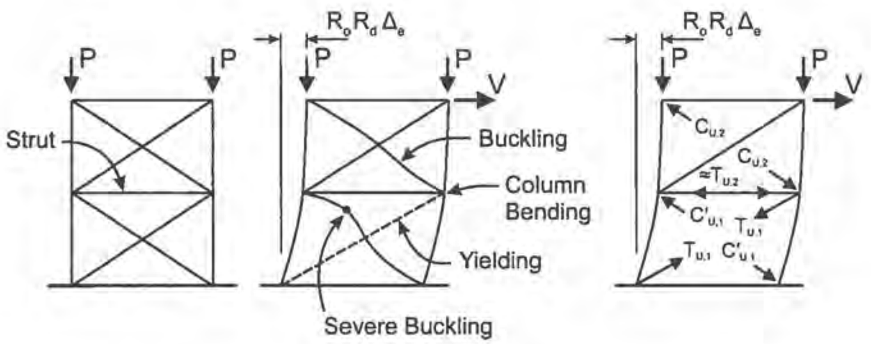
In CSA S16-09, tension-compression bracing with braces meeting at columns between adjacent diaphragm levels was permitted in Type LD concentrically braced frames. This bracing configuration, also referred to as multi-tiered braced frames, is common in tall single-storey buildings or in multi-storey buildings when it becomes impractical to use braces that extend the full storey height. In seismic design, multi-tiered bracing is also advantageous as brace lengths are reduced, which allows smaller braces to be used and, in turn, results in lower capacity design forces for brace connections and adjacent components along the lateral load path. In S16-09, a horizontal strut was required at every tier level to resist the unbalanced lateral brace loads that develop after the compression braces buckle and the forces in the tension braces increase to reach the yield strengths, T_u (Figure 2-70(a)). As shown in Figure 2-70(b), the addition of struts creates a positive continuous load path for the brace forces between adjacent floor levels, or between the roof and the ground for single-storey buildings, so that the full tensile strength of the braces can be mobilized after brace buckling without imposing direct lateral loading on the columns. However, analyses show that brace tension yielding tends to develop in only one tier, even in cases where all tiers have identical bracing members and geometry, as unavoidable initial imperfections and variations in material properties will lead to a weaker tier. This creates non-uniform tier drift demand and induces bending moments in the columns. CSA S16-09 therefore also required that the columns of multi-tiered CBFs resist the bending moments that are induced when brace tension yielding develops in any one tier when the storey drift reaches the anticipated value including inelastic deformation effects, i.e. $R_d R_o \Delta_e$ (where Δ_e is the elastic storey drift under the NBCC base shear for the purpose of calculating deflections). In addition, the columns had to resist concomitant out-of-plane moments from notional loads, applied at every brace-to-column intersecting points, equal to 10% of the forces in compression members (Figure 2-70(c)).

In S16-09, use of multi-tiered concentrically braced steel frames was limited to the Type LD category, and the number of tiers was restricted to limit the ductility demand on the braces located in the critical tier and to prevent premature brace fracture. In light of more recent research findings in this area (Imanpour *et. al.*, 2012a, 2012b, 2013; Imanpour and Tremblay, 2012a, 2014a), S16-14 permits the use of concentrically braced steel frames with braces meeting columns between horizontal diaphragms in Type MD CBF category for frames having up to 3 tiers. The design requirements introduced in S16-09 still apply, except that the out-of-plane notional loads have been reduced to 2% of the axial compression force acting in the column below the brace-to-column intersecting point.

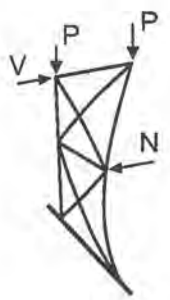
Columns of multi-tiered braced frames should first be designed for the loading condition where all compression-acting braces attain their probable compressive resistance, C_u, while tension braces develop tension forces equal to T_u, as defined in Clause 27.5.3.4. The columns' resistance must then be verified for the combination of axial loads, in-plane bending moments due to non-uniform drifts, and out-of-plane bending moments due to notional loads after the tension brace in the critical tier has yielded, and the storey drift reaches the maximum anticipated drift including inelastic deformations. For this loading case, axial load and in-plane moment demands can be determined using one of three analysis methods: 1) nonlinear dynamic time history analysis, 2) nonlinear static (pushover) analysis, and 3) pseudo-nonlinear static analysis. Concomitant gravity loads must be included in the analysis. The last two methods are generally preferred for design, as the analysis is performed to obtain the conditions at the storey drift $R_d R_o \Delta_e$. In the nonlinear static analysis, brace yielding and buckling nonlinear response must be included in the analysis model, and the rate of brace compressive strength degradation in the post-buckling range must be accentuated to simulate cyclic brace response (Imanpour and Tremblay, 2014b). The pseudo-nonlinear static analysis is simpler because braces are modelled using elastic elements. Elastic static analysis is therefore performed except that the



(a) Without Struts



(b) With a Horizontal Strut



(c) Out-of-Plane Moments from Notional Loads

Figure 2-70
Two-Tiered X-Bracing

tension and compression braces in the critical panel, where brace tension yielding is likely to initiate, are replaced by forces representing their expected tensile (T_u) and post-buckling (C'_u) resistances, respectively. The analysis is performed by applying a lateral load at the frame top until the storey drift reaches $R_d R_o \Delta_e$. Alternatively, that storey drift can be directly imposed on the frame in the analysis. After the analysis, the compression braces in the other panels that resist forces exceeding their buckling strength (C_u) must be replaced by a force C_u, and the analysis must be repeated. For this method, the critical tier of the frame must have been identified previously. It corresponds to the tier that has the lowest storey shear resistance as provided by the braces reaching their probable resistances C_u in compression and T_u in tension. When the frame includes identical tiers, several critical tier scenarios should be considered by slightly varying the probable resistances of the braces in each analysis. This last comment also applies to nonlinear dynamic and static (pushover) analyses.

As an alternative to the above analysis methods, in-plane moments in columns at the maximum anticipated storey drift can be determined by assuming a deformed shape for the frame and by assuming that inelastic lateral deformations occur only in the critical tier. As shown in Figure 2-70(b), the drift in the critical tier is equal to $R_d R_o \Delta_e$ minus the drifts in the non-critical tiers. These non-critical tier drifts can be taken equal to R_o times the elastic tier drifts or may be determined by using a more refined method based on brace axial deformations under the storey shear as limited by the critical tier. Using the deformed frame configuration, column moments can be deducted using any stiffness-based method such as the three-moment equations (Imanpour and Tremblay, 2012).

In Clause 27.5.6, it is assumed that brace tension yielding develops in only one tier over the frame height, which is the common situation. In frames with stiff and strong columns, it is possible that brace tension yielding will be triggered in two tiers before the design storey drift is reached. This behaviour is acceptable and can lead to a most economical design, as the distribution of brace tension yielding over several tiers generally results in reduced in-plane flexural demands on the columns. This response is automatically captured by nonlinear dynamic or static (pushover) analysis. In pseudo-nonlinear static analysis, the behaviour will produce tension brace forces exceeding their probable yield tensile strength in a second tier. In that case, the analysis should be interrupted at the point where the brace force reaches T_u in that tier, which presents the condition for maximum column moments.

Tier drifts beyond 1.5-2.0% may lead to premature brace fracture because of low-cycle fatigue. Past studies showed that well-proportioned multi-tiered braced frames are not expected to develop tier drifts in excess of these values. In cases where excessive drifts are obtained in the critical tier, spreading brace tension yielding in multiple tiers can mitigate tier drifts. This behaviour can be achieved by stiffening the columns or by mobilizing the flexural stiffness of adjacent gravity columns. For the latter, the gravity columns must be connected to the braced frame by means of horizontal struts at every tier level. The gravity columns and the struts should then be designed to resist forces resulting from this interaction (Imanpour *et al.*, 2014).

Columns in multi-tiered CBFs should be restrained against rotation about their longitudinal axis at each tier level, so that the unsupported length for lateral-torsional buckling can be reduced to the tier height. Torsional bracing can be provided by the horizontal (out-of-plane) flexural stiffness and strength of the struts. In that case, the struts must be rigidly connected to the columns against rotations in the horizontal plane, and the torsional stiffness per column can be conservatively taken as equal to $2EI/L$, where I is the moment of inertia of the struts for bending in the horizontal plane and L is the length of the struts. Minimum stiffness and strength requirements for torsional bracing by Helwig and Yura (1999) for inelastic columns can be used to proportion the struts.

27.5.7 Protected Zones

Bracing members are considered as protected zones over their full length, because yielding in tension is expected to occur at any location along the braces. Brace connections are designated as protected zones, as they are likely to sustain high strain and inelastic rotational demands upon brace buckling and under large storey drifts.

27.6 Type LD (Limited-Ductility) Concentrically Braced Frames, $R_d = 2.0$, $R_o = 1.3$

27.6.1 General

Braced frames of this category are designed with an R_d factor of 2.0 and are thus expected to undergo lower inelastic response than Type MD braced frames. However, inelastic response is still restricted to bracing members and beams of low-rise chevron braced frames. The frames must therefore be designed according to Clause 27.5, except that some relaxation is permitted in view of the lower anticipated ductility demand.

27.6.2 Bracing Systems

Frames provided with higher lateral resistance are less prone to soft-storey response, and taller buildings are permitted in this frame category. Frames with heights up to 80% of the height limits specified in NBCC can be expected to perform satisfactorily with no further inelastic analysis needed. Those within the height range of 80% to 100% of the NBCC limits are required to be designed for additional forces stipulated in this clause.

27.6.2.1 Tension-Compression Bracing

This clause applies to all tension-compression bracing configurations, including Chevron bracing systems. Bracing configurations, consisting of pairs of compression and tension braces meeting a column on one side at one or more elevations between horizontal diaphragms, may be used in limited-ductility frames, provided that the specific requirements of Clause 27.6.6 are satisfied.

27.6.2.2 Chevron Bracing

Chevron bracing with no special beam capacity requirements is permitted up to 20 m in this category. During an earthquake, the beams in such frames lose the vertical support provided by the braces and must then be capable of supporting their tributary gravity loads without the help of the braces. Significant plastic hinging is expected in these beams, and beam-to-column connections must be designed to sustain the forces that develop when the probable nominal flexural resistance of the beams is reached (Tremblay and Robert 2000). Chevron braced frames so proportioned exhibit severe deterioration of their storey shear resistance after brace buckling and cannot be used in structures taller than 20 m without risk of soft-storey response. Chevron bracing with a strong-beam design as specified in Clause 27.5.2.4 should be used for these taller structures.

27.6.2.3 Tension-Only Bracing

Compared with Type MD tension-only braced frames, columns in this system are required to be fully continuous and have a constant cross-section over only two storeys.

27.6.3 Diagonal Bracing Members

27.6.3.1 Single- and two-storey braced frames with more slender braces having KL/r up to 300 are permitted in this frame category. Other requirements for ductile braced frames, including minimum brace connection resistance, still apply, however.

27.6.3.2 Limited inelastic compressive strains are expected in braces with KL/r greater than 200, as permitted by Clause 27.6.3.1. Therefore, stringent width-to-thickness ratios do not apply for these braces. The inelastic demand anticipated in frames with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) less than 0.45 is also small, and Class 2 sections are permitted in these locations.

27.6.4 Bracing Connections

Clause 27.5.4.3 need not apply to slender braces in frames located in lower seismic hazard categories, as low rotational demand is expected at the ends of such braces.

27.6.5 Columns, Beams and Other Connections

Abrupt changes in the inter-storey drift angle from one storey to another is not expected in frames of this category when located in low seismicity regions. For such frames, column splice connections as currently fabricated and built in practice should provide sufficient shear capacity to ensure integrity of the gravity columns, and no minimum shear force is prescribed for splices in these columns.

27.6.6 Columns with Braces Intersecting Between Horizontal Diaphragms

Type LD braced frames with braces intersecting columns between diaphragms must be designed in accordance with the requirements for Type MD concentrically braced frames as given in Clause 27.5.6, except that the limit on the number of tiers for Type LD frames is extended to 5 tiers. This relaxation is permitted in view of the less severe concentration of inelastic demand over the frame height as a result of the higher design seismic loads.

27.7 Type D (Ductile) Eccentrically Braced Frames, R_d = 4.0, R_o = 1.5

27.7.1 General

Ductile eccentrically braced frames (EBF) are designed to dissipate energy by yielding of links which form part of the beam in braced bays, and other members of the frame are designed to respond elastically while the links are yielding and strain-hardening. Some common configurations of EBF are shown in Figure 2-71. The load in each brace is limited by the fact that a link is located at one or possibly both ends. The brace is designed to remain elastic under the maximum load the link can sustain, and hence the uncertain load-carrying capacity of compression braces following yield or buckling is not a concern.

27.7.2 Link Beam

Short links will yield in shear prior to flexural hinging at the link ends, whereas long links will yield in flexure before shear. Either mode is acceptable, although short links are easier to design and have somewhat more stable and predictable post-yield behaviour (Kasai and Popov 1986, Engelhardt and Popov 1992). Long links must be Class 1 sections as flexural hinging is expected at link ends, whereas short links may have Class 2 flanges, provided the web is Class 1 (Engelhardt, 2005). The link beam will normally carry high axial forces as well as high bending moments, and the axial forces cannot be neglected in the design. For a general discussion of EBF behaviour, see Popov *et al.* (1989).

For short- and moderate-length links in particular, the web is expected to undergo severe cyclic inelastic action with straining well into the strain-hardening range. For this reason, discontinuities such as openings, splices, and stress raisers such as welded attachments (except stiffeners) must be avoided. Splices within the link are not acceptable and should also be avoided in the outer parts of the link beam near the link ends (with the exception of links attached directly to columns). The webs should be of uniform depth to maintain the same shear capacity throughout the link length, thus avoiding confined yielding.

In earlier research and past applications of the system, link beams were segments of beams made of W-shapes, and design and detailing rules in previous editions of CSA S16 had been developed for W-shape link beams. Based on the work of Berman and Bruneau (2008a), provisions for link beams made of built-up rectangular hollow sections were introduced in CSA S16-09. The use of built-up beam sections allows links to be sized to match closely the design force

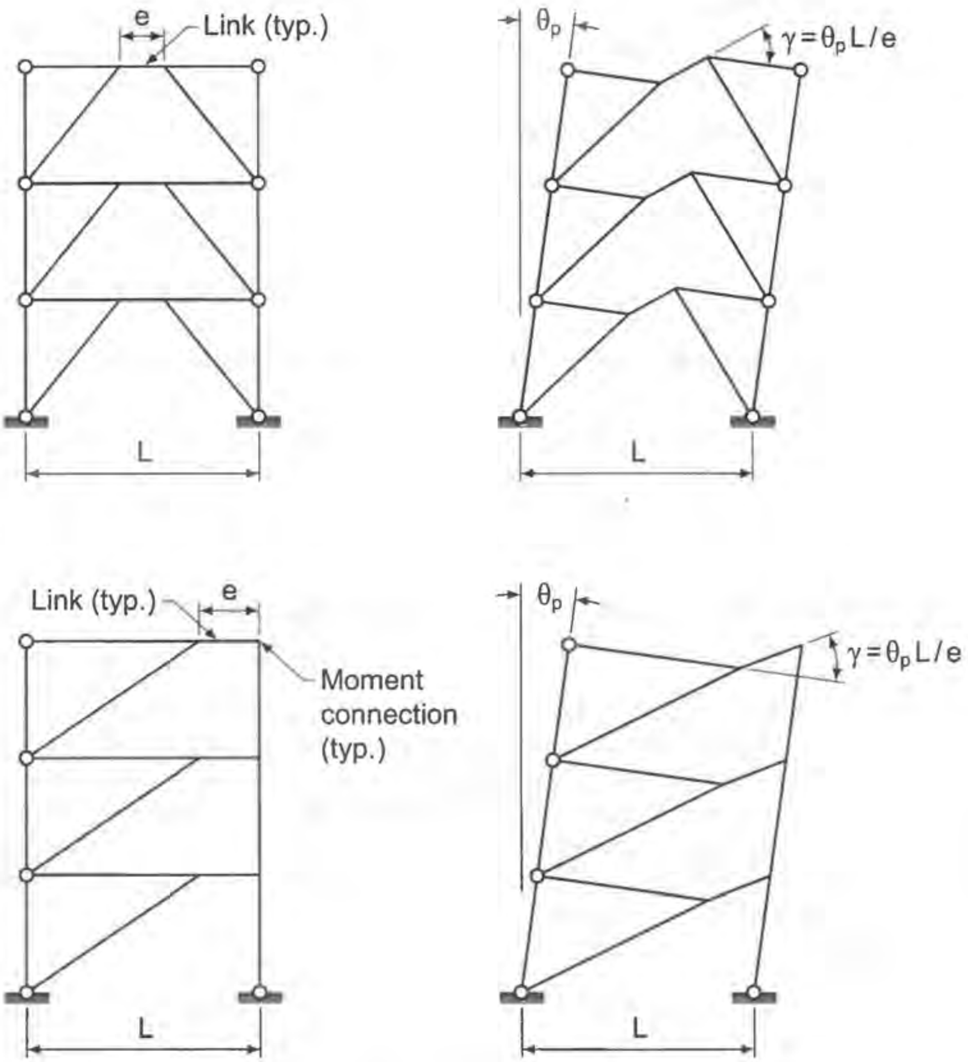
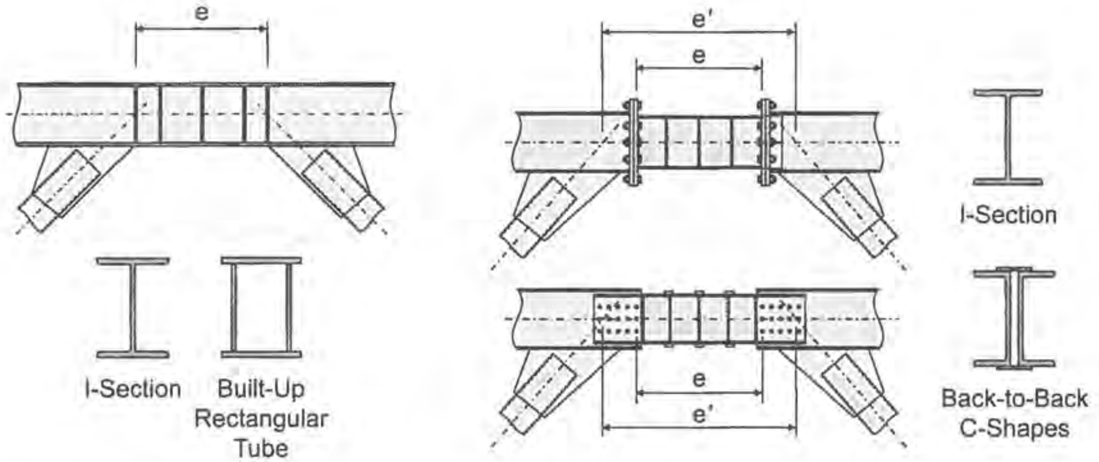


Figure 2-71
Common Configurations of Eccentrically Braced Frames

demand and, hence, to minimize the capacity design force demand requirement. Furthermore, when properly sized, tubular links do not require lateral bracing. This option offers an attractive solution in situations where lateral bracing is impossible or impractical, for example in a braced bent along an exterior column line, or next to an elevator or stairway shaft.

W-shaped and tubular links are segments of the beams (Figure 2-72(a)). Provisions for two types of modular link beams have been introduced in CSA S16-14: I-shaped link beams connected to the beam with unstiffened end-plate moment connections, and links made from back-to-back C-sections with eccentrically loaded web-bolted connections to the beam (Figure 2-72(b)). The C-sections may be channels or I-shapes with the flanges cut flush with the web on one side. Modular links are distinct from the beams, and both elements can therefore be designed independently to obtain structurally effective EBF solutions. As shown in Figure 2-72(b), the link length e can be set shorter than the brace eccentricity e', allowing greater flexibility when proportioning the frame. The system allows for EBF segments to be prefabricated



(a) I-Shaped and Built-Up Tubular Link Sections

(b) Modular Links Made from I-Sections with End Plate Connections and Back-to-Back C-Sections with Eccentrically Loaded Web-Bolted Connections

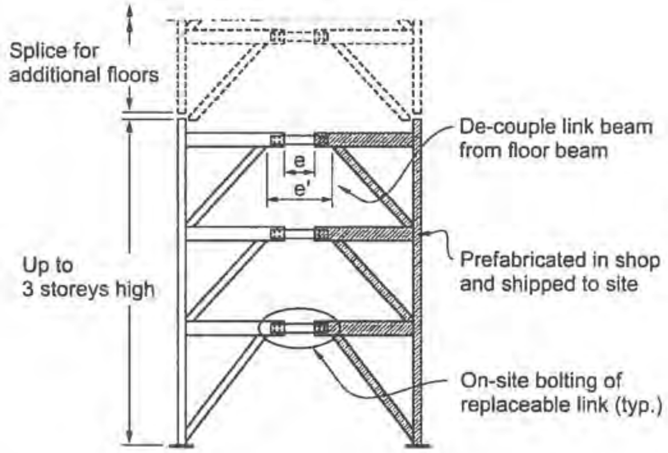
Figure 2-72
EBF Link Beams

and assembled on site (Figure 2-73(a)). Hence, critical connections between beams, braces, and columns can be shop-welded to produce more compact connections. After a severe earthquake, modular links that have sustained large inelastic deformations can be more readily replaced to reduce downtime periods and make the system more resilient.

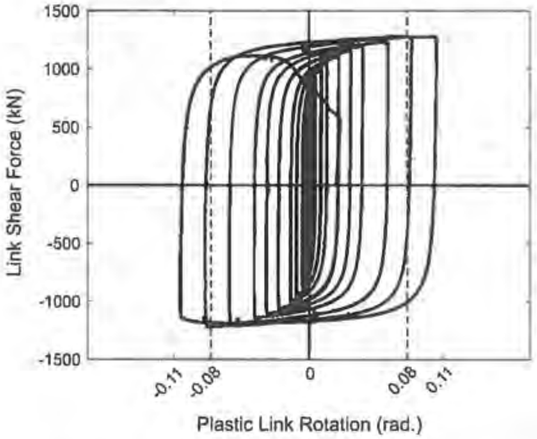
Mansour *et al.* (2011) proposed design procedures for modular links. A key requirement is that the links must be detailed to yield in shear so that the flexural demand on the end connections is reduced and controlled. For the C-section web-connected links, lateral support to the top and bottom flanges is required to prevent lateral-torsional buckling of the individual C-sections. Angles welded to the flanges of each C-section and bolted together through their vertical legs can provide lateral support. Flange reinforcement plates may be added to increase the flexural resistance of web-connected links and to ensure shear yielding.

Mansour et al. experimentally verified their design approach for both types of modular links. The performance of links with end plate connections has also been demonstrated in past tests on traditional EBF links (e.g., Stratan and Dubina, 2004; Okasaki and Engelhardt, 2007). Their seismic inelastic response is therefore similar to that of EBF links that are part of the beams, as shown in Figure 2-73(b). Web-connected link specimens by Mansour *et al.* were sized so that bolt slip and inelastic bearing deformations could develop in the bolted web connections. This resulted in pinched hysteretic response, but the links could sustain larger plastic rotations compared to traditional and end-plate-connected links (Figure 2-73(c)). Replaceability of damaged links and repairability of the concrete floor slab after severe reversed cyclic loading were also reported in their test programs.

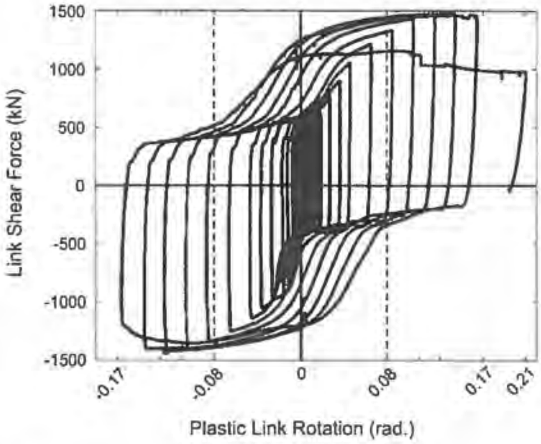
In S16, specific requirements are typically presented in parallel for all link types. No reference to link cross-sections or types is made; however, requirements equally apply to EBFs of either type.



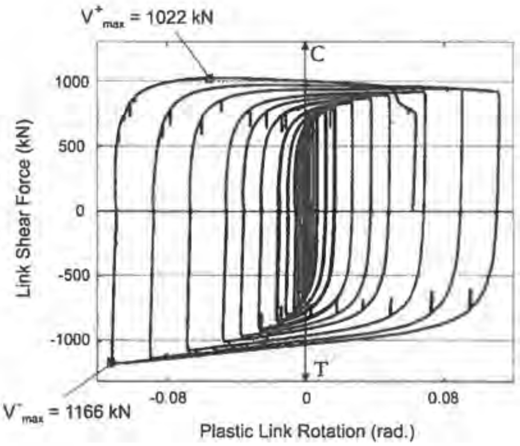
(a) Prefabricated EBF with Modular Link Beams



(b) Hysteretic Behaviour of Individual Modular Link with End Plate Connections



(c) Hysteretic Behaviour of Individual Modular Link with Eccentrically Loaded Bolted Web Connections



(d) Hysteretic Behaviour of One-Storey EBF Sub-Assemblage with End-Plate-Connected Modular Link Subjected to Axial Loading (Adapted from Mansour 2010)

Figure 2-73
Modular Link Beams

27.7.3 Link Resistance

The nominal resistances of the link are defined by taking into account the axial force, but this may be neglected if it is low. The interaction between bending moment and shearing force has been found to be negligible and is in fact neglected. The factored values of these resistances (nominal resistance times ϕ) are used when proportioning link beams for the factored load effects.

Nominal values are also used to determine probable resistances of links and capacity design forces applied on other frame members (see Clauses 27.7.9 to 27.7.13). In S16-14, equations for the probable resistance of links have been moved into a new clause 27.7.3.2. The values are unchanged compared to previous codes, except that the increase in shear strength due to axial tension is now accounted for, as was observed in tests by Mansour *et al.* (2011) and shown in Figure 2-73(d).

Forces due to strain-hardening of a wide-flange link and a modular link are taken as $1.3\,R_y$ times the nominal strength of the link. The 1.3 factor accounts for the increase above the yield value due to strain-hardening, and R_y accounts for the probable yield stress exceeding the minimum specified value. For links with a built-up tubular cross-section, Berman and Bruneau (2008a) reported a higher strain-hardening response than for wide-flange links. Built-up rectangular box links can develop a maximum strength that is typically 11% larger than for wide-flange links, and forces associated with strain-hardening for that link type are taken as $1.45\,R_y$ times the nominal strength of the links.

In S16, the strain-hardening factor of 1.3 or 1.45 applies to all link lengths, i.e. whether yield is related to shear or bending moment.

27.7.4 Link Length

Link lengths are defined for all link types.

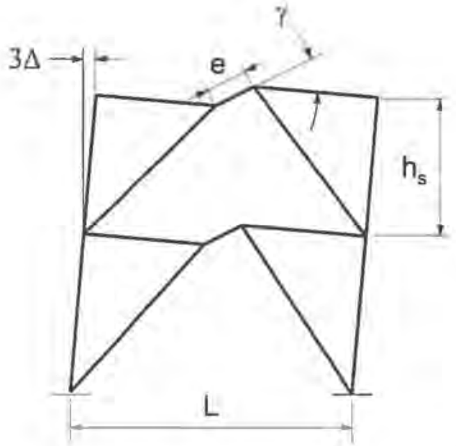
Very short links are proscribed, since they tend to undergo very high shearing deformations and develop very high and unpredictable forces.

Upper limits on the length are needed when the link is subjected to axial force. These are based on Engelhardt and Popov (1989).

27.7.5 Inelastic Link Rotation

The inelastic link rotation must be limited as specified in this Clause, to ensure that the ductile capacity of the link is not exceeded. The limits in CSA S16 were based on earlier test programs on links made of ASTM A36 steel with $F_y = 248$ MPa. Recent tests by Okazaki *et al.* (2005) and Okazaki and Engelhardt (2007) showed that the limits also apply to links made of the higher strength steel ASTM A992 ($F_y = 345$ MPa). The same limits apply to all link types. Until additional test data for web-connected modular links become available, the additional inelastic rotation capacity due to bolt slip and bearing deformations observed for those links is not considered in design. The inelastic link rotation is computed for each storey in the following way:

- Elastic interstorey deflections δ_e are obtained from an elastic analysis of the structure under lateral loads corresponding to the NBCC base shear distributed according to NBCC, for the purpose of calculating deflections (either based on the static method or the distribution obtained from modal analysis).
- These deflections are multiplied by 3 to give an estimate of the maximum inelastic deflections expected under severe shaking.



Inelastic link rotation:

$$\gamma = (L / e)\,\theta_p$$

where:

$$\theta_p = 3\Delta / h_s$$

Figure 2-74
Inelastic Drift Angle vs Link Rotation in an Eccentrically Braced Frame (Rigid Plastic Mechanism Shown)

- Assuming the frame undergoes an interstorey drift corresponding to the calculated inelastic deflections as a rigid plastic mechanism, with deformations confined to the link, the link rotation angle (i.e. the angle between the link and the link beam outside the link) is obtained. As illustrated in Figure 2-74, γ is determined as a function of the inelastic interstorey drift corresponding to the calculated inelastic deflections.

Figure 2-71 shows rigid plastic mechanisms for two common EBF configurations. This procedure gives reasonable results for frames having relatively low height-to-width aspect ratios, such as those shown in Figures 2-71 and 2-74. However, axial deformation of columns due to overturning effect (chord drift) contributes significantly to interstorey drifts in upper storeys for frames with higher aspect ratios but does not affect the link rotations. This chord drift effect can be eliminated by making the columns axially rigid (i.e. modelled with very large cross-sectional areas) in the elastic analysis described above.

27.7.6 Link Stiffeners

27.7.6.1 Links with Wide-Flange Cross-Sections

Full-depth stiffeners on both sides of the web are required to clearly define the end of the link and to transfer the high shearing forces over the full web depth. Requirements for intermediate web stiffeners are based on physical test results and are needed to ensure the ductile performance of the link. For short links, stiffeners control shear buckling of the yielding web, while for long links, stiffeners required near the ends control flange buckling.

Flange-to-stiffener welds of the link end stiffeners are required to develop the full stiffener yield capacity because of the very high forces that must be transferred between the brace and link at a point where high shear and bending loads occur.

Tests by Okasaki *et al.* (2005) showed that fracture of I-shaped link web can be delayed and link inelastic rotation enhanced by increasing the distance between the upper end of the stiffener-to-link web weld and the k-line of link sections. Stiffener welds for I-shaped links must therefore be terminated a distance from the transition radius between the web and the flanges of the link. In any case, it is good practice to terminate the fillet welds at a short distance from the ends of the stiffeners in cyclically loaded structures.

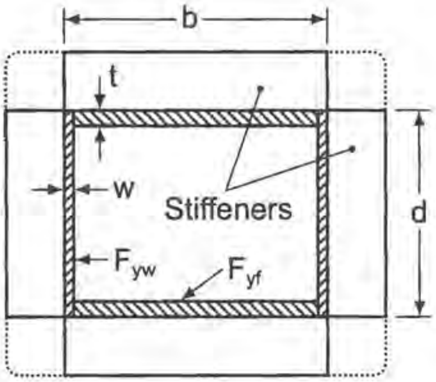


Figure 2-75
Built-Up Tubular Link Cross-Section with Intermediate Stiffener

27.7.6.2 Links with Built-Up Tubular Cross-Sections

As is the case for wide-flange links, full-depth stiffeners are required at the ends of links. These stiffeners are provided on one side of each link web at the diagonal brace connection.

Full-depth intermediate stiffeners are also needed for shear-yielding built-up tubular links ($e \leq 1.6\ M_p / V_p$). As for wide-flange links, the required stiffener spacing depends on the magnitude of the link rotation angle. In CSA S16, only the equation for the spacing needed to develop a link rotation angle of 0.08 radian ($20w - (d - 2t)/8$) by Berman and Bruneau (2005) is given, as experimental and analytical data is only available to support this closer stiffener spacing. A similar expression ($37w - (d - 2t)/8$) has been proposed for a 0.02 radian rotation, but the more restrictive stiffener spacing is required for all links until other data become available. The presence of intermediate web stiffeners was shown to be significant for shear-yielding built-up box links with h/w greater than $0.64\sqrt{E/F_y}$ and less than or equal to $1.67\sqrt{E/F_y}$ (Berman and Bruneau 2008a). For shear links with h/w less than or equal to $0.64\sqrt{E/F_y}$, flange buckling is the controlling limit state, and intermediate stiffeners have no effect.

For links with lengths exceeding $1.6\ M_p / V_p$, compression local buckling of both webs and flanges (resulting from compressive stresses associated with the development of the plastic moment) dominates link strength degradation. This buckling resistance is unaffected by the presence of intermediate web stiffeners. As a result, intermediate web stiffeners are not required for long links, provided that the webs and flanges have a width-to-thickness ratio not exceeding $0.64\sqrt{E/F_y}$, as they are both subjected to large compressive stresses.

The built-up box beams tested and simulated numerically by Berman and Bruneau (2008a) had intermediate stiffeners welded to both webs and flanges. A typical cross-section is shown in Figure 2-75. However, the presence of stiffeners did not influence flange buckling. Whereas web stiffeners in wide-flange links may also provide stability to the flanges (Malley and Popov 1983), this is not the case with built-up box cross-sections. Therefore, for built-up box section links, weld attachment of intermediate flange stiffeners is not required. In particular, intermediate stiffeners may be welded to the inside faces, enhancing architectural appeal and improving resistance to corrosion by reducing the risk of debris accumulation between stiffeners in exposed applications.

27.7.6.3 Modular Links

The requirements for intermediate stiffeners for modular links are the same as for links of continuous I-shaped link beams. End plates of links with end plate connections serve as end stiffeners. Full-depth end stiffeners are required for both C-sections of the replaceable web-connected links.

27.7.7 Lateral Support for Link

The required capacity of lateral bracing for wide-flange links is much greater than is usually the case for beams, because of the anticipated large inelastic deformations and accompanying forces amplified by strain-hardening. No lateral bracing is required for links with rectangular built-up tubular cross-sections, provided that the moment of inertia of the links about the vertical axis in the plane of the EBF is not less than 0.67 times the moment of inertia about the axis perpendicular to the plane of the EBF, as specified in Clause 27.7.2.6.

27.7.8 Link Beam-to-Column Connection

Links are often connected directly to the column face in order to accommodate doorways adjacent to columns. This configuration causes severe straining of the link, connection welds, and column flanges as the link deforms. Tests by Okazaki *et al.* (2006) on link-to-column connections designed and fabricated using pre-Northridge practices showed poor performance. Test specimens with improved welding details alone did not develop the level of inelastic rotation intended in design. Until joint details exhibiting satisfactory inelastic behaviour are developed, link connections must be demonstrated to meet the performance criteria defined in this clause. As for moment-resisting frames, this demonstration can be provided by cyclic tests of full-scale prototypes of the link and column assemblage, following the procedures given in AISC (2005).

If the connection region is reinforced so that a short length of beam adjacent to the column remains elastic under the action of strain-hardened link forces, such demonstration may not be necessary. For this to be acceptable the link must be short, thus limiting the flange forces, and have full-depth stiffeners at the end of the elastic region. In this case, the link ends at the stiffeners.

Link beam-to-column connections can be avoided by adopting a chevron bracing configuration, thereby locating the links away from the columns.

27.7.9 Beam Outside the Link

The forces in the outer beam segment caused by the strain-hardened link forces must be calculated; if reinforcement of the outer beam segment is to be avoided, it will often be necessary to provide a moment-resisting connection between brace and link beam, so that the brace can relieve the outer beam segment of some of the resulting bending moment. It should be noted that this part of the beam will normally also carry a high axial force. When the beam segment considered in this clause is part of the same member as the link, (1) its resistance can be increased by the factor R_y thus, in this case, nullifying any effect of an enhanced yield stress, and (2) the nominal, rather than factored, resistance is used since most of the uncertainties associated with the resistance factor, ϕ, affect both load and resistance identically. This increase in resistance does not apply to frames with modular links, as the links and beams are built from different shapes.

The outer beam segment is subject to bending and axial loads, and must be adequately laterally braced. If a plastic hinge is expected at the link end of this segment, bracing must conform to Clause 13.7(a), which requires bracing within a specified distance of the hinge. While a floor slab will often be present to provide support to the top flange, the bottom flange at this location must also be braced (or torsional restraint provided). The likelihood of a plastic hinge at the link end of the outer beam segment can be determined by examining the distribution of the link end moment between beam and brace according to their relative elastic stiffnesses.

In tests by Mansour *et al.* (2011) on EBF specimens with a concrete floor slab, regularly spaced studs were used on the beam except over the modular links. Cracking of the slab and pulling of the studs closest to the link were observed in the tests, which may affect the integrity of the slab acting as a diaphragm. Studs should therefore be placed at a minimum distance away from the links. This detail will also likely reduce the contribution of the slab to the shear resistance of the links.

27.7.11 Diagonal Braces

The forces used for the design of braces and their connections are consistent with those specified for the outer beam segment in Clause 27.7.9. Although expected to respond elastically, the brace section is restricted to Classes 1 or 2 because of the uncertain stress distribution in the brace-to-beam connection and the possibility of excessive strains in part of the brace cross-section.

27.7.13 Columns

Column design can be based on lower strain-hardening factors than braces and beams since, except for the top several storeys, the cumulative effect of a number of yielding links will be less than the sum of their maximum possible developed forces. A recent study on axial loads in EBF columns is presented by Richards (2009).

Column moments under gravity and lateral loads induced by eccentric shears and moment-resisting beam connections can be calculated. Those arising from variations in inelastic drifts between adjacent storeys cannot be predicted unless an inelastic dynamic analysis is performed. Columns serve an important role by providing an alternative means of resisting storey shear due especially to link yield. Under these conditions, columns can be effective in preventing soft-storey deformations. Column continuity is therefore desirable, and design of the connections should take into account the shear and bending that may occur. On the basis of numerical studies of the dynamic response of a variety of EBF structures (Kasai and Han (1997), Han (1998), Koboevic (2000)), inelastic dynamic analysis may be avoided if the additional end bending moments specified in CSA S16 are combined with the bending moments acting in the plane of the frame and obtained from a linear elastic analysis.

The requirements for column splices containing partial-joint-penetration groove welds follow those for ductile moment frames.

27.7.14 Protected Zone

Links in EBFs are designated as protected zones and shall satisfy the requirements of Clause 27.1.9. Studs are not permitted in links and must be kept at some distance away from links.

27.8 Type D (Ductile) Buckling-Restrained Braced Frames, $R_d = 4.0$, $R_o = 1.2$

27.8.1 General

Buckling-restrained braced frames are essentially concentrically braced steel frames that are constructed with bracing members specifically designed and detailed so as not to buckle. A typical buckling-restrained brace is illustrated in Figure 2-76. The brace has a steel core. A segment of this core is fabricated with a reduced cross-section where axial yielding is expected to develop in both compression and tension. The core is prevented from buckling by means of a restraining system. In the example shown in Figure 2-76, a steel tube filled with mortar is used for this purpose. Unbonding material is placed at the interface between the core and the mortar,

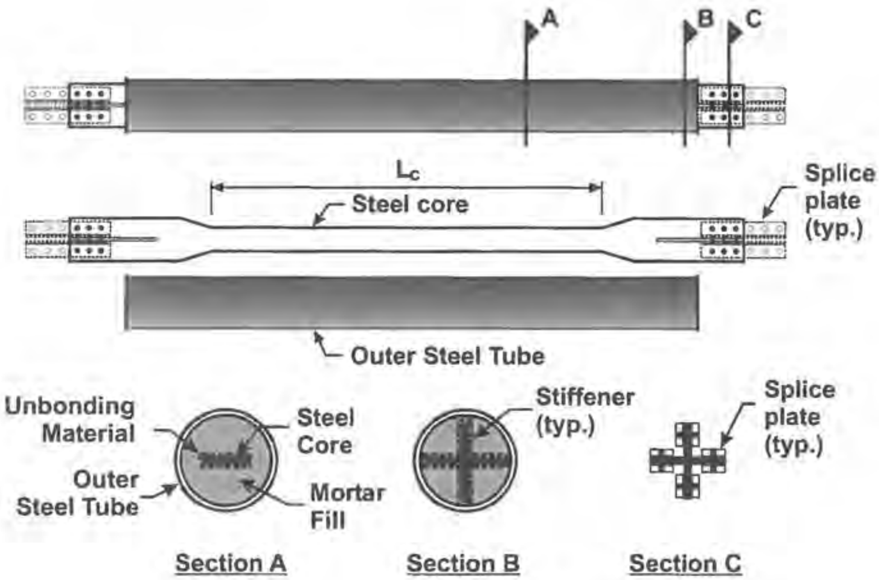


Figure 2-76
Buckling-Restrained Bracing Member (Typical)

so that axial loads are resisted by the core only. In a severe earthquake, energy dissipation is therefore provided by yielding of the brace core in compression and tension. Several alternative designs and systems of buckling-restrained bracing have been developed. More information on the system can be found in Sabelli (2004) and López and Sabelli (2004).

27.8.2 Bracing Systems

When loaded well into the inelastic range, the brace's compression capacity typically exceeds its tension capacity significantly. For this reason, bracing configurations consisting of braces intersecting columns from one side only should intersect the columns at the roof and floor elevations.

Buckling-restrained bracing (BRB) members exhibit very stable hysteretic response with large energy dissipation capacity. However, S16 restricts the application of BRB to frames not exceeding 40 metres in height, except where the specified short-period spectral acceleration ratio is less than 0.35. Concentration of inelastic demand and soft-storey response of taller frames, without beam-to-column rigid connections, have been observed in analytical studies. For these taller structures, use of this system is permitted only when inelastic dynamic stability is demonstrated. Alternative design solutions for tall frame stability have been reported in the literature (Merzouq and Tremblay 2006, Tremblay 2003, Tremblay and Poncet 2007).

27.8.3 Bracing Members

27.8.3.1 Foreign researchers have reported satisfactory performance in a variety of buckling-restrained bracing designs (e.g. Uang and Nakashima 2004, Xie 2005). In the U.S., proprietary BRB products are distributed by specialty suppliers. In Canadian applications, BRB members made of mortar-filled tubes have been fabricated and installed by steel fabricators (Tremblay *et al.* 1999, 2006). Provisions in CSA S16 focus on general performance-based design requirements together with qualification testing requirements for the bracing members. Specific bracing design and detailing required to achieve the specified performance are not given in CSA S16. Typically, BRB manufacturers supply their products in compliance with S16 requirements and project-specific requirements as specified by the structural engineer for the project, such as dimensions and other geometric details, strength, and deformation capacities, etc.

In CSA S16, the compression and tension resistances of a BRB member are assumed equal in magnitude. Hence, the same expression is used to determine the factored axial resistances in tension and compression. Contrary to other structural steel elements, the factored resistance of BRB members can be determined using the yield stress value obtained from coupon testing. This explains the relatively low value of the overstrength-related seismic force modification factor, R_o, specified in NBCC for the system – 1.2 versus 1.3 or greater for the other seismic force-resisting systems. Minimum ductility requirements are specified for the brace core material to prevent premature fracture under inelastic cyclic loading.

Results from nonlinear time-history analyses have shown that axial deformations of buckling-restrained members can exceed significantly the values corresponding to the anticipated total deflections (including inelastic response), as defined in NBCC. Therefore, 2.0 times the NBCC value is required.

Strain hardening is expected to develop following yielding of the core of BRB members. In compression, friction between the core and the restraining mechanism and Poisson's effects are expected to enhance the brace resistance at large deformations. These factors must be considered when determining probable brace resistances for capacity design purposes. These effects vary depending on the type of BRB system and will typically be more important as the cyclic brace deformation increases. Therefore, they must be determined based on qualification tests specified in CSA S16 for the bracing system and deformation demands applicable to the project.

27.8.4 Brace Connections

Brace connections must be designed and detailed to resist brace forces corresponding to the attainment of probable brace resistances. The steel core projections at the ends of BRB members and connection elements must also be capable of resisting the local force demand and of accommodating local deformations that accompany large lateral frame deformations (Tsai and Hsiao 2008, Mahin *et al.* 2004). Information on BRB connections can be found in Berman and Bruneau (2009), Fahnestock *et al.* (2007), and Tremblay *et al.* (2006).

27.8.5 Beams, Columns, and Connections Other Than Brace Connections

As in concentrically braced frames, beams, columns, and other connections should resist the effects of gravity loads, if any, together with forces corresponding to the tensile and compressive brace resistances determined in Clause 27.8.3. Differences in compressive and tensile brace resistances must be accounted for in the calculations.

Columns in multi-storey BRB frames serve to distribute vertically the inelastic demand in the structure. They must therefore be designed to account for the effects due to the redistribution of loads when bracing members develop their probable tensile and compressive resistances.

27.8.6 Testing

Performance of the buckling-restrained brace members and the buckling-restrained braced frame system to be used in a construction project must be verified by means of full-scale qualification cyclic tests. Two tests are required: a test of a brace subassemblage (including brace connection rotational demands at the specified performance) and a uniaxial or subassembly test. The purpose of brace subassemblage tests is to demonstrate that a BRB member and its connections can accommodate cyclic force and deformation demands up to twice the design storey drift. The test is typically performed on a frame specimen that includes the bracing member as well as the beams and columns (or parts of beams and columns) to which it is connected.

End connections of the BRB member must conform, as closely as is practical, to those used in construction, and the specimen must be subjected to the cyclic rotation demand corresponding to twice the design storey drift. The objective of testing individual braces is to verify that they can develop the specified strength and deformation capacities, without buckling, and to determine the maximum expected brace forces for capacity design purposes. Uniaxial testing is typically used for an individual brace test. Brace specimens must conform to the material properties, pertinent design, detailing, and construction features of those used in the construction project.

27.8.7 Protected Zone

The steel cores of buckling-restrained braces, as well as the elements used to connect the brace core to beams and columns, are protected zones and must comply with the requirements of Clause 27.1.9.

27.9 Type D (Ductile) Plate Walls, $R_d = 5.0$, $R_o = 1.6$

27.9.1 General

Plate walls are built of relatively thin infill plates connected at every level to the surrounding beam and column framing members. The infill plates provide for the resistance to storey shear forces, whereas the overturning moment is resisted by the columns. Shear buckling of the infill plates can be controlled by means of stiffeners (Alinia and Dastfan 2007, Chusilp and Usami 2002, Chen *et al.* 2006, Sabouri-Ghomi *et al.* 2008) or by encasing them into concrete walls or panels (Zhao and Astaneh 2004). Although this leads to higher initial stiffness and shear capacity being delivered by the infill plates, recent research and practice in North America have shown that unstiffened plate walls can represent an effective design strategy for resisting lateral wind and seismic loads. Provisions in Clause 27 of S16 have therefore been developed for unstiffened plate wall systems.

Stable hysteretic behaviour under cyclic lateral loading has been demonstrated in several past experimental studies (Tromposch and Kulak 1987, Kulak 1991, Driver *et al.* 1997, 1998a, 1998b, Lubell *et al.* 2000, Berman and Bruneau 2003, 2005b, Qu *et al.* 2008, Vian *et al.* 2009). Much of the cyclic energy imparted to a wall is dissipated by the yielding of infill plates in tension along inclined lines. Upon load reversal, the tension field forces reduce, then the plate buckles under low compressive load, and a new tension field develops in a manner consistent with the shear force in the opposite direction. The advantages of plate walls consist in their high lateral strength and stiffness, which make them very suitable for high seismic applications.

If the beams of a plate wall are attached to the columns using standard, simple shear connections, the hysteretic behaviour is pinched. The behaviour can be improved if moment connections are provided between the beams and columns surrounding the infill plate panels. This Standard distinguishes between Type D (ductile) plate walls, in which rigid frame action contributes to the overall lateral load resistance, and Type LD (limited-ductility) plate walls, in which rigid connections are optional. Type LD walls must satisfy all requirements for Type D walls, except as indicated in Clause 27.10.

For ductile framed plate walls, energy is dissipated during earthquakes by tensile yielding of the infill plates and the development of plastic flexural hinges at the ends of the beams and at the column bases. The provisions in Clause 27 aim at achieving this behaviour. Additional design guidance for the system can be found in Sabelli and Bruneau (2007).

27.9.2 Infill Plates

The infill plate at every level is designed to resist 100% of the factored storey shear force. The equation for the factored shear resistance is based on the shear yielding capacity of the infill plate, assuming full tension field response is developed (Berman and Bruneau 2003, Sabouri-Ghomi and Roberts 1991):

$$V_y = 0.5\, F_y\, w\, L \sin 2\alpha$$

where α is the angle of inclination of the tension field with respect to the vertical. This angle is determined in accordance with Clause 20.4. Of particular interest is the fact that the shear strength of plate walls designed according to this standard has been found not to be sensitive to the inclination of the tension field and that using a single value of 40° throughout the wall height can give accurate predictions of the shear strength of the plates (Shishkin *et al.* 2005). The resistance V_y corresponds to the full yield capacity of the infill plates. Tension stresses in infill plates are not uniformly distributed, and yielding develops progressively upon increasing lateral loads. The capacity V_y is reached only at large lateral deformations under lateral loads equal to 1.1 to 1.5 times the lateral loads initiating yielding in the plates (Berman and Bruneau 2003). In NBCC and S16, the factored resistance of seismic force-resisting systems is typically based on the lateral strength at onset of yielding of the system, the difference between the fully developed lateral capacity and the factored lateral resistance being taken into account by the overstrength-related force modification factor, R_o (Mitchell *et al.* 2003). To make the design consistent with these code assumptions, the factor 0.5 in the equation for V_y is reduced to 0.4 in the equation used to determined V_r in Clause 27.9.2.1.

In capacity design, beams, columns, and connections in plate walls must be designed to resist tensile yielding forces that will develop in the infill plates at large deformations ($R_y F_y w$, shown as ω in Figure 2-77. Also see comments below on Clauses 27.9.3 and 27.9.4). These capacity design forces need not exceed forces corresponding to $R_o R_d = 1.3$ (see Clause 27.1.2).

Engineers and fabricators often select a minimum infill plate thickness to ease workmanship and handling or to maintain reasonable flatness. This practice may lead to capacity design forces that significantly exceed the seismic force demand when the design storey shear is low compared to the factored resistance (upper levels, wall width dictated by architectural layout, etc.). When reasons other than structural requirements dictate the minimum plate thickness, design forces can be reduced by using low yield stress steel for the infill plates (Vian and Bruneau 2004). In that case, the probable yield stress should be taken as an average yield stress, obtained in accordance with CSA G40.20 (see Clause 27.1.7), and the availability of the steel must be verified. Alternatively, circular perforations may be introduced in the infill plate to reduce its capacity. As illustrated in Figure 2-78(a), the perforations must be uniformly distributed and aligned so that diagonal tension strips can form upon buckling of the plates. Clause 27.9.2.3 provides an equation for calculating the factored shear resistance of such perforated infill plates. Minimum detailing requirements are also specified in the Clause.

Infill plates made of thin sheet steel (0.91 mm) have been successfully used, and their adequate inelastic seismic performance has been demonstrated by Berman and Bruneau (2005b). S16 does not include any provision for this application. If this innovative approach is contemplated, caution must be exercised to ensure that the sheet steel used meets the minimum ductility requirements specified in Clause 27.1.5, the fabrication and installation are consistent with methodologies used in supporting research, and reliable welds are consistently provided, etc.

The stiffness of regularly perforated infill plates can be estimated using an effective plate thickness, w_{eff}, given by:

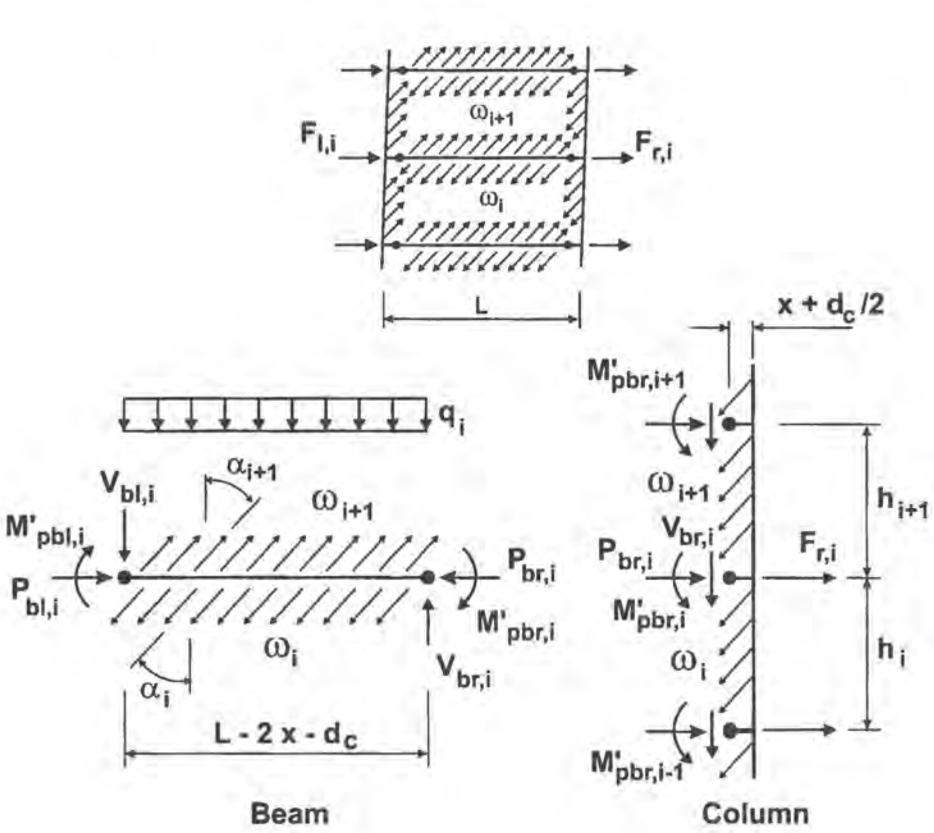
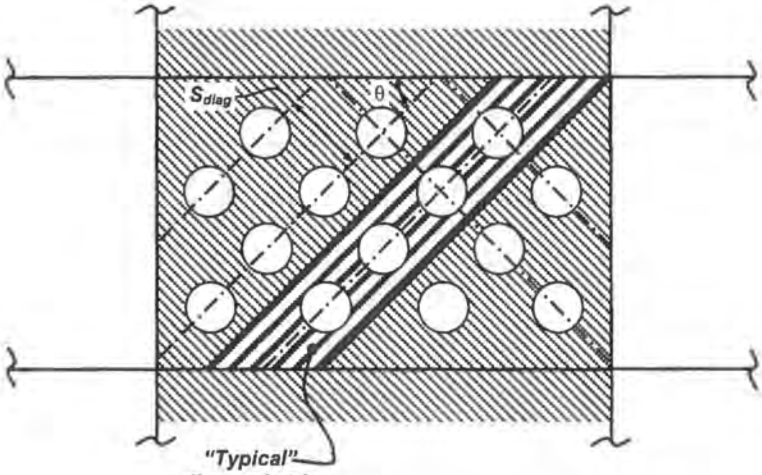


Figure 2-77
Forces Due to Tension Yielding of the Infill Plate and Plastic Hinging at the Beam Ends

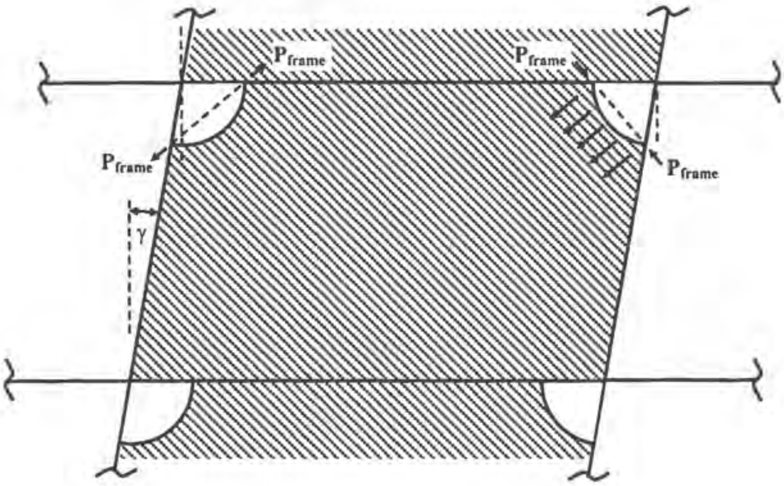
$$w_{eff} = \frac{1 - \frac{\pi}{4}\left(\frac{D}{S_{diag}}\right)}{1 - \frac{\pi}{4}\left(\frac{D}{S_{diag}}\right)\left(1 - \frac{N_r D \sin\theta}{H_c}\right)} w$$

where N_r is the number of perforations along the strips (4 in Figure 2-78(a)), θ is the inclination of the strips, and H_C is the infill plate clear height. Other parameters are defined in Figure 2-78(a). Additional design information and supporting analytical and experimental evidences on the behaviour of plate walls with perforated infill plates can be found in Roberts and Sabouri-Ghomi (1992), Purba and Bruneau (2009), and Vian *et al.* (2009). It is noted that the perforations allow the passage of mechanical and electrical ducts and pipes. In S16-14, a minimum number of lines of perforations that reflects the conditions present in the reference test programs is specified.

An alternative solution for the passage of utilities in infill plates is to utilize quarter-circle cut-outs in the plate corners, as permitted in Clause 27.9.2.4 and illustrated in Figure 2-78(b). In that case, the original shear strength and stiffness of the infill plates are preserved, provided that the cut-outs are suitably reinforced with arching plates and meet geometrical requirements.



(a) Perforated Infill Plate



(b) Infill Plate with Cut-Out Corners

Figure 2-78

Forces acting in the reinforcing arch are caused by a combination of effects, including arching action under tension forces due to infill plate yielding in tension and thrusting action due to change of angle at the corner of the frame. The factored tensile force, T_f, induced in the arch by tension field action in the infill plate can be taken equal to:

$$T_f = \frac{R_y F_y w R^2}{4e}$$

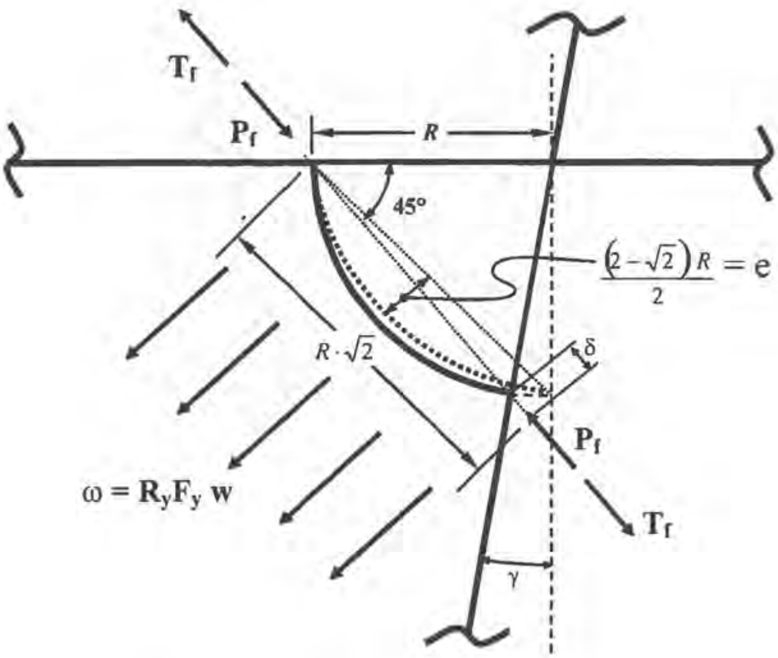


Figure 2-79
Infill Plate with Reinforced Cut-Out Corner – Arch End Reactions Due to Frame Deformations and Tension Field Forces on the Arches

where the radius R and the distance e are defined in Figure 2-79. For thrusting action due to frame deformation, the arch must resist the combined effect of a factored axial load P_f (P_{frame} in Figure 2-78(b)) and a bending moment $M_f = P_f e$, where P_f is given by:

$$P_f = \frac{15EI_y}{16e^2}\frac{\Delta}{h_s}$$

In the expression for P_f, I_y is the moment of inertia of the reinforcement, Δ is the design storey drift, and h_s is the storey height. It is noted that the arch plate width is irrelevant in that calculation, and it is instead conservatively obtained by considering the strength required to resist the axial component of force in the arch due to the panel forces at the closing corner. The design for the two loading cases, i.e., T_f, and P_f and M_f, can be done independently, because the components of arch forces due to tension field action (T_f) forces are opposing those due to frame corner opening (P_f) (see Figures 2-78(b) and 2-79). Beams and columns must resist the tension and compression forces acting at the ends of the arching reinforcement. Further details are given in Vian *et al.* (2009) and Purba and Bruneau (2007, 2009).

27.9.3 Beams

Beams are expected to develop plastic hinges at their ends. Past tests have shown that lateral resistance and energy dissipation capacity under cyclic loading is essentially supplied by moment-resisting frame response once the infill plate has been stretched in larger cycles. To achieve minimum frame response, CSA S16 requires the boundary moment-resisting frame to be designed for a factored storey shear resistance, $V_{r,\,MRF}$ = 25% of the design seismic storey shear. This factored resistance is taken as $V_{r,\,MRF} = 2\,M_{rb}/h_s$, where M_{rb} is the beam factored resistance in bending in the absence of axial loads, and h_s is the storey height (Berman and Bruneau 2003, Qu and Bruneau 2009).

Beams must also be designed to resist the combined effects of axial loads, shear forces, and bending moments due to gravity loads together with infill plate yielding in tension and plastic hinging at the ends of the beams, as depicted in Figure 2-77. Beam axial loads are due to the horizontal components of the plate yield loads acting both along the beams and the columns (Berman and Bruneau 2008b). When calculating the plastic hinge resistance of the beams, the axial load effects should be taken into account. Beam and column forces can be determined by manual calculations. Alternatively, a static incremental (push-over) analysis can be performed using the infill plate strip model by Thorburn *et al.* (1983) with inelastic response assigned to the strips, beams, and columns (Berman and Bruneau 2003, 2008b).

In Clause 27.9.2, the shear resistance of the infill plates is based on the assumption that full tension field response can eventually develop in the plates. For this to occur, the horizontal boundary members at the base and top of plate walls should meet the minimum flexural stiffness requirement, as specified in Clause 20.5.2. Alternatively, the plate panel at the wall base can be attached to a steel member embedded in the foundations. For long walls, Sabelli and Bruneau (2007) suggest that vertical struts could be added at the center of the wall to provide a vertical support to the top beam.

27.9.4 Columns

Columns must be designed to remain essentially elastic once yielding develops in the infill plates and beams. Axial loads, shear forces, and bending moments arising from yielding of the infill plate and beams, as illustrated in Figure 2-77, must therefore be added to the effects of gravity loads on the columns (Sabelli and Bruneau 2007, Berman and Bruneau 2008b). Plastic hinges in columns are permitted only at the column bases.

Bending moments and shear forces induced by infill plate forces can be significant. Column shear yielding must be considered. Li *et al.* (2009) proposed and verified through testing the use of struts between floors to reduce shear and bending moment demands on columns. Composite columns inherently possess high axial and flexural strength and stiffness, and can therefore represent an effective design solution (Astaneh-Asl 2001, Deng *et al.* 2008).

Columns must satisfy the minimum flexural stiffness requirement of Clause 20.5.1, to ensure adequate infill plate tension field response.

When plastic hinging is expected at the column bases, the columns must be detailed so that plastic rotation develops above the base plate or the foundation beam. Premature local buckling in the plastic hinge region must also be prevented (Driver *et al.* 1997).

27.9.5 Minimum Stiffness for Beams and Columns

Minimum stiffness is required to develop uniform yielding of the infill plate.

27.9.6 Column Joint Panel Zones

Joint panel zones in plate walls must satisfy the design and detailing requirements specified for panel zones used in ductile moment-resisting frames.

27.9.7 Beam-to-Column Joints and Connections

Beam-to-column connections must be designed to resist forces anticipated in the infill plates, beams, and columns. Plate walls inherently possess high lateral stiffness, and the anticipated storey drifts are less than anticipated in ductile moment-resisting frames; this is reflected in the limited rotation capacity requirement for the beam-to-column joint (0.02 rad). Maximum moments imposed by beam hinging must however be taken equal to $1.1\,R_y$ times the plastic moment of the beam. Effects of axial loads acting in the beams may be accounted for when determining the moments imposed by beam hinging.

Reduced beam section (RBS) beam-to-column connections have been used in past cyclic test programs (e.g. Vian *et al.* 2009, Qu *et al.* 2008). Well proportioned RBS connections exhibit good plastic rotation capacity and help to minimize shear forces in beams, and flexural and axial load demands on columns.

27.9.8 Protected Zones

Components of the plate walls that are expected to develop large inelastic deformations, such as infill plates, hinges in beams and columns, and their connections, are designated as protected zones and must satisfy the requirements of Clause 27.1.9.

27.10 Type LD (Limited-Ductility) Plate Walls, $R_d = 2.0$, $R_o = 1.5$

For plate walls with limited ductility, seismic energy input is expected to be dissipated primarily by yielding of the infill plate panels. Rigid frame connections are not necessary. However, capacity design requirements for beams, columns, and connections apply.

Commencing in the 2014 edition of the Standard, all requirements for Type LD plate walls have been incorporated in Clause 27.10 to form a stand-alone set of provisions for user friendliness and clarity.

Type LD plate walls are expected to sustain lower inelastic deformation demands compared to Type D plate walls, and several relaxations are permitted. For the beams, Class 1 and Class 2 sections are permitted, and lateral bracing requirements are reduced. When rigid beam-to-column connections are used, the moment demand imposed by beam plastic hinging used for the design of the columns and beam-to-column joints can be based on $R_y\,M_{pb}$ rather than $1.1R_y\,M_{pb}$. Design forces for column splices in structures located in low and moderate seismic regions need not satisfy the requirement of Clause 27.1.4.

27.11 Conventional Construction, $R_d = 1.5$, $R_o = 1.3$

In its 2001 edition, the standard introduced provisions for structures of Conventional Construction. The provisions were considered necessary because it was recognized that Conventional Construction would be used for many low-rise structures subjected to considerable seismic hazard, and that most steel structure failures in seismic events are associated with brittle connection details. Provisions related to connections and diaphragms were introduced to prevent brittle failure either by providing ductile connection details, or increasing the design loads. These provisions still apply for seismic-force-resisting systems with specified short-period spectral acceleration ratios ($I_E\,F_a\,S_a(0.2)$) greater than 0.45.

Connections of primary framing members forming the seismic-force-resisting system are typically beam-to-column connections in the moment-resisting frame or braced frame, including member splices subjected to seismic forces in tension or shear, or both, and connections to the foundations. In braced frames, they also include brace-to-beam, brace-to-column, and brace-to-brace connections. Beams acting as collectors, chords, and struts in diaphragms are also primary framing members.

Connections that may be considered ductile if appropriately proportioned include extended-end-plate moment connections, flange-plate moment connections, gusset plates proportioned for ductility (Cheng and Grondin 1999), and bolted connections in which the governing failure mode corresponds to bolt bearing failure. Tests (Tremblay *et al.* 2009) showed that welded connections comprising fillet welds may not possess sufficient ductility to prevent fracture, regardless of load direction. They should also be designed for the amplified loads.

The failure of steel deck diaphragms is typically controlled by failure of the connections between the individual deck sheets and between the deck sheets and the supporting structure. Diaphragms designed and constructed using connections that have been shown by testing to be ductile can be designed using the factored forces calculated for Conventional Construction, while those diaphragms with connections that have not been shown to be ductile should be designed using forces calculated using $R_d R_o = 1.3$. Button-punched side lap connections or arc-spot welded connections commonly used for steel decks have not shown adequate ductile behaviour under cyclic loading. Research investigation into diaphragm designs for more ductile response is underway. Test results reported by Essa *et al.* (2003), Tremblay *et al.* (2004) and Hilti (2007) suggest that diaphragms made of thin steel deck sheets (0.76 mm and 0.91 mm) with power-actuated frame fasteners and screwed sidelaps can accommodate some inelastic deformations through screw tilting and bearing, and tearing of the steel deck sheets at frame fasteners. Welded connections with washers, when properly fabricated, can also sustain inelastic deformation demand (Peuler *et al.* 2002), although this approach is generally less appealing from a practical standpoint.

Cantilever column structures composed of single or multiple beam-columns fixed at the base and pin-connected or free at their upper ends can be designated as Conventional Construction, provided that they are proportioned to satisfy the specific requirements in this clause.

In NBCC 2005, the use of Conventional Construction for steel buildings subject to moderate and high seismicities was restricted to buildings not exceeding 15 metres in height. This restriction was intended to retain the traditional 3-storey height limit stipulated in previous editions of the NBCC. In NBCC 2010, the height limit for steel seismic force-resisting systems of the Conventional Construction category was extended to 60 m in moderate seismic regions and 40 m when subjected to higher seismicities. This relaxation applies to all building occupancies except assembly occupancy. Structures such as stadia, large exhibition halls, arenas, convention centres, and other similar structures must comply with the 15 m height restrictions.

Conventional Construction was permitted for certain buildings in the User's Guide to NBCC 2005, and special requirements and height restrictions were introduced in S16-09 to ensure proper response and to prevent premature failure and non-ductile behaviour for these taller structures. The additional requirements are maintained in CSA S16-14. Amplified design seismic loads are specified to compensate for the greater uncertainty in the prediction of the force demand in taller structures. Response spectrum or time-history dynamic analysis must be used to determine forces and deformations. Minimum ductility requirements for steel material and notch-toughness for thick plates, heavy shapes, and weld metal apply to these structures, and more stringent cross-section stockiness requirements are prescribed to delay local buckling. Amplified design forces are specified for columns, in view of the consequences of column buckling. Higher design loads for columns should encourage yielding in adjacent members such as beams, braces, etc. A special requirement is given to prevent overloading of columns that serve as part of two or more systems intersecting in plan. To avoid premature connection failure, a member's end connections should resist the lesser of its gross cross-sectional probable capacity and the amplified connection design forces given in this Clause. In addition, unless yielding is expected in the adjoining members, connections must also be designed and detailed for a minimum inelastic deformation capacity. This could be achieved through plate yielding or bolt bearing. Higher seismic design loads are also specified for diaphragms so that they remain essentially elastic and can maintain their capacity to distribute seismic forces among the vertical elements of the seismic force-resisting system. Lastly, a minimum out-of-plane force is specified at unbraced member intersections to prevent excessive out-of-plane deformations and/or instability.

27.12 Special Seismic Construction

Many different types of alternative structural systems have been developed to dissipate seismic energy in a ductile and stable manner. One such system, the Special Truss Moment Frames (Goel and Itani 1994, Goel *et al.* 1998), can sustain significant inelastic deformations within a specially designed and detailed segment of the truss. The AISC Seismic Provisions (AISC 2010a) provide design and detailing guidance for this system. Design provisions for seismically isolated structures are available (BSSC 2003). In these cases the provisions could be modified as appropriate to provide a level safety and seismic performance comparable to that implied by the S16 requirements.

28. SHOP AND FIELD FABRICATION AND COATING

This clause and the clauses on erection and inspection serve to show that design cannot be considered in isolation but is part of the design and construction sequence. The resistance factors used in this Standard and the methods of analysis are related to tolerances and good practices in fabrication, erection, and inspection procedures.

CISC Quality Certification for plant fabrication of structural steel is an option for project teams and owners requiring a proven level of fabrication quality and control over processes. This quality management system written specifically for the Canadian structural steel industry is third-party audited by independent auditors. CISC Quality Certification is globally recognized and available to steel fabricators within and outside of Canada. A list of CISC Certified companies is available at *www.cisc-icca.ca*.

28.1 Cambering, Curving, and Straightening

CSA Standard W59 specifies that the temperature of the heated areas shall not exceed 650°C in general and not more than 590°C for QT plate.

28.3 Sheared or Thermally Cut Edge Finish

28.3.2 The use of sheared edges is restricted because the micro-cracking induced may reduce the ductility.

28.4 Fastener Holes

28.4.1 The thickness of 700Q steels that can be punched is restricted because of the excessive damage that occurs at the edge of the hole. The maximum plate thickness for thermally cut holes, as allowed in Clause 28.4.3, is dependent upon the thermal cutting process and equipment used.

28.4.2 The restriction of this clause is similar to that of Clause 28.3.2.

28.4.3 Thermally cut holes are allowed for static load applications when subject to the restrictions of this Clause. Iwankiw and Schlafly (1982) found no significant difference in the connection strength of double lap joints with holes made by punching, drilling, and flame cutting.

28.5 Joints in Contact Bearing

Milling techniques will realistically result in some measurable deviation. Tests by Popov and Stephen (1977a) on columns with intentionally introduced gaps at milled splice joints indicated that the compressive resistance of spliced columns is similar to that of unspliced columns. Local yielding reduces the gap. While in these tests column splice gaps of 1.6 mm were left unshimmed, the Standard is more restrictive and defines full contact as a separation not exceeding 0.5 mm. Because shims will be subjected to either biaxial or triaxial stress fields, mild steel shims may be used regardless of the grade of the main material.

28.6 Member Tolerances

The resistance factors given in this Standard, particularly for compression members, are consistent with the distribution of out-of-straightness of members produced to the straightness tolerances given here (Kennedy and Gad Aly 1980, Chernenko and Kennedy 1991).

28.7 Cleaning, Surface Preparation, and Shop Coating

Throughout this section, the word "painting" has been replaced by "coating" to accommodate coating systems other than paint.

There are five instances where steelwork need not be or should not be coated:

- steelwork concealed by an interior building finish or in a limited corrosive environment;
- steelwork encased in concrete;
- faying surfaces of slip-critical joints, except as permitted by Clause 23;
- surfaces finished to bear unless otherwise specified;
- steelwork where any coating could be detrimental to achieving a sound weldment; and,
- surfaces in an enclosed space entirely sealed off from an external source of oxygen.

Specific requirements are provided in Clause 28.7.4.3 for a limited number of applications where welding over coating is permitted.

28.7.5 Metallic Zinc Coatings

These represent coatings other than paint and include hot-dip galvanizing and zinc metallized coatings, both of which are to comply with the relevant CSA Standards.

29. ERECTION

29.3 Erection Tolerances

This entire clause provides helpful definitions of tolerances for the location of the ends of members with respect to their theoretical locations. Tolerances are given for column base plates and for the alignment and elevations of horizontal or sloping members. For column splice tolerances, also see the Commentary on Clause 28.5.

Clauses 29.3.4, 29.3.5, 29.3.6, and 29.3.8 are written in a parallel manner, in that the offset of one end relative to the other, or the elevation of one end relative to the other, both with respect to their theoretical locations shown on the drawings (e.g. the member is not plumb or not level), is expressed as a function of the length but with upper and lower limits. The lower limit represents a realistic assessment of adequate positioning, and the upper limit is a maximum not to be exceeded by the largest members, as illustrated in Figure 2-80 for horizontal alignment of spandrel beams.

29.3.7 Alignment of Braced Members

This clause is an outgrowth of the extensive work on restructuring Clause 9 on Stability of Structures and Members during the preparation of S16-01. Clause 9.2.1 requires the structure to be brought into line so that the initial misalignment of members at any brace point when the brace is installed does not exceed the limits of Clause 29.3. Thus, the initial misalignment at the particular brace point, Δ_0, relative to the adjacent ones is established, and the analyses given in Clause 9 can follow confidently. This Clause again emphasizes that design, fabrication, and erection are inextricably linked.

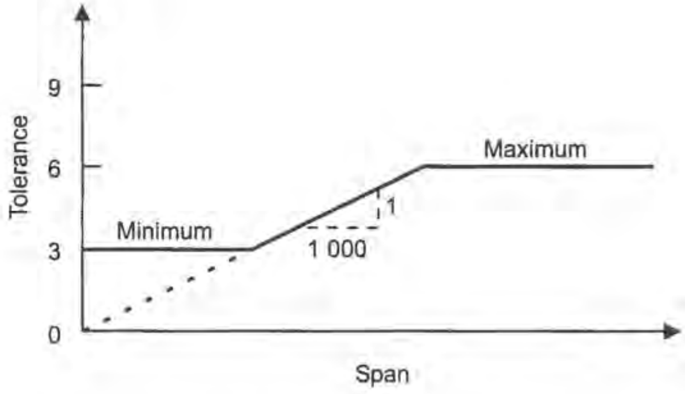


Figure 2-80
Horizontal Alignment Tolerances of Spandrel Beams

30. INSPECTION

This clause outlines quality assurance practices with the objective of ensuring that all shop work and field erection work are in essential compliance with this Standard, in order to provide a structure that is fit for purpose with the requisite strength and stiffness.

30.5 Welding Inspection

30.5.1.1 General

CSA W59 requires the welding company (fabricator or erector) to visually inspect all welds as part of its quality control process. This internal inspection may be performed by the company's own personnel in accordance with its quality control process and may use either competent persons and/or inspection technologies built into the process. Formal welding inspector certification such as CSA W178.2 is not required for the welding company.

Non-destructive examination (NDE), other than the standard visual inspection (by the fabricator or erector) specified by CSA W59, is deemed to be a special and extra requirement and therefore must be specified in the project specifications. The type, location, extent and personnel qualifications of the NDE, as well as the party responsible (owner or other) for performing these inspections, must also be specified in the project specifications.

The CISC "Accredited Steel Inspector – Buildings" accreditation provides objective evidence that an inspector has a minimum competency in steel fabrication and erection inspection. This CISC accreditation is considered to be a complementary competency record, to be paired with CSA W178.2 (welding) if needed.

30.5.2 Competency of inspection personnel

This clause refers to the requirements of all third-party NDE personnel (including visual inspection) and company-employed NDE personnel other than visual.

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PART THREE

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GENERAL INFORMATION

Part 3 contains tables, examples, dimensions and general information of assistance to designers, detailers and others concerned with the design and detailing of connections and tension members according to the requirements of Clauses 12, 13.2, 13.11, 13.12, 13.13, 21, 22 and 23 of CSA S16-14. Information is provided primarily for Imperial series bolts, although all design data are given in SI units. While the basic steel grade for W shapes is ASTM A992, detail material (angles and plates) is still CSA G40.21-300W.

For convenience, Part 3 is divided into seven main sections:

Bolt Data

Pages 3-5 to 3-39 contain information on diameter, area and strength of bolts, including bolt resistances and unit resistances, for evaluating bolts in bearing-type connections, slip-critical connections, and bolts subjected to tension and prying action. Tables are also included for evaluating eccentric loads on various bolt groups.

Weld Data

Pages 3-40 to 3-59 contain information on the factored resistance of welds, including values for various sizes of fillet welds. Tables are included for evaluating eccentric loads on various weld groups and configurations.

Framed Beam Shear Connections

Pages 3-60 to 3-79 contain information on common types of beam shear connections traditionally considered standard in the industry. Included are double-angle beam connections, simple end-plate connections, single-angle beam connections, shear tab beam connections and tee-type beam connections. Information on all-bolted single-angle connections has been incorporated in this (11th) edition of the Handbook.

Seated Beam Shear Connections

Pages 3-80 to 3-86 contain information on unstiffened and stiffened seated beam shear connections of a type commonly used in practice, where direct framing of the supported beam is either not desirable or possible.

Moment Connections

Pages 3-87 to 3-97 contain examples of welded and welded/bolted moment connections, and information for the design of stiffeners on supporting columns.

Hollow Structural Section Connections

Pages 3-98 to 3-107 contain information regarding the connecting of HSS sections.

Tension Members

Pages 3-108 to 3-116 contain tables and examples for calculating net effective areas and for evaluating the tensile resistance of bolted and welded tension members.

BOLT DATA

General

In this (11th) Edition, bolt data in Part 3 are provided for the imperial-series bolts and assemblies based on ASTM Specifications A325, A490, F1852, F2280 and A307 as referenced by CSA S16-14 (see note below). General information on bolts, imperial and metric series, is provided in Part 6. At the time of preparation of this Handbook, metric-series bolts are generally unavailable unless a very large order is placed with advance notice.

This section includes the following:

Bolt Data

Table 3-1 lists the specified minimum tensile strengths for bolts and bolt assemblies. Table 3-2 provides the nominal diameter (mm) and nominal area (mm^2) for bolt sizes from ½ inch to 1½ inch diameter.

Bolts in Bearing-Type Connections

Tables 3-3 to 3-7 list values of bearing and bolt resistances computed in accordance with Clause 13.12.1. Tables 3-8 and 3-9 assist in evaluating combined shear and tension on bolts.

Bolts in Slip-Critical Connections

Tables 3-10 and 3-11 list resistances for use with bolts in slip-critical connections, computed in accordance with Clause 13.12.2.

Bolts in Tension and Prying Action

Tables and design aids (Table 3-13 and Figure 3-1) assist in evaluating the effects of prying action on bolts loaded in tension.

Eccentric Loads on Bolt Groups

Tables 3-14 to 3-20 are provided for evaluating eccentric loads on bolts in bearing-type and slip-critical connections for various bolt group configurations.

Note: ASTM F3125, a consolidation and replacement of six standards (A325, A325M, A490, A490M, F1852, and F2280) was published in January 2015. The name of each bolt standard becomes a bolt grade in this "umbrella" standard, F3125; e.g. A490 becomes F3125 Grade A490. The design of bolted connections must comply with CSA S16-14, which specifies the bolt strength and resistances, and references the ASTM bolt standards prior to the consolidation. F3125 is not referenced in Part 3 of the Handbook. New purchase orders, however, may be placed in accordance with the ordering requirements in ASTM F3125 as summarized in *High-Strength Bolts – Purchase Order Information* in Part 6.

SPECIFIED MINIMUM TENSILE STRENGTHS*, F_u (MPa) — Table 3-1

Bolt, Bolt Assembly	Specified Minimum Tensile Strength F_u
A325, F1852 (d ≤ 1")	**825**
A325, F1852 (d > 1")	**725**
A490, F2280	**1035**
A307 †	**410**
A325M	830
A490M	1040

* CSA S16-14 Clause 13.12.1.2

† Use of A307 bolts in connections is covered in CSA S16-14 Clause 23.6(b).

BASIC BOLT DATA — Table 3-2

Bolt Size *		Nominal Diameter of Bolt	Nominal Area A_b
Imperial	Metric		
in.	mm	mm	mm²
½		**12.70**	**127**
⅝		**15.88**	**198**
	M16	16.00	201
¾		**19.05**	**285**
	M20	20.00	314
	M22	22.00	380
⅞		**22.23**	**388**
	M24	24.00	452
1		**25.40**	**507**
	M27	27.00	573
1⅛		**28.58**	**641**
	M30	30.00	707
1¼		**31.75**	**792**
	M36	36.00	1018
1½		**38.10**	**1140**

Notes: See Table 3-1 for specified minimum tensile strengths, F_u.

* Maximum bolt diameter for ASTM F1852 and F2280 is 1¼ in. See Table 3-48.

BOLTS IN BEARING-TYPE CONNECTIONS

General

Connections are generally detailed as bearing-type, unless the designer has specified that the connection is slip-critical. Bearing-type connections are designed for factored loads at the ultimate limit states (ULS). In Part 3, bearing-type connections are assumed unless noted otherwise. Although tension-control bolt assemblies, F1852 and F2280, are typically used in slip-critical and other pretensioned joints, bolt data for ULS are provided for them because these connections must also satisfy ULS requirements

Tables 3-3 to 3-9 on the following pages assist in evaluating the requirements of Clause 13.12.1 of CSA S16-14. Clause 22.3.5.2 lists the size and type of holes permitted with bearing-type connections.

Table 3-3 (below) summarizes the requirements of Clause 13.12.1.2 for bolts in shear and Clause 13.12.1.3 for bolts in tension, and lists expressions for the factored resistance and unit factored resistance of bolts in bearing-type connections.

Table 3-4 lists factored shear and tensile resistances in kN/bolt. Table 3-5 lists values of the specified minimum tensile strength, F_u, for common grades of structural steel, and values of unit factored bearing resistances at bolt holes.

Tables 3-6 and 3-7 list factored bearing resistances in kN/bolt for different values of F_u for the connected material. Bearing resistances in these tables are given in terms of the steel grade and thickness, and bolt size.

Tables 3-8 and 3-9 assist in evaluating bolts in combined shear and tension according to Clause 13.12.1.4.

The tearing out of material beyond a bolt or group of bolts is governed by Clause 13.11 (Block Shear). Other examples of "block shear" failure modes in bolted connections are illustrated in *Tension Members* at the end of Part 3.

Bearing-Type Connections — Table 3-3

CSA S16-14 Summary

Bolt Situation in Joint	Factored Resistance (ϕ_b = 0.80, ϕ_{br} = 0.80)	Factored Resistance Per Unit of Bolt Area (A_b) or Unit of Bearing Area (t·d)	Clause Reference
BOLTS IN SHEAR			13.12.1.2
Shear on bolts with threads excluded from shear plane	$V_r = 0.60 \phi_b n m A_b F_u$	$0.48 F_u$	
Shear on bolts with threads intercepted by shear plane	$V_r = 0.42 \phi_b n m A_b F_u$	$0.336 F_u$	
For long lap joints with L ≥ 760 mm:			13.12.1.2(c)
- Threads excluded from shear plane	$V_r = 0.50 \phi_b n m A_b F_u$	$0.40 F_u$	
- Threads intercepted by shear plane	$V_r = 0.35 \phi_b n m A_b F_u$	$0.28 F_u$	
Bearing on Bolt Hole:			
- Other than long slotted holes	$B_r = 3.0 \phi_{br} t d n F_u$	$2.40 F_u$	13.12.1.2(a)
- Long slotted holes perpendicular to slot	$B_r = 2.4 \phi_{br} t d n F_u$	$1.92 F_u$	13.12.1.2(b)
BOLTS IN TENSION	$T_r = 0.75 \phi_b n A_b F_u$	$0.60 F_u$	13.12.1.3

Note: See Clause 22.3.5.2 of CSA S16-14 regarding the use of oversize or slotted bolt holes.

FACTORED SHEAR AND TENSILE RESISTANCES PER BOLT

Table 3-4

$\phi_b = 0.80$

Bolt Diameter	Nominal Area, A_b	Factored Shear Resistance † - Single Shear ** (kN)						Factored Tensile Resistance (kN)		
		Threads Excluded			Threads Intercepted ††					
in.	mm²	A325 F1852 *	A490 F2280 *	A307	A325 F1852 *	A490 F2280 *	A307	A325 F1852 *	A490 F2280 *	A307
½	127	50.3	63.1		35.2	44.2		62.9	78.9	
⅝	198	78.4	98.4	39.0	54.9	68.9	27.3	98.0	123	48.7
¾	285	**113**	142	56.1	**79.0**	99.1	39.3	**141**	177	70.1
⅞	388	**154**	193	76.4	**108**	135	53.5	**192**	241	95.4
1	507	201	**252**		141	**176**		251	**315**	
1⅛	641	223	**318**		156	**223**		279	**398**	
1¼	792	276	393		193	275		345	492	
1½	1140	397	566		278	396		496	708	

* Maximum bolt diameter for ASTM F1852 and F2280 is 1¼ in. See Table 3-48 for further information.

** For double shear (m = 2), multiply tabulated values by 2.

† Resistance for lap splices with L ≥ 760 mm shall be reduced by one-sixth. See CSA S16-14 Clause 13.12.1.2.

†† Threads may be intercepted if the thin ply is next to the nut, especially when detailed for minimum bolt stick-through.

UNIT FACTORED BEARING RESISTANCE
At Bolt Holes

Table 3-5

$\phi_{br} = 0.80$

Standard	Steel Grade: Rolled Shapes	Steel Grade: Plates and Bars	Specified Minimum Tensile Strength, F_u (MPa)	$3\phi_{br}F_u$ (MPa)	$2.4\phi_{br}F_u$* (MPa)
CSA G40.21		260W, 260WT	410	984	787
	300W	**300W**, 300WT	**440**	**1056**	845
	300WT, **350W**, 345WM, 345WMT	**350W**, 350WT	**450**	**1080**	864
	350A, 350AT, 350WT, 380W	**350A**, **350AT** 380W	**480**	**1152**	922
		400W, 400WT, 400A, 400AT	520	1248	998
		450W, 450WT	550	1320	1056
		480W, 480WT, 480A, 480AT	590	1416	1133
		550W, 550WT, 550A, 550AT	620	1488	1190
		700Q, 700QT	760	1824	1459
ASTM	A36	A36	400	960	768
	A572 Gr. 50 (345) A709M Gr. 345S A913 Gr. 50 (345) **A992**	A572 Gr. 50 (345)	**450**	**1080**	864
	A588	A709M Grades 345W, HPS 345W	485	1164	931
	A913 Gr. 65 (450)		550	1320	1056
		A790M Gr. HPS 485W	585	1404	1123
	A913 Gr. 70 (485)		620	1488	1190

* Factored bearing resistance perpendicular to long slotted holes

FACTORED BEARING RESISTANCE PER BOLT, B_r* (kN) — Table 3-6a

CSA G40.21 350W Plates and Shapes; 350WT Plates; 300WT, 345WM and 345WMT Shapes
ASTM A992, A709M Gr. 345S and A913 Gr. 50 Shapes; A572 Gr. 50 Plates and Shapes (F_u = 450MPa)

t (mm)	Bolt Diameter, in.							
	½	⅝	¾	⅞	1	1⅛	1¼	1½
4	54.9	68.6	82.3	96.0	110	123	137	165
4.5	61.7	77.2	92.6	108	123	139	154	185
5	68.6	85.7	103	120	137	154	171	206
6	82.3	103	123	144	165	185	206	247
7	96.0	120	144	168	192	216	240	288
8	110	137	165	192	219	247	274	329
9	123	154	185	216	247	278	309	370
10	137	171	206	240	274	309	343	411
11		189	226	264	302	339	377	453
12		206	247	288	329	370	411	494
13			267	312	357	401	446	535
14			288	336	384	432	480	576
15				360	411	463	514	617
16				384	439	494	549	658
17				408	466	525	583	700
18					494	555	617	741
19					521	586	652	782
20						617	686	823
21						648	720	864
22							754	905
23							789	946
24								988
25								1029
26								1070
27								1111
28								1152

F_u = 450 MPa

Table 3-6b

CSA G40.21 300W Plates and Shapes; 300WT Plates (F_u = 440MPa)

t (mm)	Bolt Diameter, in.							
	½	⅝	¾	⅞	1	1⅛	1¼	1½
4	53.6	67.1	80.5	93.9	107	121	134	161
4.5	60.4	75.4	90.5	106	121	136	151	181
5	67.1	83.8	101	117	134	151	168	201
6	80.5	101	121	141	161	181	201	241
7	93.9	117	141	164	188	211	235	282
8	107	134	161	188	215	241	268	322
9	121	151	181	211	241	272	302	362
10	134	168	201	235	268	302	335	402
11		184	221	258	295	332	369	443
12		201	241	282	322	362	402	483
13			262	305	349	392	436	523
14			282	329	376	422	469	563
15			302	352	402	453	503	604
16				376	429	483	536	644
17				399	456	513	570	684
18					483	543	604	724
19					510	573	637	764
20						604	671	805
21						634	704	845
22						664	738	885
23							771	925
24							805	966
25								1006
26								1046
27								1086
28								1127

F_u = 440 MPa

Note: Shear resistance of bolt, shear rupture or block shear of structural steel may govern.
* $B_r = 3\,\phi_{br}\,t\,d\,F_u$ for one bolt, where ϕ_{br} = 0.80. For joints with long slotted holes, see S16-14 Clause 13.12.1.2(b).

FACTORED BEARING RESISTANCE PER BOLT, B_r* (kN)

Table 3-7a

ASTM A36 Plates and Shapes (F_u = 400 MPa)

t (mm)	Bolt Diameter, in. ½	⅝	¾	⅞	1	1⅛	1¼	1½
4	48.8	61.0	73.2	85.3	97.5	110	122	146
4.5	54.9	68.6	82.3	96.0	110	123	137	165
5	61.0	76.2	91.4	107	122	137	152	183
6	73.2	91.4	110	128	146	165	183	219
7	85.3	107	128	149	171	192	213	256
8	97.5	122	146	171	195	219	244	293
9	110	137	165	192	219	247	274	329
10	122	152	183	213	244	274	305	366
11	134	168	201	235	268	302	335	402
12		183	219	256	293	329	366	439
13		198	238	277	317	357	396	475
14			256	299	341	384	427	512
15			274	320	366	411	457	549
16			293	341	390	439	488	585
17				363	415	466	518	622
18				384	439	494	549	658
19				405	463	521	579	695
20					488	549	610	732
21					512	576	640	768
22						604	671	805
23						631	701	841
24						658	732	878
25							762	914
26							792	951

F_u = 400 MPa

Table 3-7b

CSA G40.21 260W and 260WT Plates (F_u = 410 MPa)

t (mm)	Bolt Diameter, in. ½	⅝	¾	⅞	1	1⅛	1¼	1½
4	50.0	62.5	75.0	87.5	100	112	125	150
4.5	56.2	70.3	84.4	98.4	112	127	141	169
5	62.5	78.1	93.7	109	125	141	156	187
6	75.0	93.7	112	131	150	169	187	225
7	87.5	109	131	153	175	197	219	262
8	100	125	150	175	200	225	250	300
9	112	141	169	197	225	253	281	337
10	125	156	187	219	250	281	312	375
11	137	172	206	241	275	309	344	412
12		187	225	262	300	337	375	450
13		203	244	284	325	366	406	487
14			262	306	350	394	437	525
15			281	328	375	422	469	562
16			300	350	400	450	500	600
17				372	425	478	531	637
18				394	450	506	562	675
19					475	534	594	712
20					500	562	625	750
21					525	590	656	787
22						619	687	825
23						647	719	862
24							750	900
25							781	937
26							812	975

F_u = 410 MPa

* $B_r = 3\phi_{br}\,t\,d\,F_u$ for one bolt, where ϕ_{br} = 0.80. For joints with long slotted holes, see S16-14 Clause 13.12.1.2(b). Shear resistance of bolt, shear rupture or block shear of structural steel may govern.

FACTORED BEARING RESISTANCE PER BOLT, B_r* (kN) — Table 3-7c

CSA G40.21 350A, 350AT and 380W Plates and Shapes; 350WT Shapes (F_u = 480 MPa)

t (mm)	Bolt Diameter, in.							
	½	⅝	¾	⅞	1	1⅛	1¼	1½
4	58.5	73.2	87.8	102	117	132	146	176
4.5	65.8	82.3	98.8	115	132	148	165	198
5	73.2	91.4	110	128	146	165	183	219
6	87.8	110	132	154	176	198	219	263
7	102	128	154	179	205	230	256	307
8	117	146	176	205	234	263	293	351
9	132	165	198	230	263	296	329	395
10		183	219	256	293	329	366	439
11		201	241	282	322	362	402	483
12			263	307	351	395	439	527
13			285	333	380	428	475	571
14				358	410	461	512	614
15				384	439	494	549	658
16				410	468	527	585	702
17					497	560	622	746
18					527	593	658	790
19						625	695	834
20						658	732	878
21							768	922
22							805	966
23								1009
24								1053
25								1097
26								1141

F_u = 480 MPa

Table 3-7d

CSA G40.21 400W, 400WT, 400A and 400AT Plates (F_u = 520 MPa)

t (mm)	Bolt Diameter, in.							
	½	⅝	¾	⅞	1	1⅛	1¼	1½
4	63.4	79.2	95.1	111	127	143	158	190
4.5	71.3	89.2	107	125	143	160	178	214
5	79.2	99.1	119	139	158	178	198	238
6	95.1	119	143	166	190	214	238	285
7	111	139	166	194	222	250	277	333
8	127	158	190	222	254	285	317	380
9		178	214	250	285	321	357	428
10		198	238	277	317	357	396	475
11			262	305	349	392	436	523
12			285	333	380	428	475	571
13				361	412	464	515	618
14				388	444	499	555	666
15					475	535	594	713
16					507	571	634	761
17						606	674	808
18						642	713	856
19							753	903
20							792	951
21								999
22								1046
23								1094
24								1141
25								
26								

F_u = 520 MPa

* $B_r = 3\phi_{br}\, t\, d\, F_u$ for one bolt, where ϕ_{br} = 0.80. For joints with long slotted holes, see S16-14 Clause 13.12.1.2(b). Shear resistance of bolt, shear rupture or block shear of structural steel may govern.

FACTORED BEARING RESISTANCE PER BOLT, B_r* (kN) — Table 3-7e

CSA G40.21 450W and 450WT Plates
A913 Gr. 65 Shapes (F_u = 550 MPa)

t (mm)	Bolt Diameter, in. ½	⅝	¾	⅞	1	1⅛	1¼	1½
4	67.1	83.8	101	117	134	151	168	201
4.5	75.4	94.3	113	132	151	170	189	226
5	83.8	105	126	147	168	189	210	251
6	101	126	151	176	201	226	251	302
7	117	147	176	205	235	264	293	352
8	134	168	201	235	268	302	335	402
9		189	226	264	302	339	377	453
10		210	251	293	335	377	419	503
11			277	323	369	415	461	553
12			302	352	402	453	503	604
13				381	436	490	545	654
14				411	469	528	587	704
15					503	566	629	754
16					536	604	671	805
17						641	712	855
18							754	905
19							796	956
20								1006
21								1056
22								1106
23								1157
24								
25								
26								

F_u = 550 MPa

Table 3-7f

CSA G40.21 550W and 550WT Plates
A913 Gr. 70 Shapes (F_u = 620 MPa)

t (mm)	Bolt Diameter, in. ½	⅝	¾	⅞	1	1⅛	1¼	1½
4	75.6	94.5	113	132	151	170	189	227
4.5	85.0	106	128	149	170	191	213	255
5	94.5	118	142	165	189	213	236	283
6	113	142	170	198	227	255	283	340
7	132	165	198	231	265	298	331	397
8		189	227	265	302	340	378	454
9		213	255	298	340	383	425	510
10			283	331	378	425	472	567
11			312	364	416	468	520	624
12				397	454	510	567	680
13					491	553	614	737
14					529	595	661	794
15						638	709	850
16							756	907
17							803	964
18								1020
19								1077
20								1134
21								
22								
23								
24								
25								
26								

F_u = 620 MPa

* $B_r = 3\phi_{br}\, t\, d\, F_u$ for one bolt, where ϕ_{br} = 0.80. For joints with long slotted holes, see S16-14 Clause 13.12.1.2(b). Shear resistance of bolt, shear rupture or block shear of structural steel may govern.

Bolts in Combined Shear and Tension (Bearing-Type Connections)

Clause 13.12.1.4 of CSA S16-14 requires that bolts subjected to shear and tension satisfy the expression:

$$\left(\frac{V_f}{V_r}\right)^2 + \left(\frac{T_f}{T_r}\right)^2 \leq 1$$

where V_f is the factored shear load on the bolt and T_f is the factored tensile load including prying effects. If the shear-tension ratio X is defined as:

$$X = \frac{V_f}{T_f},$$

solving for V_f and T_f gives $V_f = X\,T_f$, and

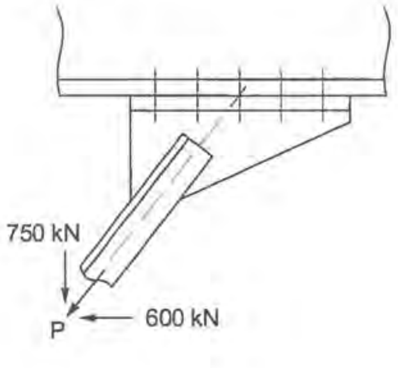
$$T_f = \sqrt{\frac{V_r^2\,T_r^2}{X^2\,T_r^2 + V_r^2}}$$

Combined shear and tension usually occurs for the threads-excluded case, since a plate or flange thin enough to include threads in the shear plane (about 10 mm) has little capacity to transmit tension. Table 3-8 gives values of V_f and T_f for various shear-tension ratios X for ¾, ⅞, 1, 1⅛ and 1¼-inch A325 bolts, with threads excluded from the shear plane. Table 3-9 gives values for A490 bolts.

Example

Given:

A bracing connection consisting of a tee section resists an inclined factored load P, with a tension component T_f of 750 kN and a shear component V_f of 600 kN. Check the number of ¾-inch, A325 bolts required (threads excluded).



Solution:

Shear-tension ratio is $X = 600 / 750 = 0.80$.

From Table 3-8, permitted V_f = 79.8 kN and permitted T_f = 99.8 kN per bolt.

Therefore, the number of bolts required = 600 / 79.8 or 750 / 99.8 = 7.52 ≈ 8.

Prying action should also be checked to complete the design calculations. See *Bolts in Tension and Prying Action* for further information.

BEARING-TYPE IN SHEAR AND TENSION — Table 3-8

Factored Resistances (kN)

A325 Bolts and F1852 Assemblies, Threads Excluded, ϕ_b = 0.80

Shear-Tension Ratio $X = V_f / T_f$		Bolt Size *									
		¾		⅞		1		1⅛		1¼	
X	1 / X	V_f	T_f	V_f	T_f	V_f	T_f	V_f	T_f	V_f	T_f
0	T_r	0	141	0	192	0	251	0	279	0	344
0.10	10.00	14.0	140	19.1	191	24.9	249	27.7	277	34.2	342
0.20	5.00	27.4	137	37.2	186	48.6	243	54.2	271	66.8	334
0.30	3.33	39.6	132	54.0	180	70.5	235	78.3	261	96.6	322
0.40	2.50	50.4	126	68.8	172	89.6	224	100	250	123	308
0.50	2.00	60.0	120	81.5	163	107	213	119	237	146	292
0.60	1.67	67.8	113	92.4	154	121	201	134	223	166	276
0.70	1.43	74.2	106	102	145	132	189	147	210	181	259
0.80	1.25	79.8	99.8	109	136	142	177	158	197	195	244
0.90	1.11	84.3	93.7	115	128	150	167	167	185	206	229
1.00	1.00	88.1	88.1	120	120	157	157	174	174	215	215
1.11	0.90	91.6	82.4	124	112	163	147	181	163	223	201
1.25	0.80	95.1	76.1	130	104	169	135	188	150	233	186
1.43	0.70	98.4	68.9	134	93.8	176	123	194	136	240	168
1.67	0.60	102	61.1	139	83.1	182	109	202	121	248	149
2.00	0.50	105	52.4	143	71.3	186	93.2	208	104	256	128
2.50	0.40	108	43.0	146	58.5	191	76.4	213	85.0	263	105
3.33	0.30	110	32.9	149	44.8	195	58.5	217	65.1	268	80.4
5.00	0.20	112	22.3	152	30.3	198	39.6	221	44.1	272	54.4
10.00	0.10	113	11.3	153	15.3	200	20.0	222	22.2	275	27.5
V_r	0	113	0	154	0	201	0	223	0	276	0

* See Table 3-48 for F1852 assemblies.

A490 and F2280, Threads Excluded, ϕ_b = 0.80 — Table 3-9

Shear-Tension Ratio $X = V_f / T_f$		Bolt Size *									
		¾		⅞		1		1⅛		1¼	
X	1 / X	V_f	T_f	V_f	T_f	V_f	T_f	V_f	T_f	V_f	T_f
0	T_r	0	177	0	241	0	315	0	398	0	492
0.10	10.00	17.6	176	23.9	239	31.2	312	39.5	395	48.8	488
0.20	5.00	34.4	172	46.8	234	61.0	305	77.2	386	95.4	477
0.30	3.33	49.8	166	67.8	226	88.5	295	112	373	138	460
0.40	2.50	63.2	158	86.0	215	112	281	142	356	176	440
0.50	2.00	75.0	150	102	204	134	267	169	338	209	417
0.60	1.67	85.2	142	116	193	151	252	191	319	236	393
0.70	1.43	93.1	133	127	181	166	237	210	300	259	370
0.80	1.25	100	125	136	170	178	223	226	282	278	348
0.90	1.11	106	118	144	160	188	209	239	265	294	327
1.00	1.00	111	111	150	150	197	197	249	249	307	307
1.11	0.90	114	103	157	141	204	184	259	233	319	287
1.25	0.80	119	95.4	163	130	213	170	269	215	331	265
1.43	0.70	124	86.5	169	118	220	154	279	195	343	240
1.67	0.60	128	76.6	173	104	227	136	287	172	355	213
2.00	0.50	131	65.7	179	89.5	234	117	296	148	366	183
2.50	0.40	135	53.9	184	73.4	240	95.9	303	121	375	150
3.33	0.30	138	41.3	187	56.2	245	73.4	310	92.9	383	115
5.00	0.20	140	28.0	191	38.1	249	49.7	315	62.9	389	77.7
10.00	0.10	141	14.1	192	19.2	251	25.1	318	31.8	392	39.2
V_r	0	142	0	193	0	252	0	319	0	393	0

* See Table 3-48 for F2280 assemblies.

BOLTS IN SLIP-CRITICAL CONNECTIONS

General

The name slip-critical emphasizes that this type of connection is required only when the consequences of slip are critical to the performance of the structure. Clause 22.2.2(a) of CSA S16-14 requires slip-critical connections where slippage into bearing cannot be tolerated, such as structures sensitive to deflection, or subject to fatigue or frequent load reversals. In accordance with Clause 13.12.2, slip-critical shear joints transfer the specified loads by the slip resistance (friction) of the clamped faying surfaces, which is a function of the slip coefficient of the contact surfaces and the clamping force. Table 3 of S16-14 provides the mean slip coefficients, k_s, for Class A and Class B surfaces, whereas the resistance factor for slip, c_s, accounts for the clamping force that depends on the installation method.

In addition to the slip resistance of the joints, their factored shear resistance as bearing-type joints under factored loads for all applicable ultimate limit states must also be checked.

Tables

Tables 3-10 and 3-11 are based on Clause 13.12.2.2 of S16-14 for bolts in slip-critical connections.

Table 3-10 lists values of c_s and values of unit slip resistance ($0.53\, c_s k_s F_u$) for combinations of contact surfaces (Class A and Class B), and A325 and A490 bolts installed by the turn-of-nut method and by using F959 washer-type direct tension indicators. Table 3-10 also gives corresponding values when using twist-off-type bolt assemblies, F1852 and F2280.

Table 3-11 lists slip resistance values ($V_s = 0.53\, c_s k_s m n A_b F_u$) for bolted joints with a single faying surface (m = 1) for Class A and Class B contact surfaces for the combinations of bolts and installation methods covered in Table 3-10.

Example

Given:

A single shear connection is subject to 370 kN at the specified load level and 550 kN at the factored load level. Select the number of ¾ inch A325 bolts required for a slip-critical connection. Steel is G40.21-350W, 6 mm thick, and the surface is clean mill scale (Class A). Assume 80 mm bolt pitch and 30 mm bolt end distance.

Solution:

(a) For specified loads:

From Table 3-11, V_s = 37.4 kN (¾ inch A325 bolt for clean mill scale). The number of bolts required is $370 / 37.4 = 9.9$. Use 10 (say 2 lines of 5, parallel to the force).

(b) Confirm the connection at factored loads. This includes checking bolts for shear resistance, checking material for bolt bearing, and checking material for block shear.

From Table 3-4, V_r = 79.0 kN (¾ inch A325, threads intercepted). The factored shear resistance of the bolts is $10 \times 79.0 = 790$ kN > 550 kN

From Table 3-6, the factored bearing resistance at one ¾ inch bolt in 6 mm thick 350W material is 123 kN. 10 bolts give a resistance of $123 \times 10 = 1230$ kN > 550 kN

The connection also has to be confirmed for different modes of block shear. See S16-14 Clause 13.11.

UNIT SLIP RESISTANCE [1], $0.53 c_s k_s F_u$ — Table 3-10

For Specified Loads

Bolt Assembly and Installation Method	Bolt Properties		Contact Surfaces of Bolted Parts			
	Diameter	F_u	Class A, $k_s = 0.30$ Unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel or hot-dipped galvanized and roughened surfaces		Class B, $k_s = 0.52$ Unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel	
	in.	MPa	c_s	$0.53 c_s k_s F_u$ (MPa)	c_s	$0.53 c_s k_s F_u$ (MPa)
A325 by Turn-of-Nut	½ to 1	825	1.00	**131**	1.04	**236**
	1⅛ to 1½	725	1.00	**115**	1.04	**208**
F1852 [2], A325 with F959	½ to 1	825	0.78	**102**	0.81	**184**
	1⅛ to 1½	725	0.78	**89.9**	0.81	**162**
A490 by Turn-of-Nut	½ to 1½	1035	0.92	**151**	0.96	**274**
F2280 [2], A490 with F959			0.78	**128**	0.81	**231**

1. See S16-14 Clause 13.12.2.2 for values of c_s and k_s.
2. Maximum bolt diameter for ASTM F1852 and F2280 is 1¼ in. See Table 3-48.

SLIP RESISTANCE PER BOLT, V_s, (kN) — Table 3-11

For Specified Loads and Single Shear [1]

Bolt Diameter	Nominal Area A_b	Class A Surfaces				Class B Surfaces			
		A325 by Turn-of-Nut	F1852 [2], A325 with F959	A490 by Turn-of-Nut	F2280 [2], A490 with F959	A325 by Turn-of-Nut	F1852 [2], A325 with F959	A490 by Turn-of-Nut	F2280 [2], A490 with F959
in.	mm^2								
½	127	16.7	13.0	19.2	16.3	30.0	23.4	34.8	29.3
⅝	198	26.0	20.3	30.0	25.4	46.8	36.5	54.2	45.7
¾	285	**37.4**	**29.2**	43.1	36.6	**67.4**	**52.5**	78.0	65.8
⅞	388	**50.9**	**39.7**	58.7	49.8	**91.7**	**71.5**	106	89.6
1	507	66.5	51.9	**76.8**	**65.1**	120	93.4	**139**	**117**
1⅛	641	73.9	57.6	**97.0**	**82.3**	133	104	**176**	**148**
1¼	792	91.3	71.2	120	102	165	128	217	183
1½	1140	131	103	173	146	237	185	312	263

Note: These resistances are for use with specified loads in accordance with CSA S16-14 Clause 13.12.

[1] For double shear (m = 2), multiply tabulated values by 2.

[2] Maximum bolt diameter for ASTM F1852 and F2280 is 1¼ in. See Table 3-48.

Bolts in Combined Shear and Tension — Slip-Critical Connections

Clause 13.12.2.3 of CSA S16-14 requires that bolts subjected to both shear and tension in a slip-critical connection satisfy the following relationship for specified loads:

$$\frac{V}{V_s} + 1.9 \frac{T}{n\, A_b\, F_u} \leq 1.0$$

The above relationship can conservatively be expressed (see Commentary on Clause 13.12.2 in Part 2 of this Handbook) as:

$$\frac{V}{V_s} + \frac{T}{T_i} \leq 1.0$$

where T_i is the specified installed tension.

If the shear-tension ratio V/T on the bolts is X, solving for V and T gives $V = XT$, and $T = V_s/(X + V_s/T_i)$.

Table 3-12a lists values of V and T for various shear-tension ratios X for Class A contact surfaces (clean mill scale or blast cleaned with Class A coatings, $k_s = 0.30$) using A325 bolts in single shear installed by the turn-of-nut method. Table 3-12b lists values of V and T for various shear-tension ratios X for Class A contact surfaces ($k_s = 0.30$) using F1852 twist-off-type bolts (or A325 bolts installed with F959 washer-type direct tension indicators) in single shear. These tables can be used to establish directly the number of bolts required to satisfy the interaction equation for slip-critical connections subjected to a combination of shear and tension.

Example

Given:

Find the number of ¾ inch A325 bolts required in a slip-critical connection to resist a specified tension force of 320 kN and a specified shear force of 400 kN. The single faying surface consists of clean mill scale.

Solution:

Prying is not a factor when making the specified shear vs. specified tension interaction check. Within permitted loadings, prying is only a redistribution of the contact forces between the material surfaces, having no significant effect on slip resistance.

Shear-tension ratio is $400 / 320 = 1.25$

From Table 3-12a, for ¾ inch A325 bolts and $V/T = 1.25$,

permitted V and T are 30.1 kN and 24.1 kN, respectively, per bolt.

Therefore, number of bolts required is $400 / 30.1$ or $320 / 24.1 = 13.3$

Try 14 bolts.

The connection also has to be confirmed for strength, including bolt prying and flange bending, as a bearing-type connection at factored loads.

SPECIFIED SHEAR AND TENSION (kN) — Table 3-12a

Slip-Critical Connections, Class A Surfaces — $k_s = 0.30$

A325 Bolts Installed by Turn-of-Nut Method — $c_s = 1.00$

Shear / Tension Ratio X = V/T		Bolt Size									
		¾		⅞		1		1⅛		1¼	
X	1 / X	V	T	V	T	V	T	V	T	V	T
0.5		23.3	46.6	31.7	63.4	41.4	82.9	46.1	92.2	56.9	114
0.6		24.9	41.4	33.8	56.4	44.2	73.7	49.2	81.9	60.7	101
0.7		26.1	37.3	35.5	50.8	46.4	66.3	51.6	73.8	63.8	91.1
0.8		27.1	33.9	36.9	46.2	48.2	60.3	53.7	67.1	66.2	82.8
0.9		28.0	31.1	38.1	42.3	49.8	55.3	55.3	61.5	68.3	75.9
1.0	1.0	28.7	28.7	39.1	39.1	51.0	51.0	56.8	56.8	70.1	70.1
1.11	0.9	29.4	26.5	40.0	36.0	52.3	47.0	58.1	52.3	71.8	64.6
1.25	0.8	30.1	24.1	41.0	32.8	53.5	42.8	59.5	47.6	73.5	58.8
1.43	0.7	30.9	21.6	42.0	29.4	54.9	38.4	61.0	42.7	75.3	52.7
1.67	0.6	31.7	19.0	43.1	25.8	56.3	33.8	62.6	37.5	77.3	46.4
2.00	0.5	32.5	16.2	44.2	22.1	57.7	28.9	64.2	32.1	79.3	39.6
2.50	0.4	33.4	13.3	45.4	18.2	59.3	23.7	66.0	26.4	81.4	32.6
3.33	0.3	34.3	10.3	46.7	14.0	60.9	18.3	67.8	20.3	83.7	25.1
5.00	0.2	35.3	7.1	48.0	9.6	62.7	12.5	69.7	13.9	86.1	17.2
10.00	0.1	36.3	3.6	49.4	4.9	64.5	6.5	71.8	7.2	88.6	8.9
V_s	0	37.4	0	50.9	0	66.5	0	73.9	0	91.3	0

$V = XT, \quad T = \dfrac{V_s}{X + V_s/T_i}$

SPECIFIED SHEAR AND TENSION (kN) — Table 3-12b

Slip-Critical Connections, Class A Surfaces — $k_s = 0.30$

F1852 Assemblies, A325 Bolts Installed with F959 — $c_s = 0.78$

Shear / Tension Ratio X = V/T		Bolt Size*									
		¾		⅞		1		1⅛		1¼	
X	1 / X	V	T	V	T	V	T	V	T	V	T
0.5		19.8	39.6	27.0	54.0	35.2	70.5	39.2	78.4	48.4	96.8
0.6		20.9	34.9	28.5	47.5	37.2	62.0	41.4	69.0	51.1	85.2
0.7		21.8	31.2	29.7	42.4	38.8	55.4	43.1	61.6	53.3	76.1
0.8		22.5	28.2	30.7	38.3	40.0	50.1	44.5	55.7	55.0	68.7
0.9		23.1	25.7	31.5	35.0	41.1	45.7	45.7	50.8	56.4	62.7
1.0	1.0	23.6	23.6	32.1	32.1	42.0	42.0	46.7	46.7	57.6	57.6
1.11	0.9	24.1	21.7	32.7	29.5	42.8	38.5	47.6	42.8	58.7	52.9
1.25	0.8	24.5	19.6	33.4	26.7	43.6	34.9	48.5	38.8	59.9	47.9
1.43	0.7	25.0	17.5	34.1	23.9	44.5	31.2	49.5	34.6	61.1	42.8
1.67	0.6	25.6	15.3	34.8	20.9	45.4	27.3	50.5	30.3	62.4	37.4
2.00	0.5	26.1	13.0	35.5	17.8	46.4	23.2	51.6	25.8	63.7	31.8
2.50	0.4	26.7	10.7	36.3	14.5	47.4	19.0	52.7	21.1	65.1	26.0
3.33	0.3	27.2	8.2	37.1	11.1	48.4	14.5	53.9	16.2	66.5	19.9
5.00	0.2	27.9	5.6	37.9	7.6	49.5	9.9	55.1	11.0	68.0	13.6
10.00	0.1	28.5	2.8	38.8	3.9	50.7	5.1	56.3	5.6	69.5	7.0
V_s	0	29.2	0	39.7	0	51.8	0	57.7	0	71.2	0

$V = XT, \quad T = \dfrac{V_s}{X + V_s/T_i}$

* See Table 3-48.

BOLTS IN TENSION AND PRYING ACTION

General

Connections with fasteners loaded in tension occur in many common situations, such as hanger and bracing connections with tee-type gussets, and end-plate moment connections. When bolts are loaded in direct tension, Clause 13.12.1.3 of CSA S16-14 requires that the effects of prying action be taken into account in proportioning the bolts and connected parts. This clause also requires that the connection be arranged to minimize prying forces when subjected to tensile cyclic loading.

The actual stress distribution in the flange of a tee-type connection is extremely complex as it depends on the bolt size and arrangement, and on the strength and dimensions of the connecting flange. Consequently, various design methods have been proposed in the technical literature for proportioning such connections. The procedures given in this section are based on the recommendations contained in the *Guide to Design Criteria for Bolted and Riveted Joints,* by Kulak, Fisher and Struik, second edition, page 285.

The procedures include a set of seven equations for selecting a trial section and for evaluating the bolt forces and flange capacity. Equation (4) uses the full tensile resistance, T_r, of the bolts to determine α for use in equation (5) which provides the maximum connection capacity. Similarly, equation (6) uses the applied factored tensile load per bolt, P_f, to determine α for use in the amplified bolt force expressed by equation (7). This provides a value for the factored load per bolt (including prying), T_f.

Based on these equations, Table 3-13 and Figure 3-1 provide aids for preliminary design and checking purposes. They indicate the effect of applied factored tensile load per bolt and flange geometry for various bolt sizes, assuming static loads.

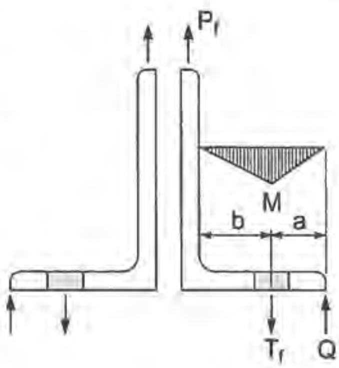
In general, prying effects can be minimized by dimensioning for minimum practical gauge distance and for maximum permissible edge distance. For repeated loading the flange must be made sufficiently thick and stiff so that flange deformation is virtually eliminated. In addition, special attention must be paid to bolt installation to ensure that the bolts are properly pretensioned to provide the required clamping force.

The expressions for prying effects are based on tests carried out on tees. For angles, assuming the distribution of moment shown on the accompanying figure, the moment equilibrium equation can be derived from statics as follows:

$$P_f b = Q\, a$$

Therefore,

$$\frac{Q}{P_f} = \frac{b}{a}$$



Equations

$$K = \frac{4 \times 10^3\, b'}{\phi\, p\, F_y} \qquad (1)$$

$$\delta = 1 - \frac{d'}{p} \qquad (2)$$

$$\text{Range of } t = \sqrt{\frac{K P_f}{1+\delta\alpha}} \qquad (3)$$

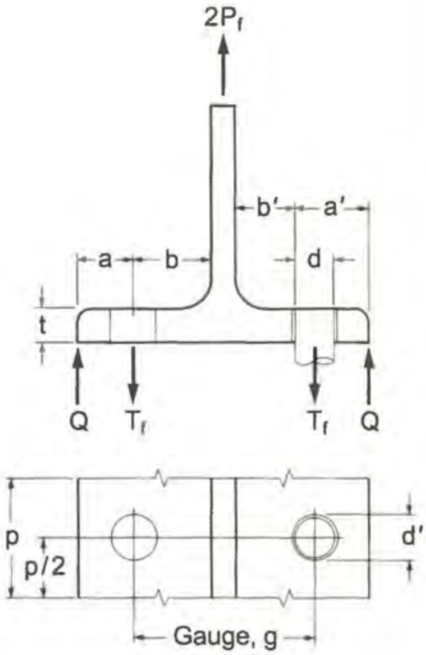
t_{min} when $\alpha = 1.0$, t_{max} when $\alpha = 0.0$

$$\alpha = \left(\frac{K T_r}{t^2} - 1\right)\frac{a'}{\delta(a'+b')}, \quad 0 \le \alpha \le 1.0 \qquad (4)$$

$$\text{Connection capacity} = \frac{t^2}{K}(1+\delta\alpha)n \qquad (5)$$

$$\alpha = \left(\frac{K P_f}{t^2} - 1\right)\frac{1}{\delta} \quad \text{(for use in Eq. 7)} \qquad (6)$$

$$T_f \approx P_f\left[1 + \frac{b'}{a'}\left(\frac{\delta\alpha}{1+\delta\alpha}\right)\right] \le T_r \qquad (7)$$



Nomenclature

K	=	Parameter as defined in Eq. 1
P_f	=	Applied factored tensile load per bolt, (kN)
Q	=	Prying force per bolt at factored load, $Q = T_f - P_f$, (kN)
T_f	=	Factored load per bolt including prying (amplified bolt force), (kN)
T_r	=	Factored tensile resistance per bolt, $0.75\phi_b A_b F_u$, (kN)
F_y	=	Yield strength of flange material, (MPa)
a	=	Distance from bolt line to edge of tee flange, not more than 1.25 b, (mm)
a'	=	$a + d/2$, (mm)
b	=	Distance from bolt line (gauge line) to face of tee stem, (mm)
b'	=	$b - d/2$, (mm)
d	=	Bolt diameter, (mm)
d'	=	Nominal hole diameter, (mm).
n	=	Number of flange bolts in tension
p	=	Length of flange tributary to each bolt, or bolt pitch, (mm)
t	=	Thickness of flange, (mm)
α	=	Ratio of sagging moment at bolt line to hogging moment at stem of tee
δ	=	Ratio of net to gross flange area along a longitudinal line of bolts (see Eq. 2)
ϕ	=	Resistance factor for the tee material, (0.9)

Preliminary Design Tables

Table 3-13 lists the maximum and minimum values of flange thickness t calculated with Eq. 3 using $\alpha = 0.0$ and $\alpha = 1.0$ for a range of values of P_f. Results are tabulated for various flange bolt patterns and bolt sizes.

The maximum and minimum values of t indicate a range of flange thickness within which the bolts and flange are in equilibrium for the particular flange geometry, and in which the effects of flange flexure and prying reduce the effective tension capacity of the bolts. When the flange thickness is greater than the larger value of t, ($\alpha = 0.0$), the flange is generally sufficiently thick and stiff to virtually eliminate prying action, and the connection capacity will be limited by the tensile resistance of the bolts. When the flange thickness is less than the smaller value of t, ($\alpha = 1.0$), the flange thickness will govern the connection capacity, and the bolts will usually have excess capacity to resist the applied tension load in spite of prying effects.

Within the range of flange thickness for $0.0 < \alpha < 1.0$, with the bolts and flange in equilibrium, the ratio T_f/P_f will increase from unity for t_{max} to a maximum value for t_{min}. In this range, the bolts control the capacity with the flange strength being increasingly consumed as the flange thickness decreases. It can be helpful to note that the typical ratio of maximum-to-minimum flange thickness is about 1.33, and that at the minimum flange thickness, the prying ratio T_f/P_f is about the same. When the maximum flange thickness is used, there is essentially no prying and the ratios $t_{max}/t_{req'd}$ and T_f/P_f are both 1.0.

The bolt pitch p should be approximately 4 to 5 times the bolt size ($4\,d \leq p \leq 5\,d$) and the gauge g should be kept as small as practicable. Also, dimension a for design purposes must not exceed $1.25\,b$.

Figure 3-1 graphs the amplified bolt force T_f for various applied loads P_f, flange thicknesses t, and four different values of b (40 mm, 45 mm, 50 mm and 55 mm) with ¾, ⅞ and 1-inch A325 bolts. These graphs can be used to evaluate the effects of flange thickness, gauge distance and bolt size on the amplified bolt force, and to establish reasonable trial connection parameters. The graphs are based on a value of a (distance from bolt line to edge of tee flange) taken equal to b, and are intended to be used within the range $b \leq a \leq 1.25\,b$.

Design Procedure

Trial Section

1) Select an intended number and size of bolts as a function of the applied factored tensile load per bolt P_f and the anticipated prying ratio.
2) With P_f, the bolt size, and trial values of b' and p, use equations 1, 2 and 3 (with $\alpha = 0.0$ and $\alpha = 1.0$) to identify a range of acceptable flange thicknesses. Alternatively, use Table 3-13.
3) Identify an intended flange thickness.

Figure 3-1 may also be used to identify an intended bolt size and flange geometry based on the amplified bolt force being less than the bolt tensile resistance.

Design Check

1) Recalculate K, if necessary, and use Eq. 4 to determine α for use in Eq. 5.

2) Calculate the connection capacity with Eq. 5. (If α from Eq. 4 < 0.0, use $\alpha = 0.0$, and if $\alpha > 1.0$, use $\alpha = 1.0$.)

3) Equations 6 and 7 can be used if desired to determine the total bolt tension, including prying (amplified bolt force), that results from the applied load.

Note

CSA S16-14 Clause 22.2.2(e) requires that all bolts subject to tensile loadings be pretensioned when installed, and Clause 13.12.1.3 requires that connections with tensile cyclic loads on bolts be arranged to minimize prying forces.

Example 1

Given:

Design a tension tee connection with 4 ASTM A325 bolts in tension for a factored static load of 480 kN assuming the bolts are on a 100 mm gauge, at a pitch of 100 mm, with the tee connected to rigid supports. Use ASTM A992 steel (F_y = 345 MPa).

Solution:

Trial Section

Applied load per bolt = P_f = 480 / 4 = 120 kN

Assume ⅞ inch bolts, 24 mm nominal hole diameter, 15 mm web. T_r = 192 kN

$b = (100 - 15) / 2 = 42.5$ mm $b' = 42.5 - 22.23 / 2 = 31.4$ mm

$K = 4 \times 31.4 \times 10^3 / (0.9 \times 100 \times 345) = 4.05$ (Eq. 1)

$\delta = 1 - (24 / 100) = 0.760$ (Eq. 2)

$$t_{min} = \sqrt{\frac{4.05 \times 120}{1.760}} = 16.6 \text{ mm}; \quad t_{max} = \sqrt{\frac{4.05 \times 120}{1.0}} = 22.0 \text{ mm} \qquad \text{(Eq. 3)}$$

(Alternatively, the range of t, by interpolation from Table 3-13, could be seen to be 16.6 to 22.0 mm.)

One possible solution is W410x85 (t = 18.2 mm).

Design Check

Try W410x85 with ⅞ inch bolts: d = 22.23 mm, d' = 24 mm

t = 18.2 mm, w = 10.9 mm, flange width = 181 mm

$b = (100 - 10.9)/2 = 44.6$ mm; $b' = 44.6 - 22.23/2 = 33.5$ mm; $1.25\,b = 55.8$ mm

$a = (181 - 100)/2 = 40.5 < 1.25\,b$; $a' = 40.5 + 22.23/2 = 51.6$ mm; $a' + b' = 85.1$ mm

$K = 4 \times 33.5 \times 10^3 / (0.9 \times 100 \times 345) = 4.32$ (Eq. 1)

$\delta = 0.760$ (as above) (Eq. 2)

$$\alpha = \left(\frac{4.32 \times 192}{18.2^2} - 1\right) \times \frac{51.6}{0.760 \times 85.1} = 1.20 > 1.0\text{, so use } \alpha = 1.0 \qquad \text{(Eq. 4)}$$

$\delta\alpha = 0.760 \times 1.0 = 0.760$

Connection capacity $= (18.2^2 / 4.32)(1.760)\,4 = 540$ kN > 480 kN (Eq. 5)

To find actual bolt load (including prying), if desired:

$$\alpha = \left(\frac{4.32 \times 120}{18.2^2} - 1\right) \times \frac{1}{0.760} = 0.743 \qquad \text{(Eq. 6)}$$

$\delta\alpha = 0.760 \times 0.743 = 0.565$

$$T_f = 120\left[1 + \frac{33.5}{51.6}\left(\frac{0.565}{1 + 0.565}\right)\right] = 148 \text{ kN} < 192 \text{ kN} \qquad \text{(Eq. 7)}$$

Prying ratio: $T_f / P_f = 148 / 120 = 1.23$

Tee stem capacity is $0.9\,(2 \times 100)\,10.9 \times 345 = 677$ kN > 480 kN

Example 2

Given:

Use Table 3-13 and Figure 3-1 to select the bolt size and trial dimensions for a tee cut from a W460x177 section of ASTM A992 steel (F_y = 345 MPa). The factored tensile load is 560 kN and the bolt gauge is 130 mm. Confirm the trial design.

Solution:

Since Table 3-13 and Figure 3-1 are intended only for the selection of a trial section that must be checked with Eqs. 1, 2, 4 and 5 (illustrated in the previous example), precise interpolation is not necessary.

For W460x177: $t = 26.9$ mm, $w = 16.6$ mm, flange width = 286 mm

For $g = 130$ mm, $b = (130 - 16.6)/2 = 56.7$ mm (Use $b = 55$ in Table 3-13)

With 4 bolts, $P_f = 560 / 4 = 140$ kN, and $T_r / P_f = 192 / 140 = 1.37$ for ⅞ inch A325 bolts.

Table 3-13, with $b = 55$, ⅞ inch bolts and $P_f = 140$ kN, $p = 90$ mm gives:

$t_{min} = 22.5$ mm, $t_{max} = 29.7$ mm.

Alternatively, Figure 3-1 can be used to select the bolt size based on the flange thickness and the amplified bolt force.

Use graph for $b = 55$ mm ($b = 56.7$ mm, see above)

Enter graph at applied load per bolt of 140 kN and flange thickness $t \approx 27.0$ mm

With ⅞ inch bolts, amplified bolt force, $T_f \approx 160$ kN $< T_r = 192$ kN

Proceed with the design check using ⅞ inch bolts; $d = 22.23$ mm, $d' = 24$ mm

$4\,d < p = 90 \text{ mm} < 5\,d$

$b = 56.7$ mm; $b' = 56.7 - 22.23 / 2 = 45.6$ mm; $1.25\,b = 70.9$ mm

$a = (286 - 130)/2 = 78.0$ mm $> 1.25\,b = 70.9$ mm, therefore $a = 70.9$ mm

$a' = 70.9 + 22.23 / 2 = 82.0$ mm; $a' + b' = 82.0 + 45.6 = 127.6$ mm

$$K = 4 \times 45.6 \times 10^3 / (0.9 \times 90 \times 345) = 6.53 \qquad \text{(Eq. 1)}$$

$\delta = 1 - (24/90) = 0.733$ (Eq. 2)

$$\alpha = \left(\frac{6.53 \times 192}{26.9^2} - 1\right) \times \frac{82.0}{0.733 \times 127.6} = 0.642, \quad 0 \le \alpha \le 1.0 \quad \delta\alpha = 0.471 \qquad \text{(Eq. 4)}$$

Connection capacity = $(26.9^2 / 6.53)\,(1.471)\,4 = 652$ kN > 560 kN (Eq. 5)

Check total bolt load (amplified bolt force):

$$\alpha = \left(\frac{6.53 \times 140}{26.9^2} - 1\right) \times \frac{1}{0.733} = 0.359 \qquad \delta\alpha = 0.263 \qquad \text{(Eq. 6)}$$

$$T_f = 140\left[1 + \frac{45.6}{82.0}\left(\frac{0.263}{1 + 0.263}\right)\right] = 156\ \text{kN} < 192\ \text{kN} \qquad \text{(Eq. 7)}$$

RANGE OF t

$$t = \sqrt{\frac{K\,P_f}{(1+\delta\alpha)}}$$

Table 3-13

t_{min} when $\alpha = 1.0$, t_{max} when $\alpha = 0.0$

Bolt Size (in.)	b (mm)	P_f = 80 kN, pitch p (mm) 80	90	100	P_f = 100 kN, pitch p (mm) 80	90	100	P_f = 120 kN, pitch p (mm) 80	90	100	P_f = 140 kN, pitch p (mm) 80	90	100
¾ *	35	13.7	12.8	12.1	15.4	14.4	13.5	16.8	15.7	14.8	18.2	17.0	16.0
		18.1	17.1	16.2	20.3	19.1	18.1	22.2	20.9	19.8	24.0	22.6	21.4
	40	15.0	14.1	13.2	16.8	15.7	14.8	18.4	17.2	16.2	19.9	18.6	17.5
		19.8	18.7	17.7	22.2	20.9	19.8	24.3	22.9	21.7	26.2	24.7	23.4
	45	16.2	15.2	14.3	18.1	17.0	16.0	19.9	18.6	17.5	21.5	20.1	18.9
		21.4	20.2	19.1	23.9	22.5	21.4	26.2	24.7	23.4	28.3	26.7	25.3
	50	17.3	16.2	15.3	19.4	18.1	17.1	21.2	19.8	18.7	22.9	21.4	20.2
		22.8	21.5	20.4	25.5	24.1	22.8	28.0	26.4	25.0	30.2	28.5	27.0
	55	18.4	17.2	16.2	20.5	19.2	18.1	22.5	21.0	19.8	24.3	22.7	21.4
		24.2	22.8	21.6	27.1	25.5	24.2	29.6	27.9	26.5	32.0	30.2	28.6

Bolt Size (in.)	b (mm)	P_f = 120 kN, pitch p (mm) 90	100	110	P_f = 140 kN, pitch p (mm) 90	100	110	P_f = 160 kN, pitch p (mm) 90	100	110	P_f = 180 kN, pitch p (mm) 90	100	110
⅞	40	16.9	15.9	15.1	18.3	17.2	16.3	19.5	18.4	17.4	20.7	19.5	18.5
		22.3	21.1	20.1	24.1	22.8	21.8	25.7	24.4	23.3	27.3	25.9	24.7
	45	18.3	17.3	16.3	19.8	18.6	17.7	21.2	19.9	18.9	22.4	21.1	20.0
		24.1	22.9	21.8	26.1	24.7	23.6	27.9	26.4	25.2	29.5	28.0	26.7
	50	19.6	18.5	17.5	21.2	20.0	18.9	22.7	21.3	20.2	24.0	22.6	21.4
		25.8	24.5	23.4	27.9	26.5	25.2	29.8	28.3	27.0	31.7	30.0	28.6
	55	20.9	19.6	18.6	22.5	21.2	20.1	24.1	22.7	21.5	25.5	24.0	22.8
		27.5	26.0	24.8	29.7	28.1	26.8	31.7	30.1	28.7	33.6	31.9	30.4

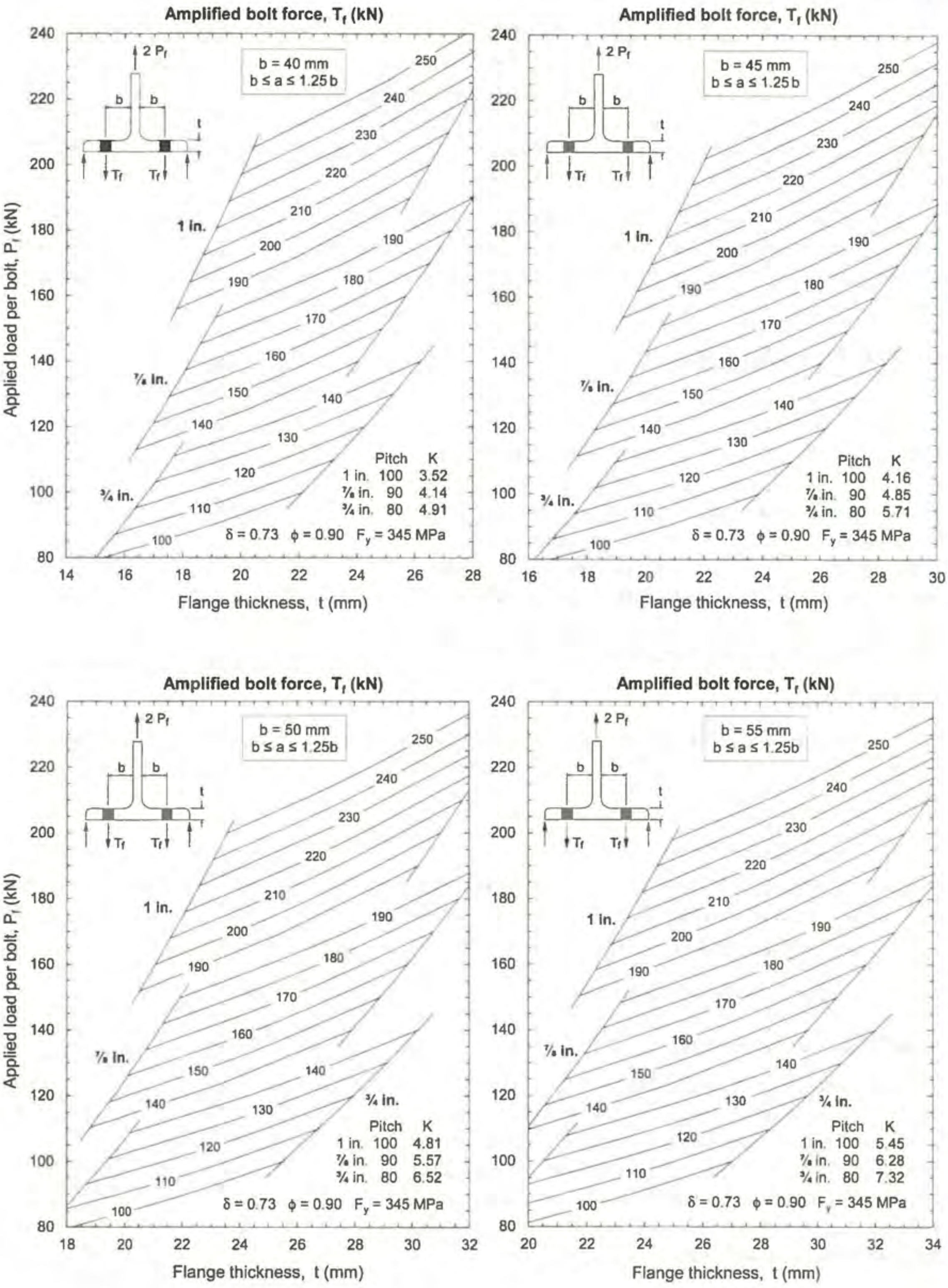
Bolt Size (in.)	b (mm)	P_f = 160 kN, pitch p (mm) 100	110	120	P_f = 180 kN, pitch p (mm) 100	110	120	P_f = 200 kN, pitch p (mm) 100	110	120	P_f = 220 kN, pitch p (mm) 100	110	120
1	40	18.0	17.1	16.3	19.1	18.1	17.2	20.2	19.1	18.2	21.1	20.0	19.1
		23.7	22.6	21.7	25.2	24.0	23.0	26.5	25.3	24.2	27.8	26.5	25.4
	45	19.6	18.6	17.7	20.8	19.7	18.8	21.9	20.8	19.8	23.0	21.8	20.7
		25.8	24.6	23.6	27.4	26.1	25.0	28.8	27.5	26.3	30.3	28.8	27.6
	50	21.1	20.0	19.0	22.4	21.2	20.2	23.6	22.3	21.2	24.7	23.4	22.3
		27.7	26.4	25.3	29.4	28.0	26.8	31.0	29.6	28.3	32.5	31.0	29.7
	55	22.4	21.3	20.2	23.8	22.5	21.5	25.1	23.8	22.6	26.3	24.9	23.7
		29.5	28.2	27.0	31.3	29.9	28.6	33.0	31.5	30.1	34.6	33.0	31.6

Bolt Size (in.)	b (mm)	P_f = 200 kN, pitch p (mm) 100	120	140	P_f = 220 kN, pitch p (mm) 100	120	140	P_f = 240 kN, pitch p (mm) 100	120	140	P_f = 260 kN, pitch p (mm) 100	120	140
1⅛	45	21.6	19.4	17.8	22.6	20.4	18.7	23.6	21.3	19.5	24.6	22.1	20.3
		28.1	25.7	23.8	29.5	26.9	24.9	30.8	28.1	26.0	32.1	29.3	27.1
	50	23.3	20.9	19.2	24.4	22.0	20.1	25.5	22.9	21.0	26.5	23.9	21.9
		30.3	27.7	25.6	31.8	29.0	26.9	33.2	30.3	28.1	34.6	31.6	29.2
	55	24.8	22.3	20.5	26.1	23.4	21.5	27.2	24.5	22.4	28.3	25.5	23.4
		32.4	29.6	27.4	34.0	31.0	28.7	35.5	32.4	30.0	36.9	33.7	31.2
	60	26.3	23.7	21.7	27.6	24.8	22.8	28.8	25.9	23.8	30.0	27.0	24.7
		34.3	31.3	29.0	36.0	32.9	30.4	37.6	34.3	31.8	39.1	35.7	33.1

Bolt Size (in.)	b (mm)	P_f = 240 kN, pitch p (mm) 120	140	160	P_f = 260 kN, pitch p (mm) 120	140	160	P_f = 280 kN, pitch p (mm) 120	140	160	P_f = 300 kN, pitch p (mm) 120	140	160
1¼	45	20.9	19.1	17.7	21.7	19.9	18.4	22.5	20.6	19.1	23.3	21.3	19.8
		27.4	25.4	23.7	28.5	26.4	24.7	29.6	27.4	25.6	30.6	28.4	26.5
	50	22.6	20.7	19.2	23.5	21.5	20.0	24.4	22.3	20.7	25.2	23.1	21.4
		29.7	27.5	25.7	30.9	28.6	26.7	32.0	29.7	27.7	33.2	30.7	28.7
	55	24.2	22.1	20.5	25.2	23.0	21.4	26.1	23.9	22.2	27.0	24.7	23.0
		31.7	29.4	27.5	33.0	30.6	28.6	34.3	31.7	29.7	35.5	32.9	30.7
	60	25.7	23.5	21.8	26.7	24.5	22.7	27.7	25.4	23.5	28.7	26.3	24.4
		33.7	31.2	29.2	35.1	32.5	30.4	36.4	33.7	31.5	37.7	34.9	32.6

$K = 4 \times 10^3\, b' / (\phi\, p\, F_y)$ where $\phi = 0.90$ and $F_y = 345$ MPa

* Nominal bolt hole diameter, d' = 21 mm.



Amplified bolt force, T_f (kN)
b = 40 mm
b ≤ a ≤ 1.25 b
Pitch K
1 in. 100 3.52
⅞ in. 90 4.14
¾ in. 80 4.91
δ = 0.73 φ = 0.90 F_y = 345 MPa
Applied load per bolt, P_f (kN)
Flange thickness, t (mm)
Amplified bolt force, T_f (kN)
b = 45 mm
b ≤ a ≤ 1.25 b
Pitch K
1 in. 100 4.16
⅞ in. 90 4.85
¾ in. 80 5.71
δ = 0.73 φ = 0.90 F_y = 345 MPa
Flange thickness, t (mm)
Amplified bolt force, T_f (kN)
b = 50 mm
b ≤ a ≤ 1.25b
Pitch K
1 in. 100 4.81
⅞ in. 90 5.57
¾ in. 80 6.52
δ = 0.73 φ = 0.90 F_y = 345 MPa
Applied load per bolt, P_f (kN)
Flange thickness, t (mm)
Amplified bolt force, T_f (kN)
b = 55 mm
b ≤ a ≤ 1.25b
Pitch K
1 in. 100 5.45
⅞ in. 90 6.28
¾ in. 80 7.32
δ = 0.73 φ = 0.90 F_y = 345 MPa
Flange thickness, t (mm)

ECCENTRIC LOADS ON BOLT GROUPS

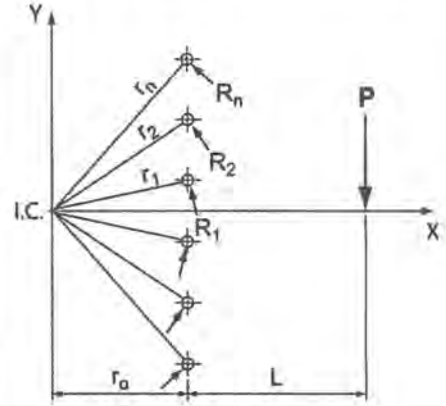
General

A bolted connection is eccentrically loaded when the line of action of the applied load passes outside the centroid of the bolt group. When the bolts are subjected to shear forces only, the effect of this eccentricity is to cause rotation about a single point called the instantaneous centre of rotation. The location of the instantaneous centre is obtained when the connection satisfies the three equilibrium equations for statics, $\Sigma F_x = 0$, $\Sigma F_y = 0$ and $\Sigma M = 0$ about the instantaneous centre.

Calculation of the instantaneous centre described in the references is a trial-and-error process, and the tables included in this section permit rapid evaluation of common bolt groups subjected to various eccentricities. All tables are based on symmetrical arrangements of bolts.

Bearing-Type Connections

For bearing-type connections, a method of analysis is described by Kulak *et al.* (1987). At the time the ultimate load is reached, it is assumed that the bolt furthest from the instantaneous centre will just reach its failure load. The resistance of each bolt is assumed to act on a line perpendicular to the radius joining the bolt to the instantaneous centre, and Δ is assumed to vary linearly with the length of the radius. The resistance of each bolt is calculated according to the load-deformation relationship:



Forces on Eccentrically Loaded Connection

$$R = R_u (1 - e^{-\mu \Delta})^{\lambda}$$

and the ultimate load is reached when $\Delta = \Delta_{max}$ for the bolt furthest from the instantaneous centre, where

R = bolt load at any given deformation

R_u = ultimate bolt load

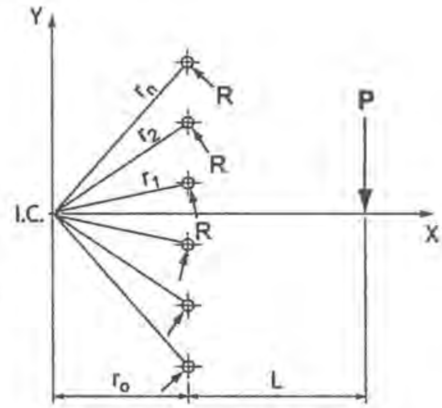
Δ = shearing, bending and bearing deformation of the bolt, and local deformation of the connecting material

μ, λ = regression coefficients

e = base of natural logarithms

Slip-Critical Connections

For slip-critical connections, the method of analysis is essentially the same as that for bearing-type, except that the limiting slip resistance of the joint is reached when the maximum slip resistance of each individual bolt is reached as expressed by the relationship, $R = V_s = 0.53\, c_s\, k_s\, m\, n\, A_b\, F_u$ and the slip resistance of each bolt is assumed to be equal.



Forces on Eccentrically Loaded
Slip-Critical Connection

Tables

Tables 3-14 to 3-20 have been developed using the method described for bearing-type connections. Values tabulated are non-dimensional coefficients C and may be used for bolts of any diameter. In determining the coefficients C, the following values were used: $R_u = 74$ kips (329 kN), $\mu = 10.0$, $\lambda = 0.55$, $\Delta_{max} = 0.34$ inches (8.64 mm). These values were obtained experimentally for ¾ inch diameter A325 bolts and are reported by Crawford and Kulak (1971). The ultimate load P for each bolt group and eccentricity was computed and then divided by the maximum value of R (when $\Delta = \Delta_{max}$) to obtain the values of C.

The tables may thus be used to obtain the factored resistance, expressed as a vertical load P, of a connection by multiplying the coefficient C, for any particular bolt group and eccentricity, by the factored shear resistance of a single bolt. i.e. $P_f = C\,V_r$.

Coefficients were developed in a similar way for slip-critical connections, except that the individual bolt resistances for all bolts in the group were assumed to be equal. The coefficients calculated in this way were from 5% to 10% higher than those for bearing-type connections. Thus only one set of tables, based on the bearing-type connections, is provided for use with both bearing-type and slip-critical connections.

Use of Tables

Bearing-Type Connections

1) To obtain the coefficient C required for a given geometry of bolts and eccentricity of load, divide the factored load P_f by the factored shear resistance V_r of a single bolt for the appropriate shear condition, i.e. $C = P_f / V_r$.

2) To determine the connection capacity, multiply the coefficient C for the bolt group and eccentricity by the appropriate bolt shear resistance value V_r of a single bolt: $P_f = C\,V_r$.

   V_r is the factored shear resistance of the bolt from Table 3-4. Used in this way these tables provide a margin of safety which is consistent with bolts in joints less than 760 mm long and subjected to shear produced by concentric loads only.

Slip-Critical Connections

Although developed using the method for bearing-type connections, these tables can also be used for slip-resistant connections using the *specified* load P and the appropriate slip resistance value V_s for the bolt size and condition of the faying surface.

1) Required $C = P / V_s$
2) Capacity $P = C\,V_s$

V_s is the slip resistance determined from Tables 3-10 and 3-11.

References

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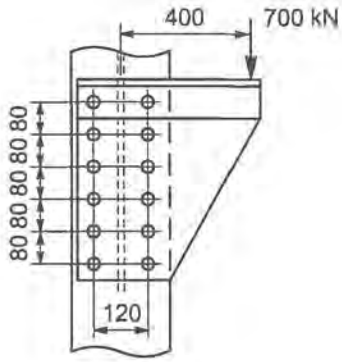
Example

1. Given:

A double column bracket must be designed to support a factored load of 700 kN at an eccentricity of 400 mm. Find the number of ¾ inch A325 bolts per flange required for a gauge dimension of 120 mm and a pitch of 80 mm assuming a bearing-type connection.



Solution:

$P_f = 700 / 2 = 350$ kN $\quad L = 400$ mm

$V_r = 113$ kN (Table 3-4, single shear, threads excluded)

Required $C = 350 / 113 = 3.10$

From Tables 3-15 and 3-16, for 2 lines of bolts

6 rows at 80 mm gauge, $C = 3.49$

at 320 mm gauge, $C = 4.77$

Interpolating for 120 mm gauge $C = 3.49 + (4.77 - 3.49) \times 40 / 240 = 3.70$

Use 6 rows of bolts (total 12 bolts)

Capacity is $3.70 \times 113 = 418$ kN per side

The connected material should be thick enough to provide bearing capacity for the 113 kN resistance of the bolts in accordance with CSA S16-14, Clause 13.12.1.2. Minimum edge distances must conform with Clause 22.3.2.

Note: In double-angle beam connections, the eccentricity may be neglected in the web-framing leg when connected with a single row of bolts.

2. Given:

Find the number of ⅞ inch bolts required for a similar bracket assuming a slip-critical connection with clean mill scale and a specified load of 550 kN.

Solution:

$P = 550 / 2 = 275$ kN $\quad L = 400$ mm

$V_s = 50.9$ kN (Table 3-11) $\quad$ Required $C = 275 / 50.9 = 5.40$

From Tables 3-15 and 3-16, for 2 lines of bolts:

8 rows at 80 mm gauge, $C = 5.89$

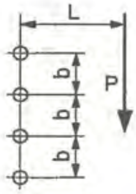
at 320 mm gauge, $C = 7.17$

Interpolating for 120 mm gauge, $C = 5.89 + (7.17 - 5.89) \times 40 / 240 = 6.10$

Use 8 rows of bolts (total 16 bolts)

Capacity is $6.10 \times 50.9 = 310$ kN per side

The ultimate strength of the joint would also be checked in bearing and shear for factored loads.

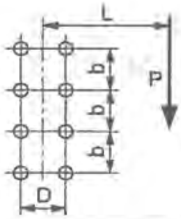


$$C = \frac{P_f}{V_r}, \text{ or } \frac{P}{V_s}$$

ECCENTRIC LOADS ON BOLT GROUPS
Coefficients C

Table 3-14

Moment Arm, L, mm												Number of Bolts	Pitch b mm
50	75	100	125	150	175	200	250	300	400	500	600		
1.20	0.89	0.70	0.57	0.48	0.42	0.37	0.30	0.25	0.19	0.15	0.12	2	
2.26	1.78	1.42	1.17	0.98	0.85	0.74	0.60	0.50	0.37	0.30	0.25	3	
3.37	2.86	2.40	2.04	1.76	1.54	1.37	1.11	0.93	0.71	0.57	0.47	4	
4.47	3.96	3.46	3.00	2.63	2.32	2.07	1.69	1.43	1.08	0.87	0.72	5	
5.54	5.07	4.55	4.05	3.60	3.22	2.90	2.40	2.04	1.56	1.26	1.05	6	
6.59	6.16	5.65	5.13	4.64	4.20	3.82	3.19	2.73	2.10	1.69	1.42	7	75
7.63	7.24	6.75	6.23	5.72	5.24	4.80	4.07	3.50	2.71	2.20	1.85	8	
8.67	8.30	7.85	7.34	6.81	6.31	5.83	5.00	4.34	3.39	2.76	2.33	9	
9.69	9.36	8.93	8.44	7.92	7.40	6.89	5.99	5.24	4.13	3.38	2.85	10	
10.7	10.4	10.0	9.53	9.02	8.50	7.98	7.02	6.19	4.93	4.05	3.43	11	
11.7	11.4	11.1	10.6	10.1	9.60	9.08	8.07	7.18	5.78	4.78	4.05	12	
1.25	0.94	0.74	0.61	0.51	0.44	0.39	0.32	0.26	0.20	0.16	0.13	2	
2.33	1.87	1.50	1.24	1.05	0.90	0.79	0.64	0.53	0.40	0.32	0.27	3	
3.44	2.96	2.51	2.14	1.86	1.63	1.45	1.18	0.99	0.75	0.61	0.51	4	
4.52	4.06	3.58	3.14	2.76	2.45	2.19	1.80	1.52	1.15	0.92	0.77	5	
5.59	5.16	4.68	4.20	3.76	3.38	3.06	2.54	2.16	1.66	1.34	1.12	6	
6.64	6.25	5.78	5.29	4.82	4.39	4.00	3.37	2.89	2.23	1.80	1.51	7	80
7.67	7.32	6.88	6.40	5.91	5.44	5.01	4.28	3.70	2.88	2.34	1.97	8	
8.70	8.38	7.97	7.50	7.01	6.53	6.06	5.24	4.57	3.59	2.93	2.47	9	
9.73	9.43	9.04	8.60	8.11	7.62	7.14	6.26	5.50	4.37	3.59	3.03	10	
10.7	10.5	10.1	9.68	9.22	8.73	8.24	7.30	6.48	5.20	4.30	3.64	11	
11.8	11.5	11.2	10.8	10.3	9.83	9.34	8.38	7.50	6.08	5.05	4.30	12	
1.41	1.11	0.89	0.74	0.63	0.55	0.48	0.39	0.33	0.25	0.20	0.17	2	
2.52	2.13	1.78	1.50	1.28	1.12	0.98	0.79	0.66	0.50	0.40	0.33	3	
3.62	3.25	2.86	2.51	2.21	1.96	1.76	1.45	1.23	0.93	0.75	0.63	4	
4.68	4.35	3.96	3.58	3.22	2.90	2.63	2.19	1.86	1.43	1.15	0.96	5	
5.72	5.43	5.07	4.68	4.29	3.93	3.60	3.06	2.63	2.04	1.66	1.39	6	
6.76	6.49	6.16	5.78	5.39	5.01	4.64	4.00	3.48	2.73	2.23	1.87	7	100
7.78	7.54	7.24	6.88	6.50	6.10	5.72	5.01	4.41	3.50	2.88	2.43	8	
8.80	8.59	8.30	7.97	7.60	7.21	6.81	6.06	5.39	4.34	3.59	3.05	9	
9.82	9.62	9.36	9.04	8.69	8.31	7.92	7.14	6.42	5.24	4.37	3.72	10	
10.8	10.6	10.4	10.1	9.77	9.41	9.02	8.24	7.48	6.19	5.20	4.45	11	
11.8	11.7	11.4	11.2	10.8	10.5	10.1	9.34	8.56	7.18	6.08	5.24	12	
1.56	1.28	1.06	0.89	0.77	0.67	0.60	0.48	0.41	0.31	0.25	0.21	2	
2.67	2.37	2.06	1.78	1.55	1.36	1.21	0.98	0.82	0.62	0.50	0.42	3	
3.74	3.47	3.17	2.86	2.58	2.32	2.11	1.76	1.50	1.15	0.93	0.78	4	
4.79	4.56	4.27	3.96	3.65	3.36	3.09	2.63	2.27	1.76	1.43	1.20	5	
5.82	5.62	5.36	5.07	4.76	4.45	4.15	3.60	3.15	2.49	2.04	1.72	6	
6.84	6.66	6.43	6.16	5.86	5.55	5.24	4.64	4.12	3.30	2.73	2.31	7	125
7.85	7.70	7.49	7.24	6.95	6.65	6.34	5.72	5.15	4.20	3.50	2.98	8	
8.87	8.72	8.53	8.30	8.04	7.75	7.44	6.82	6.21	5.16	4.34	3.72	9	
9.87	9.75	9.57	9.36	9.11	8.83	8.54	7.92	7.29	6.16	5.24	4.52	10	
10.9	10.8	10.6	10.4	10.2	9.91	9.63	9.02	8.39	7.20	6.19	5.38	11	
11.9	11.8	11.6	11.4	11.2	11.0	10.7	10.1	9.49	8.27	7.18	6.28	12	
1.66	1.41	1.20	1.03	0.89	0.79	0.70	0.57	0.48	0.37	0.30	0.25	2	
2.76	2.52	2.26	2.01	1.78	1.59	1.42	1.17	0.98	0.74	0.60	0.50	3	
3.81	3.62	3.38	3.12	2.86	2.62	2.40	2.04	1.76	1.37	1.11	0.93	4	
4.85	4.68	4.47	4.22	3.96	3.70	3.46	3.00	2.63	2.07	1.69	1.43	5	
5.87	5.72	5.54	5.31	5.07	4.81	4.55	4.05	3.60	2.90	2.40	2.04	6	
6.88	6.76	6.59	6.39	6.16	5.91	5.65	5.13	4.64	3.82	3.19	2.73	7	150
7.89	7.78	7.63	7.45	7.24	7.00	6.75	6.23	5.72	4.80	4.07	3.50	8	
8.90	8.80	8.67	8.50	8.30	8.08	7.85	7.34	6.81	5.83	5.00	4.34	9	
9.91	9.82	9.69	9.54	9.36	9.15	8.93	8.44	7.92	6.89	5.99	5.24	10	
10.9	10.8	10.7	10.6	10.4	10.2	10.0	9.53	9.02	7.98	7.02	6.19	11	
11.9	11.8	11.7	11.6	11.4	11.3	11.1	10.6	10.1	9.08	8.07	7.18	12	

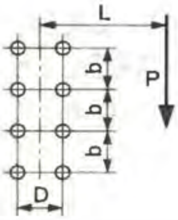


$$C = \frac{P_f}{V_r}, \text{ or } \frac{P}{V_s}$$

ECCENTRIC LOADS ON BOLT GROUPS
Coefficients C

Table 3-15

D = 80 mm												Bolts per Vertical Row	Pitch b mm
Moment Arm, L, mm													
50	75	100	125	150	175	200	250	300	400	500	600		
0.89	0.70	0.57	0.48	0.42	0.37	0.33	0.28	0.23	0.18	0.15	0.12	1	
2.66	2.15	1.78	1.52	1.31	1.16	1.03	0.85	0.72	0.55	0.44	0.37	2	
4.66	3.88	3.25	2.77	2.41	2.13	1.91	1.58	1.34	1.03	0.83	0.70	3	
6.82	5.93	5.13	4.47	3.93	3.50	3.14	2.60	2.22	1.70	1.38	1.15	4	
8.97	8.08	7.18	6.36	5.66	5.06	4.57	3.80	3.24	2.50	2.03	1.71	5	
11.1	10.3	9.33	8.42	7.60	6.87	6.25	5.25	4.51	3.49	2.84	2.39	6	
13.2	12.4	11.5	10.6	9.66	8.83	8.09	6.87	5.92	4.61	3.75	3.17	7	80
15.3	14.6	13.7	12.7	11.8	10.9	10.1	8.64	7.51	5.89	4.82	4.07	8	
17.4	16.7	15.9	14.9	14.0	13.0	12.1	10.5	9.23	7.30	5.99	5.07	9	
19.4	18.8	18.0	17.1	16.2	15.2	14.3	12.5	11.1	8.83	7.29	6.18	10	
21.4	20.9	20.1	19.3	18.4	17.4	16.4	14.6	13.0	10.5	8.69	7.39	11	
23.5	23.0	22.3	21.4	20.5	19.6	18.6	16.7	15.0	12.2	10.2	8.69	12	
2.78	2.26	1.89	1.61	1.39	1.23	1.10	0.90	0.76	0.58	0.47	0.39	2	
4.85	4.10	3.47	2.97	2.59	2.29	2.05	1.69	1.44	1.11	0.90	0.75	3	
7.02	6.22	5.45	4.79	4.24	3.79	3.41	2.84	2.42	1.86	1.51	1.26	4	
9.16	8.39	7.57	6.79	6.09	5.49	4.97	4.15	3.55	2.74	2.23	1.88	5	
11.3	10.6	9.74	8.91	8.12	7.41	6.77	5.74	4.95	3.85	3.14	2.64	6	
13.4	12.7	11.9	11.1	10.2	9.46	8.73	7.48	6.49	5.08	4.15	3.50	7	90
15.4	14.8	14.1	13.3	12.4	11.6	10.8	9.38	8.21	6.50	5.33	4.51	8	
17.5	16.9	16.3	15.5	14.6	13.8	12.9	11.4	10.1	8.03	6.63	5.62	9	
19.5	19.0	18.4	17.6	16.8	16.0	15.1	13.5	12.0	9.70	8.05	6.85	10	
21.6	21.1	20.5	19.8	19.0	18.2	17.3	15.6	14.0	11.5	9.59	8.19	11	
23.6	23.2	22.6	21.9	21.2	20.3	19.5	17.8	16.1	13.3	11.2	9.62	12	
2.89	2.37	1.99	1.70	1.48	1.30	1.17	0.96	0.81	0.62	0.50	0.42	2	
5.01	4.30	3.68	3.17	2.77	2.45	2.20	1.82	1.55	1.19	0.97	0.81	3	
7.17	6.46	5.74	5.09	4.53	4.07	3.67	3.06	2.62	2.02	1.64	1.38	4	
9.30	8.64	7.90	7.16	6.48	5.88	5.35	4.50	3.86	2.99	2.43	2.05	5	
11.4	10.8	10.1	9.32	8.58	7.89	7.26	6.20	5.37	4.20	3.43	2.89	6	
13.5	12.9	12.3	11.5	10.7	10.0	9.30	8.05	7.04	5.55	4.55	3.85	7	100
15.5	15.0	14.4	13.7	12.9	12.2	11.4	10.0	8.87	7.08	5.84	4.95	8	
17.6	17.1	16.5	15.9	15.1	14.4	13.6	12.1	10.8	8.74	7.25	6.17	9	
19.6	19.2	18.7	18.0	17.3	16.6	15.8	14.3	12.8	10.5	8.79	7.51	10	
21.6	21.3	20.8	20.2	19.5	18.7	18.0	16.4	14.9	12.4	10.4	8.96	11	
23.6	23.3	22.8	22.3	21.6	20.9	20.2	18.6	17.1	14.4	12.2	10.5	12	
3.07	2.58	2.19	1.88	1.64	1.46	1.30	1.07	0.91	0.70	0.56	0.47	2	
5.25	4.64	4.06	3.55	3.12	2.78	2.49	2.06	1.76	1.36	1.10	0.93	3	
7.40	6.83	6.21	5.61	5.06	4.58	4.17	3.51	3.02	2.34	1.90	1.60	4	
9.50	8.99	8.40	7.77	7.15	6.57	6.05	5.16	4.46	3.48	2.84	2.39	5	
11.6	11.1	10.6	9.96	9.32	8.70	8.10	7.04	6.17	4.88	4.01	3.39	6	
13.6	13.2	12.7	12.1	11.5	10.9	10.2	9.06	8.03	6.45	5.33	4.52	7	120
15.7	15.3	14.8	14.3	13.7	13.1	12.4	11.2	10.0	8.17	6.81	5.81	8	
17.7	17.4	17.0	16.4	15.9	15.3	14.6	13.3	12.1	10.0	8.43	7.23	9	
19.7	19.4	19.0	18.6	18.0	17.4	16.8	15.5	14.3	12.0	10.2	8.78	10	
21.7	21.5	21.1	20.7	20.2	19.6	19.0	17.7	16.4	14.0	12.0	10.4	11	
23.7	23.5	23.2	22.8	22.3	21.8	21.2	19.9	18.6	16.1	14.0	12.2	12	
3.36	2.92	2.54	2.22	1.97	1.75	1.58	1.31	1.12	0.86	0.70	0.58	2	
5.53	5.10	4.64	4.18	3.76	3.39	3.07	2.57	2.19	1.69	1.38	1.16	3	
7.64	7.28	6.84	6.37	5.90	5.45	5.04	4.33	3.77	2.97	2.43	2.05	4	
9.70	9.40	9.01	8.56	8.09	7.61	7.14	6.28	5.55	4.42	3.64	3.09	5	
11.7	11.5	11.1	10.7	10.3	9.81	9.33	8.39	7.53	6.14	5.12	4.37	6	
13.8	13.5	13.2	12.9	12.5	12.0	11.5	10.6	9.63	8.01	6.76	5.81	7	160
15.8	15.6	15.3	15.0	14.6	14.2	13.7	12.8	11.8	10.0	8.56	7.41	8	
17.8	17.6	17.4	17.1	16.7	16.3	15.9	15.0	14.0	12.1	10.5	9.16	9	
19.8	19.7	19.4	19.2	18.8	18.5	18.0	17.2	16.2	14.3	12.5	11.0	10	
21.8	21.7	21.5	21.2	20.9	20.6	20.2	19.3	18.4	16.4	14.6	13.0	11	
23.8	23.7	23.5	23.3	23.0	22.7	22.3	21.5	20.6	18.6	16.7	15.0	12	

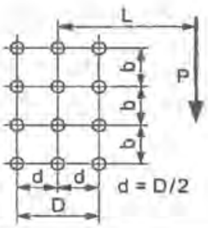


$$C = \frac{P_f}{V_r}, \text{ or } \frac{P}{V_s}$$

ECCENTRIC LOADS ON BOLT GROUPS
Coefficients C

Table 3-16

D = 320 mm												Bolts per Vertical Row	Pitch b mm
Moment Arm, L, mm													
50	75	100	125	150	175	200	250	300	400	500	600		
1.52	1.36	1.23	1.12	1.03	0.95	0.89	0.78	0.69	0.57	0.48	0.42	1	
3.20	2.89	2.63	2.41	2.22	2.06	1.91	1.68	1.50	1.22	1.04	0.90	2	
5.01	4.56	4.16	3.82	3.53	3.27	3.04	2.67	2.38	1.95	1.65	1.43	3	
6.94	6.37	5.86	5.40	5.00	4.65	4.34	3.82	3.41	2.79	2.36	2.04	4	
8.94	8.30	7.69	7.12	6.62	6.17	5.76	5.08	4.53	3.72	3.15	2.73	5	
11.0	10.3	9.62	8.97	8.37	7.82	7.33	6.49	5.80	4.77	4.04	3.50	6	
13.1	12.4	11.6	10.9	10.2	9.59	9.01	8.01	7.17	5.91	5.01	4.34	7	80
15.1	14.4	13.7	12.9	12.2	11.5	10.8	9.65	8.67	7.17	6.08	5.27	8	
17.2	16.5	15.8	15.0	14.2	13.4	12.7	11.4	10.3	8.52	7.24	6.28	9	
19.3	18.6	17.9	17.1	16.3	15.4	14.7	13.2	12.0	9.97	8.50	7.38	10	
21.3	20.7	20.0	19.2	18.4	17.5	16.7	15.1	13.8	11.5	9.84	8.55	11	
23.3	22.8	22.1	21.3	20.5	19.6	18.8	17.1	15.6	13.2	11.3	9.81	12	
3.22	2.92	2.66	2.44	2.25	2.08	1.94	1.70	1.52	1.24	1.05	0.91	2	
5.08	4.63	4.24	3.90	3.60	3.34	3.11	2.74	2.44	2.00	1.69	1.47	3	
7.05	6.50	6.00	5.55	5.15	4.79	4.48	3.95	3.52	2.89	2.44	2.11	4	
9.08	8.48	7.89	7.34	6.84	6.38	5.97	5.28	4.71	3.87	3.28	2.84	5	
11.1	10.5	9.88	9.25	8.66	8.12	7.63	6.77	6.07	5.00	4.23	3.66	6	
13.2	12.6	11.9	11.3	10.6	9.98	9.40	8.39	7.53	6.22	5.28	4.57	7	90
15.3	14.7	14.0	13.3	12.6	11.9	11.3	10.1	9.13	7.58	6.44	5.59	8	
17.3	16.8	16.1	15.4	14.7	14.0	13.3	12.0	10.8	9.03	7.69	6.68	9	
19.4	18.9	18.2	17.5	16.8	16.0	15.3	13.9	12.7	10.6	9.06	7.88	10	
21.4	20.9	20.3	19.6	18.9	18.1	17.4	15.9	14.6	12.3	10.5	9.16	11	
23.5	23.0	22.4	21.8	21.0	20.3	19.5	18.0	16.5	14.0	12.1	10.5	12	
3.25	2.95	2.69	2.47	2.28	2.11	1.97	1.73	1.54	1.26	1.06	0.92	2	
5.14	4.71	4.32	3.98	3.68	3.41	3.18	2.80	2.49	2.05	1.73	1.50	3	
7.15	6.63	6.14	5.69	5.29	4.93	4.61	4.07	3.63	2.98	2.52	2.18	4	
9.20	8.65	8.08	7.55	7.05	6.60	6.18	5.47	4.90	4.03	3.41	2.96	5	
11.3	10.7	10.1	9.52	8.95	8.41	7.92	7.05	6.33	5.23	4.43	3.84	6	
13.3	12.8	12.2	11.6	10.9	10.3	9.78	8.76	7.89	6.53	5.55	4.81	7	100
15.4	14.9	14.3	13.7	13.0	12.4	11.7	10.6	9.59	7.99	6.80	5.91	8	
17.5	17.0	16.4	15.8	15.1	14.4	13.8	12.5	11.4	9.55	8.15	7.09	9	
19.5	19.1	18.5	17.9	17.2	16.5	15.9	14.5	13.3	11.2	9.62	8.39	10	
21.5	21.1	20.6	20.0	19.4	18.7	18.0	16.6	15.3	13.0	11.2	9.78	11	
23.6	23.2	22.7	22.1	21.5	20.8	20.1	18.7	17.3	14.9	12.9	11.3	12	
3.31	3.02	2.76	2.54	2.34	2.17	2.03	1.78	1.59	1.30	1.10	0.95	2	
5.27	4.86	4.48	4.14	3.84	3.57	3.33	2.93	2.61	2.14	1.82	1.57	3	
7.32	6.86	6.40	5.96	5.57	5.20	4.88	4.32	3.87	3.18	2.69	2.33	4	
9.39	8.93	8.43	7.93	7.45	7.01	6.59	5.87	5.26	4.34	3.68	3.19	5	
11.5	11.0	10.5	9.99	9.46	8.95	8.47	7.60	6.85	5.69	4.84	4.19	6	
13.5	13.1	12.6	12.1	11.5	11.0	10.5	9.46	8.58	7.16	6.10	5.30	7	120
15.6	15.2	14.7	14.2	13.7	13.1	12.5	11.4	10.4	8.79	7.53	6.56	8	
17.6	17.3	16.8	16.3	15.8	15.2	14.6	13.5	12.4	10.5	9.07	7.91	9	
19.6	19.3	18.9	18.4	17.9	17.4	16.8	15.6	14.4	12.4	10.7	9.40	10	
21.7	21.4	21.0	20.5	20.0	19.5	18.9	17.7	16.5	14.3	12.5	11.0	11	
23.7	23.4	23.1	22.6	22.2	21.6	21.1	19.9	18.7	16.3	14.3	12.7	12	
3.43	3.15	2.90	2.67	2.48	2.30	2.15	1.90	1.69	1.38	1.17	1.01	2	
5.48	5.13	4.78	4.45	4.15	3.88	3.63	3.20	2.86	2.35	1.99	1.73	3	
7.56	7.21	6.83	6.45	6.07	5.72	5.40	4.82	4.34	3.59	3.04	2.64	4	
9.62	9.31	8.93	8.53	8.12	7.72	7.33	6.61	5.99	4.98	4.24	3.68	5	
11.7	11.4	11.0	10.7	10.2	9.81	9.39	8.58	7.83	6.60	5.65	4.93	6	
13.7	13.5	13.1	12.8	12.4	11.9	11.5	10.6	9.81	8.36	7.21	6.30	7	160
15.7	15.5	15.2	14.9	14.5	14.1	13.7	12.8	11.9	10.3	8.93	7.84	8	
17.8	17.6	17.3	17.0	16.6	16.2	15.8	14.9	14.0	12.3	10.8	9.51	9	
19.8	19.6	19.4	19.1	18.7	18.4	17.9	17.1	16.2	14.3	12.7	11.3	10	
21.8	21.6	21.4	21.1	20.8	20.5	20.1	19.2	18.3	16.5	14.7	13.2	11	
23.8	23.6	23.4	23.2	22.9	22.6	22.2	21.4	20.5	18.6	16.8	15.1	12	

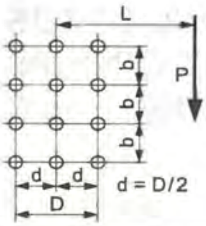


$$C = \frac{P_f}{V_r}, \text{ or } \frac{P}{V_s}$$

ECCENTRIC LOADS ON BOLT GROUPS
Coefficients C

Table 3-17

D = 160 mm												Bolts per Vertical Row	Pitch b mm
Moment Arm, L, mm													
50	75	100	125	150	175	200	250	300	400	500	600		
1.79	1.49	1.28	1.11	0.98	0.87	0.77	0.63	0.53	0.40	0.32	0.27	1	
4.24	3.57	3.06	2.67	2.35	2.10	1.89	1.57	1.34	1.03	0.83	0.70	2	
7.07	6.06	5.24	4.60	4.09	3.67	3.32	2.78	2.38	1.83	1.49	1.25	3	
10.2	8.98	7.92	7.02	6.27	5.65	5.13	4.31	3.70	2.87	2.34	1.97	4	
13.4	12.1	10.9	9.73	8.76	7.93	7.22	6.11	5.28	4.11	3.36	2.83	5	
16.6	15.3	14.0	12.7	11.6	10.5	9.66	8.23	7.13	5.59	4.57	3.86	6	
19.7	18.5	17.2	15.9	14.6	13.4	12.3	10.6	9.20	7.26	5.96	5.04	7	80
22.8	21.7	20.4	19.1	17.7	16.4	15.2	13.2	11.5	9.15	7.54	6.39	8	
25.9	24.9	23.7	22.3	20.9	19.6	18.3	16.0	14.1	11.2	9.27	7.88	9	
29.0	28.1	26.9	25.6	24.2	22.8	21.4	18.9	16.8	13.5	11.2	9.53	10	
32.1	31.2	30.1	28.8	27.4	26.0	24.6	22.0	19.6	15.9	13.3	11.3	11	
35.2	34.3	33.3	32.1	30.7	29.3	27.9	25.1	22.6	18.5	15.5	13.3	12	
4.35	3.68	3.17	2.76	2.44	2.18	1.96	1.63	1.39	1.07	0.87	0.73	2	
7.29	6.31	5.49	4.83	4.29	3.86	3.50	2.93	2.51	1.94	1.58	1.32	3	
10.5	9.35	8.31	7.41	6.65	6.01	5.47	4.61	3.96	3.08	2.51	2.12	4	
13.7	12.5	11.4	10.3	9.32	8.47	7.73	6.56	5.68	4.45	3.64	3.07	5	
16.8	15.8	14.6	13.4	12.3	11.3	10.4	8.87	7.71	6.07	4.98	4.21	6	
20.0	19.0	17.8	16.6	15.4	14.3	13.2	11.4	9.98	7.90	6.51	5.52	7	90
23.1	22.2	21.1	19.8	18.6	17.4	16.2	14.2	12.5	9.99	8.26	7.01	8	
26.1	25.3	24.3	23.1	21.9	20.6	19.4	17.1	15.2	12.3	10.2	8.66	9	
29.2	28.4	27.5	26.3	25.1	23.9	22.6	20.2	18.1	14.7	12.3	10.5	10	
32.3	31.6	30.7	29.6	28.4	27.1	25.9	23.4	21.1	17.3	14.6	12.5	11	
35.3	34.7	33.8	32.8	31.6	30.4	29.2	26.6	24.2	20.1	17.0	14.6	12	
4.46	3.80	3.28	2.86	2.53	2.26	2.04	1.70	1.45	1.12	0.91	0.76	2	
7.49	6.55	5.73	5.05	4.50	4.05	3.68	3.09	2.65	2.06	1.67	1.40	3	
10.7	9.67	8.68	7.79	7.01	6.36	5.80	4.90	4.23	3.30	2.69	2.27	4	
13.9	12.9	11.8	10.8	9.83	8.98	8.24	7.01	6.08	4.78	3.92	3.31	5	
17.0	16.1	15.1	14.0	12.9	11.9	11.0	9.49	8.29	6.55	5.39	4.57	6	
20.1	19.3	18.3	17.2	16.1	15.0	14.0	12.2	10.7	8.55	7.06	6.00	7	100
23.2	22.5	21.5	20.5	19.3	18.2	17.1	15.1	13.4	10.8	8.97	7.64	8	
26.3	25.6	24.7	23.7	22.6	21.5	20.4	18.2	16.3	13.3	11.1	9.44	9	
29.3	28.7	27.9	26.9	25.9	24.8	23.6	21.4	19.3	15.9	13.3	11.4	10	
32.4	31.8	31.1	30.1	29.1	28.0	26.9	24.6	22.4	18.7	15.8	13.6	11	
35.4	34.9	34.2	33.3	32.4	31.3	30.2	27.9	25.6	21.6	18.4	15.9	12	
4.66	4.02	3.49	3.07	2.72	2.44	2.20	1.84	1.58	1.22	0.99	0.83	2	
7.82	6.98	6.19	5.50	4.91	4.43	4.03	3.41	2.94	2.29	1.86	1.57	3	
11.0	10.2	9.31	8.47	7.70	7.03	6.44	5.49	4.76	3.73	3.05	2.58	4	
14.2	13.4	12.5	11.6	10.7	9.92	9.17	7.90	6.89	5.44	4.48	3.80	5	
17.3	16.6	15.8	14.9	14.0	13.0	12.2	10.7	9.39	7.50	6.21	5.27	6	
20.4	19.8	19.0	18.1	17.2	16.3	15.4	13.6	12.1	9.80	8.15	6.95	7	120
23.4	22.9	22.2	21.4	20.5	19.5	18.6	16.8	15.1	12.4	10.4	8.86	8	
26.5	26.0	25.4	24.6	23.7	22.8	21.9	20.0	18.2	15.1	12.8	11.0	9	
29.5	29.1	28.5	27.8	27.0	26.1	25.2	23.2	21.4	18.0	15.3	13.3	10	
32.6	32.1	31.6	30.9	30.2	29.3	28.4	26.5	24.6	21.0	18.1	15.7	11	
35.6	35.2	34.7	34.1	33.4	32.6	31.7	29.8	27.9	24.2	21.0	18.3	12	
5.01	4.42	3.91	3.47	3.11	2.80	2.54	2.14	1.84	1.43	1.16	0.98	2	
8.25	7.61	6.95	6.31	5.72	5.20	4.75	4.04	3.50	2.75	2.25	1.90	3	
11.4	10.9	10.2	9.52	8.84	8.20	7.61	6.60	5.79	4.59	3.79	3.21	4	
14.5	14.0	13.5	12.8	12.1	11.4	10.7	9.47	8.39	6.75	5.59	4.76	5	
17.6	17.2	16.6	16.0	15.4	14.7	14.0	12.6	11.3	9.29	7.78	6.66	6	
20.6	20.3	19.8	19.2	18.6	17.9	17.2	15.8	14.4	12.1	10.2	8.80	7	160
23.7	23.3	22.9	22.4	21.8	21.2	20.5	19.1	17.7	15.0	12.9	11.2	8	
26.7	26.4	26.0	25.6	25.0	24.4	23.8	22.4	20.9	18.2	15.8	13.8	9	
29.7	29.4	29.1	28.7	28.2	27.6	27.0	25.7	24.2	21.4	18.8	16.5	10	
32.7	32.5	32.2	31.8	31.3	30.8	30.2	28.9	27.5	24.6	21.9	19.5	11	
35.7	35.5	35.2	34.9	34.4	33.9	33.4	32.2	30.8	27.9	25.1	22.5	12	

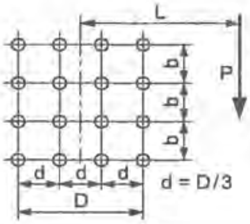


$$C = \frac{P_f}{V_r}, \text{ or } \frac{P}{V_s}$$

ECCENTRIC LOADS ON BOLT GROUPS
Coefficients C

Table 3-18

D = 320 mm												Bolts per Vertical Row	Pitch b mm
Moment Arm, L, mm													
50	75	100	125	150	175	200	250	300	400	500	600		
2.22	1.98	1.79	1.63	1.49	1.38	1.28	1.11	0.98	0.77	0.63	0.53	1	
4.69	4.21	3.81	3.47	3.18	2.93	2.71	2.35	2.07	1.65	1.36	1.15	2	
7.39	6.67	6.05	5.52	5.06	4.68	4.34	3.77	3.32	2.66	2.20	1.87	3	
10.3	9.39	8.57	7.85	7.23	6.68	6.20	5.41	4.77	3.84	3.19	2.72	4	
13.3	12.3	11.3	10.4	9.61	8.90	8.28	7.25	6.42	5.18	4.32	3.69	5	
16.4	15.3	14.2	13.2	12.2	11.4	10.6	9.32	8.27	6.70	5.60	4.79	6	
19.6	18.4	17.3	16.1	15.0	14.0	13.1	11.6	10.3	8.39	7.03	6.03	7	80
22.7	21.6	20.4	19.2	18.0	16.9	15.9	14.0	12.5	10.3	8.61	7.40	8	
25.8	24.8	23.6	22.3	21.1	19.9	18.7	16.7	14.9	12.3	10.3	8.90	9	
28.9	27.9	26.7	25.5	24.2	22.9	21.7	19.5	17.5	14.5	12.2	10.5	10	
32.0	31.0	29.9	28.7	27.4	26.1	24.8	22.4	20.2	16.8	14.2	12.3	11	
35.0	34.2	33.1	31.9	30.6	29.3	27.9	25.4	23.1	19.3	16.4	14.2	12	
4.74	4.26	3.86	3.51	3.22	2.97	2.75	2.38	2.10	1.67	1.38	1.17	2	
7.51	6.80	6.17	5.64	5.18	4.79	4.44	3.87	3.41	2.74	2.27	1.93	3	
10.5	9.61	8.80	8.09	7.46	6.90	6.42	5.60	4.95	3.99	3.32	2.83	4	
13.6	12.6	11.7	10.8	9.98	9.26	8.62	7.56	6.71	5.43	4.53	3.88	5	
16.7	15.7	14.7	13.7	12.7	11.9	11.1	9.78	8.70	7.07	5.92	5.07	6	
19.8	18.8	17.8	16.7	15.7	14.7	13.8	12.2	10.9	8.90	7.48	6.42	7	90
22.9	22.0	21.0	19.8	18.7	17.7	16.6	14.8	13.3	10.9	9.20	7.92	8	
26.0	25.2	24.1	23.0	21.9	20.7	19.6	17.6	15.9	13.1	11.1	9.56	9	
29.1	28.3	27.3	26.2	25.1	23.9	22.7	20.6	18.6	15.5	13.1	11.4	10	
32.2	31.4	30.5	29.4	28.3	27.1	25.9	23.6	21.5	18.0	15.3	13.3	11	
35.2	34.5	33.6	32.6	31.5	30.3	29.1	26.7	24.5	20.7	17.7	15.4	12	
4.79	4.31	3.91	3.56	3.27	3.01	2.79	2.42	2.13	1.70	1.40	1.19	2	
7.62	6.93	6.31	5.76	5.30	4.90	4.55	3.97	3.50	2.81	2.34	1.99	3	
10.7	9.82	9.03	8.32	7.69	7.13	6.63	5.80	5.13	4.14	3.45	2.95	4	
13.8	12.9	12.0	11.1	10.3	9.61	8.97	7.87	7.00	5.68	4.75	4.07	5	
16.9	16.0	15.1	14.1	13.2	12.4	11.6	10.2	9.13	7.44	6.25	5.36	6	
20.0	19.2	18.2	17.2	16.2	15.3	14.4	12.8	11.5	9.40	7.92	6.82	7	100
23.1	22.3	21.4	20.4	19.4	18.4	17.4	15.6	14.0	11.6	9.79	8.44	8	
26.2	25.5	24.6	23.6	22.6	21.5	20.5	18.5	16.8	13.9	11.8	10.2	9	
29.2	28.6	27.7	26.8	25.8	24.7	23.6	21.6	19.7	16.5	14.0	12.2	10	
32.3	31.7	30.9	30.0	29.0	27.9	26.9	24.7	22.7	19.2	16.4	14.3	11	
35.3	34.8	34.0	33.2	32.2	31.2	30.1	27.9	25.8	22.0	18.9	16.5	12	
4.89	4.42	4.01	3.66	3.36	3.10	2.87	2.49	2.20	1.76	1.45	1.23	2	
7.84	7.18	6.57	6.02	5.55	5.13	4.77	4.17	3.69	2.98	2.48	2.11	3	
10.9	10.2	9.47	8.78	8.15	7.58	7.07	6.21	5.51	4.46	3.73	3.19	4	
14.1	13.3	12.5	11.8	11.0	10.3	9.64	8.51	7.59	6.20	5.20	4.47	5	
17.2	16.5	15.7	14.9	14.0	13.2	12.5	11.1	9.98	8.19	6.91	5.95	6	
20.3	19.6	18.9	18.1	17.2	16.3	15.5	14.0	12.6	10.4	8.82	7.62	7	120
23.3	22.8	22.1	21.3	20.4	19.5	18.6	17.0	15.4	12.9	11.0	9.49	8	
26.4	25.9	25.2	24.5	23.6	22.7	21.8	20.1	18.4	15.5	13.3	11.5	9	
29.5	29.0	28.4	27.6	26.8	26.0	25.1	23.3	21.5	18.3	15.8	13.8	10	
32.5	32.0	31.5	30.8	30.0	29.2	28.3	26.5	24.7	21.3	18.4	16.2	11	
35.5	35.1	34.6	34.0	33.2	32.4	31.6	29.7	27.9	24.3	21.2	18.7	12	
5.09	4.64	4.23	3.88	3.57	3.30	3.07	2.67	2.35	1.89	1.57	1.34	2	
8.18	7.63	7.07	6.54	6.06	5.63	5.24	4.60	4.09	3.32	2.78	2.38	3	
11.3	10.8	10.2	9.57	8.99	8.43	7.92	7.02	6.27	5.13	4.31	3.70	4	
14.4	13.9	13.4	12.7	12.1	11.5	10.9	9.74	8.76	7.22	6.11	5.28	5	
17.5	17.1	16.6	16.0	15.3	14.6	14.0	12.7	11.6	9.67	8.23	7.13	6	
20.6	20.2	19.7	19.1	18.5	17.9	17.2	15.8	14.6	12.3	10.6	9.20	7	160
23.6	23.3	22.8	22.3	21.7	21.1	20.4	19.1	17.7	15.2	13.2	11.5	8	
26.6	26.3	25.9	25.5	24.9	24.3	23.7	22.3	20.9	18.3	16.0	14.1	9	
29.7	29.4	29.0	28.6	28.1	27.5	26.9	25.6	24.2	21.4	18.9	16.8	10	
32.7	32.4	32.1	31.7	31.2	30.7	30.1	28.8	27.4	24.6	22.0	19.6	11	
35.7	35.5	35.2	34.8	34.3	33.8	33.3	32.1	30.7	27.9	25.1	22.6	12	

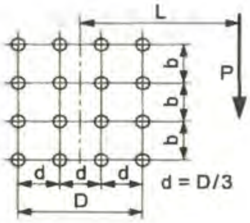


$$C = \frac{P_f}{V_r}, \text{ or } \frac{P}{V_s}$$

ECCENTRIC LOADS ON BOLT GROUPS
Coefficients C

Table 3-19

D = 240 mm												Bolts per Vertical Row	Pitch b mm
Moment Arm, L, mm													
50	75	100	125	150	175	200	250	300	400	500	600		
2.70	2.33	2.04	1.79	1.58	1.41	1.27	1.05	0.90	0.70	0.57	0.48	1	
5.91	5.14	4.52	4.00	3.58	3.22	2.92	2.46	2.11	1.64	1.34	1.13	2	
9.56	8.40	7.43	6.65	5.98	5.42	4.94	4.17	3.59	2.80	2.28	1.93	3	
13.6	12.1	10.9	9.77	8.84	8.05	7.37	6.27	5.43	4.25	3.48	2.94	4	
17.7	16.2	14.6	13.3	12.1	11.0	10.2	8.70	7.57	5.96	4.89	4.14	5	
22.0	20.3	18.7	17.1	15.7	14.4	13.3	11.5	10.0	7.94	6.54	5.54	6	
26.2	24.6	22.9	21.2	19.6	18.1	16.8	14.5	12.8	10.2	8.41	7.14	7	80
30.3	28.9	27.2	25.4	23.7	22.0	20.5	17.9	15.8	12.7	10.5	8.94	8	
34.5	33.1	31.5	29.7	27.9	26.1	24.5	21.6	19.1	15.4	12.8	10.9	9	
38.6	37.3	35.7	34.0	32.2	30.4	28.6	25.4	22.6	18.4	15.4	13.1	10	
42.7	41.5	40.0	38.3	36.5	34.6	32.8	29.4	26.4	21.6	18.1	15.5	11	
46.8	45.7	44.2	42.6	40.8	39.0	37.1	33.5	30.3	25.0	21.0	18.1	12	
6.01	5.24	4.61	4.09	3.66	3.31	3.00	2.53	2.17	1.69	1.38	1.16	2	
9.79	8.65	7.67	6.87	6.20	5.63	5.14	4.35	3.76	2.93	2.39	2.02	3	
13.9	12.5	11.3	10.2	9.26	8.45	7.75	6.61	5.74	4.51	3.70	3.13	4	
18.1	16.7	15.2	13.9	12.7	11.6	10.7	9.23	8.05	6.36	5.23	4.43	5	
22.3	20.9	19.4	17.9	16.5	15.3	14.1	12.2	10.7	8.54	7.05	5.98	6	
26.5	25.2	23.7	22.1	20.6	19.1	17.8	15.5	13.7	11.0	9.10	7.74	7	90
30.6	29.4	28.0	26.4	24.8	23.2	21.8	19.2	17.0	13.7	11.4	9.73	8	
34.8	33.6	32.2	30.7	29.1	27.5	25.9	23.0	20.5	16.7	13.9	11.9	9	
38.9	37.8	36.5	35.0	33.4	31.8	30.1	27.0	24.3	19.9	16.7	14.3	10	
42.9	42.0	40.7	39.3	37.7	36.1	34.4	31.2	28.3	23.4	19.7	17.0	11	
47.0	46.1	45.0	43.6	42.1	40.4	38.8	35.5	32.3	27.0	22.9	19.8	12	
6.12	5.35	4.71	4.19	3.76	3.39	3.08	2.60	2.24	1.74	1.42	1.20	2	
10.0	8.89	7.92	7.11	6.43	5.85	5.35	4.54	3.92	3.06	2.50	2.11	3	
14.2	12.9	11.7	10.6	9.66	8.84	8.13	6.96	6.05	4.77	3.92	3.31	4	
18.4	17.1	15.8	14.5	13.3	12.2	11.3	9.75	8.54	6.78	5.58	4.73	5	
22.6	21.4	20.0	18.6	17.3	16.0	14.9	13.0	11.4	9.13	7.55	6.42	6	
26.8	25.6	24.3	22.9	21.5	20.1	18.8	16.5	14.6	11.8	9.79	8.35	7	100
30.9	29.9	28.6	27.2	25.7	24.3	22.9	20.3	18.1	14.7	12.3	10.5	8	
35.0	34.0	32.9	31.5	30.1	28.6	27.1	24.3	21.9	17.9	15.1	12.9	9	
39.1	38.2	37.1	35.8	34.4	32.9	31.4	28.5	25.8	21.4	18.1	15.5	10	
43.1	42.3	41.3	40.1	38.7	37.3	35.8	32.8	29.9	25.1	21.3	18.4	11	
47.2	46.4	45.5	44.3	43.0	41.6	40.1	37.1	34.2	28.9	24.7	21.4	12	
6.32	5.56	4.93	4.40	3.95	3.58	3.26	2.76	2.38	1.86	1.52	1.28	2	
10.4	9.36	8.41	7.58	6.87	6.27	5.76	4.92	4.26	3.34	2.74	2.31	3	
14.6	13.5	12.4	11.4	10.4	9.61	8.87	7.65	6.68	5.30	4.36	3.70	4	
18.8	17.8	16.7	15.5	14.4	13.4	12.4	10.8	9.49	7.60	6.29	5.34	5	
23.0	22.1	21.0	19.8	18.6	17.4	16.3	14.4	12.8	10.3	8.57	7.31	6	
27.1	26.3	25.3	24.1	22.9	21.7	20.5	18.3	16.3	13.3	11.1	9.55	7	120
31.2	30.4	29.5	28.4	27.2	26.0	24.8	22.4	20.2	16.7	14.0	12.1	8	
35.3	34.6	33.7	32.7	31.6	30.4	29.1	26.6	24.3	20.3	17.2	14.8	9	
39.3	38.7	37.9	36.9	35.9	34.7	33.5	30.9	28.5	24.1	20.6	17.9	10	
43.4	42.8	42.1	41.2	40.1	39.0	37.8	35.3	32.8	28.1	24.2	21.1	11	
47.4	46.9	46.2	45.4	44.4	43.3	42.2	39.7	37.1	32.2	28.0	24.6	12	
6.69	5.98	5.36	4.82	4.36	3.97	3.63	3.09	2.68	2.10	1.72	1.46	2	
10.9	10.1	9.28	8.49	7.76	7.12	6.57	5.66	4.95	3.92	3.22	2.73	3	
15.1	14.4	13.6	12.7	11.8	11.0	10.3	8.97	7.92	6.35	5.27	4.49	4	
19.3	18.6	17.9	17.0	16.1	15.2	14.3	12.7	11.3	9.19	7.68	6.57	5	
23.4	22.8	22.1	21.3	20.4	19.5	18.6	16.8	15.2	12.5	10.6	9.07	6	
27.5	27.0	26.3	25.6	24.8	23.9	22.9	21.1	19.3	16.2	13.7	11.9	7	160
31.5	31.1	30.5	29.8	29.0	28.2	27.3	25.4	23.5	20.1	17.3	15.0	8	
35.5	35.2	34.6	34.0	33.3	32.5	31.6	29.8	27.9	24.2	21.1	18.5	9	
39.6	39.2	38.8	38.2	37.5	36.8	35.9	34.1	32.2	28.5	25.0	22.1	10	
43.6	43.3	42.8	42.3	41.7	41.0	40.2	38.5	36.6	32.8	29.2	26.0	11	
47.6	47.3	46.9	46.4	45.8	45.2	44.5	42.8	41.0	37.2	33.4	30.0	12	



$$C = \frac{P_f}{V_r}, \text{ or } \frac{P}{V_s}$$

ECCENTRIC LOADS ON BOLT GROUPS
Coefficients C

Table 3-20

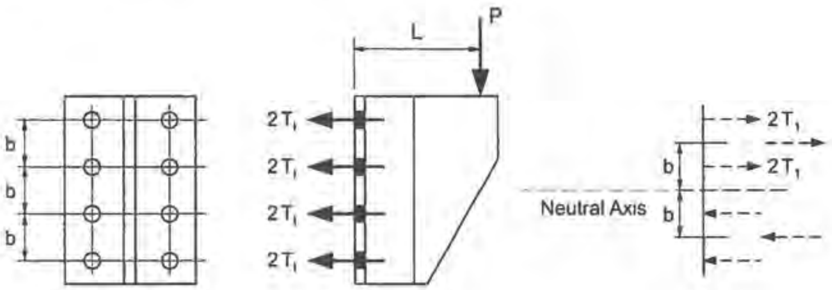
D = 480 mm												Bolts per Vertical Row	Pitch b mm
Moment Arm, L, mm													
50	75	100	125	150	175	200	250	300	400	500	600		
3.18	2.91	2.70	2.50	2.33	2.18	2.04	1.79	1.58	1.27	1.05	0.90	1	
6.54	6.01	5.56	5.16	4.81	4.49	4.20	3.70	3.29	2.66	2.22	1.90	2	
10.1	9.31	8.61	8.01	7.48	6.99	6.56	5.80	5.17	4.20	3.52	3.01	3	
13.9	12.8	11.9	11.1	10.4	9.73	9.13	8.10	7.25	5.93	4.98	4.27	4	
17.8	16.6	15.5	14.4	13.5	12.7	11.9	10.6	9.54	7.84	6.60	5.68	5	
21.8	20.5	19.2	18.0	16.9	15.9	15.0	13.4	12.1	9.95	8.41	7.25	6	
25.9	24.5	23.2	21.8	20.5	19.4	18.3	16.4	14.8	12.3	10.4	8.98	7	80
30.1	28.7	27.2	25.7	24.3	23.0	21.8	19.6	17.7	14.8	12.5	10.9	8	
34.2	32.8	31.3	29.8	28.3	26.8	25.5	23.0	20.8	17.4	14.9	12.9	9	
38.3	37.0	35.5	33.9	32.3	30.8	29.3	26.6	24.2	20.3	17.4	15.1	10	
42.4	41.2	39.7	38.1	36.5	34.9	33.3	30.3	27.7	23.4	20.1	17.5	11	
46.5	45.3	43.9	42.3	40.7	39.0	37.4	34.2	31.4	26.6	22.9	20.0	12	
6.57	6.05	5.60	5.20	4.84	4.52	4.23	3.73	3.32	2.69	2.25	1.92	2	
10.2	9.42	8.72	8.12	7.58	7.09	6.65	5.89	5.26	4.28	3.59	3.07	3	
14.0	13.1	12.1	11.3	10.6	9.94	9.34	8.30	7.43	6.09	5.12	4.41	4	
18.0	16.9	15.8	14.8	13.9	13.0	12.3	11.0	9.85	8.12	6.85	5.90	5	
22.1	20.9	19.7	18.5	17.4	16.4	15.5	13.9	12.5	10.4	8.78	7.58	6	
26.3	25.0	23.7	22.4	21.2	20.0	18.9	17.0	15.4	12.8	10.9	9.44	7	90
30.4	29.2	27.8	26.5	25.1	23.8	22.6	20.4	18.5	15.5	13.2	11.5	8	
34.5	33.4	32.0	30.6	29.2	27.8	26.5	24.0	21.9	18.4	15.7	13.7	9	
38.6	37.5	36.2	34.8	33.4	31.9	30.5	27.8	25.4	21.5	18.5	16.1	10	
42.7	41.7	40.4	39.0	37.6	36.1	34.6	31.8	29.1	24.7	21.3	18.7	11	
46.8	45.8	44.6	43.3	41.8	40.3	38.8	35.8	33.0	28.2	24.4	21.4	12	
6.61	6.09	5.64	5.24	4.88	4.56	4.27	3.77	3.35	2.72	2.27	1.95	2	
10.3	9.53	8.84	8.22	7.68	7.20	6.75	5.99	5.35	4.37	3.66	3.14	3	
14.2	13.3	12.4	11.6	10.8	10.1	9.55	8.50	7.62	6.26	5.28	4.54	4	
18.3	17.2	16.1	15.1	14.2	13.4	12.6	11.3	10.2	8.40	7.11	6.13	5	
22.4	21.3	20.1	19.0	17.9	16.9	16.0	14.4	13.0	10.8	9.16	7.92	6	
26.5	25.4	24.2	23.0	21.8	20.7	19.6	17.7	16.0	13.4	11.4	9.91	7	100
30.6	29.6	28.4	27.1	25.9	24.6	23.5	21.3	19.4	16.3	13.9	12.1	8	
34.8	33.8	32.6	31.3	30.0	28.7	27.4	25.0	22.9	19.3	16.6	14.5	9	
38.9	37.9	36.8	35.6	34.2	32.9	31.6	29.0	26.6	22.6	19.5	17.1	10	
42.9	42.1	41.0	39.8	38.5	37.1	35.8	33.1	30.5	26.1	22.6	19.8	11	
47.0	46.2	45.2	44.0	42.7	41.4	40.0	37.2	34.5	29.8	25.9	22.8	12	
6.69	6.18	5.72	5.32	4.96	4.63	4.34	3.84	3.42	2.78	2.33	2.00	2	
10.5	9.76	9.08	8.46	7.92	7.42	6.97	6.20	5.55	4.55	3.82	3.28	3	
14.5	13.6	12.8	12.0	11.3	10.6	9.98	8.91	8.02	6.62	5.59	4.83	4	
18.6	17.7	16.8	15.8	14.9	14.1	13.3	11.9	10.8	8.98	7.63	6.60	5	
22.8	21.9	20.9	19.9	18.9	17.9	17.0	15.3	13.9	11.6	9.93	8.62	6	
26.9	26.1	25.1	24.0	22.9	21.9	20.9	19.0	17.3	14.6	12.5	10.9	7	120
31.0	30.2	29.3	28.2	27.1	26.0	24.9	22.8	20.9	17.7	15.3	13.4	8	
35.1	34.4	33.5	32.5	31.4	30.2	29.1	26.9	24.8	21.2	18.3	16.1	9	
39.2	38.5	37.7	36.7	35.6	34.5	33.4	31.0	28.8	24.8	21.6	19.0	10	
43.2	42.6	41.8	40.9	39.9	38.8	37.6	35.3	32.9	28.6	25.1	22.1	11	
47.3	46.7	46.0	45.1	44.1	43.1	41.9	39.6	37.2	32.6	28.7	25.4	12	
6.86	6.36	5.91	5.51	5.14	4.81	4.52	4.00	3.58	2.92	2.46	2.11	2	
10.9	10.2	9.56	8.95	8.39	7.89	7.44	6.65	5.98	4.94	4.17	3.59	3	
15.0	14.3	13.6	12.8	12.1	11.5	10.9	9.78	8.84	7.37	6.27	5.43	4	
19.1	18.5	17.7	17.0	16.2	15.4	14.6	13.3	12.1	10.2	8.69	7.57	5	
23.3	22.7	22.0	21.2	20.3	19.5	18.7	17.1	15.7	13.3	11.5	10.0	6	
27.3	26.8	26.2	25.4	24.6	23.7	22.9	21.2	19.6	16.8	14.5	12.8	7	160
31.4	30.9	30.3	29.6	28.9	28.0	27.2	25.4	23.7	20.5	17.9	15.8	8	
35.5	35.0	34.5	33.8	33.1	32.3	31.4	29.7	27.9	24.5	21.6	19.1	9	
39.5	39.1	38.6	38.0	37.3	36.6	35.7	34.0	32.2	28.6	25.4	22.6	10	
43.5	43.2	42.7	42.1	41.5	40.8	40.0	38.3	36.5	32.8	29.4	26.4	11	
47.5	47.2	46.8	46.3	45.7	45.0	44.2	42.6	40.8	37.1	33.5	30.3	12	

ECCENTRIC LOAD ON BOLT GROUPS — SPECIAL CASE

High-Strength Bolts

For connections where the eccentric load causes both shear and tension in the bolts, the following design method may be used when the fasteners are high-strength bolts that have been tightened to the specified minimum initial tension.

A bracket connected by means of bolts with an initial tension T_i is shown below. Both simple and unwieldy methods are available for determining tension that is applied to the upper bolts by the load on the bracket. Generally, the simpler solutions are considerably more conservative than the more accurate but unwieldy ones. The solution presented here is easy to use and conservative.



A neutral axis is assumed through the centre of gravity of the bolt group. Those bolts above the axis are said to carry the tension while those below are considered to be in "compression", so that the applied moment is resisted by a couple applied at the resultants of the upper and the lower bolts. The upper bolts are all taken to be equally loaded; this plastic stress distribution is justified by results that are still conservative compared to more precise methods.

Bolt tension from the applied moment is therefore:

$$T_1 = \frac{P\,L}{n'\,d_m}$$

n' = number of bolts above the neutral axis

d_m = moment arm between resultants of the tensile and compressive forces.

Bolt shear from the applied load is:

$$V = \frac{P}{n}$$

Fasteners in the top half of the connection are subjected to tension, from both the applied moment and from prying (if any), and shear. Bolts in the bottom half are subjected to shear only, with top and bottom bolts participating equally.

The connection should be proportioned so that the bolt tension T_1 due to the moment PL (plus bolt tension due to prying), when combined with the bolt shear, meets the requirements of CSA S16-14 for bolts subjected to combined shear and tension. The relevant clauses are 13.12.1.4 for bearing-type connections and 13.12.2.3 for slip-critical connections.

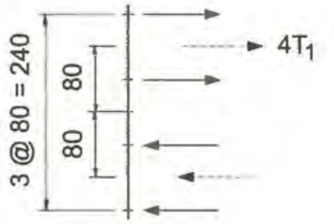
Example 1

Given:

Check the adequacy of eight ¾-inch, A325 bolts (2 rows of 4, at 80 mm pitch) for the connection shown on the previous page for a factored load P_f of 300 kN at an eccentricity L of 150 mm. Assume the material thickness is adequate so that prying action on the bolts is not significant.

Solution:

Factored tension in one bolt:



$$T_1 = \frac{P_f\, L}{n'\, d_m} = \frac{300 \times 150}{4(2 \times 80)} = 70.3 \text{ kN}$$

< 141 kN (Table 3-4)

Factored shear in one bolt:

$$V_f = \frac{P_f}{n} = \frac{300}{8} = 37.5 \text{ kN} < 113 \text{ kN} \quad \text{(Table 3-4, threads excluded case assumed)}$$

Check combined shear and tension for $V_f \,/\, T_f = 37.5 \,/\, 70.3 = 0.53$

From Table 3-8, for bearing-type connections,

permissible $V_f = 62.3$ kN (by interpolation) > 37.5 kN
and permissible $T_f = 118$ kN (by interpolation) > 70.3 kN

Example 2

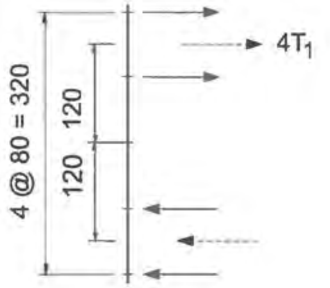
Given:

Determine the number of ¾-inch, A325 bolts required to design the connection in Example 1 as a slip-critical connection for a specified load of 200 kN. Assume clean mill scale faying surfaces and bolts installed by the turn-of-the-nut method.

Solution:

Try 10 bolts (2 rows of 5, at 80 mm pitch)

Specified tension in one bolt:



$$T_1 = \frac{P\, L}{n'\, d_m} = \frac{200 \times 150}{4(2 \times 120)} = 31.3 \text{ kN}$$

$$T_f = 1.5 \times 31.3 = 47.0 \text{ kN} < 141 \text{ kN} \quad \text{(Table 3-4)}$$

Specified shear in one bolt:

$$V = \frac{P}{n} = \frac{200}{10} = 20.0 \text{ kN} < 37.4 \text{ kN} \quad \text{(Table 3-11)}$$

Check combined shear and tension for $V / T = 20.0 / 31.3 = 0.64$

From Table 3-12a,

permissible $V = 25.4$ kN (by interpolation) > 20.0 kN
and permissible $T = 39.8$ kN (by interpolation) > 31.3 kN

WELD DATA

General

Tables in this section are based on CSA S16-14 and other pertinent standards it references. Information on weld resistances and rated electrode tensile strengths in CSA Standard W48 may be found in Table 3-22. Although S16-14 permits the use of non-matching electrodes where permitted in CSA W59, all weld data are provided for matching conditions under static loading, unless noted otherwise.

Tables

Table 3-21 summarizes weld resistances as a function of type of load and type of weld.

Table 3-22 provides information on matching electrode conditions and gives unit factored weld resistances for various rated electrode tensile strengths.

Table 3-23 gives factored shear resistances for a range of effective throats per millimetre of weld length, for various rated electrode tensile strengths and matching electrode applications.

Table 3-24 lists factored shear resistances of a range of fillet weld sizes per millimetre of weld length parallel to the force, for various rated electrode tensile strengths and matching electrode applications.

Table 3-25 shows fillet weld resistances as a function of the angle between the axis of the weld and the direction of the load for matching electrode applications.

Tables 3-26 to 3-33 present the resistance of fillet weld groups in various configurations when they are loaded eccentrically in the plane of the welds.

Table 3-34 presents weld resistances for two parallel fillet welds when the eccentric load is in a plane perpendicular to the plane of the welds.

FACTORED RESISTANCE OF WELDS* Table 3-21

Type of Load	Type of Weld	Factored Resistance
Shear (including tension or compression-induced shear in fillet welds)	Complete and partial joint penetration groove welds, and plug and slot welds	Lesser of: base metal, $V_r = 0.67\,\phi_w A_m F_u$ weld metal, $V_r = 0.67\,\phi_w A_w X_u$
	Fillet welds	Weld metal: $V_r = 0.67\,\phi_w A_w X_u\,(1.00 + 0.50 \sin^{1.5}\theta)\,M_w$ (1) But if over-matched electrodes are used (5), not greater than: $V_r = 0.67\,\phi_w A_m F_u$.
Tension (normal to axis of load)	Complete joint penetration groove weld (made with matching electrodes) (2)	Same as the base metal
	Partial joint penetration groove weld (made with matching electrodes) (2)	$T_r = \phi_w A_n F_u \le \phi A_g F_y$ (3)
	Partial joint penetration groove weld combined with a fillet weld (made with matching electrodes) (2)	$T_r = \phi_w \sqrt{(A_n F_u)^2 + (A_w X_u)^2} \le \phi A_g F_y$
Compression (normal to axis of load)	Complete joint penetration groove weld (made with matching electrodes) (2)	Same as the base metal
	Partial joint penetration groove weld (made with matching electrodes) (2)	Same as the base metal, for the nominal area of the fusion face normal to the compression plus the area of the base metal fitted in contact bearing. (4)

* The detail design of welded joints is to conform to the requirements of CSA Standard W59.

A_m = shear area of effective fusion face.

A_w = area of effective weld throat, plug or slot.

A_n = nominal area of fusion face normal to the tensile force.

θ = angle of axis of weld with the line of action of force (0° for a longitudinal weld and 90° for a transverse weld).

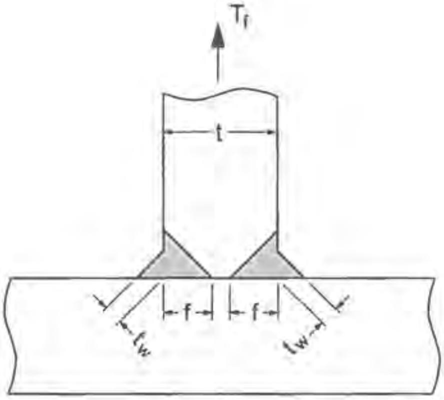
(1) M_w is the strength reduction factor for multi-orientation fillet welds. See CSA S16-14 Clause 13.13.2.2.

(2) The base metal resistance need not be checked for matching electrodes. For information on matching electrodes, see CSA S16-14 Table 4.

(3) When overall ductile behaviour is desired (member yielding before weld fracture) $A_n F_u > A_g F_y$.

(4) See CSA S16-14, Clause 28.5.

(5) However, when electrodes stronger than matching are permitted and used, CSA W59-13 restricts the maximum design value of X_u to that of the matching electrode.



f: to be used for A_n (see CSA Standard W59 for effective fusion face)

t_w: to be used for A_w (see CSA Standard W59 for effective weld throat)

t: to be used for A_g

Application of expression $T_r = \phi_w \sqrt{(A_n F_u)^2 + (A_w X_u)^2} \le \phi A_g F_y$

UNIT FACTORED WELD RESISTANCE — Table 3-22

Matching Electrode Conditions [1]

$\phi_w = 0.67$

WELD METAL			BASE METAL [3]				
Rated Electrode Ultimate Tensile Strength X_u	Unit Factored Shear Resistance on Weld Metal [2]: On Effective Throat $A_w 0.67\phi_w X_u$	Unit Factored Shear Resistance on Weld Metal [2]: Per Unit Area Based on Fillet Size, D $0.67\phi_w X_u/\sqrt{2}$	Standard	Specification and Grade	Specified Minimum Strength: Tensile Strength F_u	Specified Minimum Strength: Yield Stress F_y	Unit Factored Resistance $0.67\phi_w F_u$
MPa	MPa	MPa			MPa	MPa	MPa
430	193	136	CSA G40.21	260W, 260WT	410	260	184
490	**220**	**156**	CSA G40.21	**300W**, 300WT	**440** [(4)]	300	**198**
				300WT (Shapes)	450	300	202
				350W, 350WT [5]	**450**	350	**202**
				350WM, 350WMT	450	345	202
				350WT (Shapes)	480	350	215
				350A, 350AT, 350R	480	350	215
				380W	480	380	215
550	247	175	CSA G40.21	400W, 400WT 400A, 400AT	520	400	233
620	278	197	CSA G40.21	480W, 480WT 480A, 480AT	590	480	265
430	193	136	ASTM	A36	400	250	180
490	**220**	**156**	ASTM	A500 Gr. C	427		192
				Round HSS		317	
				Square and Rectangular		345	
				A572 Gr. 50	450	345	202
				A709M Gr. 345S	450	345	202
				A913 Gr. 50	450	345	202
				A992	**450**	345	**202**
				A588 F_y = 345 MPa	485	345	218
				A709M Gr. 345W, HPS 345W	485	345	218
550	247	175	ASTM	A913 Gr. 65	550	450	247
620	278	197	ASTM	A709M Gr. HPS 485W	585	485	263

1. For more information concerning matching electrode conditions, refer to Table 4 of CSA S16-14 and CSA W59.
2. Factored weld resistance of fillet welds not parallel to force may be increased, whereas the resistance of weld groups comprising multi-orientation fillet segments shall be reduced in accordance with S16-14, Clause 13.13.2.2.
3. The base metal resistance need not be checked for fillet welds in matching electrode conditions. See CSA S16-14 Clause 13.13.2.2 and Table 4.
4. F_u = 410 MPa for 300W HSS; 450 MPa for 300WT Shapes.
5. F_u = 480 MPa for 350WT Shapes.

FACTORED SHEAR RESISTANCE — Table 3-23

On Effective Throat Per Millimetre of Weld Length (kN/mm)

Rated Electrode Tensile Strength, X_u (MPa)	Unit Shear Resistance (MPa)	Effective Throat Thickness (mm): 2	3	4	5	6	7	8	10	12	15	20	25	30
430	193	0.386	0.579	0.772	0.965	1.16	1.35	1.54	1.93	2.32	2.90	3.86	4.83	5.79
490	220	0.440	0.660	0.880	1.10	1.32	1.54	1.76	2.20	2.64	3.30	4.40	5.50	6.60
550	247	0.494	0.741	0.988	1.23	1.48	1.73	1.98	2.47	2.96	3.70	4.94	6.17	7.41
620	278	0.557	0.835	1.11	1.39	1.67	1.95	2.23	2.78	3.34	4.17	5.57	6.96	8.35

FACTORED SHEAR RESISTANCE OF FILLET WELDS

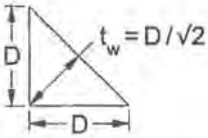


Table 3-24a

Per Millimetre of Weld Length
Parallel to Force ($\theta^* = 0^\circ$)

Matching Electrode Applications

Metric Size Fillet Welds				
Fillet Weld Size, D	Rated Electrode Tensile Strength, X_u (MPa)			
	430	**490**	550	620
mm	kN/mm			
5	0.682	**0.778**	0.873	0.984
6	0.819	**0.933**	1.05	1.18
8	1.09	**1.24**	1.40	1.57
10	1.36	**1.56**	1.75	1.97
12	1.64	**1.87**	2.09	2.36
14	1.91	**2.18**	2.44	2.76
16	2.18	**2.49**	2.79	3.15
18	2.46	**2.80**	3.14	3.54
20	2.73	**3.11**	3.49	3.94

* CSA S16-14 Clause 13.13.2.2: $V_r = 0.67\,\phi_w A_w X_u (1.0 + 0.5 \sin^{1.5}\theta) M_w$

FACTORED SHEAR RESISTANCE

Table 3-24b

of Fillet Welds Per Millimetre of Weld Length for Angle θ

X_u = 490 MPa

Matching Electrode Applications

Weld Size	Angle θ between weld axis and force direction									
	0°	10°	20°	30°	40°	50°	60°	70°	80°	90°
mm	kN/mm									
5	**0.778**	0.806	0.855	0.915	0.978	1.04	1.09	1.13	1.16	1.17
6	**0.933**	0.967	1.03	1.10	1.17	1.25	1.31	1.36	1.39	1.40
8	**1.24**	1.29	1.37	1.46	1.56	1.66	1.75	1.81	1.85	1.87
10	**1.56**	1.61	1.71	1.83	1.96	2.08	2.18	2.26	2.32	2.33
12	**1.87**	1.93	2.05	2.20	2.35	2.49	2.62	2.72	2.78	2.80
14	**2.18**	2.26	2.40	2.56	2.74	2.91	3.05	3.17	3.24	3.27
16	**2.49**	2.58	2.74	2.93	3.13	3.32	3.49	3.62	3.70	3.73
18	**2.80**	2.90	3.08	3.29	3.52	3.74	3.93	4.07	4.17	4.20
20	**3.11**	3.22	3.42	3.66	3.91	4.15	4.36	4.53	4.63	4.67

Note: Only single weld orientations are considered (M_w = 1). For loads on specific weld patterns, see Tables 3-26 to 3-33.

NOTES

STRENGTH REDUCTION FACTOR FOR MULTI-ORIENTATION FILLET WELDS, M_w

Table 3-25

θ_1 (Deg.) \ θ_2 (Deg.)	0°	5°	10°	15°	20°	25°	30°	35°	40°	45°	50°	55°	60°	65°	70°	75°	80°	85°	90°
90°																			1.00
85°																		1.00	.992
80°																	1.00	.992	.983
75°																1.00	.992	.983	.975
70°															1.00	.991	.983	.975	.967
65°														1.00	.991	.983	.975	.966	.958
60°													1.00	.991	.983	.974	.966	.958	.950
55°												1.00	.991	.983	.974	.966	.958	.950	.942
50°											1.00	.991	.982	.974	.966	.957	.949	.941	.933
45°										1.00	.991	.982	.974	.965	.957	.949	.941	.933	.925
40°									1.00	.991	.982	.973	.965	.957	.948	.940	.932	.924	.917
35°								1.00	.991	.982	.973	.965	.956	.948	.940	.932	.924	.916	.908
30°							1.00	.991	.982	.973	.964	.956	.947	.939	.931	.923	.915	.908	.900
25°						1.00	.991	.982	.973	.964	.955	.947	.939	.930	.922	.915	.907	.899	.892
20°					1.00	.991	.981	.972	.964	.955	.946	.938	.930	.922	.914	.906	.898	.891	.883
15°				1.00	.991	.981	.972	.963	.955	.946	.938	.929	.921	.913	.905	.897	.890	.882	.875
10°			1.00	.990	.981	.972	.963	.954	.945	.937	.929	.920	.912	.904	.897	.889	.881	.874	.867
5°		1.00	.990	.981	.972	.963	.954	.945	.936	.928	.920	.912	.904	.896	.888	.880	.873	.866	.858
0°	1.00	.990	.981	.971	.962	.953	.944	.936	.927	.919	.911	.903	.895	.887	.879	.872	.864	.857	.850

θ_1 = Angle of axis of weld segment under consideration, with respect to the line of action of applied force

θ_2 = Angle of axis of weld segment in the joint that is nearest to 90°, with respect to the line of action of applied force

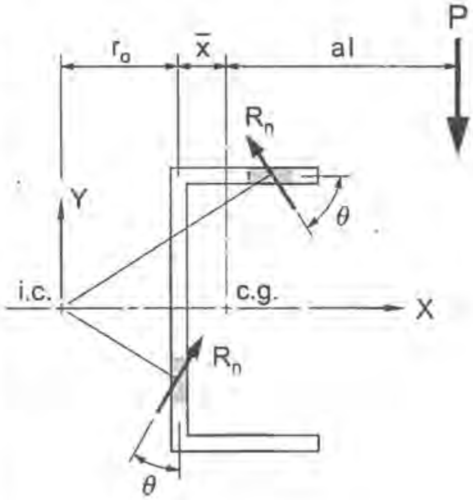
See CSA S16-14 Clause 13.13.2.2.

ECCENTRIC LOADS ON WELD GROUPS

When the line of action of a load on a weld group does not pass through the centre of gravity of the group, the connection is eccentrically loaded. The elastic method of analysis was the traditional approach used in the first two editions of this Handbook.

The third edition incorporated the work of Butler *et al.* (1972), which showed that the margins of safety for eccentrically loaded weld groups analysed elastically were both high and variable. They suggested a method of analysis based on the load-deformation characteristics of the weld and the instantaneous centre of rotation analogy similar to that for eccentrically loaded bolt groups. For this method of analysis, the weld group is considered to be divided into a discrete number of finite weld elements. The resistance of the weld group to the external eccentric load is provided by the combined resistances of the weld elements.

The resistance of each weld element is assumed to act on a line perpendicular to the radius extending from the instantaneous centre of rotation to the centroid of the weld element, as shown on the accompanying figure, where θ is the angle between the axis of the weld and the direction of the weld resistance, R_n. The ultimate load is obtained when the ultimate strength and deformation of some weld element is reached. The resistance of the remaining weld elements is then computed by assuming that deformations vary linearly with the distance from the instantaneous centre. The correct location of the instantaneous centre is assured when the connection is in equilibrium, that is, when the three equations of statics, $\Sigma F_x = 0$, $\Sigma F_y = 0$ and $\Sigma M = 0$ are simultaneously satisfied.



Beginning with the ninth edition of this Handbook, design tables for the factored resistances of eccentrically loaded weld groups (Tables 3-26 to 3-33 on the following pages) were calculated using the work of Lesik and Kennedy (1990). This method of analysis was also based on the instantaneous centre of rotation method and featured refined load-deformation characteristics. Notably, the shear strength of a fillet weld, V_θ, at an angle θ from the line of action of the applied load is expressed by:

$$\frac{V_\theta}{V_o} = (1 + 0.5 \sin^{1.5} \theta)$$

where V_o is the shear strength of a longitudinal weld.

Tables

1. Use of Tables

The coefficients C listed in Tables 3-26 to 3-33 are based on a matching electrode, $X_u = 490$ MPa, and a resistance factor for welded connections, $\phi_w = 0.67$. The tables are applicable to matching electrode applications only. The base metal resistance has not been included; therefore, the tables are not suitable for over-matched applications. For further information, see CSA S16-14 Clause 13.13.2.2 and Table 4.

(a) To determine the capacity P of the eccentrically loaded weld group in kN, multiply the appropriate coefficient C by the number of millimetres of weld size D and the length of the weld L, in millimetres.

(b) To determine the required number of millimetres of weld size D, divide the factored load P, in kN, by the appropriate coefficient C and the length of the weld L, in mm.

2. Other Weld Configurations

For situations not covered by the tables of *Eccentric Loads on Weld Groups*, interpolating between weld configurations in the tables which "bracket" the situation being evaluated will often be sufficient to confirm adequacy.

Example

For an example on the use of these tables, see the design example following Table 3-33.

References

BUTLER, L.J., PAL, S., and KULAK, G.L. 1972. Eccentrically loaded welded connections. ASCE Journal of the Structural Division, **98**(ST5), May.

KULAK, G.L., and TIMLER, P.A. 1984. Tests on eccentrically loaded fillet welds. Structural Engineering Report No. 124, December, University of Alberta.

LESIK, D.F., and KENNEDY, D.J.L. 1990. Ultimate strength of fillet welded connections loaded in plane. Canadian Journal of Civil Engineering, **17**(1), February.

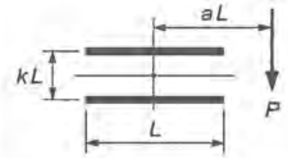
SWANNELL, P., and SKEWES, I.C. 1977. Design of welded brackets loaded in-plane: general theoretical ultimate load techniques and experimental programme. Australian WRA, RC #46, December, University of Queensland.

ECCENTRIC LOADS ON WELD GROUPS

Table 3-26

Coefficients C

Matching Electrode X_u = 490 MPa



P = Factored eccentric load, kN
L = Length of each weld, mm
D = Size of fillet weld, mm
C = Coefficients tabulated below
$P = CDL$

Required Minimum $C = \dfrac{P}{DL}$

Required Minimum $D = \dfrac{P}{CL}$

Required Minimum $L = \dfrac{P}{CD}$

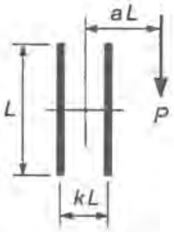
a	k															
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	.467	.467	.467	.467	.467	.467	.467	.467	.467	.467	.467	.467	.467	.467	.467	.467
0.05	.395	.400	.410	.422	.432	.440	.446	.451	.454	.456	.457	.460	.461	.462	.462	.463
0.10	.355	.359	.369	.383	.396	.409	.420	.429	.436	.441	.445	.451	.454	.457	.458	.459
0.15	.321	.324	.334	.346	.362	.377	.391	.403	.413	.422	.428	.438	.445	.449	.452	.455
0.20	.290	.294	.303	.315	.330	.346	.362	.376	.389	.400	.409	.423	.433	.440	.445	.448
0.25	.264	.267	.276	.288	.302	.318	.335	.351	.365	.378	.389	.407	.419	.429	.436	.441
0.30	.241	.244	.253	.265	.278	.293	.310	.326	.342	.356	.368	.389	.405	.416	.425	.432
0.35	.221	.224	.232	.243	.256	.271	.287	.304	.320	.334	.349	.371	.389	.403	.414	.423
0.40	.204	.207	.215	.225	.238	.252	.267	.283	.299	.314	.329	.354	.374	.390	.402	.412
0.45	.189	.192	.199	.209	.221	.234	.249	.264	.280	.296	.311	.336	.358	.376	.390	.401
0.50	.175	.178	.185	.195	.206	.219	.232	.247	.263	.278	.293	.321	.343	.362	.378	.390
0.60	.153	.156	.162	.171	.181	.193	.206	.219	.233	.247	.262	.290	.314	.335	.353	.368
0.70	.136	.138	.144	.152	.161	.172	.184	.196	.209	.222	.235	.263	.288	.310	.329	.346
0.80	.121	.124	.129	.136	.145	.155	.166	.177	.188	.201	.213	.239	.264	.287	.306	.324
0.90	.110	.112	.117	.124	.131	.141	.150	.161	.172	.183	.195	.219	.243	.265	.286	.304
1.00	.100	.102	.106	.113	.120	.129	.138	.148	.158	.168	.179	.201	.224	.246	.267	.285
1.20	.085	.086	.090	.096	.102	.110	.118	.126	.135	.144	.153	.173	.193	.214	.233	.251
1.40	.074	.075	.079	.083	.089	.096	.103	.110	.118	.126	.134	.152	.169	.187	.206	.224
1.60	.065	.066	.070	.074	.079	.084	.091	.097	.105	.111	.119	.135	.151	.167	.184	.200
1.80	.058	.059	.062	.066	.070	.076	.081	.087	.094	.100	.107	.121	.135	.150	.165	.180
2.00	.053	.054	.056	.060	.064	.069	.074	.079	.085	.091	.097	.110	.123	.136	.150	.164
2.20	.048	.049	.051	.054	.058	.063	.067	.073	.078	.083	.089	.100	.112	.125	.137	.150
2.40	.044	.045	.047	.050	.054	.058	.062	.067	.071	.077	.082	.093	.103	.115	.127	.138
2.60	.041	.042	.044	.046	.050	.053	.057	.062	.066	.071	.076	.086	.096	.107	.117	.128
2.80	.038	.039	.041	.043	.046	.050	.054	.058	.062	.066	.071	.080	.089	.099	.109	.120
3.00	.036	.036	.038	.040	.043	.046	.050	.054	.058	.062	.066	.075	.084	.093	.102	.112

When over-matched electrodes are used, the base metal capacity should also be checked (S16-14 Clause 13.13.2.2).

ECCENTRIC LOADS ON WELD GROUPS

Matching Electrode X_u = 490 MPa

Coefficients C



P = Factored eccentric load, kN
L = Length of each weld, mm
D = Size of fillet weld, mm
C = Coefficients tabulated below

$$P = CDL$$

Required Minimum $C = \frac{P}{DL}$

Required Minimum $D = \frac{P}{CL}$

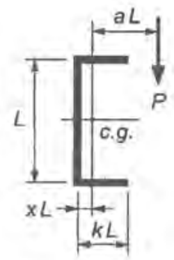
Required Minimum $L = \frac{P}{CD}$

a	k															
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	.311	.311	.311	.311	.311	.311	.311	.311	.311	.311	.311	.311	.311	.311	.311	.311
0.05	.311	.311	.311	.311	.311	.311	.311	.309	.308	.307	.306	.305	.303	.302	.302	.301
0.10	.311	.311	.311	.311	.309	.307	.305	.303	.301	.300	.299	.297	.295	.295	.294	.294
0.15	.309	.307	.305	.302	.299	.297	.295	.293	.292	.290	.289	.288	.287	.286	.286	.286
0.20	.296	.294	.291	.288	.286	.284	.282	.281	.280	.279	.279	.278	.278	.278	.278	.279
0.25	.278	.276	.274	.272	.271	.269	.268	.268	.267	.267	.267	.268	.268	.269	.270	.271
0.30	.259	.257	.256	.255	.255	.254	.254	.254	.255	.255	.256	.257	.259	.260	.262	.263
0.35	.240	.239	.238	.238	.238	.239	.240	.241	.242	.243	.244	.247	.249	.252	.254	.256
0.40	.222	.221	.222	.222	.223	.225	.227	.228	.230	.232	.234	.237	.240	.243	.246	.248
0.45	.205	.205	.206	.208	.210	.212	.214	.216	.218	.221	.223	.227	.231	.235	.238	.241
0.50	.191	.191	.192	.194	.197	.200	.202	.205	.208	.211	.214	.219	.223	.227	.231	.234
0.60	.165	.166	.168	.171	.175	.178	.182	.186	.189	.192	.195	.202	.208	.213	.217	.221
0.70	.145	.146	.148	.152	.156	.160	.165	.169	.173	.176	.180	.187	.194	.200	.205	.209
0.80	.129	.130	.133	.137	.141	.145	.150	.155	.159	.163	.167	.174	.181	.187	.193	.198
0.90	.116	.117	.120	.124	.128	.133	.138	.142	.147	.151	.155	.163	.170	.177	.183	.188
1.00	.105	.106	.109	.113	.118	.122	.127	.132	.137	.141	.145	.153	.160	.168	.174	.179
1.20	.089	.090	.092	.096	.101	.105	.110	.115	.119	.124	.128	.136	.144	.151	.158	.163
1.40	.076	.077	.080	.084	.088	.093	.097	.101	.106	.110	.114	.122	.130	.137	.144	.149
1.60	.067	.068	.070	.074	.078	.082	.086	.091	.095	.099	.103	.111	.119	.126	.133	.138
1.80	.060	.061	.063	.066	.070	.074	.078	.082	.086	.090	.094	.102	.109	.116	.123	.128
2.00	.054	.055	.057	.060	.064	.067	.071	.075	.079	.083	.087	.094	.101	.107	.114	.119
2.20	.049	.050	.052	.055	.058	.062	.065	.069	.073	.076	.080	.087	.094	.100	.106	.112
2.40	.045	.046	.048	.050	.053	.057	.060	.064	.067	.071	.074	.081	.087	.094	.100	.106
2.60	.042	.042	.044	.047	.049	.053	.056	.059	.063	.066	.069	.076	.082	.088	.094	.100
2.80	.039	.039	.041	.043	.046	.049	.052	.055	.059	.062	.065	.071	.077	.083	.089	.094
3.00	.036	.037	.038	.041	.043	.046	.049	.052	.055	.058	.061	.067	.073	.079	.084	.089

When over-matched electrodes are used, the base metal capacity should also be checked (S16-14 Clause 13.13.2.2).
The effect of eccentricity is negligible for cases above the solid horizontal line.

Coefficients C

Matching Electrode X_u = 490 MPa



P = Factored eccentric load, kN
L = Length of weld parallel to load, mm
D = Size of fillet weld, mm
C = Coefficients tabulated below
xL = Distance from vertical weld to centre of gravity of weld group

$$P = CDL$$

Required Minimum $C = \dfrac{P}{DL}$

Required Minimum $D = \dfrac{P}{CL}$

Required Minimum $L = \dfrac{P}{CD}$

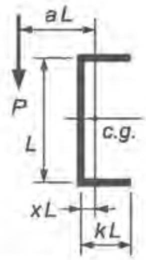
a	k															
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	.156	.179	.226	.272	.319	.366	.412	.459	.505	.552	.599	.692	.785	.879	.972	1.065
0.05	.156	.179	.226	.272	.319	.366	.412	.459	.505	.552	.599	.692	.785	.879	.972	1.065
0.10	.156	.179	.226	.272	.319	.366	.412	.459	.505	.552	.599	.692	.785	.877	.967	1.058
0.15	.155	.179	.226	.272	.319	.364	.408	.452	.496	.540	.584	.672	.760	.848	.936	1.025
0.20	.148	.179	.223	.264	.305	.347	.388	.430	.473	.515	.557	.643	.728	.814	.900	.986
0.25	.139	.173	.210	.248	.287	.327	.366	.407	.447	.488	.529	.611	.694	.778	.862	.947
0.30	.129	.162	.196	.232	.269	.306	.344	.383	.421	.460	.500	.580	.660	.743	.825	.909
0.35	.120	.150	.182	.216	.251	.286	.322	.359	.396	.433	.472	.549	.628	.708	.790	.872
0.40	.111	.139	.170	.201	.234	.267	.301	.336	.372	.408	.445	.521	.598	.676	.755	.836
0.45	.103	.130	.158	.187	.218	.249	.282	.316	.350	.385	.420	.493	.569	.646	.724	.804
0.50	.095	.121	.147	.175	.204	.234	.264	.297	.330	.363	.398	.468	.542	.617	.693	.773
0.60	.083	.105	.128	.153	.179	.206	.234	.263	.293	.324	.356	.424	.492	.566	.639	.716
0.70	.073	.093	.113	.136	.159	.183	.209	.235	.264	.292	.323	.386	.452	.521	.591	.665
0.80	.065	.083	.101	.121	.142	.165	.188	.213	.238	.265	.293	.352	.416	.482	.550	.620
0.90	.058	.074	.091	.109	.129	.149	.171	.194	.217	.243	.269	.325	.384	.446	.514	.581
1.00	.053	.067	.083	.100	.117	.136	.156	.177	.200	.223	.248	.301	.358	.416	.478	.544
1.20	.044	.057	.070	.084	.099	.115	.133	.152	.171	.192	.214	.262	.312	.365	.422	.482
1.40	.038	.049	.060	.073	.086	.101	.116	.133	.150	.169	.189	.230	.276	.324	.375	.431
1.60	.034	.043	.053	.064	.076	.089	.103	.117	.133	.150	.168	.205	.247	.291	.338	.387
1.80	.030	.038	.048	.057	.068	.079	.092	.105	.119	.134	.151	.185	.223	.263	.306	.353
2.00	.027	.035	.043	.052	.062	.072	.083	.095	.108	.122	.137	.168	.202	.240	.280	.323
2.20	.025	.032	.039	.047	.056	.066	.076	.087	.099	.112	.125	.154	.185	.221	.257	.297
2.40	.022	.029	.036	.043	.052	.060	.070	.080	.091	.103	.116	.142	.172	.203	.238	.275
2.60	.021	.027	.033	.040	.047	.056	.065	.074	.084	.095	.107	.132	.159	.189	.221	.256
2.80	.019	.025	.031	.037	.044	.052	.060	.069	.079	.089	.099	.123	.148	.176	.207	.239
3.00	.018	.023	.029	.035	.042	.048	.056	.065	.074	.083	.093	.115	.139	.166	.194	.225
x	0	.008	.029	.056	.089	.125	.164	.204	.246	.289	.333	.424	.516	.610	.704	.800

When over-matched electrodes are used, the base metal capacity should also be checked (S16-14 Clause 13.13.2.2).
The effect of eccentricity is negligible for cases above the solid horizontal line.

ECCENTRIC LOADS ON WELD GROUPS

Matching Electrode X_u = 490 MPa

Coefficients C



P = Factored eccentric load, kN
L = Length of weld parallel to load, mm
D = Size of fillet weld, mm
C = Coefficients tabulated below
xL = Distance from vertical weld to centre of gravity of weld group

$$P = CDL$$

$$\text{Required Minimum } C = \frac{P}{DL}$$

$$\text{Required Minimum } D = \frac{P}{CL}$$

$$\text{Required Minimum } L = \frac{P}{CD}$$

a	k															
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	.156	.179	.226	.272	.319	.366	.412	.459	.505	.552	.599	.692	.785	.879	.972	1.065
0.05	.156	.179	.226	.272	.319	.366	.412	.459	.505	.552	.599	.686	.772	.857	.943	1.029
0.10	.156	.179	.226	.272	.319	.366	.412	.454	.496	.537	.579	.663	.746	.829	.912	.996
0.15	.155	.179	.226	.272	.314	.354	.395	.435	.475	.516	.556	.636	.717	.799	.880	.962
0.20	.148	.179	.221	.260	.299	.337	.376	.414	.453	.492	.531	.609	.688	.767	.847	.928
0.25	.139	.173	.209	.245	.282	.319	.355	.393	.430	.468	.505	.581	.658	.736	.815	.894
0.30	.129	.162	.195	.230	.264	.300	.335	.371	.407	.443	.480	.554	.629	.705	.783	.861
0.35	.120	.151	.182	.214	.248	.281	.316	.350	.385	.420	.455	.527	.601	.675	.752	.829
0.40	.111	.140	.170	.200	.232	.264	.296	.330	.363	.397	.432	.502	.574	.647	.721	.797
0.45	.103	.130	.158	.187	.217	.247	.279	.311	.343	.376	.410	.478	.548	.620	.693	.768
0.50	.095	.121	.147	.174	.203	.232	.263	.293	.324	.356	.388	.456	.524	.595	.667	.740
0.60	.083	.105	.129	.153	.179	.206	.233	.262	.291	.321	.352	.415	.480	.547	.617	.688
0.70	.073	.093	.114	.136	.160	.184	.210	.237	.264	.291	.320	.380	.442	.506	.574	.642
0.80	.065	.083	.102	.122	.144	.167	.190	.215	.240	.266	.294	.350	.408	.471	.535	.601
0.90	.058	.075	.092	.111	.131	.152	.173	.196	.220	.245	.270	.323	.380	.439	.500	.564
1.00	.053	.068	.084	.101	.119	.139	.160	.181	.203	.227	.251	.301	.355	.410	.468	.530
1.20	.044	.057	.071	.085	.101	.118	.137	.156	.176	.197	.218	.263	.311	.363	.416	.472
1.40	.038	.049	.061	.074	.088	.103	.119	.136	.154	.173	.192	.233	.276	.324	.373	.425
1.60	.034	.043	.054	.065	.077	.091	.105	.120	.137	.154	.171	.209	.249	.292	.337	.385
1.80	.030	.039	.048	.058	.069	.081	.094	.108	.123	.138	.155	.189	.226	.265	.307	.352
2.00	.027	.035	.043	.052	.062	.073	.085	.097	.111	.125	.141	.172	.206	.243	.281	.323
2.20	.025	.032	.039	.048	.057	.067	.077	.089	.101	.114	.128	.158	.190	.224	.260	.299
2.40	.022	.029	.036	.044	.052	.061	.071	.081	.093	.105	.118	.146	.176	.208	.242	.278
2.60	.021	.027	.033	.040	.048	.057	.066	.075	.086	.097	.109	.135	.163	.193	.225	.259
2.80	.019	.025	.031	.037	.045	.053	.061	.070	.080	.090	.102	.126	.153	.181	.211	.243
3.00	.018	.023	.029	.035	.042	.050	.057	.066	.075	.084	.095	.118	.143	.170	.198	.229
x	0	.008	.029	.056	.089	.125	.164	.204	.246	.289	.333	.424	.516	.610	.704	.800

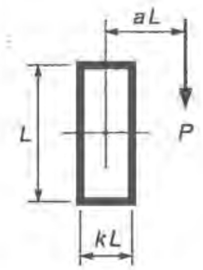
When over-matched electrodes are used, the base metal capacity should also be checked (S16-14 Clause 13.13.2.2). The effect of eccentricity is negligible for cases above the solid horizontal line.

ECCENTRIC LOADS ON WELD GROUPS

Table 3-30

Coefficients C

Matching Electrode X_u = 490 MPa



P = Factored eccentric load, kN

L = Length of longer welds, mm

D = Size of fillet weld, mm

C = Coefficients tabulated below

Note: When load P is perpendicular to longer side L, use table on facing page.

$$P = CDL$$

$$\text{Required Minimum } C = \frac{P}{DL}$$

$$\text{Required Minimum } D = \frac{P}{CL}$$

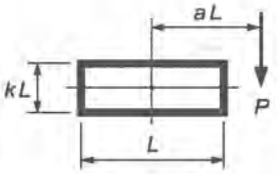
$$\text{Required Minimum } L = \frac{P}{CD}$$

a	k										
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
0.00	.311	.311	.358	.404	.451	.498	.544	.591	.638	.684	.731
0.05	.311	.311	.358	.404	.451	.498	.544	.591	.638	.684	.731
0.10	.311	.311	.358	.404	.451	.498	.544	.591	.638	.684	.731
0.15	.309	.311	.358	.404	.451	.498	.544	.587	.628	.670	.711
0.20	.296	.311	.358	.403	.441	.480	.519	.559	.599	.639	.680
0.25	.278	.311	.345	.380	.416	.454	.492	.530	.569	.608	.648
0.30	.259	.290	.323	.356	.392	.427	.464	.501	.539	.577	.615
0.35	.240	.270	.301	.333	.367	.402	.438	.473	.510	.547	.584
0.40	.222	.250	.280	.312	.344	.378	.412	.447	.483	.519	.555
0.45	.206	.233	.261	.291	.322	.355	.388	.422	.457	.492	.529
0.50	.191	.217	.244	.273	.303	.335	.366	.400	.434	.468	.503
0.60	.166	.189	.214	.240	.268	.297	.328	.360	.392	.425	.459
0.70	.145	.167	.189	.214	.240	.268	.296	.325	.356	.387	.420
0.80	.129	.148	.169	.192	.217	.243	.270	.298	.326	.356	.386
0.90	.116	.133	.153	.174	.198	.221	.246	.273	.299	.327	.357
1.00	.105	.121	.139	.159	.180	.203	.226	.251	.277	.303	.331
1.20	.089	.102	.118	.136	.154	.174	.196	.218	.241	.265	.288
1.40	.076	.088	.102	.118	.134	.152	.171	.191	.211	.233	.255
1.60	.067	.078	.090	.104	.119	.135	.152	.170	.189	.208	.228
1.80	.060	.069	.080	.093	.107	.121	.137	.153	.170	.188	.206
2.00	.054	.062	.072	.084	.097	.110	.124	.139	.154	.171	.188
2.20	.049	.057	.066	.077	.088	.100	.113	.127	.142	.157	.173
2.40	.045	.052	.061	.070	.081	.092	.105	.117	.131	.145	.160
2.60	.042	.048	.056	.065	.075	.086	.097	.109	.121	.134	.148
2.80	.039	.045	.052	.061	.070	.080	.090	.101	.113	.125	.138
3.00	.036	.042	.049	.057	.065	.074	.084	.095	.106	.117	.129

When over-matched electrodes are used, the base metal capacity should also be checked (S16-14 Clause 13.13.2.2).
The effect of eccentricity is negligible for cases above the solid horizontal line.

Matching Electrode X_u = 490 MPa

Coefficients C



P = Factored eccentric load, kN
L = Length of longer welds, mm
D = Size of fillet weld, mm
C = Coefficients tabulated below

Note: When load P is parallel to longer side L, use table on facing page.

$$P = CDL$$

Required Minimum $C = \frac{P}{DL}$

Required Minimum $D = \frac{P}{CL}$

Required Minimum $L = \frac{P}{CD}$

a	k = 0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
0.00	.467	.493	.519	.546	.572	.599	.625	.652	.678	.705	.731
0.05	.395	.430	.469	.508	.548	.587	.624	.652	.678	.705	.731
0.10	.355	.389	.425	.465	.506	.547	.588	.627	.666	.704	.731
0.15	.321	.353	.389	.427	.467	.509	.550	.591	.632	.672	.711
0.20	.290	.322	.357	.395	.432	.473	.514	.556	.598	.639	.680
0.25	.264	.295	.328	.365	.403	.441	.482	.523	.564	.606	.648
0.30	.241	.270	.303	.338	.375	.412	.451	.491	.532	.574	.615
0.35	.221	.249	.280	.314	.350	.386	.423	.462	.503	.543	.584
0.40	.204	.230	.260	.292	.328	.362	.398	.436	.475	.515	.555
0.45	.189	.214	.242	.274	.306	.341	.375	.412	.451	.489	.529
0.50	.175	.199	.227	.256	.288	.322	.356	.390	.428	.465	.503
0.60	.153	.175	.200	.227	.256	.287	.320	.353	.387	.421	.459
0.70	.136	.155	.178	.203	.230	.259	.289	.320	.353	.386	.420
0.80	.121	.139	.160	.184	.208	.235	.263	.292	.323	.354	.386
0.90	.110	.127	.146	.167	.191	.215	.241	.269	.296	.326	.357
1.00	.100	.115	.133	.153	.175	.197	.222	.248	.275	.303	.331
1.20	.085	.098	.114	.131	.150	.170	.191	.214	.238	.263	.288
1.40	.074	.085	.099	.114	.131	.149	.168	.188	.209	.232	.255
1.60	.065	.075	.088	.101	.116	.133	.150	.168	.187	.207	.228
1.80	.058	.068	.078	.091	.104	.119	.135	.151	.169	.187	.206
2.00	.053	.061	.071	.083	.095	.108	.122	.137	.153	.170	.188
2.20	.048	.056	.065	.075	.087	.099	.112	.126	.141	.156	.173
2.40	.044	.051	.060	.069	.080	.091	.103	.116	.130	.145	.160
2.60	.041	.047	.055	.064	.074	.085	.096	.108	.121	.134	.148
2.80	.038	.044	.051	.060	.069	.079	.089	.100	.113	.125	.138
3.00	.036	.041	.048	.056	.065	.074	.084	.094	.105	.117	.129

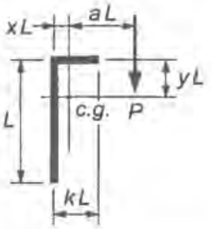
When over-matched electrodes are used, the base metal capacity should also be checked (S16-14 Clause 13.13.2.2).
The effect of eccentricity is negligible for cases above the solid horizontal line.

ECCENTRIC LOADS ON WELD GROUPS

Table 3-32

Coefficients C

Matching Electrode X_u = 490 MPa



P = Factored eccentric load, kN
L = Length of weld parallel to load, mm
D = Size of fillet weld, mm
C = Coefficients tabulated below
xL = Distance from vertical weld to centre of gravity of weld group
yL = Distance from horizontal weld to centre of gravity of weld group

$P = CDL$

Required Minimum $C = \frac{P}{DL}$

Required Minimum $D = \frac{P}{CL}$

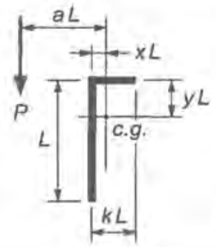
Required Minimum $L = \frac{P}{CD}$

a	k															
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	.156	.156	.179	.202	.226	.249	.272	.296	.319	.342	.366	.412	.459	.505	.552	.599
0.05	.156	.156	.179	.202	.226	.249	.272	.296	.319	.342	.366	.412	.459	.505	.552	.599
0.10	.156	.156	.179	.202	.226	.249	.272	.296	.319	.342	.366	.412	.459	.505	.552	.599
0.15	.155	.156	.179	.202	.226	.249	.272	.296	.319	.341	.362	.406	.449	.493	.538	.582
0.20	.148	.156	.179	.202	.223	.243	.263	.283	.304	.324	.345	.387	.429	.472	.516	.559
0.25	.139	.156	.174	.192	.210	.229	.248	.267	.286	.306	.326	.367	.409	.451	.494	.537
0.30	.129	.145	.162	.179	.196	.214	.232	.250	.269	.288	.307	.347	.388	.430	.472	.514
0.35	.120	.134	.150	.167	.183	.199	.216	.234	.252	.270	.289	.328	.368	.408	.451	.493
0.40	.111	.124	.139	.154	.170	.186	.202	.218	.235	.253	.271	.309	.349	.389	.430	.472
0.45	.103	.115	.129	.143	.158	.173	.188	.204	.220	.238	.255	.291	.330	.369	.410	.451
0.50	.095	.107	.119	.133	.147	.161	.176	.191	.206	.223	.240	.276	.314	.352	.392	.433
0.60	.083	.093	.104	.115	.128	.141	.154	.168	.182	.198	.213	.247	.283	.321	.359	.399
0.70	.073	.082	.091	.102	.113	.125	.137	.149	.163	.177	.192	.223	.257	.294	.330	.369
0.80	.065	.073	.081	.090	.100	.111	.123	.134	.147	.159	.174	.203	.236	.270	.306	.342
0.90	.058	.065	.073	.081	.090	.100	.111	.121	.133	.145	.158	.187	.217	.249	.283	.318
1.00	.053	.059	.066	.074	.082	.091	.101	.111	.122	.133	.145	.171	.201	.231	.264	.297
1.20	.044	.050	.056	.062	.069	.077	.086	.095	.104	.114	.125	.149	.174	.202	.231	.263
1.40	.038	.043	.048	.054	.060	.067	.074	.082	.090	.099	.109	.130	.153	.178	.205	.234
1.60	.034	.038	.042	.047	.052	.059	.066	.073	.080	.088	.097	.115	.137	.159	.184	.210
1.80	.030	.034	.038	.042	.047	.052	.059	.065	.072	.079	.087	.104	.123	.144	.167	.191
2.00	.027	.030	.034	.038	.042	.047	.053	.059	.065	.071	.079	.095	.112	.131	.152	.174
2.20	.025	.028	.031	.035	.038	.043	.048	.054	.059	.065	.072	.086	.102	.121	.140	.160
2.40	.022	.025	.028	.032	.035	.040	.044	.049	.055	.060	.066	.080	.095	.111	.129	.149
2.60	.021	.024	.026	.029	.033	.036	.041	.046	.050	.056	.061	.074	.088	.103	.120	.138
2.80	.019	.022	.024	.027	.030	.034	.038	.042	.047	.052	.057	.069	.082	.096	.112	.129
3.00	.018	.020	.023	.025	.029	.032	.036	.040	.044	.048	.053	.064	.077	.090	.105	.121
x	0	.005	.017	.035	.057	.083	.113	.144	.178	.213	.250	.327	.408	.492	.579	.667
y	.500	.455	.417	.385	.357	.333	.313	.294	.278	.263	.250	.227	.208	.192	.179	.167

When over-matched electrodes are used, the base metal capacity should also be checked (S16-14 Clause 13.13.2.2).
The effect of eccentricity is negligible for cases above the solid horizontal line.

Matching Electrode X_u = 490 MPa

Coefficients C



P = Factored eccentric load, kN
L = Length of weld parallel to load, mm
D = Size of fillet weld, mm
C = Coefficients tabulated below
xL = Distance from vertical weld to centre of gravity of weld group
yL = Distance from horizontal weld to centre of gravity of weld group

$P = CDL$

Required Minimum $C = \frac{P}{DL}$

Required Minimum $D = \frac{P}{CL}$

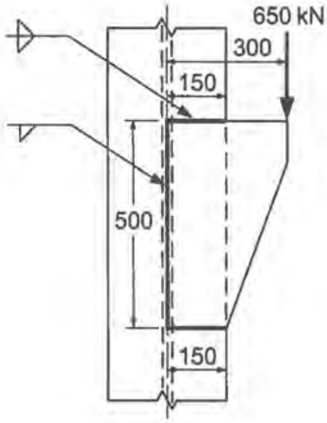
Required Minimum $L = \frac{P}{CD}$

a	k															
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	.156	.156	.179	.202	.226	.249	.272	.296	.319	.342	.366	.412	.459	.505	.552	.599
0.05	.156	.156	.179	.202	.226	.249	.272	.296	.319	.342	.366	.404	.441	.479	.518	.557
0.10	.156	.156	.179	.202	.226	.249	.272	.295	.313	.330	.348	.384	.420	.458	.496	.535
0.15	.155	.156	.179	.202	.226	.245	.262	.279	.296	.312	.329	.364	.400	.437	.475	.514
0.20	.148	.156	.179	.198	.214	.230	.246	.261	.277	.294	.310	.344	.380	.417	.455	.494
0.25	.139	.155	.170	.185	.200	.215	.229	.245	.260	.276	.293	.326	.362	.399	.436	.475
0.30	.129	.144	.158	.172	.186	.200	.214	.229	.244	.259	.276	.310	.345	.381	.419	.457
0.35	.120	.134	.147	.160	.173	.186	.200	.214	.229	.244	.260	.294	.328	.365	.403	.440
0.40	.111	.124	.136	.148	.161	.173	.187	.201	.215	.230	.247	.279	.314	.350	.386	.425
0.45	.103	.115	.126	.138	.150	.162	.175	.189	.203	.218	.233	.266	.299	.335	.372	.410
0.50	.095	.107	.117	.128	.140	.152	.164	.178	.191	.206	.221	.253	.286	.321	.357	.395
0.60	.083	.093	.102	.113	.123	.134	.146	.159	.171	.185	.200	.230	.263	.296	.331	.367
0.70	.073	.082	.090	.099	.109	.120	.131	.142	.155	.168	.182	.211	.242	.274	.308	.343
0.80	.065	.073	.081	.089	.098	.108	.118	.129	.141	.153	.166	.194	.224	.254	.287	.320
0.90	.058	.065	.073	.080	.089	.098	.107	.118	.129	.140	.153	.179	.207	.237	.268	.301
1.00	.053	.059	.066	.073	.081	.089	.098	.108	.118	.129	.141	.166	.193	.222	.251	.283
1.20	.044	.050	.056	.062	.069	.076	.084	.092	.102	.111	.122	.145	.169	.195	.223	.252
1.40	.038	.043	.048	.053	.059	.066	.073	.081	.089	.098	.107	.128	.150	.174	.200	.227
1.60	.034	.038	.042	.047	.052	.058	.065	.071	.079	.087	.096	.114	.135	.157	.181	.205
1.80	.030	.034	.038	.042	.047	.052	.058	.064	.071	.078	.086	.103	.122	.142	.164	.188
2.00	.027	.030	.034	.038	.042	.047	.052	.058	.064	.071	.078	.094	.111	.130	.150	.172
2.20	.025	.028	.031	.034	.038	.043	.048	.053	.059	.065	.072	.086	.102	.120	.139	.159
2.40	.022	.025	.028	.032	.035	.039	.044	.049	.054	.060	.066	.080	.095	.111	.129	.148
2.60	.021	.023	.026	.029	.033	.036	.041	.045	.050	.055	.061	.074	.088	.104	.120	.138
2.80	.019	.022	.024	.027	.030	.034	.038	.042	.047	.052	.057	.069	.082	.097	.112	.130
3.00	.018	.020	.023	.025	.028	.032	.035	.039	.044	.048	.053	.065	.077	.091	.106	.122
x	0	.005	.017	.035	.057	.083	.113	.144	.178	.213	.250	.327	.408	.492	.579	.667
y	.500	.455	.417	.385	.357	.333	.313	.294	.278	.263	.250	.227	.208	.192	.179	.167

When over-matched electrodes are used, the base metal capacity should also be checked (S16-14 Clause 13.13.2.2).
The effect of eccentricity is negligible for cases above the solid horizontal line.

Example

Given:



A column bracket of G40.21-300W steel supports a factored load of 650 kN. The width of the bracket is 300 mm. Welds are made using matching electrodes $X_u = 490$ MPa. For the weld configuration shown, find the required weld size.

Solution:

Referring to Table 3-28,

$$D = \frac{P}{C\,L}$$

= number of millimetres of fillet weld leg size

$k = 150 / 500 = 0.3$

From the bottom line of Table 3-28, for $k = 0.3 \quad x = 0.056$

Referring to the figure in Table 3-28, $a\,L + x\,L = 300$

For $L = 500$, $500\,a + 0.056\,(500) = 300$, $a = 0.544$

For $a = 0.544$ and $k = 0.3$, $C = 0.165$ by interpolation

Therefore, $D = \dfrac{650}{0.165 \times 500} = 7.88 \quad$ say 8 mm

Notes:

1. The final choice of the fillet weld size to be used in an actual connection will also depend on the minimum and maximum sizes required by a) the physical thickness of the parts joined and b) the requirements of Standard CSA W59.
2. The strength of an actual connection will also depend on the resistances of the connected parts.

ECCENTRIC LOADS ON WELD GROUPS
SHEAR AND MOMENT

Two configurations involving a vertical load applied out-of-plane with respect to the fillet weld group are shown in Figure 3-2. In Figure 3-2(a), a plate is welded to the flange of a column with a pair of vertical fillet welds. The eccentricity of the load, P, with respect to the weld group is denoted by $a\,L$, where L is the weld length. In Figure 3-2(b), a stiffened seat is welded to the column using a tee-shaped weld configuration. The length of the horizontal welds is denoted by kL.

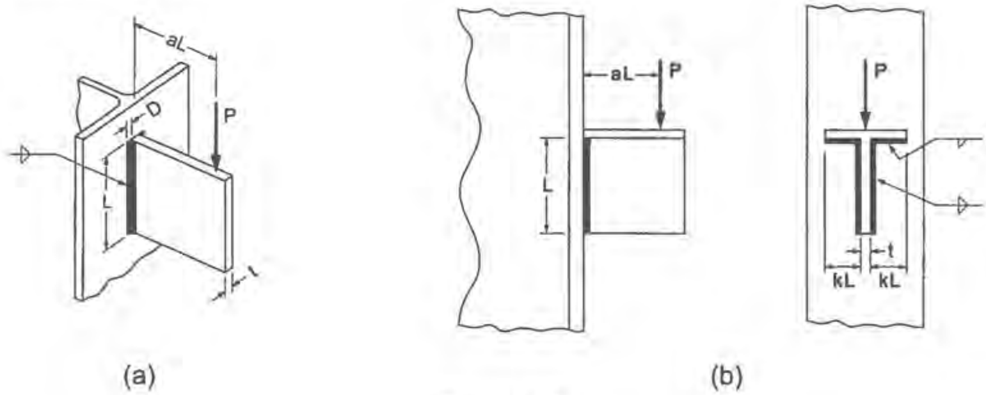


Figure 3-2

In both types of connections, the lower portions of the welded parts are assumed to bear against each other at the ultimate load. The closed-form solution given below for the welded connection shown in Figure 3-2(a) was developed by Kwan *et al.* (2010).

1. Pair of vertical welds (k = 0), Figure 3-2(a)

(a) For $a/Q > 0.53$, the factored load resistance (based on weld failure) is given by:

$$P_r = \frac{0.711\phi F_y t L}{a(Q+1.421)} \quad \text{where } a = \text{eccentricity ratio and } Q = \frac{F_y t}{X_u D} \tag{1a}$$

(b) For $a/Q \leq 0.53$, the factored load resistance (based on weld failure) is given by:

$$P_r = P_{ro}[1-1.89(a/Q)] + 1.89(a/Q)P_{r53} \quad \text{where } P_{ro} = 2(0.67)\phi 0.7071 X_u D L \tag{1b}$$

and P_{r53} is obtained using equation (1a) for an eccentricity a that yields a value of a/Q of 0.53 for the applicable value of Q.

(c) For all values of a/Q, the factored load resistance based on failure in the plate (due to material yield only – instability is not considered) is given by:

$$P_r = \frac{2\phi V_p\left(\sqrt{a^2L^2V_p^2+3M_p^2}-aLV_p\right)}{3M_p}, \text{ where } M_p = \frac{tL^2F_u}{4}, \text{ and } V_p = \frac{tLF_u}{2} \tag{1c}$$

For cases (a), (b) and (c) above, Kwan *et al.* (2010) demonstrated that designs using a resistance factor of 0.58 for welds and 0.71 for steel correspond to a reliability index, $\beta = 4.5$. A resistance factor, $\phi = 0.58$, for both welds and steel is adopted for this application in this Handbook. A design table is given in Table 3-34 for $X_u = 490$ MPa, $F_y = 300$ MPa and $F_u = 440$ MPa. The tabulated coefficients are given by: $C' = P/L$.

For the case of concentric loading ($a = 0$), the weld resistances were taken from Table 3-27 (with $k = 0$).

Plate stability and the resistance of the supporting steel part must also be verified since these modes of behaviour are beyond the scope of the method developed by Kwan *et al.*

2. Tee-shaped configuration, Figure 3-2(b)

The method described above may be applied for the tee-shape configuration shown in Figure 3-2(b), conservatively ignoring the horizontal welds and the seat plate.

Example

Given:

A 12 mm plate carrying a 265 kN factored load is welded to a column with a pair of vertical fillet welds 250 mm long. Find the fillet weld size required if the 265 kN load acts at an eccentricity of 110 mm.

Solution:

$L = 250$ mm, $aL = 110$ mm; therefore, $a = 110/250 = 0.44$

C' required is $P/L = 265/250 = 1.06$ Try a 6 mm fillet weld

From Table 3-34, for plate thickness, $t = 12$ mm, and weld size, $D = 6$ mm:

$C' = 1.13$ for $a = 0.40$, and 1.02 for $a = 0.50$

Therefore, for $a = 0.44$, $C' = 1.09$ (by interpolation) $>$ 1.06

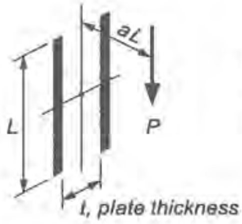
The minimum weld size based on the thickness of the materials joined and the resistance of the connected parts must also be checked.

References:

DAWE, J.L., and KULAK, G.L. 1974. Welded connections under combined shear and moment. ASCE Journal of the Structural Division, **100**(ST4), April.

KWAN, Y.K., GOMEZ, I.R., GRONDIN, G.Y. and KANVINDE, A.M. 2010. Strength of welded joints under combined shear and out-of-plane bending. Canadian Journal of Civil Engineering, 37(2): 250-261.

Table 3-34 ECCENTRIC LOADS ON WELD GROUPS
Coefficients C'



P = Factored eccentric load, kN
L = Length of each weld, mm
C' = Coefficients tabulated below

$P = C'L$
Required Minimum $C' = P/L$
Required Minimum $L = P/C'$

Plate Thickness, t		8 mm	10 mm		12 mm			16 mm		
Weld Size, D		5	5	6	5	6	8	6	8	10
	0.0	1.56	1.56	1.87	1.56	1.87	2.49	1.87	2.49	3.11
	0.1	1.05	1.25	1.31	1.26	1.50	1.58	1.52	2.00	2.10
	0.2	0.938	1.15	1.17	1.17	1.38	1.41	1.42	1.85	1.88
	0.3	0.839	1.05	1.05	1.08	1.26	1.26	1.31	1.68	1.68
	0.4	0.754	0.942	0.942	0.993	1.13	1.13	1.21	1.51	1.51
	0.5	0.681	0.851	0.851	0.905	1.02	1.02	1.11	1.36	1.36
	0.6	0.617	0.767	0.772	0.816	0.920	0.926	1.01	1.23	1.23
	0.7	0.563	0.668	0.704	0.728	0.802	0.844	0.913	1.07	1.13
	0.8	0.515	0.585	0.633	0.642	0.701	0.774	0.813	0.935	1.03
	0.9	0.458	0.520	0.563	0.571	0.624	0.705	0.720	0.831	0.916
a	1.0	0.412	0.468	0.507	0.514	0.561	0.635	0.648	0.748	0.825
	1.2	0.344	0.390	0.422	0.428	0.468	0.529	0.540	0.624	0.687
	1.4	0.294	0.334	0.362	0.367	0.401	0.453	0.463	0.534	0.589
	1.6	0.258	0.292	0.317	0.321	0.351	0.397	0.405	0.468	0.515
	1.8	0.229	0.260	0.282	0.285	0.312	0.353	0.360	0.416	0.458
	2.0	0.206	0.234	0.253	0.257	0.281	0.317	0.324	0.374	0.412
	2.2	0.187	0.213	0.230	0.233	0.255	0.288	0.295	0.340	0.375
	2.4	0.172	0.195	0.211	0.214	0.234	0.264	0.270	0.312	0.344
	2.6	0.159	0.180	0.195	0.198	0.216	0.244	0.249	0.288	0.317
	2.8	0.147	0.167	0.181	0.183	0.200	0.227	0.232	0.267	0.294
	3.0	0.137	0.156	0.169	0.171	0.187	0.212	0.216	0.249	0.275

Plate Thickness, t		20 mm			25 mm				40 mm			
Weld Size, D		8	10	12	8	10	12	14	10	12	14	16
	0.0	2.49	3.11	3.73	2.49	3.11	3.73	4.35	3.11	3.73	4.35	4.98
	0.1	2.02	2.50	2.63	2.03	2.52	3.00	3.28	2.56	3.05	3.54	4.03
	0.2	1.88	2.31	2.34	1.91	2.35	2.78	2.93	2.43	2.88	3.32	3.75
	0.3	1.74	2.10	2.10	1.79	2.17	2.55	2.62	2.30	2.70	3.09	3.48
	0.4	1.60	1.88	1.88	1.66	2.00	2.32	2.36	2.17	2.52	2.87	3.20
	0.5	1.46	1.70	1.70	1.54	1.83	2.09	2.13	2.03	2.35	2.64	2.92
	0.6	1.32	1.53	1.54	1.42	1.65	1.87	1.93	1.90	2.17	2.42	2.65
	0.7	1.19	1.34	1.41	1.30	1.48	1.64	1.76	1.77	1.99	2.19	2.37
	0.8	1.05	1.17	1.27	1.17	1.31	1.43	1.54	1.64	1.82	1.97	2.09
	0.9	0.931	1.04	1.13	1.05	1.16	1.27	1.37	1.51	1.64	1.74	1.86
a	1.0	0.838	0.935	1.01	0.930	1.05	1.15	1.23	1.37	1.46	1.56	1.68
	1.2	0.699	0.779	0.845	0.773	0.873	0.956	1.03	1.11	1.19	1.30	1.40
	1.4	0.599	0.668	0.724	0.663	0.748	0.819	0.879	0.913	1.02	1.11	1.20
	1.6	0.524	0.585	0.633	0.580	0.655	0.717	0.769	0.799	0.893	0.976	1.05
	1.8	0.466	0.520	0.563	0.515	0.582	0.637	0.683	0.710	0.794	0.867	0.931
	2.0	0.419	0.468	0.507	0.464	0.524	0.573	0.615	0.639	0.715	0.780	0.838
	2.2	0.381	0.425	0.461	0.422	0.476	0.521	0.559	0.581	0.650	0.710	0.762
	2.4	0.349	0.390	0.422	0.386	0.437	0.478	0.513	0.533	0.596	0.650	0.699
	2.6	0.322	0.360	0.390	0.357	0.403	0.441	0.473	0.492	0.550	0.600	0.645
	2.8	0.299	0.334	0.362	0.331	0.374	0.410	0.439	0.457	0.511	0.557	0.599
	3.0	0.279	0.312	0.338	0.309	0.349	0.382	0.410	0.426	0.476	0.520	0.559

Matching electrode X_u = 490 MPa

Base metal: F_y = 300 MPa, F_u = 440 MPa

FRAMED BEAM SHEAR CONNECTIONS

General

This section of Part 3 contains information on five common types of beam shear connections traditionally considered standard in the industry. Double-angle, simple end-plate, single-angle, shear tab and tee-type connections are included. In its Eleventh Edition, this Handbook has incorporated information on all-bolted single-angle connections.

Connections of these types are generally designed for strength requirements under factored static gravity loads. The capacities of welds and of bolts in bearing-type connections are based on their ultimate limit states (ULS) factored resistances.

Tabulated bolt capacities for bearing-type connections are based on threads intercepted by the shear planes (unless noted otherwise) and have been calculated according to CSA S16-14, Clause 13.12.1.2. Starting with the 1989 edition, S16 no longer implies that threads intercept the shear plane when the material thickness adjacent to the nut is less than 10 mm. Without special precautions, however, such thin plies may allow threads to be intercepted. For practical reasons, it is suggested that shear connections be designed on the assumption of intercepted threads when combinations of thin material and detailing for minimum bolt stick-through (the nuts) are expected.

Slip-critical bolt capacities are included for double-angle and end-plate connections for use with connections such as those subjected to fatigue or frequent load reversal. However, for fatigue and dynamic load applications, suitability of the connections and capacity of the welded joints are beyond the scope of Part 3. Values are based on Class A (clean mill scale, blast-cleaned with Class A coatings or hot-dip galvanized and roughened) contact surfaces (k_s = 0.30). Values for use with twist-off type bolt assemblies, ASTM F1852, and direct tension indicators, ASTM F959, have been incorporated in this Eleventh Edition. The slip-critical bolt capacities are to be used with *specified loads only*.

Tables of bolt and weld capacities are based on ¾ and ⅞-inch diameter A325 bolts and F1852 assemblies, and on matching electrodes with X_u = 490 MPa, except that values based on 1-inch bolts are also provided for tee-type and bolted seat connections. The tables are based on the use of angles and plates with a specified minimum yield strength, F_y = 300 MPa, and a specified minimum tensile strength, F_u = 440 MPa. For the supported and supporting members, F_y = 345 MPa and F_u = 450 MPa are assumed.

Although based on specific arrangements of bolts and welds, the tables are general in nature when used for common applications. These tables may help steel fabricators to prepare drawing office and shop standards, design authorities to check fabricator standards, and educational institutions to teach structural steel design and detailing.

The standard connections of individual fabricators will depend on fabrication methods and material sources. They may differ from those shown in the tables with respect to steel grade, length and size of angles and other detail material, as well as gauge and pitch of bolts.

Minimum Material Thickness

Together with the tabulated capacities of welds and bolts in bearing-type connections, information is provided concerning the minimum required thickness of supporting and supported material to develop the full connector capacities. All supported beams are assumed to be uncoped. These minimum thicknesses were generally determined in the following manner.

For bolts in bearing-type connections, the minimum material thickness was derived by equating the bearing capacity of the material to the shear capacity of the bolts while assuming that supported beams are not coped. For webs of beams (both supporting and supported) and for webs and flanges of (supporting) columns, the factored bearing resistance has been calculated according to Clause 13.12.1.2(a).

For welded connections, the weld shear resistance for each weld size was equated to the shear resistance of the supported beam web within the weld length, and the equation solved for the web material thickness. The weld resistance is based on S16-14 Clause 13.13.2.2, and the web resistance (Clause 13.4.1.1) is $V_r = \phi A_w F_s$, with $F_s = 0.66\ F_y$ and A_w = web area.

Block shear failure was also considered in calculating the minimum required material thickness for bolted connections. This mode of failure was evaluated according to Clause 13.11, taking into account the failure patterns as illustrated below.

Block Shear Failure

S16-14 Clause 13.11

Other Failure Modes

Possible failure modes other than those mentioned above, such as effects due to bending, shear and axial forces in the supporting column, in combination as applicable, should also be considered in the design.

DOUBLE-ANGLE BEAM CONNECTIONS

Tables 3-37 and 3-38 list capacities of bolted and welded double-angle beam connections. At the bottom of each table are values for minimum material thickness required to develop the connector capacities listed in the corresponding columns, as described in the preceding pages. For material thicknesses less than those listed, the corresponding connector capacities must be reduced by the ratio of the thickness of material supplied to the thickness of material listed. Any combination of welded or bolted legs can be selected from the tables.

Consistent with long-standing North American practice, bolt capacities are based on concentric loading as tests have shown that eccentricity does not influence the ultimate strength of the bolts in connections using a single line of bolts in the web-framing leg. Weld capacities include the effect of eccentricity for connection angles.

The connection angle length, L, is based on a bolt pitch of 80 mm assuming an end distance of 35 mm. For connection angles with both legs welded, the angle lengths can be adjusted and capacities interpolated in accordance with the length used. Nominal minimum and maximum depths of supported beams appropriate to each length of connection angle are included. The suggested maximum depth assumes a connection length not less than half the beam depth to provide some measure of stiffness and stability. It should be recognized that these depths may not always be appropriate for a particular structure.

Table 3-37 lists bolt capacities for bearing-type and slip-critical connections for three sizes of angle (width of leg and gauge dimension) and includes values for 2 to 13 bolts per vertical line based on a bolt pitch of 80 mm. For web-framing legs, bolt capacities are based on the "double shear" condition and for outstanding legs on the "single shear" condition. Thus two vertical lines of bolts in the outstanding legs (one line in each angle leg) have the same capacity as one vertical line in the web-framing leg. When beams are connected to both sides of the supporting material, the total bolt capacity in the outstanding legs is double that listed, provided the thickness of the supporting material is equal to or greater than that listed for the web of the supported beam.

For connection angles, the minimum required thickness to develop the bolt capacities is listed in the table. To ensure connection flexibility, the angle thickness selected should not be greater than necessary, with a minimum thickness of 6 mm for practical reasons. Bolt capacities are provided separately for both conditions of threads intercepted and threads excluded. See Clause 13.12.1.2 of CSA S16-14.

Table 3-38 lists weld capacities for web-framing legs and for outstanding legs. Values are tabulated for four sizes of fillet weld and are based on the length (L) and size (angle width, W) of connection angles listed. For the web-framing leg welds, capacities were calculated using the instantaneous centre of rotation method. For the outstanding leg welds, the out-of-plane eccentricity between weld lines and the vertical beam reaction, taken to be 65 mm (the largest g_l value listed), was taken into account. It is good practice to select an angle thickness no less than $D/0.75$, where D = weld size, due to the rolled edges of angles.

Design of bearing-type connections for types and sizes of bolts other than those shown in Table 3-37 will be facilitated by looking up the resistances listed in Tables 3-3 to 3-7, and for slip-critical connections in Tables 3-10 and 3-11.

Encroachment by Framing Angles on Beam Fillets

The maximum length of framing angles needs to be compatible with the clear distance, *T*, between the flange fillets of a beam. In compact situations, it is customary to tolerate a modest amount of encroachment by the angles onto the toes of the fillets. Encroachments that create a gap not greater than 1 mm under the end of an angle are listed, as a function of the fillet radius, in Table 3-35.

The minimum nominal depth of supported beam given in Tables 3-37 and 3-38 has been adjusted for the fillet radius indicated in mill catalogs, taking into account the encroachment onto the fillets.

Supported Beams with Copes

When copes are required at the ends of supported beams to avoid interference with the supporting material, the capacity of the beam in the vicinity of the connection and/or the capacity of the connection may be reduced. When selecting the beam size, the designer should consider the effect of copes on the load-carrying capacity of the beam, and the detailer should be aware that copes often reduce the capacity of connections on beams with thin webs.

With reference to the beam, the *Steel Construction Manual* (AISC 2011) provides guidance for a variety of situations that include shear at the reduced section, flexural yielding of the coped section due to bending, and web buckling in the vicinity of the cope due to shear and bending. The shear resistance of the web is calculated according to S16-14 Clause 13.4.3.

For top-coped beams, block shear or "block tear-out" is often the failure mode. See S16-14 Clause 13.11. Block shear takes a different pattern when connection material is bolted to the supported beam than it does when connection material is welded to the beam. In the former case, the pattern is usually a tension tear along a horizontal plane from the end of the beam to the bottom bolt hole of the connection combined with a vertical shearing tangential to the bolt holes from the bottom hole to the cope (see Table 3-36 and the accompanying figure). For welded connection angles, there are corresponding tension and shear planes, but along the toes of the welds (although it is common and conservative practice to take these planes along the edges of the angle in the design). The vertical shearing extends all the way to the cope, with the result that the weld across the top of the angles does not participate in the connection resistance.

A detailing aid for evaluating the block shear resistance of a bolted connection on a coped beam is presented in Table 3-36. The two coefficients C_1 and C_2 were calculated based on S16-14 Clause 13.11 for combined tension and shear failure. Coefficient C_1 is a function of the horizontal and vertical edge distances, L_h and L_v, to the beam end and the cope, respectively. Coefficient C_2 is a function of the bolt diameter and the number of bolts. The sum of the coefficients multiplied by the web thickness gives the block shear resistance in kN.

Tests cited by Yura *et al* (1980) have shown that capacities of single-line bolted connections computed assuming failure along the "block tear-out" line are conservative, but when two lines of bolts are used in the web-framing leg, the effects of eccentricity should be taken into account.

References

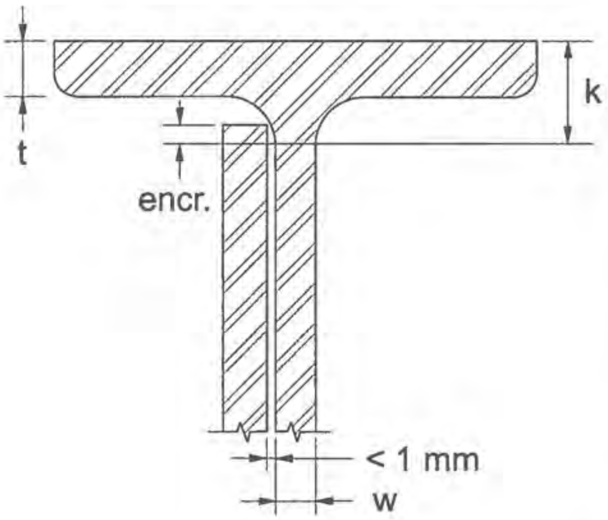
AISC. 2011. Steel Construction Manual, 14th Edition, American Institute of Steel Construction.

BIRKEMOE, P.C. and GILMOR, M.I. 1978. Behaviour of bearing-critical double-angle beam connections. Engineering Journal, Fourth Quarter, AISC.

YURA, J.A., BIRKEMOE, P.E. and RICLES, J.M. 1980. Beam web shear connections – an experimental study. Beam-to-Column Building Connections: State of the Art, Preprint 80-179, April, ASCE.

Fillet Encroachment

Table 3-35



Fillet Radius k – t (mm)	Encroachment (mm)
8	3
9	4
10	4
12	4
14	5
16	5
18	5
20	6
22	6
24	6
26	7

BLOCK SHEAR IN TOP-COPED BEAMS — Table 3-36

Coefficients C_1 and C_2
Standard Holes and 80 mm Bolt Pitch *
ASTM A992, A572 grade 50, CSA G40.21-350W

Coefficient C_1

L_v (mm)	L_h (mm)												
	25	26	28	30	32	34	38	45	52	59	66	73	80
25	12.1	12.4	13.0	13.6	14.2	14.8	16.0	18.1	20.3	22.4	24.5	26.6	28.8
26	12.2	12.5	13.2	13.8	14.4	15.0	16.2	18.3	20.4	22.6	24.7	26.8	29.0
28	12.6	12.9	13.5	14.1	14.7	15.3	16.6	18.7	20.8	22.9	25.1	27.2	29.3
30	13.0	13.3	13.9	14.5	15.1	15.7	16.9	19.0	21.2	23.3	25.4	27.5	29.7
32	13.3	13.6	14.2	14.8	15.4	16.1	17.3	19.4	21.5	23.6	25.8	27.9	30.0
34	13.7	14.0	14.6	15.2	15.8	16.4	17.6	19.8	21.9	24.0	26.1	28.3	30.4
38	14.4	14.7	15.3	15.9	16.5	17.1	18.3	20.5	22.6	24.7	26.8	29.0	31.1
45	15.6	15.9	16.6	17.2	17.8	18.4	19.6	21.7	23.8	26.0	28.1	30.2	32.3
52	16.9	17.2	17.8	18.4	19.0	19.6	20.8	23.0	25.1	27.2	29.3	31.5	33.6
59	18.1	18.5	19.1	19.7	20.3	20.9	22.1	24.2	26.3	28.5	30.6	32.7	34.9
66	19.4	19.7	20.3	20.9	21.5	22.1	23.3	25.5	27.6	29.7	31.9	34.0	36.1
73	20.7	21.0	21.6	22.2	22.8	23.4	24.6	26.7	28.9	31.0	33.1	35.2	37.4
80	21.9	22.2	22.8	23.4	24.0	24.6	25.9	28.0	30.1	32.2	34.4	36.5	38.6

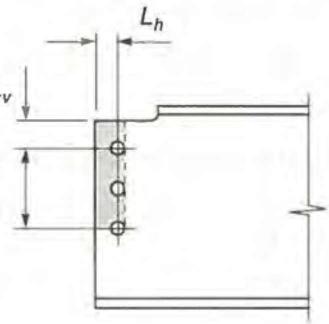
Coefficient C_2

n	Bolt		
	¾ in.	⅞ in.	1 in.
2	10.7	10.4	9.9
3	25.0	24.7	24.2
4	39.3	39.0	38.5
5	53.6	53.3	52.8
6	67.9	67.6	67.1
7	82.2	81.9	81.5
8	96.5	96.2	95.8
9	110.8	110.5	110.1
10	125.1	124.8	124.4

$\phi_u = 0.75$ $U_t = 0.9$ $F_y = 345$ MPa $F_u = 450$ MPa

pitch = 80 mm

n = number of bolts



Block shear

$T_r = \phi_u [U_t A_n F_u + 0.6 A_{gv} (F_y + F_u) / 2]$ (S16-14 Clause 13.11)

$T_r = (C_1 + C_2)\, w$ (using coefficients in Table 3-36)

Design example

W460x89 beam using four ¾ in. bolts with: $L_v = 45$ mm, $L_h = 38$ mm and $w = 10.5$ mm

From Table 3-36: $C_1 = 19.6$ $C_2 = 39.3$

$T_r = (C_1 + C_2)\, w = (19.6 + 39.3)\, 10.5 = 618$ kN, where:

T_r = factored resistance for block shear
w = thickness of the beam web
A_n = net area in tension
A_{gv} = gross area in shear
L_h = distance, centre of hole to beam end
L_v = distance, center of hole to cope
n = number of bolts
d_h = design hole diameter *

* The design hole sizes include allowance for punched holes as given in Table 3-47 and S16-14, Clause 12.3.2. Coefficient C_2 was calculated using d_h = 24 mm for ¾-in. bolts, 26 mm for ⅞-in. bolts and 29 mm for 1-in. bolts.

Example 1

Double angles bolted to beam web, bearing-type, welded to column flange.

Given:

W530x92 beam connected to the flange of a W250x73 column, both ASTM A992 steel.

Reaction due to factored loads = 450 kN

Beam web thickness = 10.2 mm; column flange thickness = 14.2 mm.

Detail material G40.21-300W steel, ¾-inch A325 bolts, X_u = 490 MPa matching electrodes.

Solution:

Web-framing legs – bolted (Table 3-37)

Vertical line with four bolts (threads intercepted) provides a capacity of:

632 kN > 450 kN OK

Supported beam web thickness, based on bearing, required for steels with F_u = 450 MPa

$= 7.7 \times 450/632 = 5.5 \text{ mm} < 10.2 \text{ mm}$ OK

Angle thickness required:

$= 7.1 \times 450/632 = 5.1$ mm. Try 6.4 mm.

Minimum angle length required = 310 mm. (OK for 4 bolts and 530 mm beam depth)

Outstanding legs – welded (Table 3-38)

With L = 310 mm, W = 89 or 76 mm

6 mm fillet welds provide a capacity of 480 kN > 450 kN OK

Minimum angle thickness (recommended good practice due to rolled edges of angles):
$t \geq D/0.75 = 6/0.75 = 8$ mm

Also, $t \geq D + 2 \text{ mm} = 6 + 2 = 8$ mm. Increase angle thickness to $t = 7.94 \text{ mm} \approx 8.0$ mm

Flange thickness of supporting column $= 4.6 \times 450/480 = 4.3 \text{ mm} < 14.2 \text{ mm}$ OK

Use:

76x76x7.9 connection angles, 310 mm long, four ¾-inch A325 bolts in web-framing legs and 6 mm fillet welds on outstanding legs.

Example 2

Double angles welded to the beam web, bolted to the column flange, bearing-type.

Given:

Same as example 1

Solution:

Web-framing legs – welded (Table 3-38)

6 mm fillet welds provide a capacity of 846 kN with angle length L = 310 mm and W = 76 mm

Supported beam web thickness required for 6 mm fillet welds, $L = 310$ mm

$= 9.1 \times 450 / 846 = 4.8$ mm $<$ 10.2 mm OK

Minimum angle thickness = 7.94 mm (same as example 1).

Outstanding legs – bolted (Table 3-37)

For L = 310 mm, W = 89 or 76 mm, four bolts (threads intercepted) per vertical line, bolt shear capacity is 632 kN > 450 kN OK

Angle thickness required $= 7.1 \times 450/632 = 5.1$ mm $<$ 7.94 mm OK

The required flange thickness of the supporting column, with a beam framing from one side, is one half the required thickness of the supported beam web:

14.2 mm $> 0.5\,(7.7 \times 450/632) = 2.7$ mm OK

Use:

89x76x7.9 connection angles, 310 mm long, 89 mm outstanding legs, g = 130 mm with eight ¾-inch A325 bolts (4 per vertical line) and 6 mm fillet welds on web-framing legs.

Example 3

Double angles bolted to the beam web and bolted to both sides of the supporting member, supported beams not coped, bearing-type.

Given:

W530x92 beam of ASTM A992 steel, factored reaction 450 kN, framing from both sides of 11.9 mm web of W760x134 girder of A992 steel.

Detail material – G40.21-300W steel, ¾-inch A325 bolts.

Solution:

Web framing legs – same as example 1

Outstanding legs – bolted to both sides of supporting member (Table 3-37)

Total reaction on girder web is $2 \times 450 = 900$ kN

For beams connected to both sides of the supporting member, the bolt capacity is double that listed in the table:

$2 \times 632 = 1260$ kN $>$ 900 kN

Required web thickness of supporting member, based on bearing, is the same as that given for web thickness of supported beam. For angle L = 310 mm, W = 89 or 76 mm, four ¾-inch A325 bolts per vertical line (threads included), the girder web thickness is:

11.9 mm $> 7.7 \times 450/632 = 5.5$ mm OK

Use:

89x89x7.9 connection angles, 310 mm long, four ¾-inch A325 bolts per vertical line in both web-framing and outstanding legs.

BOLTED DOUBLE-ANGLE BEAM CONNECTIONS Table 3-37

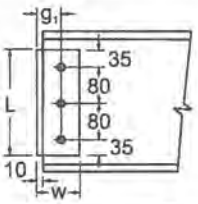
A325 Bolts, F1852[1] Assemblies

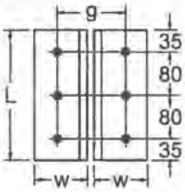
CSA G40.21-300W Angles

ASTM A992, A572 Gr. 50, CSA G40.21-350W Beams and Supporting Members

BOLT GROUP RESISTANCE - EITHER LEG WITH BOLTS

Web-Framing Legs





Outstanding Legs

Angle Width and Gauge		
W	g	g_1
102	140	65
89	130	60
76	100	45

Nominal Depth of Supported Beam (mm)		Conn. Angle Length L (mm)	Bolts per Vertical Line	BEARING-TYPE CONNECTIONS ULS Factored Load Resistance (kN)				SLIP-CRITICAL CONNECTIONS[2] Specified Load Resistance (kN)			
				Threads Intercepted		Threads Excluded		A325 Installed by Turn-of-Nut (c_s = 1.0)		A325 Installed with F959[5], F1852[1] (c_s = 0.78)	
				Bolt Size (in.)		Bolt Size (in.)		Bolt Size (in.)		Bolt Size (in.)	
min.	max.			¾	⅞	¾	⅞	¾	⅞	¾	⅞
200	310	150	2	**316**	**430**	451	615	150	204	117	159
310	460	230	3	**474**	**645**	677	922	224	305	175	238
410	610	310	4	**632**	**860**	903	1230	299	407	233	318
460	760	390	5	**790**	**1080**	1130	1540	374	509	292	397
610	920	470	6	**948**	**1290**	1350	1840	449	611	350	476
690	1100	550	7	**1110**	**1510**	1580	2150	523	712	408	556
760		630	8	**1260**	**1720**	1810	2460	598	814	467	635
840		710	9	**1420**	**1940**	2030	2770	673	916	525	714
920		790	10	**1580**	**2150**	2260	3070	748	1020	583	794
1000		870	11	**1740**	**2370**	2480	3380	823	1120	642	873
1100		950	12	**1900**	**2580**	2710	3690	897	1220	700	953
Welded beam		1030	13	**2050**	**2800**	2930	3990	972	1320	758	1030

Specified Minimum Tensile Strength of Supported and Supporting Members	Minimum Required Web Thickness of Supported Beam[3,4] (mm)			
F_u = 450 MPa	7.7	9.0	11.0	12.8
Specified Minimum Yield and Tensile Strengths of Angles	**Minimum Required Thickness of Framing Angles (mm)**			
F_y = 300 MPa, F_u = 440 MPa	7.1	9.7	10.0	13.8

Tabulated values are based on the following angle leg widths:
For ¾ inch bolts, W = 76 or 89 mm
For ⅞ inch bolts, W = 89 or 102 mm

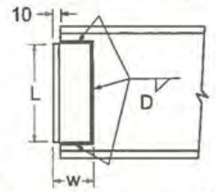
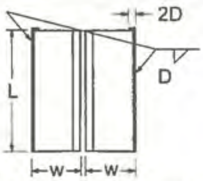
Resistance factors:
$\phi = 0.90$, $\phi_u = 0.75$, $\phi_b = \phi_{br} = 0.80$

1. ASTM F1852 twist off type tension control structural bolt/nut/washer assemblies
2. Tabulated values for slip-critical connections assume Class A contact surfaces with k_s = 0.30.
3. For supporting material with beams framing from both sides, minimum required thickness is equal to tabulated values for web thickness of supported beam. For supporting material with beams framing from one side, minimum required thickness is one-half the tabulated values.
4. Coped beams may have additional requirements. See *Double-Angle Beam Connections* in Part 3.
5. ASTM F959 compressible-washer-type direct tension indicators

WELDED DOUBLE-ANGLE BEAM CONNECTIONS Table 3-38

Fillet Welds: X_u = 490 MPa

CSA G40.21-300W Angles
ASTM A992, A572 Gr. 50, CSA G40.21-350W[1] Beams and Supporting Members

WELD GROUP RESISTANCE														
 Web-Framing Legs with Welds								 Outstanding Legs with Welds						
TWO C-SHAPE WELDS ULS Factored Load Resistance (kN)								TWO VERTICAL WELDS ULS Fact. Load Resistance (kN)				Conn. Angle Length L (mm)	Nominal Depth of Supported Beam (mm)	
Angle Width, W								Angle Width, W						
76 mm				64 mm				89 mm or 76 mm						
Fillet Size D (mm)				Fillet Size D (mm)				Fillet Size D (mm)						
5	6	8	10	5	6	8	10	5	6	8	10		min.	max.
368	442	590	737	354	425	567	709	115	139	185	231	150	200	310
532	639	852	1060	524	628	838	1050	250	300	400	500	230	310	460
705	846	1130	1410	660	792	1060	1320	400	480	640	800	310	410	610
823	987	1320	1650	768	922	1230	1540	545	654	872	1090	390	460	760
931	1120	1490	1860	874	1050	1400	1750	683	820	1090	1370	470	610	920
1030	1240	1650	2070	979	1170	1570	1960	816	980	1310	1630	550	690	1100
1140	1370	1820	2280	1080	1300	1730	2170	946	1140	1510	1890	630	760	
1250	1500	2000	2500	1190	1430	1910	2390	1070	1290	1720	2150	710	840	
1350	1620	2160	2700	1300	1550	2070	2590	1200	1440	1920	2400	790	920	
1460	1750	2340	2920	1400	1680	2240	2800	1330	1590	2120	2660	870	1000	
1570	1880	2510	3140	1510	1810	2420	3020	1450	1740	2330	2910	950	1100	
1670	2000	2670	3340	1620	1940	2590	3230	1580	1890	2530	3160	1030	Welded beam	
Minimum Required Web Thickness of Supported Beam[2] (mm)								Min. Thick. of Supporting Steel, Beam Framing on One Side[3] (mm)				F_y (MPa)		
7.6	9.1	12.1	15.2	7.6	9.1	12.1	15.2	3.8	4.6	6.1	7.6	345		

1. Resistances are based on F_y = 345 MPa.
2. Coped beams may have additional requirements. See *Double-Angle Beam Connections* in Part 3.
3. For supporting material with beams framing from both sides, use double the tabulated value.

END-PLATE CONNECTIONS

End-plate connections with the connection plate welded to the supported beam and bolted to the supporting member are commonly used because of their economy, ease of fabrication, and performance. When beams are saw-cut to length, the use of simple jigging procedures to locate and support end plates during assembly and welding makes it possible to meet the tighter fabrication tolerances required without difficulty.

Research on simple end-plate shear connections has shown that their strength and flexibility compare favourably with double-angle shear connections for similar material thickness, depth of connection, and arrangement of bolts (gauge and pitch). For practical reasons it is suggested that the minimum thickness of the end-plate be 6 mm, and for adequate flexibility, that the maximum thickness be limited to 10 mm. The gauge dimension g should preferably be between 100 mm and 150 mm for plates up to 10 mm thick, but may be as low as 80 mm for minimum thickness plates with F_y not greater than 300 MPa.

Table 3-39 lists the capacities of bolts and welds for typical end-plate connections with 2 to 8 bolts per vertical line, together with the minimum thickness of the end plate, supporting, and supported members to develop the full capacity of the bolts and welds, respectively. End-plate thicknesses are based on minimum edge distances in S16-14 Table 6.

For added safety during erection, clipped end plates with one upper corner of the end plate removed may be used. Tests at Queen's University demonstrated that clipped end-plate connections have similar moment-rotation characteristics to unclipped end-plate connections; therefore, weld capacities in Table 3-39 may be used directly for design, but tabulated bolt values must be reduced by the value of one bolt.

Table 3-39 also includes bolt capacities for slip-critical joints for those situations where bearing-type connections are not suitable.

References

VAN DALEN, K., and MACINTYRE, J.R. 1988. The rotational behaviour of clipped end-plate connections. Canadian Journal of Civil Engineering, **15**(1), February.

Example

Given:

W410x60 beam of ASTM A992 steel framing into web of W760x134 girder of A992 steel. The factored reaction is 325 kN.

Beam web thickness = 7.7 mm, girder web thickness =11.9 mm. G40.21-300W steel plate detail material, ¾-inch A325 bolts, and matching electrodes, X_u = 490 MPa

Solution:

Try 3 bolts per vertical line (threads intercepted).

Factored resistance = 474 kN > 325 kN OK

For 230 mm-long end plate, weld capacity for 5 mm fillet welds is 342 kN > 325 kN

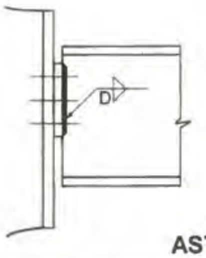
Required end-plate thickness = 6.9 × 325 / 474 = 4.7 mm. Use 6 mm minimum.

Minimum thickness of supported beam web = 7.6 × 325 / 342 = 7.2 mm < 7.7 mm

Minimum thickness of supporting girder web (beams framing from one side)
$= 3.8 \times 325 / 474 = 2.6 \text{ mm} < 11.9 \text{ mm}$ OK

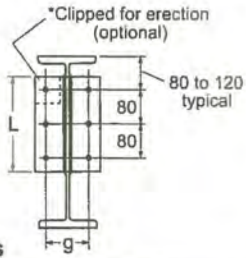
If beams were framing from both sides, the required web thickness of the girder would be twice the listed value, pro-rated for the actual factored load:
$2 \times 3.8 \times 325 / 474 = 5.2 \text{ mm} < 11.9 \text{ mm}$ OK

Use: End plate 160x6x230 mm connected to the web of the supported beam with 5 mm fillet welds, X_u = 490 MPa, and six ¾-inch A325 bolts (2 rows of 3 at 100 mm gauge).



END-PLATE BEAM CONNECTIONS
Table 3-39

A325 Bolts, F1852 Assemblies [5]
Fillet Welds X_u = 490 MPa
CSA G40.21-300W End Plates
ASTM A992, A572 Gr. 50, CSA G40.21-350W Members



Bolts per Vertical Line	BEARING-TYPE CONNECTION [1] Factored Load Resistance (kN) Threads Intercepted		WELD CAPACITY [2] Factored Load Resistance (kN) Fillet Size D (mm)			Connection Plate Length (mm)
	¾ in. Bolts	⅞ in. Bolts	5	6	8	
2	316	430	218	258	333	150
3	474	645	342	407	533	230
4	632	860	467	556	732	310
5	790	1080	591	706	931	390
6	948	1290	715	855	1130	470
7	1110	1510	840	1000	1330	550
8	1260	1720	964	1150	1530	630
Material F_u (MPa)	Minimum Required Thickness of Supporting Member with Beams Framing from One Side [3] (mm)		Minimum Required Web Thickness of Supported Beam [4] (mm)			Material F_y (MPa)
450	3.8	4.5	7.6	9.1	12.1	345

F_y = 300	Minimum Required End-Plate Thickness (mm)	
F_u = 440	6.9	9.1

Bolts per Vertical Line	SLIP-CRITICAL CONNECTION [1] Specified Load Resistance (kN) Class A Contact Surfaces, k_s = 0.30			
	A325 installed by Turn of Nut (c_s = 1.0)		A325 installed with F959[6], F1852[5] (c_s = 0.78)	
	¾ in. Bolts	⅞ in. Bolts	¾ in. Bolts	⅞ in. Bolts
2	150	204	117	159
3	224	305	175	238
4	299	407	233	318
5	374	509	292	397
6	449	611	350	476
7	523	712	408	556
8	598	814	467	635

1. For clipped end-plate connections, reduce tabulated bolt capacities by the value of one bolt.
2. The effective weld length is taken equal to the plate length minus twice the weld size.
3. Minimum required thickness of supporting member with beams framing from both sides is double that listed.
4. Coped beams may have additional requirements. See *Double-Angle Beam Connections* in Part 3.
5. ASTM F1852 twist off type tension control structural bolt/nut/washer assemblies
6. ASTM F959 compressible-washer-type direct tension indicators

SINGLE-ANGLE BEAM CONNECTIONS

For some applications, single-angle connections provide a satisfactory alternative to double-angle or end-plate connections. They are particularly suitable where limited access prevents the erection of beams with double-angle or end-plate connections, and where speed of erection is a primary consideration.

The connection angle may be either bolted or welded to the supporting and supported members. Usual practice involves shop fillet-welding to the supporting member and field-bolting to the web of the supported beam.

Bolted-Welded Connections

Tests carried out at the University of British Columbia (Lipson 1968, 1977, 1980), using 4×3×⅜ inch angles with the 4-inch leg bolted to the beam web with ¾ inch diameter A325 bolts and the 3-inch leg welded to the supporting member with ¼ inch E70XX fillet welds, demonstrated that welded-bolted single-angle connections with 2 to 12 bolts per vertical line possess adequate rotational capacity, and that in those connections loaded to ultimate capacity (2 to 8 bolts per vertical line) the failure occurred in the bolts when the weld pattern included welding along the heel and ends of the connection angle. The tests also demonstrated that the use of horizontal slotted holes in the connection angle reduced the moment at the bolts without affecting the ultimate capacity of the connection.

Table 3-40a is based on this research and assumes the use of 102x76x9.5 connection angles with the 76 mm leg welded to the supporting member and the 102 mm leg bolted to the supported web. Bolt capacities for bearing-type connections are provided for ¾ and ⅞ inch A325 bolts based on their factored shear resistance for the appropriate number of bolts. Weld capacities have been established by assuming that a connection with ¼ inch fillet welds has the same shear capacity as the ¾ inch A325 bolts (based on $\phi_b = 0.67$ in S16.1-94 and with the threads excluded), and then pro-rating for the three sizes of fillet welds shown in the table:

Weld capacity = V_r {¾ inch bolts} × $D/6.35$

where D is the fillet weld size in mm.

The bolt capacities tabulated are also valid for ¾ and ⅞ inch F1852 twist-off-type tension-control structural bolt/nut/washer assemblies.

All-Bolted Connections

While single-angle connections are commonly welded in the shop and bolted in the field, the all-bolted option is viable where proficient bolting facility is available in the shop. All-bolted connections are subjected to both in-plane and out-of-plane eccentric load effects. For connections having one vertical bolt line per angle leg, it is common practice to include the in-plane eccentric load effect on the bolts in the support-side leg only, while restricting the bolt gauge of the beam-side leg and thus reducing eccentric load effects to a non-critical value.

Table 3-40b provides the connection coefficients, C, that account for an in-plane eccentricity on the support-side leg bolt group, e, equal to 65 mm. The table is valid for *bearing-type* connections in a configuration and having dimensions as shown in the accompanying figure. In addition, the bolt gauge of the beam-side leg, g_b, must not exceed 70 mm unless it can be demonstrated otherwise. All bolts must be of the same size, type and grade. If a shear plane intercepts the threads of any of the bolts, the bolt shear resistance must be based on the thread-intercepted value. The tabulated coefficients, C, are determined by the instantaneous centre of rotation method described by Kulak *et al.* (1987); see *Eccentric Loads on Bolt Groups*. The reduced eccentrically loaded bolt group resistance is:

$$R_r = C\,V_r$$

where C = connection coefficient from Table 3-40b

V_r = factored shear resistance of a single bolt from Table 3-4

References

KULAK, G.L., FISHER, J.W., and STRUIK, J.H.A. 1987. Guide to design criteria for bolted and riveted joints, 2nd Edition, John Wiley and Sons.

LIPSON, S.L. 1980. Single-angle welded-bolted beam connections. Canadian Journal of Civil Engineering, **7**(2), June.

LIPSON, S.L. 1977. Single-angle welded-bolted connections. Journal of the Structural Division. ASCE. March.

LIPSON, S.L. 1968. Single-angle and single-plate beam framing connections. Proceedings, Canadian Structural Engineering Conference, Canadian Institute of Steel Construction, Willowdale, Ontario, February: 141–162.

Example 1

Single-angle welded-bolted beam connection (Table 3-40a)

Given

W410x60 beam ASTM A992 steel, factored reaction = 280 kN, web thickness = 7.7 mm
L102x76x7.9 connection angle of G40.21-300W steel.
¾-inch A325 bolts, electrodes rated for X_u = 490 MPa.

Solution

With threads intercepted in the shear plane, bolt capacity with four ¾-inch A325 bolts is 316 kN > 280 kN OK

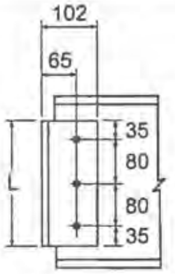
Web thickness required is $3.8 \times 280 / 316 = 3.4$ mm < 7.7 mm OK

Angle thickness required is $6.6 \times 280 / 316 = 5.8$ mm < 7.9 mm OK

Angle length required for 4 bolts is 310 mm, and weld capacity using 5 mm fillet welds (X_u = 490 MPa) is 298 kN > 280 kN OK

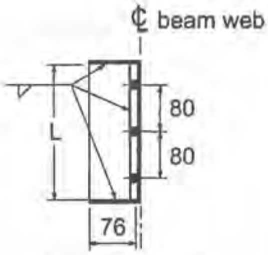
Use:

L102x76x7.9 connection angle, 310 mm long, 76 mm leg welded to supporting member with 5 mm fillet welds; 102 mm leg bolted to web of supported beam with four ¾-inch A325 bolts.



SINGLE-ANGLE BEAM CONNECTIONS BOLTED-WELDED

Table 3-40a



Web-Framing Leg (Bolted to Supported Web)

Outstanding Leg (Welded to Supporting Member)

A325 Bolts, F1852 Assemblies
Fillet Welds X_u = 490 MPa
CSA G40.21 300W Angles
ASTM A992, A572 Gr. 50, CSA G40.21-350W Members

Bolts per Vertical Line	BEARING-TYPE CONNECTION Factored Load Resistance (kN) Threads intercepted		WELD CAPACITY Factored Load Resistance (kN)			Connection Angle Length L
	Bolt Size (in.)		Fillet Size D (mm)			
	¾	⅞	5	6	8	(mm)
2	158	215	149	179	238	150
3	237	323	223	268	357	230
4	316	430	298	357	476	310
5	395	538	372	447	595	390
6	474	645	447	536	715	470
7	553	753	521	625	834	550
8	632	860	594	713	950	630
Material F_u (MPa)	Minimum Required Web Thickness of Supported Beam [1] (mm)		Minimum Required Thickness of Supporting Member with Beams Framing from One Side (mm)			Material F_y (MPa)
450	3.8	4.5	3.7	4.5	5.9	345
F_y = 300 F_u = 440	Minimum Required Thickness of Framing Angle (mm)					
	6.6	9.0				

1. Coped beams may have additional requirements. See *Double-Angle Beam Connections* in Part 3.

Example 2

Bolt group resistance of an all-bolted single-angle beam connection (Table 3-40b)

Given

A W410x54 beam is connected to end supports for a factored end reaction of 220 kN (ULS), using single-angle connections with one vertical line of bolts in both legs. Find the number of ¾-inch A325 bolts at a pitch distance of 80 mm.

Solution

$R_f = 220$ kN

$V_r = 79.0$ kN (Table 3-4, single shear, threads intercepted case assumed)

Required: $C = 220 / 79.0 = 2.78$

Try 4 bolts per leg. From Table 3-40b, $C = 3.15 > 2.78$ Use 4 bolts per leg.

Factored bolt group resistance, $R_r = 3.15 \times 79.0 = 249$ kN > 220 kN

Check support-side leg gauge distance, g_s:

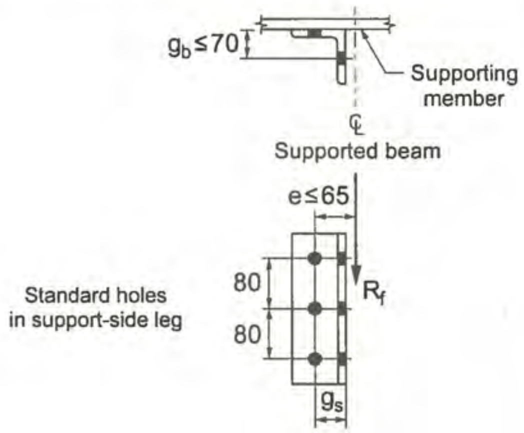
Beam web thickness = 7.5 mm (W410x54)

Maximum value of $g_s = 65 - 7.5/2 = 61.2$ mm Use $g_s = 60$ mm

To complete the connection design, the angle, the beam and its supporting member (or part) must be proportioned to satisfy other requirements, including bolt hole bearing capacity, end and edge distances, block shear and shear rupture, in accordance with S16-14 Clauses 13.12.1.2, 22.3 and 13.11, respectively. Other factors such as effects due to beam cope(s) and combined stresses in the supporting member (or part) as applicable, should also be considered.

SINGLE-ANGLE BEAM CONNECTIONS
ALL-BOLTED
Table 3-40b
Bolt Group Coefficients C *
A325, A490, F1852 and F2280 Bolts and Assemblies

Number of Bolts per Leg	Coefficient C
2	1.05
3	2.04
4	3.15
5	4.25
6	5.34
7	6.41
8	7.47
9	8.52
10	9.56
11	10.6
12	11.6



* Coefficients account for reduction of bolt group resistance due to eccentricities; other possible connection failure modes must also be considered.

SHEAR TAB BEAM CONNECTIONS

When the load magnitude does not require the strength of bolts in double shear, a simple and economical connection is a single plate welded vertically onto a supporting member with the supported member bolted to the plate. Shear tabs – as they are commonly known – were studied by Astaneh et al. (1989) in an experimental program to define a suitable design method for proportioning and rating them. Table 3-41 was prepared by following recommendations in that paper.

Astaneh et al. identified that the strength of shear tabs is a function of several variables. The first is the stiffness of the supporting member. A shear tab on a column flange is restrained from following the end rotation of the supported member, whereas a shear tab on one side of a supporting beam is freer to rotate in its own plane. This results in different effective eccentricities upon the bolts. The eccentricities are also a function of the number of bolts in the connection. Generally, shear tabs on flexible supports have larger bolt eccentricities, and therefore lower resistances, than do those on rigid supports. For shear tabs with seven bolts, however, the effective eccentricity is the same for both.

For rigid supports, efficiency in terms of capacity per bolt is a maximum for four bolts because the effective eccentricity is zero.

The test program used only standard-size holes, and the results are considered to be conservative for short slotted holes. Oversize and long slotted holes are not applicable. Holes may be either punched or drilled.

Shear tabs should be at least 6 mm thick, but no thicker than half the bolt diameter plus 2 millimetres in order to provide the potential for minor bolt hole deformation. High strength material should not be used, for the same reason.

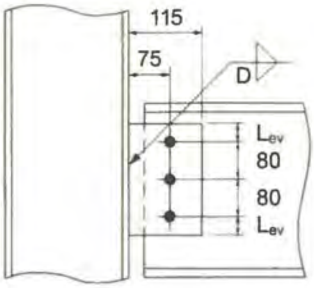
The test specimens all measured 75 mm from the plate edge at the weld to the bolt line. A minimum edge distance of 1.5 times the bolt diameter is suggested. Bolts may be either pretensioned or snug tight.

The design methodology used for Table 3-41 consisted in determining the effective eccentricity for the bolts according to Astaneh et al., finding the single shear (threads intercepted) resistance of the bolts, calculating the required thickness of the shear tab to ensure an adequate shear resistance, and selecting welds that develop the shear tab material in shear. Astaneh et al. recommended a fillet weld size equal to ¾ of the shear tab thickness.

The tabulated bolt resistances are based on a resistance factor, $\phi_b = 0.67$, found in older editions of CSA S16 and corresponding approximately to the bolt resistance incorporated into the design method proposed by Astaneh et al.

Reference

ASTANEH, A., CALL, S.M., and MCMULLIN, K.M. 1989. Design of single plate shear connections. Engineering Journal, First Quarter, American Institute of Steel Construction, Chicago, Illinois.



End distance:
L_{ev} = 40 mm for 1-inch bolts, 35 mm for smaller bolts.

SHEAR TAB BEAM CONNECTIONS

Table 3-41

FACTORED LOAD RESISTANCE (KN)
Bearing-Type Connections
Bolt Threads Intercepted

A325 Bolts, F1852 Assemblies
G40.21-300W Plates
Fillet Welds X_u = 490 MPa

RIGID SUPPORT

Number of Bolts	¾-in. Bolts				⅞-in. Bolts				1-in. Bolts			
	Plate Length (mm)	Resist-ance (kN)	Plate Thick-ness (mm)	Weld Size D (mm)	Plate Length (mm)	Resist-ance (kN)	Plate Thick-ness (mm)	Weld Size D (mm)	Plate Length (mm)	Resist-ance (kN)	Plate Thick-ness (mm)	Weld Size D (mm)
2	150	82.5	6	5	150	112	6	5	160	147	8	6
3	230	184	6	5	230	251	10	8	240	328	12	10
4	310	265	8	6	310	360	10	8	320	471	12	10
5	390	322	8	6	390	438	10	8	400	572	12	10
6	470	370	6	5	470	503	10	8	480	657	12	10
7	550	413	6	5	550	563	8	6	560	735	12	10

FLEXIBLE SUPPORT

Number of Bolts	¾-in. Bolts				⅞-in. Bolts				1-in. Bolts			
	Plate Length (mm)	Resist-ance (kN)	Plate Thick-ness (mm)	Weld Size D (mm)	Plate Length (mm)	Resist-ance (kN)	Plate Thick-ness (mm)	Weld Size D (mm)	Plate Length (mm)	Resist-ance (kN)	Plate Thick-ness (mm)	Weld Size D (mm)
2	150	62.2	6	5	150	84.6	6	5	160	111	6	5
3	230	123	6	5	230	168	6	5	240	219	8	6
4	310	196	6	5	310	266	8	6	320	348	10	8
5	390	269	6	5	390	366	8	6	400	478	10	8
6	470	342	6	5	470	465	8	6	480	607	12	10
7	550	413	6	5	550	563	8	6	560	735	12	10

TEE-TYPE BEAM CONNECTIONS

Tee-type beam connections combine some of the characteristics of single-angle connections with the web-framing leg bolted in single shear, and of double-angle connections with the outstanding legs welded to the supporting member.

Their main advantage consists in speed and ease of erection. They are also commonly used where hole making in the supporting member is undesirable (e.g. connections to HSS columns), and to avoid coping the bottom flange of the supported beam for erection purposes.

Costs are generally higher than for other types of simple beam connections because of the higher costs of fabricating the tee-sections.

Table 3-42 lists bolt capacities for bearing-type web-framing connections, and weld capacities for connections between the tee flange and its rigid support. (For a discussion of rigid versus flexible supports, see *Shear Tab Beam Connections*). The table covers connections using ¾, ⅞ and 1-inch diameter A325 bolts and F1852 assemblies. Bolt shear capacities were calculated based on the vertical reaction alone (i.e. without eccentricity), assuming that threads intercept the shear plane. Weld capacities were calculated by taking into account the out-of-plane eccentricity between the face of the support and the bolt line. These values have been computed using the same method applied to those listed in Table 3-38 for the outstanding legs of welded double-angle connections. To ensure adequate connection flexibility, the flange thickness of the tees should be held to a minimum.

Example

Tee-type beam connections with rigid supports (Table 3-42)

Given:

W460x74 beam of ASTM A992 steel, factored reaction = 375 kN

Column – HSS 254x254x13 of G40.21-350W steel

Beam web thickness = 9.0 mm

¾-inch A325 bolts, matching electrodes, X_u = 490 MPa.

Solution:

Try a tee cut from a W200x59 beam (ASTM A992); web thickness = 9.1 mm.

These thicknesses of beam web and tee web (stem) will result in threads intercepting the shear plane.

Five bolts in a vertical line provide a capacity of 395 kN > 375 kN OK

Beam web thickness required for F_u = 450 MPa is

$3.8 \times 375 / 395 = 3.6$ mm < 9.0 mm OK

Tee stem (web) thickness required is $6.0 \times 375 / 395 = 5.7$ mm < 9.1 mm OK

Length of tee required for 5 bolts is 390 mm, and weld capacity for 5 mm fillet welds is 545 kN > 375 kN OK

Clear depth of beam web between fillets, T = 391 mm > 390 mm OK

Use:

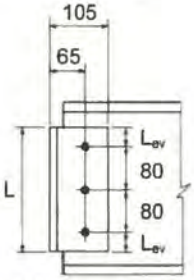
Tee cut from W200x59, 390 mm long, five ¾-inch A325 bolts connecting webs of beam and tee, and 5 mm fillet welds (with matching electrodes, X_u = 490 MPa) to supporting material.

TEE-TYPE BEAM CONNECTIONS RIGID SUPPORTS[1]

Table 3-42

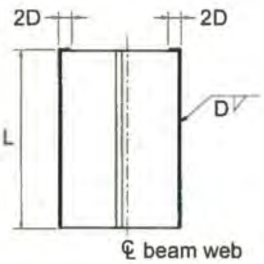
A325 Bolts, F1852 Assemblies
Fillet Welds X_u = 490 MPa

ASTM A992, A572 Gr. 50, CSA G40.21-350W Steel



Web-Framing Leg
(Bolted to Supported Web)

End distance:
L_{ev} = 40 mm for 1-inch bolts,
35 mm for smaller bolts.



Outstanding Leg
(Welded to Supporting Member)

Bolts per Vertical Line	BEARING-TYPE CONNECTIONS Factored Load Resistance (kN) Threads Intercepted						WELD CAPACITY Factored Load Resistance (kN)		
	¾-in. Bolts		⅞-in. Bolts		1-in. Bolts		Fillet Size D (mm)		
	Conn. Tee Length L (mm)	Resistance (kN)	Conn. Tee Length L (mm)	Resistance (kN)	Conn. Tee Length L (mm)	Resistance (kN)	5	6	8
2	150	158	150	215	160	281	115	139	185
3	230	237	230	323	240	421	250	300	400
4	310	316	310	430	320	562	400	480	640
5	390	395	390	538	400	702	545	654	872
6	470	474	470	645	480	843	683	820	1090
7	550	553	550	753	560	983	816	980	1310
8	630	632	630	860	640	1120	946	1140	1510

Material F_u (MPa)	Minimum Required Web Thickness of Supported Beam[2] (mm)			Approximate Required Thickness of Supporting Member with Beams Framing from One Side (F_y = 345 MPa) (mm)		
450	3.8	4.5	5.1	3.8	4.6	6.1

Material (MPa) F_y = 345 F_u = 450	Minimum Thickness of Tee Stem (mm)		
	6.0	8.3	10.5

1. For information on the distinction between rigid and flexible supports, see *Shear Tab Beam Connections.*
2. Coped beams may have additional requirements. See *Double-Angle Beam Connections.*

SEATED BEAM SHEAR CONNECTIONS

General

This section of the Handbook deals with the unstiffened angle seat and the tee-type stiffened seat designed to provide a simple beam shear connection to a supporting member. Although seated beam shear connections are designed to support vertical loads only, eccentricities produced by these connections can influence the design of supporting members as well as the connections.

Seated beam shear connections are most commonly used at beam-to-column supports. If used at beam-to-girder supports, the girder web must be checked for adequate local stability and resistance. Economy with seated beam shear connections results from ease and speed of field erection and, for unstiffened seats, simple shop fabrication.

The unstiffened angle seat consists of a relatively thick angle either shop-welded or bolted to the supporting member. They are usually bolted to column web supports due to restricted access for welding. Load capacity of an angle seat is limited by the angle thickness, unless it is stiffened. However, stiffened angle seats are more expensive to fabricate, and stiffened seats using tee-stubs built up from plate are usually more economical. Stiffened seats designed for large loads are generally referred to as brackets and are beyond the scope of this section.

A seated beam must be stabilized laterally with a flexible clip angle attached either to the top flange of the beam or to the beam web near the top of the beam. The clip angle must be thin enough to permit end rotation of the beam. Either welds or bolts can be used to connect the clip angle to the beam and supporting member. When welds are used, the fillet welds should be located along the toes of the angle.

Unstiffened Angle Seats

The capacity of unstiffened angle seats depends on the bending capacity of the seat angle. However, the beam end reaction resistances are governed by the web thickness, w, and effective bearing length of the supported beam, N, unless the beam web is stiffened. When the vertical leg of the seat angle is welded to the supporting member, the entire vertical leg is restrained by the welds, so that the capacity of the angle seat is assumed to be limited by the bending capacity of the outstanding leg. When the vertical leg is bolted to the supporting member, the top of the angle is not restrained by the bolts. Therefore, it is assumed that bending in the vertical leg, rather than the outstanding leg, controls the bending capacity of the bolted angle seat.

Tables 3-43 and 3-44 list capacities for welded and bolted unstiffened angle seats of various thicknesses for seat lengths, L = 180 mm and 230 mm, assuming beams of ASTM A992 steel (F_y = 345 MPa), seat angles of G40.21-300W steel (F_y = 300 MPa) and welds made with matching electrodes, X_u = 490 MPa. Capacities are based on unstiffened beam webs and the design models illustrated in the tables, with no allowance made for possible restraint provided by any connection between the seat and the bottom flange of the supported beam.

For detailing purposes, the gap between the beam end and the face of the supporting member is taken equal to 10 mm, although calculations are based on a gap of 20 mm.

Beam web bearing capacities in Tables 3-43 and 3-44 are based on web yielding and crippling according to Clauses 14.3.2(b)(i) and (ii) of CSA S16-14.

Vertical Leg Welded to Supporting Member

Table 3-43 lists the beam web bearing resistance values for welded seats with various angle thicknesses and beam web thicknesses, and the vertical leg weld resistance for various weld sizes and angle vertical leg lengths. The beam web bearing resistance values were calculated according to Clause 14.3.2(b), with the bearing length obtained by equating the value from Clause 14.3.2(b)(i) to the plastic bending resistance of the angle outstanding leg. The vertical weld capacity was determined using the instantaneous centre-of-rotation method, ignoring any beneficial effects due to the lower part of the angle bearing on the face of the support and weld strength increase due to weld orientation. Angle thicknesses in the top row apply to both the upper and lower parts of the table. Under each angle thickness, the largest eccentricity among all combinations of L and w is listed near the bottom and used to calculate the weld resistances.

Vertical Leg Bolted to Supporting Member

Table 3-44 lists the beam web bearing resistance values for bolted seats and the seat angle bending resistance based on the model illustrated near the top of the table. Bolt capacities are given for three sizes of bolt, two or four bolts per seat angle, for threads excluded and threads intercepted. These bolt capacities are based on assumptions thought to be conservative.

Example

Given: W530x92 beam of ASTM A992 steel, factored reaction = 185 kN
Beam web thickness = 10.2 mm, flange width = 209 mm.

Solution:

(a) *Unstiffened angle seat welded to supporting member*

Seat angle thickness and beam web bearing capacity:

From Table 3-43, a beam web thickness of 10 mm with $L = 230$ mm (to permit the 209 mm flange to be welded to the seat) and a 12.7 mm-thick angle provide a beam web bearing capacity of 210 kN > 185 kN. The 10.2 mm web is therefore adequate.

Vertical leg connection:

For an angle thickness of 12.7 mm with a vertical leg of 152 mm and a conservatively assumed eccentricity of 45 mm, 8 mm fillet welds provide connection capacity of 242 kN > 185 kN OK.

Use 152x102x12.7 seat angle 230 mm long with 152 mm leg welded to the supporting member with 8 mm fillet welds, $X_u = 490$ MPa, on each side of the vertical leg.

(b) *Unstiffened angle seat bolted to supporting member*

Seat angle thickness and beam web bearing capacity:

From Table 3-44, a 15.9 mm seat angle 230 mm long provides a capacity of:

$$183 + 0.2\,(206 - 183) = 188 \text{ kN} > 185 \text{ kN}$$

The 10.2 mm web is adequate. Four ¾-inch bolts will provide a capacity of 239 kN (threads excluded). Bolt bearing does not govern in this case (not shown).

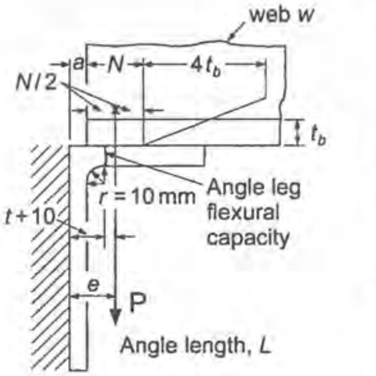
Use a 152x102x15.9 seat angle 230 mm long with 152 mm leg bolted to the supporting member with four ¾-inch A325 bolts.

WELDED UNSTIFFENED ANGLE SEATS

Table 3-43

Factored Resistances

ASTM A992, A572 Gr. 50, CSA G40.21-350W Members
CSA G40.21-300W Angles
Matching Electrode X_u = 490 MPa



Web bearing resistance (yielding)

$$P = \phi_{be} w (N + 4t_b) F_{yb}$$

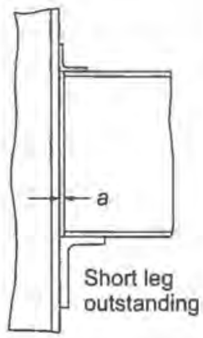
Angle leg flexural resistance

$$P = \frac{(Lt^2 / 4)\phi F_{ya}}{N/2 + a - t - r}$$

The above expressions were equated and solved for N, which was used to calculate P for the top half of the table.

For detailing purposes, gap a = 10 mm, but calculations are based on a = 20 mm.

r taken to be 10 mm
t_b taken to be 1.6w – 3 mm



Beam web factored bearing resistance (yielding and crippling, kN)	Angle t (mm)	7.9		9.5		13		16		19	
	Angle L (mm)	180	230	180	230	180	230	180	230	180	230
Beam w (mm)	5	56.0	61.6	67.2	74.0	89.5					
	6	69.0	75.1	81.5	88.8	107	116	132			
	7	84.7	90.9	98.4	106	126	136	153	166	180	
	8	103	110	118	126	147	158	177	191	206	223
	9		131	141	148	172	183	204	218	235	252
	10			166	174	199	210	233	247	266	283
	11			196	203	230	241	266	280	301	318
	12				235	264	275	301	315	338	355
	13					272	312	340	354	378	395

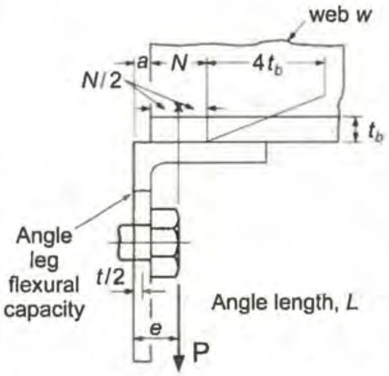
Seat vertical leg factored weld resistance (kN)	Fillet Weld D (mm)	6	6	8	8	10
Seat Angle	89x89	75.1	80.1	94.9		
	102x89	98.1	102	122		
	102x102	100	102	122	112	129
	127x89	149	149	180	166	193
	152x102	212	198	242	226	265
	178x102		251	312	293	346
	203x102			380	360	427
	Eccentricity e (mm)	34	39	45	50	55

Note: Weld resistances in the bottom half of the table were calculated using the instantaneous centre of rotation method. Bearing of the angle on the supporting member and the effect of fillet orientation on the weld strength were ignored.

BOLTED UNSTIFFENED ANGLE SEATS

Factored Resistances

ASTM A992, A572 Gr. 50, CSA G40.21-350W Members
CSA G40.21-300W Angles
A325 Bolts, F1852 Assemblies



Web bearing resistance (yielding)

$$P = \phi_{be} w (N + 4t_b) F_{yb}$$

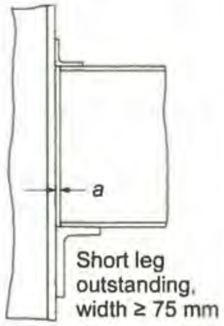
Angle leg flexural resistance

$$P = \frac{(Lt^2/4)\phi F_{ya}}{N/2 + a - t/2}$$

The above expressions were equated and solved for N, which was used to calculate P for the top half of the table.

For detailing purposes, gap a = 10 mm, but calculations are based on a = 20 mm.

t_b taken to be 1.6 w – 3 mm

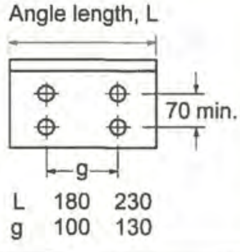


Factored Resistance – Beam Web Bearing or Seat Angle Bending (kN)

Angle t (mm)		9.5		13		16		19	
Angle L (mm)		180	230	180	230	180	230	180	230
Beam web thickness w (mm)	5	47.1	54.0	66.6	75.9	86.5			
	6		63.1	77.3	87.5	99.5	112	121	
	7			89.4	100	114	127	137	154
	8			103	115	129	144	155	172
	9				131	147	163	174	193
	10					167	183	196	215
	11						206	220	240
	12							246	267
	13							276	296
	14								329

Factored Bolt Capacity (kN)

Bolt Size (in.)	¾		⅞		1	
No. of Bolts	2	4	2	4	2	4
Threads excluded	176	239	240	326	313	425
Threads intercepted	138	210	188	286	245	374



Bolted connection capacities were calculated according to CSA S16-14 Clause 13.12.1.4. For a single row of bolts, the bolt tension is taken equal to the shear. For two rows of bolts, the top row is assumed to resist the total tensile force (taken equal to the shear) and half the total shear.

Single row of bolts (n = 2): $V_f = \dfrac{2T_r V_r}{\sqrt{T_r^2 + V_r^2}}$ Two rows of bolts (n = 4): $V_f = \dfrac{4T_r V_r}{\sqrt{T_r^2 + 4V_r^2}}$

STIFFENED SEATED BEAM CONNECTIONS

Table 3-45 lists factored resistances of stiffened seats for the tee-shaped weld configuration shown. Capacities are based on the use of matching electrodes, X_u = 490 MPa, and steel with F_y = 300 MPa. They may be used conservatively for steel with F_y = 345 MPa or 350 MPa. Factored resistances are computed using the formulas given in the section, *Eccentric Loads on Weld Groups, Shear and Moment*. For seats having a thin and narrow stiffener, yielding or crippling resistance of the stiffener as determined in accordance with S16-14 Clause 14.3.2(b) governs the tabulated values.

The figures in Table 3-45 show the general arrangement. Although the horizontal welds connecting the seat plate to the support were ignored in the calculations, they are provided for stability. Welds smaller than the vertical stiffener welds may be used, and they do not intersect the vertical welds. Generally, the seat plate is connected to the stiffener with welds having a minimum shear resistance equal to the capacity of the welds connecting the seat plate to the supporting member. Welds or bolts may be used to connect the supported beam to the seat and for attachment of the clip angle required to stabilize the beam. If welds are used, the seat should be long enough to accommodate the fillet welds as shown in the figure. If bolts are used, the seat length should match or exceed the flange width of the beam.

When stiffened seats are in line on opposite sides of a column web, the size of the vertical fillet welds (with X_u = 490 MPa) shall not exceed $F_y / 524$ times the thickness of the column web, so as not to exceed the shear resistance of the column web:

$$\frac{2\phi_w 0.67 X_u}{0.66\phi\sqrt{2}} = 524$$

As an alternative to limiting the weld size, a longer stiffener may be used to reduce the shear stresses in the column web.

Example

Given:

W530x101 beam without bearing stiffener. Factored reaction = 440 kN

Web thickness = 10.9 mm, flange width = 210 mm, flange thickness = 17.4 mm

Connected to web of W310x129 column, web thickness = 13.1 mm

Design stiffened welded seat for beams connected to both sides of column web, which is subjected to axial compression only.

ASTM A992 steel for the beam and column, G40.21-300W steel for the stiffener. Matching electrodes, X_u = 490 MPa.

Solution:

(a) *Vertical stiffener*

For an unstiffened beam web, required bearing width, N:

$B_r = \phi_{be} w (N + 4t) F_y$ Clause 14.3.2(b)(i)

$$N = \frac{B_r}{\phi_{be} w F_y} - 4t = \frac{440 \times 10^3}{0.75 \times 10.9 \times 345} - 4 \times 17.4 = 86.4 \text{ mm}$$

For $a = 20$ mm clearance, minimum stiffener width, $W = N + a = 86.4 + 20 = 106$ mm.

Check web crippling:

$$B_r = 0.60\phi_{be}w^2\sqrt{F_y E} \quad \text{Clause 14.3.2(b)(ii)}$$

$$= 0.60 \times 0.75 \times 10.9^2\sqrt{345 \times 200\,000} = 444 \text{ kN} > 440 \text{ kN}$$

For stiffeners on both sides of column web, maximum effective weld size so that shear resistance of column web is not exceeded is:

$13.1 \times 345 / 524 = 8.6$ mm

From Table 3-45, with $W = 140$ mm ($e = 80$ mm), stiffener thickness = 15.9 mm, 8 mm fillet welds and $L = 275$ mm, factored resistance of welded seat provided is:

463 kN > 440 kN OK

Use 16x140 stiffener, 275 mm long, welded to column web with 8 mm fillet welds.

(b) *Horizontal seat plate*

Try 10 mm plate and 6 mm fillet welds to attach the seat plate to the column web.

Minimum length of seat plate assuming beam is bolted to seat:

beam flange width = 210 mm

Note: If the beam is welded to the seat, the minimum length of the seat plate is:

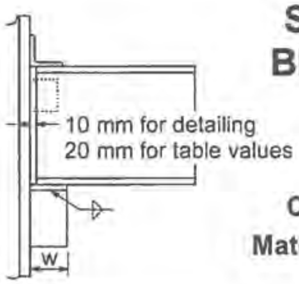
$210 + 2(2 \times 6) = 234$ mm

Use a 10x140 seat plate, 210 mm long, welded to the column web with two 60 mm-long segments of 6 mm fillet welds on the underside of the seat, and bolted to the bottom flange of the beam with two ½-inch A325 bolts.

(c) *Weld between stiffener and seat plate*

Minimum length of weld required = $2 \times 60 = 120$ mm (same as the length of the seat plate–to-column welds) for 6 mm fillets

Length available is 2×140 mm = 280 mm > 120 mm OK

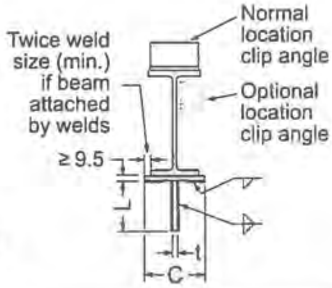


STIFFENED SEATED BEAM CONNECTIONS

Table 3-45

CSA G40.21-300W Seat Steel

Matching Electrodes X_u = 490 MPa



FACTORED RESISTANCE (kN)

Stiffener Width, Design Eccentricity (mm)	Stiffener Thickness t (mm)	Fillet Size D (mm)	Length of Stiffener, L (mm): 150	175	200	225	250	275	300	350	400	450	500
W = 100, e = 60	12.0	6	170	210	252	294	319	319	319	319	319	319	319
	15.9	8	225	279	334	389	422	422	422	422	422	422	422
	19.0	10	269	333	399	465	504	504	504	504	504	504	504
	22.0	12	311	386	462	539	584	584	584	584	584	584	584
W = 140, e = 80	12.0	6	148	187	226	267	308	350	392	427	427	427	427
	15.9	8	196	247	300	353	408	463	519	565	565	565	565
	19.0	10	235	295	358	422	488	554	620	675	675	675	675
	22.0	12	272	342	415	489	564	641	718	782	782	782	782
W = 180, e = 100	15.9	8	168	221	271	322	375	428	483	593	705	708	708
	19.0	10	205	264	323	385	448	512	577	709	842	846	846
	22.0	12	239	305	374	445	518	593	668	820	975	980	980
W = 220, e = 120	15.9	8	140	190	245	294	345	397	450	557	667	778	851
	19.0	10	171	232	293	352	412	474	537	666	797	930	1020
	22.0	12	201	273	340	407	477	549	622	771	923	1080	1180

Notes: Yielding of the stiffener controls the resistance values to the right of the thick vertical lines.

A WT section proportioned to meet the stiffened seat requirements may also be used.

MOMENT CONNECTIONS

General

Continuous construction requires moment-resisting beam-to-column connections that will maintain, virtually unchanged, the original angles between intersecting members at specified loads. Rigid moment connections can be provided by using welds, bolts or combinations of welds and bolts. In general, numerous configurations and details are possible; Figure 3-3 shows four possible arrangements.

The connections illustrated below apply to moment frames subject to gravity and wind loads. They may be used to resist seismic forces corresponding to R_d = 1.5 in frames of "Conventional Construction", subject to these conditions:

(a) Where the building height is within 15 m, Clause 27.11.1 of CSA S16-14 requires that factored seismic forces be increased for buildings with specified short-period spectral acceleration ratios, $I_E F_a S_a(0.2)$, greater than 0.45, unless connections are designed so that the expected failure mode is ductile (See Commentary) and

(b) Where the building height exceeds 15 m and $I_E F_a S_a(0.2)$ exceeds 0.35, Clause 27.11.3 provides additional connection design requirements for Conventional Construction.

For moment connections in Type D, Type MD and Type LD moment-resisting frames, ductile eccentrically braced frames and Type D plate walls, see the pertinent provisions for each respective system in Clause 27 and Annex J of S16-14, and CISC (2014).

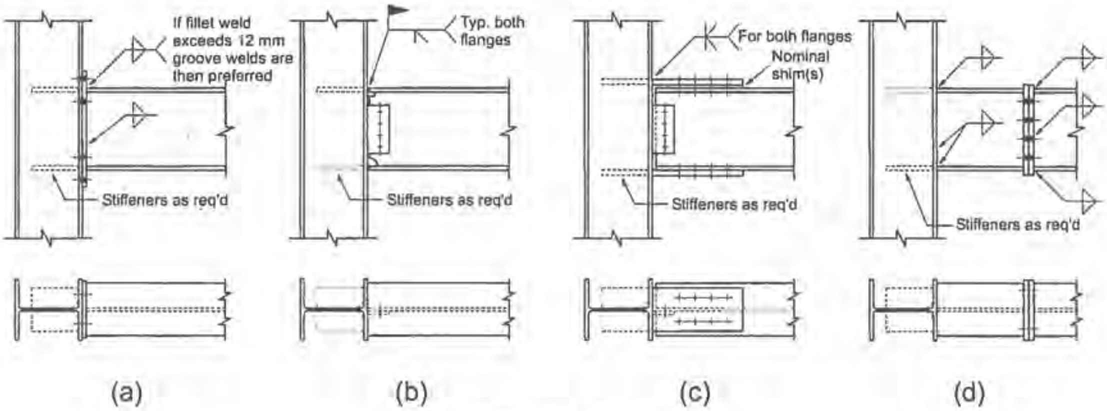


Figure 3-3

Figure 3-3(a) illustrates a heavy plate shop-welded to the end of the beam and field-bolted to the column. The end plate distributes flange forces over a greater length of column web than does a fully welded joint, but prying action must be considered.

Figure 3-3(b) illustrates beam flanges field-welded directly to the column with groove welds. Shear capacity is developed by a seat angle, web-framing angle or plate, or by welding the beam web directly to the column. Backing bars and run-off tabs for the welds may be required.

Figure 3-3(c) illustrates the use of moment plates shop-welded to the column with groove or fillet welds and fillet-welded, or preferably bolted, to the flanges of the beam. The moment plates are spaced to accommodate rolling tolerances for beam depth and flange tilt, and nominal shims are provided to fill any significant gap. Minor gaps are closed by the action of bolting.

Shear capacity is usually provided by a web plate welded to the column and field-bolted to the beam.

Figure 3-3(d) illustrates the use of short beam sections shop-welded to the column, and field-bolted to the beam near a point of contraflexure. An end-plate connection is shown but lapping splice plates for the flanges and web may be more economical, depending on the forces to be transmitted and the relative ease of achieving field fit-up.

To ensure that the connection provided is consistent with the design assumptions used to proportion members of a structure, it is important that the designer provide the connection designer with governing maximum and coincident moments, shears and axial forces to be developed at the connection. See Clause 4 in S16-14 and the Commentary. In addition, the "type" of seismic moment-resisting frame (in this case, *Conventional Construction*) should be specified.

Column Stiffeners

Where rigid connections are required, the resistance of a column section to local effects in the panel zone is important. With relatively small beams connected to heavy columns, the columns will provide the degree of fixity assumed in the design of beams. With large beams, however, the columns will usually have to be strengthened locally by means of stiffeners, doubler plates or both.

Column stiffeners are provided opposite tension flanges of the connected beams to minimize curling of the column flanges with resultant overstressing of the central portion of the weld connecting the beam flange (or moment flange plate) to the column. Opposite the compression flanges of the beams, column stiffeners are provided to prevent the column web from yielding and, for a less compact web, buckling. The most commonly used stiffeners are horizontal plates. When beams of different depths frame into opposite flanges of the column, either inclined stiffeners or horizontal plate stiffeners opposite the flange of each beam may be used. If shear generated in the column web at the moment connection exceeds the column shear capacity, "doubler" plates or diagonal plate stiffeners are used to increase the column web shear capacity locally. Clause 21.3 of S16-14 specifies requirements for web stiffeners on H-type columns when a beam is rigidly framed to the column flange.

References

The following references contain more detailed information on the design of moment connections. Some refer to allowable stress rules and must be interpreted for limit states applications.

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Examples

Note: In the following examples, the solution chosen in each case is intended to illustrate only one of several satisfactory solutions that could be used. In any given situation, the design will be influenced by the individual fabricator's experience, fabrication methods and erection procedures.

Example 1

Given:

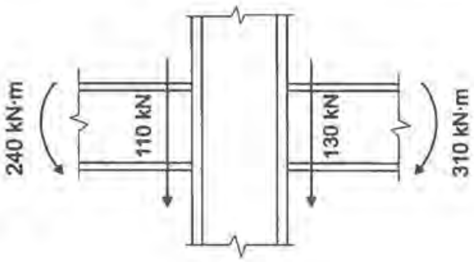
Design an interior beam-to-column connection for the following coincident forces and moments due to factored gravity loads.

Factored beam moments = 240 kN·m and 310 kN·m

Factored beam shears = 110 kN and 130 kN

Steel: ASTM A992 (W shapes), CSA G40.21 300W (plates) F_u = 440 MPa, with matching electrodes X_u = 490 MPa

W310x86 Column	W410x60 Beam
t_c = 16.3 mm	t = 12.8 mm
w_c = 9.1 mm	w = 7.7 mm
b = 254 mm	d = 407 mm
d = 310 mm	b = 178 mm
k_1 = 25 mm	Class 1 in bending
T = 234 mm	



Solution:

(a) Web Connection

The design of the connection between the beam web and the column flange need only account for the vertical shear, neglecting eccentricity. (Design for 130 kN shear.)

Two alternatives are shown to illustrate a field-welded and a field-bolted condition.

Alternative 1

Single plate field-welded to beam web, shop-welded to column flange, holes for 2 - ¾ in. erection bolts

To resist the factored shear, try 5 mm fillet welds (X_u = 490 MPa) on 6 mm plate.

Required weld length is $130 / 0.778 = 167$ mm (Table 3-24(a))

Try a 230 mm long plate, for a W410 beam (Table 3-38)

Check plate for factored shear capacity. (Clause 21.12, S16-14)

Gross plate area: $A = 230 \times 6 = 1380 \text{ mm}^2$

$$V_r = \phi\, 0.66 F_y A = 0.9 \times 0.66 \times 300 \times 1380 / 1000 = 246 \text{ kN} > 130 \text{ kN}$$

Use $6 \times 75 \times 230$ plate and 5 mm fillet welds with X_u = 490 MPa (matching condition).

Alternative 2

Single plate shop-welded to column flange, field-bolted to beam web with ⅞ in. A325 bolts (or ¾ in. bolts if also used to connect the flange plates)

From Table 3-4, factored shear resistance, single shear, threads intercepted, for ⅞ in. A325 bolts = 108 kN per bolt

For 2 bolts, $V_r = 2 \times 108 = 216 \text{ kN} > 130 \text{ kN}$

Check factored bearing resistance on beam web, w = 7.7 mm

From Table 3-6(a), B_r for t = 7 mm is 168 kN per bolt > 79.0 kN

Try 6 mm plate, 230 mm long, 2 bolts at 160 mm pitch:

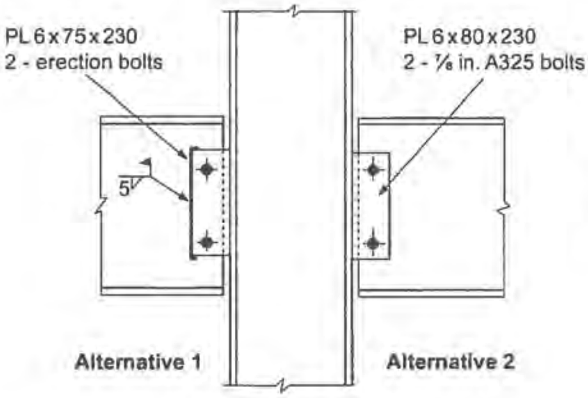
Bearing resistance on 6 mm plate, Table 3-6(b), B_r = 141 kN per bolt > 79.0 kN

Required thickness of plate (based on shear resistance, Clause 13.11) is:

$$130 \times 10^3 / [0.75 \times 0.6 \times 230\,(300 + 440)/2] = 3.4 \text{ mm} < 6 \text{ mm}$$

Block shear resistance (tension + shear, Clause 13.11) is adequate (not shown).

Use $6 \times 80 \times 230$ plate and two ⅞ in. A325 bolts at 160 mm pitch.



Alternative 2 replaces the two erection bolts with permanent high-strength bolts, and eliminates vertical field welding (likely a better solution).

(b) Flange Connection

Two alternatives are shown to illustrate field-bolted and field-welded conditions.

Alternative 1

Top and bottom moment plates shop-welded to column, field-bolted to beam flanges with A325 bolts

Flange force due to factored loads is $310 \times 1000 / 407 = 762$ kN

Bolts

Assuming joint length, $L < 760$ mm, from Table 3-4, required number of ⅞ in. A325 bolts (threads excl.) is:

$762 / 154 = 4.95$ Use 6 bolts (2 rows of 3; $L = 2(80) = 160 < 760$ mm)

Shear per bolt $= 762 / 6 = 127$ kN

Beam

Factored bearing resistance, $t = 12.8$ mm

From Table 3-6(a), for $t = 12$ mm, $B_r = 288$ kN per bolt $>$ 127 kN

Block Shear (Cl. 13.11). Try 80 mm pitch, 70 mm end distance and 35 mm edge distance.

(i) Edge block shear pattern, Figure 3-4(a)

$$T_r = 0.75\,(12.8)\,[1.0\,(2 \times 35 - 26)\,450 + 0.6\,(2)\,(70 + 2 \times 80)\,(345 + 450)/2]/1000$$

$$= 1240 \text{ kN} > 762 \text{ kN}$$

(ii) Tear-out pattern, Figure 3-4(b)

Not critical (calculation not shown).

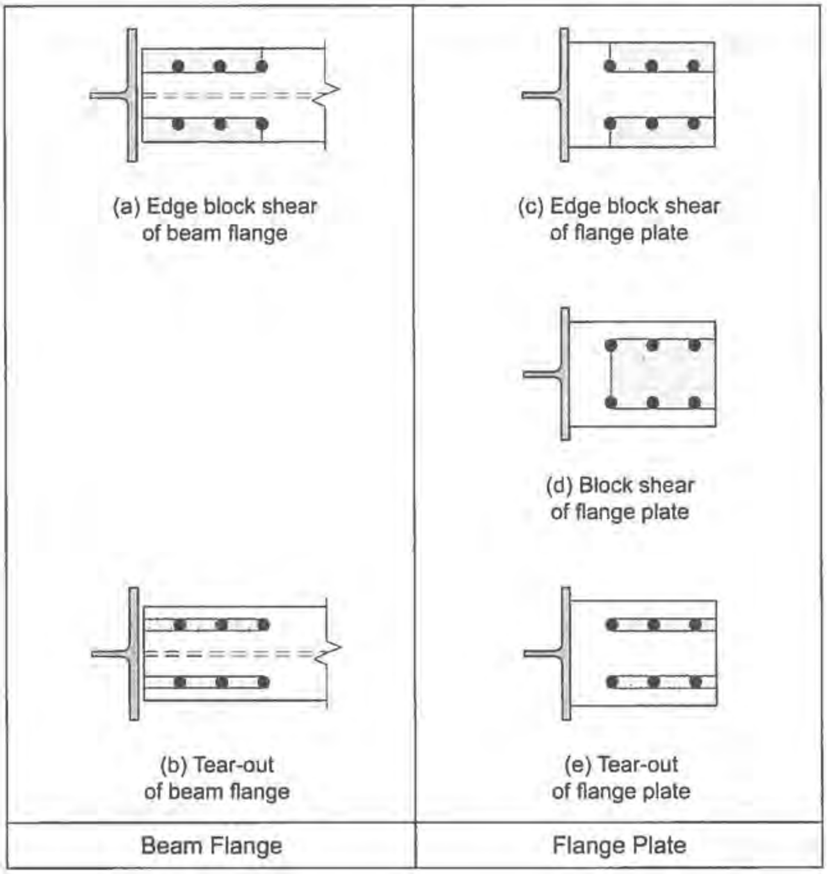
Flange Plates

Tension (Clause 13.2(a))

Required gross area $= 762 \times 10^3 / (0.9 \times 300) = 2\,820$ mm^2

Required net effective area $= 762 \times 10^3 / (0.75 \times 440) = 2\,310$ mm^2

Try 200×16 plate, and check areas.



Block Shear and Tear-Out
Tension Flange and Flange Plate

Figure 3-4

Gross area is $200 \times 16 = 3\,200 \text{ mm}^2 > 2\,820 \text{ mm}^2$

Net area is $(200 - 2 \times 26)16 = 2\,370 \text{ mm}^2 > 2\,310 \text{ mm}^2$

(Holes assumed not drilled. While 24 mm standard holes for ⅞ in. bolts are assumed here, oversized holes are usually required to facilitate erection. See S16-14, Clause 22.3.5.1(a) and Table 3-47)

Try 40 mm end distance and 10 mm beam end clearance

Plate length = $(2 \times 80) + 40 + 70 + 10 = 280$ mm

Block shear (Cl. 13.11)

(i) Edge block shear pattern, Figure 3-4(c)

$$T_r = 0.75\,(16)\,[1.0\,(200 - 178 + 2 \times 35 - 26)\,440 + 0.6\,(2)\,(40 + 2 \times 80)\,(300 + 440)/2]/1000 = 1410 \text{ kN} > 762 \text{ kN}$$

(ii) Block shear pattern, Figure 3-4(d)

$$T_r = 0.75\,(16)\,[1.0\,(178 - 2 \times 35 - 26)\,440 + 0.6\,(2)\,(40 + 2 \times 80)(300 + 440)/2]/1000 = 1500 \text{ kN} > 762 \text{ kN}$$

(iii) Tear-out pattern, Figure 3-4(e): Not critical (calculation not shown)

Bolt bearing: Not critical (calculation not shown)

Compression: Not critical (calculation not shown)

Note: The factored tensile resistance of the complete-joint-penetration groove welds with matching electrodes between the top and bottom moment plates and the column flange shall be taken as that of the base metal (S16-14, Clause 13.13.3.1).

Alternative 2

Moment plate field-welded to column flange and top flange of beam, bottom flange of beam welded directly to column flange with groove weld

As in alternative 1, the moment plate is designed to transmit the factored beam flange force of 762 kN.

Plate area required (gross) is $762 \times 10^3 / (0.9 \times 300) = 2\,820$ mm^2

Select plate width narrower than beam flange width to permit downhand welding.

Try 140 mm plate. Maximum weld size would be $(178 - 10 - 140)/2 = 14$ mm.

Plate thickness required is $2\,820 / 140 = 20.1$ mm. Use 22 mm plate

From Table 3-24(a), for matching electrodes $X_u = 490$ MPa, 12 mm fillet weld, the factored shear resistance = 1.87 kN/mm (for $\theta = 0°$)

Approx. weld length required is: $762 / 1.87 = 407$ mm. Try 400 mm.

End weld length is 140 mm, therefore length each side is $(400 - 140) / 2 = 130$ mm

The factored shear resistance of the transverse end weld ($\theta = 90°$) is 2.80 kN/mm.

(Table 3-24(b)) The base metal check is no longer required when matching electrodes are used. For the two longitudinal welds ($\theta_1 = 0°$), the strength reduction factor for multi-orientation fillet welds (Clause 13.13.2.2),

$M_w = (0.85 + 0/600)/(0.85 + 90/600) = 0.85$

$V_r = 140 \times 2.80 + 2\,(130 \times 1.87 \times 0.85) = 805 \text{ kN} > 762 \text{ kN}$

It is generally recommended that an unwelded length of plate equal to at least 1.2 times the plate width be provided, in order to ensure adequate ductility.

Therefore minimum plate length is: $130 + (1.2 \times 140) = 298$ mm.

Use $22 \times 140 \times 300$ plate welded to column flange with full penetration groove weld and welded to top flange of beam with 400 mm of 12 mm fillet welds.

If the flange plate is also subject to compression, the axial compressive resistance should be checked.

Other common alternatives include: (a) field-welding the top and bottom flanges of the beam directly to the column flange with full-penetration groove welds using backing bars fitted against the column flange and (b) extended end-plate connection.

(c) Column Shear Capacity

The column will be subject to a shear force due to the unbalanced moment. S16-14 Clause 21.3 requires stiffening of the column web if this shear exceeds

$V_r = 0.8\,\phi A_w F_s$ (Clause 13.4.2)

where F_s is calculated according to Clause 13.4.1.1

$h/w = (310 - 2 \times 16.3)/9.1 = 30.5 < 1014/\sqrt{F_y} = 54.6$

$F_s = 0.66\,F_y = 0.66 \times 345 = 228$ MPa

$V_r = 0.8 \times 0.9 \times 9.1 \times 310 \times 228 = 463$ kN

Shear force is $(310 - 240) \times 1000 / 407 = 172$ kN $<$ 463 kN OK

Thus, no reinforcing of the web is required for shear. (Shear forces from the column, above and below the moment connections, are ignored for simplicity.)

(d) Horizontal Column Panel Zone Stiffeners

Design the column stiffeners to S16-14, Clause 21.3.

Clause 21.3(a): $B_r = 0.80 \times 9.1\,[12.8 + (10 \times 16.3)]\,0.345 = 442$ kN < 762 kN

Therefore, stiffeners are required opposite the compression flange for capacity of $762 - 442 = 320$ kN

Clause 21.3(b)(i): $T_r = 7 \times 0.9 \times 16.3^2 \times 0.345 = 577$ kN $<$ 762 kN

Stiffeners are also required opposite the beam tension flange for a capacity of $762 - 577 = 185$ kN

Total stiffener area required at compression flange is:

$320 / (0.9 \times 0.300) = 1\,190$ mm^2

Maximum b/t ratio is $200/\sqrt{300} = 11.55$ (S16-14, Clause 14.4.2)

Try 90 mm wide stiffener each side of column web (beam flange is 178 mm wide).

Minimum $t = 90 / 11.55 = 7.8$ mm Try 12 mm

Effective stiffener width to clear column k_l distance is $(178/2) - 25 = 64$ mm

Effective stiffener area is $2 \times 64 \times 12 = 1\,540$ mm^2 $> 1\,190$ mm^2 OK

Use 12×90 stiffener each side of column web opposite compression flange.

Use same stiffeners opposite tension flange.

(e) Stiffener Welds

Welds connecting stiffeners to column flange must be sufficient to develop a total force in the two stiffeners of 320 kN.

For double fillet welds at stiffener ends (length 64 mm), weld resistance required is

$320 / (2 \times 64) = 2.5$ kN/mm

From Table 3-24(b), 8 mm double fillet welds with matching electrodes $X_u = 490$ MPa provide: $2 \times 1.87 = 3.74$ kN/mm OK

Welds connecting stiffeners to column web must transfer shear forces due to unbalanced beam moment of $172 / 2 = 86.0$ kN per side.

Try 150 mm weld length (T distance = 234 mm).

Weld resistance required is $86.0 / 150 = 0.573$ kN/mm (one-sided weld will do). Use single 5 mm fillet weld on each stiffener for 0.778 kN/mm. (Table 3-24(a))

See Welding Practice in Part 6 for minimum size of fillet welds.

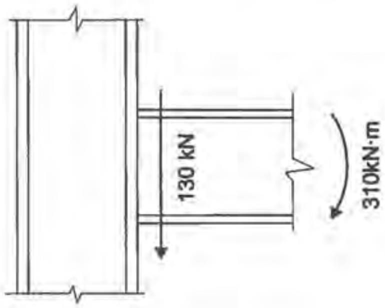
Example 2

Given:

Design an exterior beam-to-column connection for an elastically analyzed frame, in which the column size is the same as example 1, and the beam is a W460x74 having a factored end moment of 310 kN·m and a factored end shear of 130 kN.

W310x86 column
See example 1
for dimensions

W460x74 beam
t = 14.5 mm
w = 9.0 mm
d = 457 mm
b = 190 mm
Class 1 (in bending)



Solution

This example is basically an extension of Example 1, and the solutions given are intended only to provide information on other possibilities.

(a) Web Connection

Use an unstiffened seat angle shop-welded to the column to carry the beam shear and to support the beam during erection.

From Table 3-43, for a beam web of 9 mm and a seat length of 230 mm, a 9.5 mm thick angle will provide a beam web bearing capacity of 148 kN > 130 kN. Also a vertical leg of 127 mm with 6 mm fillet welds provides a vertical leg connection capacity of 149 kN > 130 kN.

Use $127 \times 89 \times 9.5$ angle $\times$ 230 mm long with 127 mm leg vertical, welded to column flange with 6 mm fillet welds with matching electrodes $X_u = 490$ MPa.

(b) Flange Connection

Assume field-welded connection with full-penetration groove welds connecting top and bottom flanges of the beam directly to the column flange (a suggested alternative in Example 1). The seat angle would serve as backing for the bottom flange weld.

(c) Column Shear Capacity

Shear force is $310 \times 1\,000 / 457 = 678$ kN (shears from column ignored)

Diagonal stiffeners will be used to carry shear in excess of the 463 kN (see Example 1) shear capacity of the column web (Use of a doubler plate is an alternative).

Horizontal component of stiffener force is 678 – 463 = 215 kN

If θ is the angle between stiffener and horizontal plane,

$\cos\theta = 310 / (310^2 + 457^2)^{1/2} = 0.561 \quad (\theta = 56°)$

Force in stiffener is $215 / \cos\theta = 215 / 0.561 = 383$ kN

Total stiffener area required is $383 / (0.9 \times 0.300) = 1\,420$ mm^2

Effective stiffener width to clear column k_l distance is $(190/2) - 25 = 70$ mm

Try 90 mm wide stiffener on each side of web.

Stiffener thickness required is $1\,420 / (2 \times 70) = 10.1$ mm   Try 12 mm.

b/t is $90/12 = 7.5 < 11.55$ maximum   OK   (see Example 1)

Use one 12 × 90 diagonal stiffener each side of column web.

(d) Horizontal Column Web Stiffeners

$B_r = 0.80 \times 9.1\,(14.5 + (10 \times 16.3))\,0.345 = 446$ kN < 678 kN   Clause 21.3(a)

Stiffeners are required opposite the compression flange for: 678 – 446 = 232 kN

$T_r = 7 \times 0.9 \times 16.3^2 \times 0.345 = 577$ kN $<$ 678 kN   Clause 21.3(b)(i)

Stiffeners are also required opposite the tension flange for: 678 – 577 = 101 kN

Design stiffeners for 232 kN; area required is: $232 / (0.9 \times 0.300) = 859$ mm^2

Use two 12 × 90 stiffeners (see Example 1).

(e) Stiffener Welds

Diagonal Stiffeners

Welds connecting the stiffeners to the column flanges must be sufficient to develop a total force in the two stiffeners of 383 kN (see above). Since the dihedral angles are within the range of 30° to 60° (i.e. 34° and 56°), partial-joint-penetration groove welds may be used to carry the calculated forces.

For double PJP groove welds at ends of stiffeners (length = 70 mm), weld resistance required is $383 / (2 \times 70) = 2.74$ kN/mm

From Table 3-23, factored shear resistance on effective throat of welds with matching electrodes, $X_u = 490$ MPa is 220 MPa. Effective throat required = $2.74 / 2 / 220 \times 1000 =$ 6.2 mm. OK

Use PJP groove welds with minimum effective throat = 6.2 mm and matching electrodes $X_u = 490$ MPa, top and bottom at each end of stiffeners (skew joints at 34° and 56° angles), and nominal 5 mm stitch fillet welds between stiffener and column web.

Note that when the dihedral angle < 45°, CSA W59 requires that the effective throat be established through procedure qualification.

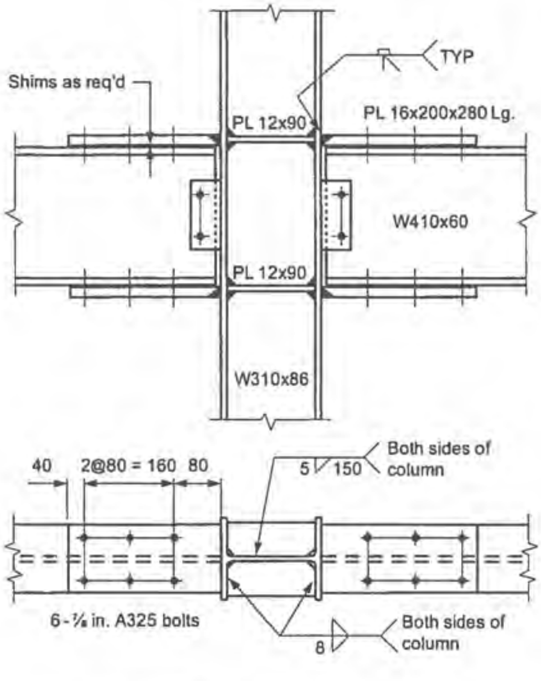
Horizontal Stiffeners

The end welds must develop total forces in the stiffeners of 232 kN, for which double 5 mm fillet welds are OK.

Welds connecting the horizontal stiffeners to the column web need transfer only a portion of the stiffener load to the column web, as most of that load proceeds up the diagonal stiffeners. However, it is conservative to size these welds to transfer the total load in the stiffeners. For a weld length of 150 mm (see Example 1), weld resistance required is:

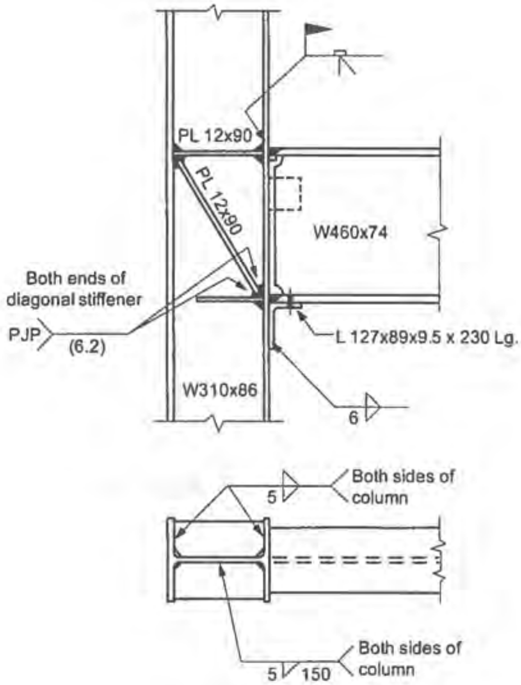
$$232/(2 \times 150) = 0.773 \text{ kN/mm}$$

for which a single 5 mm weld on each stiffener provides 0.778 kN/mm.



Shop-welded, field-bolted
Web connections are Alternative 2
Flange connections are Alternative 1

Example 1



Both shop- and field-welded

Example 2

HOLLOW STRUCTURAL SECTION CONNECTIONS

General

Hollow structural sections are frequently used for columns, trusses and space structures due to aesthetics, reduced weight for compression members and other reasons. This section of the Handbook presents sketches of some commonly used connections (Figures 3-5 to 3-9), and information for HSS welds (Figure 3-10 and Table 3-46). Since the behaviour and resistance of welded HSS connections are not always intuitive, their detail design should be undertaken only by engineers who are familiar with current literature on the subject.

The connections illustrated in Figures 3-5 and 3-6 are simple shear connections designed in a conventional manner. The recommended width-to-thickness ratio of the Tee flange is 13 or more in order to ensure suitable rotational flexibility.

The International Committee for the Study and Development of Tubular Structures (CIDECT) has played a major role in sponsoring international research that has resulted in the International Institute of Welding (IIW) making comprehensive design recommendations for HSS connections. Subsequently, a series of "state-of-the-art" design guides edited by CIDECT has been produced (see references). Based on this research, CISC has published *Hollow Structural Section Connections and Trusses—a Design Guide, 2nd Edition* (1997), which is a practical and comprehensive reference dedicated to the Canadian market with design examples that generally meet the requirements of CAN/CSA-S16.1-94.

Basic Considerations for Welded HSS Connections

A prime application of HSS members is in architecturally exposed areas where careful attention must be given to aesthetics of the connections. Simple welded connections without the use of reinforcing material often present the most pleasing and economical solutions. The following fundamentals should be kept in mind.

1. HSS members should not be selected on the basis of minimum mass. That implies that the members will need to be connected for their full capacity, which often is not possible without detail reinforcing material.
2. The force that can be transmitted from one HSS member to another is known as the "connection resistance" and is a function of the relative dimensions and wall thicknesses of the members. It is frequently less than the capacity of the connected member. Therefore, it is necessary to establish that the contemplated members have sufficient connection resistance before the member sizes can be confirmed.
3. Furthermore, design documents that specify "connect for member capacity" often have the effect of causing HSS connections to be reinforced, even if that was not the intent.
4. Square and rectangular HSS are much easier to fabricate than are round HSS because of the complexities of the connection profiles.
5. Try to avoid connections whose members are the same width. Welding is simpler and less expensive if fillet welds can be used along the sides of the connected member. On the other hand, connection resistance increases as the width of branch members approaches the width of main members, and is a maximum when the widths are the same. Therefore, to obtain optimum strength and economy with a square or rectangular HSS connection, the branch member should be as wide as possible, but not wider than the main member minus about five or six times the wall thickness of the main member (since the outer corner radius is generally between two and three times the wall thickness).

6. Connection resistance is improved when branch members have thin walls relative to the main member. A smaller-size main member with a thicker wall may not be much heavier than a larger one with a thinner wall.
7. Full-penetration welds are seldom justified (other than for member splices). They are not advantageous where connection resistance is less than the member capacity. In addition, they are not prequalified for HSS, and the certification for welders is more difficult. Inspection is much more difficult.
8. Ultrasonic inspection has limited application to HSS connections, and radiographic inspection is often only applicable to full-strength splicing of members.

Additional Considerations for HSS Trusses

1. Optimum economy can often be achieved by reducing the number of different size members that are used in a truss. It is less expensive to procure and handle a relatively large amount each of just of few sizes than a small amount each of many sizes.
2. Simple gap connections are usually the most economical when connecting pairs of web members to a truss chord. Overlap connections require additional profiling of members, more precise fitting, and sometimes interrupted fitting to perform concealed welding. Reinforced connections are generally the most expensive.
3. If fatigue is a design consideration, careful attention should be paid to the connection details. It is suggested that overlap connections of at least 50% be used for trusses subjected to fatigue loading.
4. Primary bending moments due to eccentricity e (Figure 3-7) may be ignored, with regard to *connection* design, provided the intersection of the centre lines of the web members lies within the following range measured from the centre line of the chord: 25% of the chord depth towards the outside of the truss, and 55% of the chord depth towards the inside of the truss.
5. Secondary bending moments (due to local connection deformations) may be neglected provided dimensional parameters of the connected members fall within ranges presented in Packer and Henderson (1997).
6. Since the effectiveness of load transfer from one HSS section to another is more a function of dimensional parameters of the members connected than it is of the amount of welding, Packer and Henderson (1997) outline methods to calculate connection efficiency and weld effectiveness.
7. Profiling of round members is generally required when they are joined to other members. If aesthetics allow the web members to have the ends flattened instead of profiled, cost savings may be achieved.

In HSS connections, members are usually welded all around. Table 3-46 gives the length of welds for square and rectangular web members connected to chord members at various angles θ, calculated in accordance with Clause 10.8.5.1 of AWS D1.1 (1990).

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Figure 3-5

BEAM TO HSS COLUMN CONNECTIONS

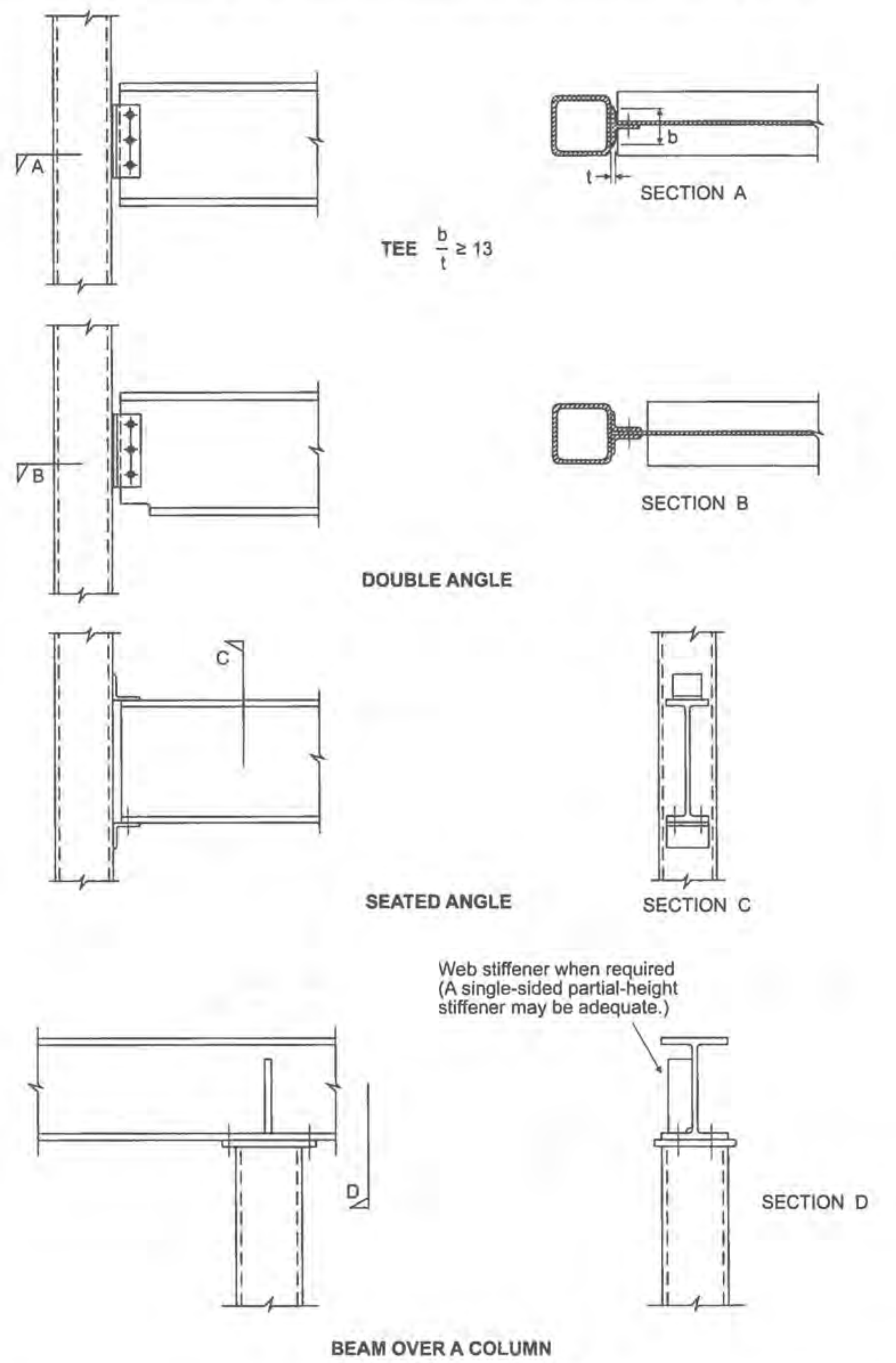
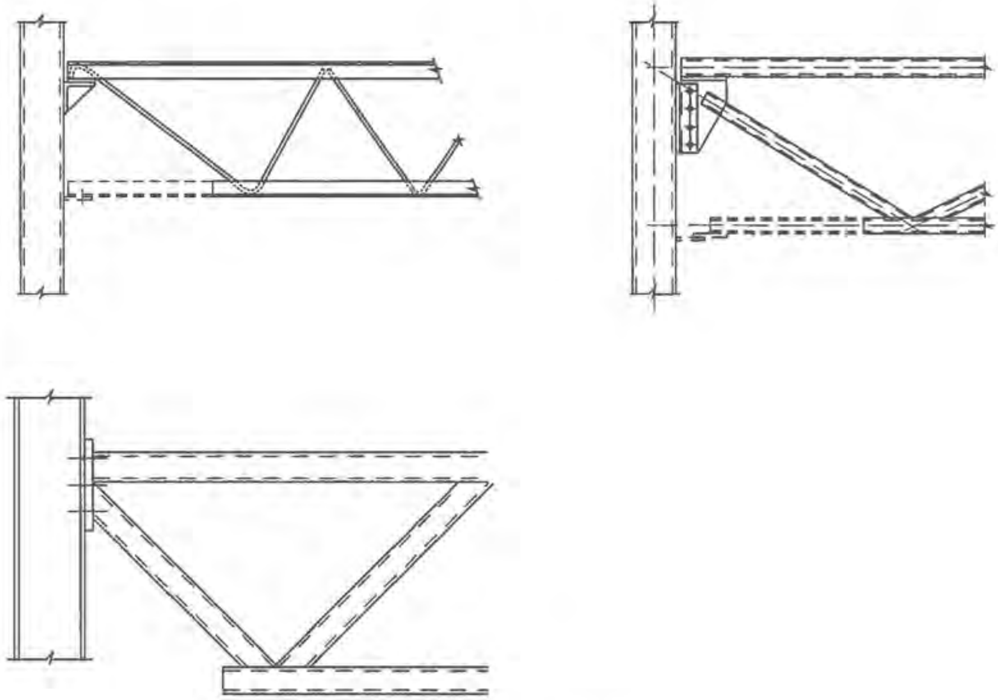
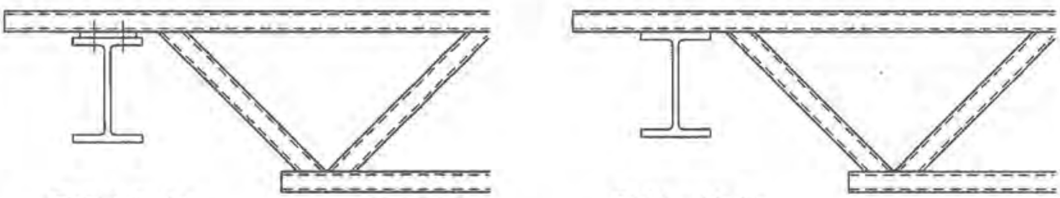


Figure 3-6

TRUSS TO COLUMN AND GIRDER CONNECTIONS

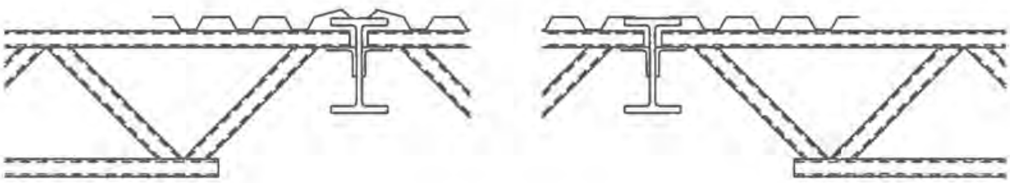


TRUSS TO COLUMN



Field-bolted to supporting member

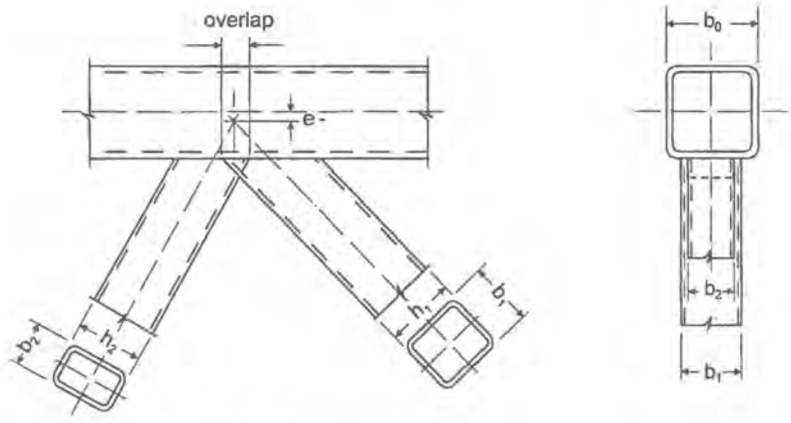
Field-welded to supporting member



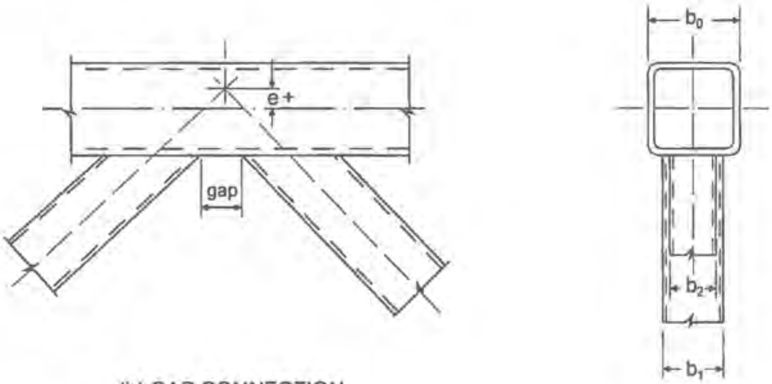
TRUSS TO GIRDER

Figure 3-7

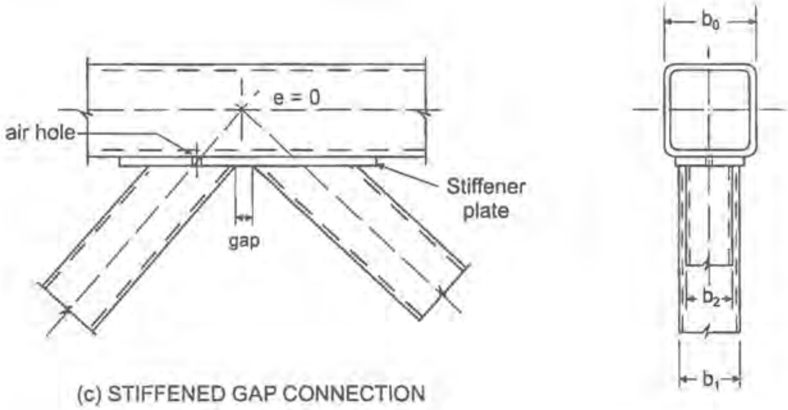
HSS TRUSS CONNECTIONS



(a) OVERLAP CONNECTION



(b) GAP CONNECTION



(c) STIFFENED GAP CONNECTION

Figure 3-8

CONNECTIONS FOR MOMENT AND SHEAR

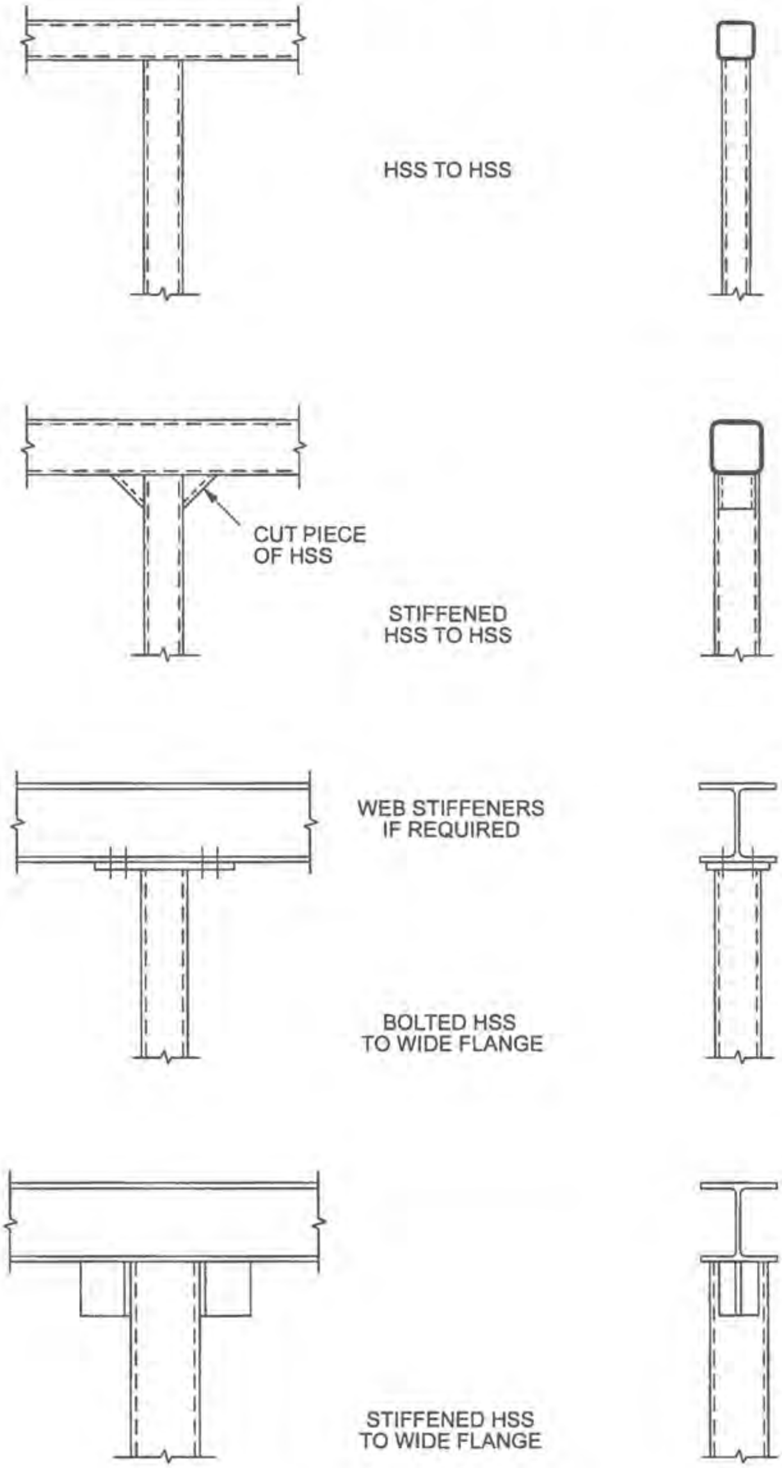


Figure 3-9

OTHER TRUSS CONFIGURATIONS

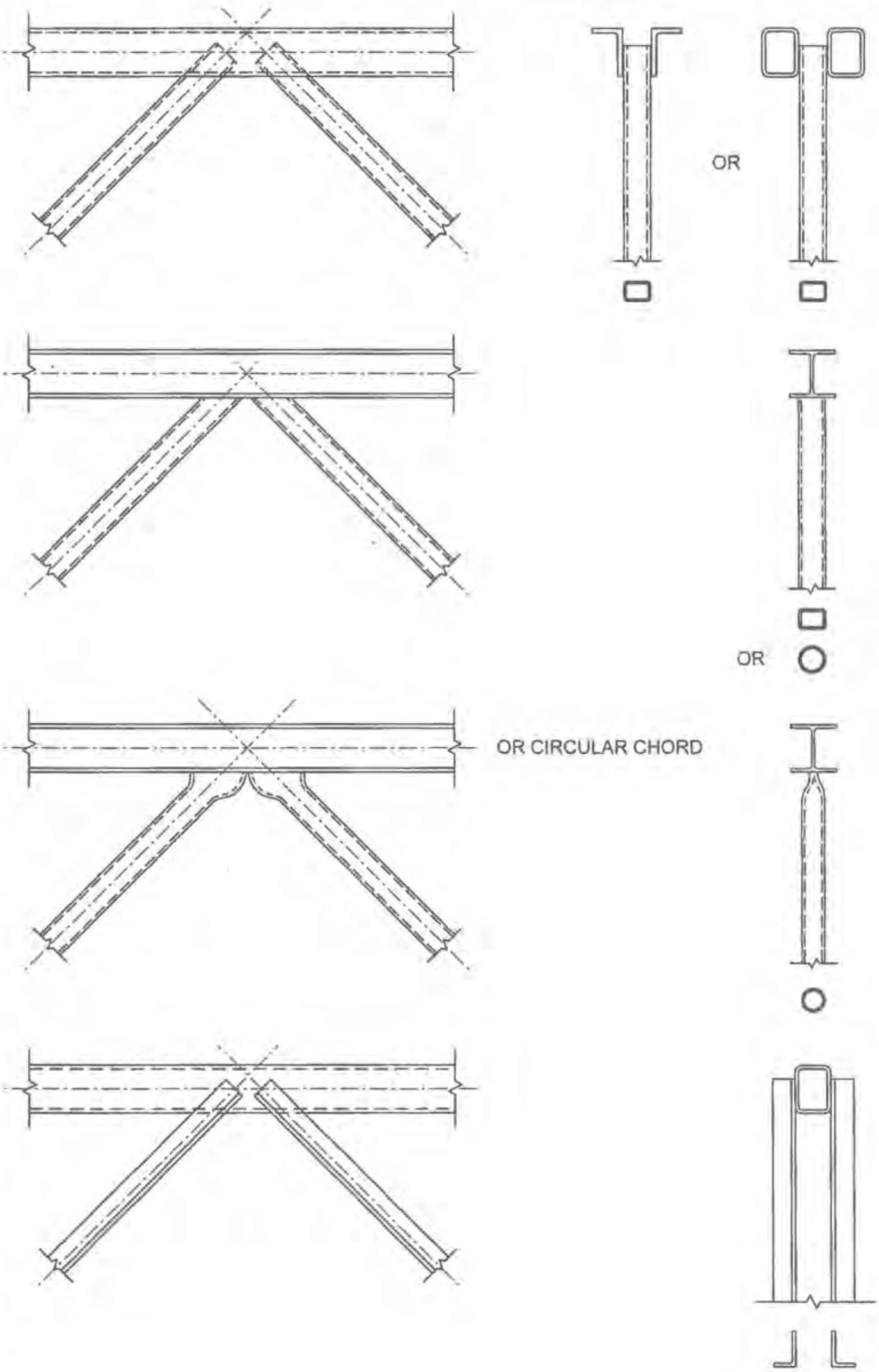
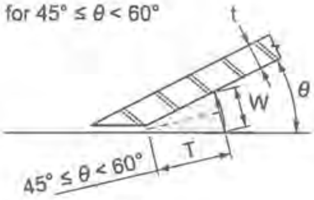


Figure 3-10

WELDING DETAILS FOR HOLLOW STRUCTURAL SECTIONS

Effective Throat:

$E = T - 3$ mm for $45° \le \theta < 60°$

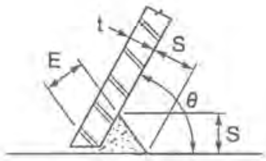


Detail A1, $45° \le \theta < 60°$

Detail A2, $30° \le \theta < 45°$: by procedure qualification

Effective Throat:

$$E = \frac{S/2}{\sin(\theta/2)}$$



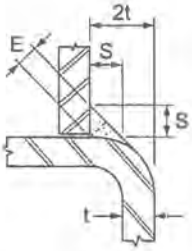
See CSA W59-13 Figure 4.8

Detail B

$60° \le \theta \le 90°$

Effective Throat:

$E = 0.707\,S$

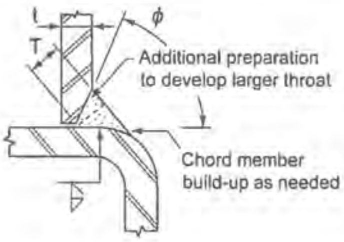


Detail C

$\theta = 90°$

Effective Throat:

$E = T$ (when $\phi \ge 60°$)

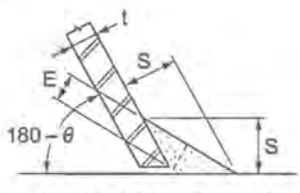


Detail D

$\theta = 90°$

Effective Throat:

$$E = \frac{S/2}{\sin(\theta/2)}$$



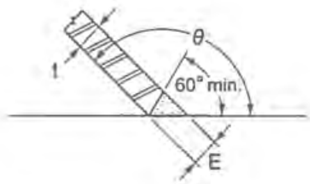
See CSA W59-13 Figure 4.8

Detail E

$90° \le \theta \le 135°$

Effective Throat:

$E = t$



Detail F

$\theta > 135°$

TOTAL LENGTH OF WELD IN MILLIMETRES — Table 3-46

HSS Web Members

HSS b x h x t (mm)	Angle θ Between Web and Chord Member									
	30°	35°	40°	45°	50°	55°	60°	65°	70°	90°
38 x 38 x 4.8	204	187	174	164	157	151	147	143	140	136
51 x 51 x 6.4	272	249	232	219	209	201	195	191	187	181
64 x 64 x 6.4	348	319	297	280	268	258	250	244	240	232
76 x 76 x 9.5	408	373	348	328	314	302	293	286	281	272
89 x 89 x 9.5	484	443	413	390	372	359	348	340	333	323
102 x 102 x 13	544	498	464	438	418	403	391	382	374	363
127 x 127 x 13	697	637	593	561	535	516	500	488	479	464
152 x 152 x 13	849	776	723	683	652	628	610	595	584	566
178 x 178 x 16	980	901	839	793	757	729	707	691	678	657
203 x 203 x 16	1 140	1 040	969	915	874	842	817	797	783	758
254 x 254 x 16	1 440	1 320	1 230	1 160	1 110	1 070	1 040	1 010	990	961
305 x 305 x 16	1 750	1 600	1 490	1 410	1 340	1 290	1 250	1 220	1 200	1 160
51 x 25 x 4.8	181	170	161	155	150	146	143	141	139	136
25 x 51 x 4.8	227	203	186	174	164	156	150	145	142	136
76 x 51 x 7.9	317	294	277	264	254	247	241	236	233	227
51 x 76 x 7.9	363	328	302	283	268	257	248	241	235	227
89 x 64 x 6.4	401	371	349	332	319	309	301	295	291	283
64 x 89 x 6.4	448	406	375	351	333	319	309	300	294	283
102 x 51 x 9.5	363	340	322	310	300	292	286	281	278	272
51 x 102 x 9.5	453	407	373	347	327	312	300	291	284	272
102 x 76 x 9.5	461	426	400	380	365	353	344	337	332	323
76 x 102 x 9.5	507	460	425	399	379	364	351	342	335	323
127 x 76 x 13	499	464	438	419	404	393	384	377	372	363
76 x 127 x 13	590	531	489	457	432	413	398	386	377	363
152 x 102 x 13	650	602	568	541	521	505	493	484	476	464
102 x 152 x 13	743	672	619	580	549	526	507	493	482	464
178 x 127 x 13	802	741	697	664	638	618	602	590	581	566
127 x 178 x 13	896	811	749	703	667	639	617	600	587	566
203 x 102 x 13	755	706	671	644	624	608	595	585	578	566
102 x 203 x 13	943	847	776	722	681	649	624	605	590	566
203 x 152 x 16	938	866	813	773	743	719	700	686	675	657
152 x 203 x 16	1 030	936	865	812	771	739	715	695	681	657
254 x 152 x 16	1 040	970	916	876	845	821	802	788	776	758
152 x 254 x 16	1 230	1 110	1 020	954	903	863	832	807	789	758
305 x 203 x 16	1 350	1 250	1 180	1 120	1 080	1 050	1 020	1 000	986	961
203 x 305 x 16	1 540	1 390	1 280	1 200	1 140	1 090	1 050	1 020	1 000	961

Notes:

1. Outside corner radius assumed equal to $2t$.
2. Perimeters calculated by: $K_a\,[4\pi t + 2(b - 4t) + 2(h - 4t)]$, where $K_a = [(h/\sin\theta) + b]/(h + b)]$
3. Weld lengths for the HSS with the thickest wall in each size group are tabulated; other sections in the group have slightly longer welds.

TENSION MEMBERS

General

Members subject to axial tension (i.e. when the resultant tensile load on the member is coincident with the longitudinal centroidal axis of the member) can be proportioned assuming a uniform stress distribution. The factored tensile resistance is calculated on the basis of yielding on the gross area and fracture on the net area (or effective net area reduced for shear lag) according to Clause 13.2 of CSA S16-14. Net area and effective net area reduced for shear lag are defined in Clause 12.3.

Net Area

Tables 3-47 and 3-49 to 3-51 are intended to simplify the calculation of net area according to the requirements of Clause 12.3.

Hole Diameters for Net Area

Table 3-47 lists the specified hole diameter for various bolt sizes according to Clause 22.3.5.2, and the diameter of holes for calculating net area according to Clause 12.3.2.

Staggered Holes in Tension Members

Table 3-49 lists values of $s^2/4g$ required to calculate the net width of any diagonal or zig-zag line of holes according to the requirements of Clause 12.3.1(b) for various pitches from 25 to 240 mm and for various gauges from 25 to 320 mm. Values of $s^2/4g$ for pitches and gauges between those listed can be interpolated.

Effective Net Area – Reduced for Shear Lag

Clause 12.3.3 of S16-14 contains provisions for determining the loss of efficiency due to shear lag when tension members are not connected by all their elements.

Shear Lag Values of $1-\bar{x}/L$

Table 3-50 lists values of $1-\bar{x}/L$ as a function of $\bar{x}$ and L for use with Clause 12.3.3.3(c) when computing the effective net area reduced for shear lag of section elements projecting from a welded connection.

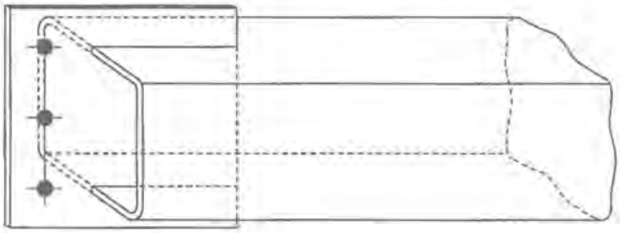
Shear Lag Values for Slotted HSS

Table 3-51 lists values of $1.1-\bar{x}'/L_w$ as a function of $\bar{x}'$ and L_w for use with Clause 12.3.3.4 when computing the effective net area reduced for shear lag of slotted HSS members welded to a plate.

Design Example - Shear Lag

Given:

An HSS152x152x6.4 of G40.21-350W material supports a factored load of 525 kN in tension and is connected by a single plate welded into slots in the HSS walls as shown. The plate will be bolted between a pair of splice plates. Design the plate using 300W steel.



Solution:

Try a 12×240 mm plate.

For G40.21-300W steel, $F_y = 300$ MPa, $F_u = 440$ MPa.

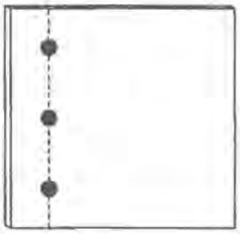
Tensile gross-area yield for the plate

$T_r = \phi\, A_g F_y$ CSA S16-14, Clause 13.2(a)(i)

$= 0.9 \times 12 \times 240 \times 300$

$= 778$ kN > 525 kN

Tensile net-area rupture (through the bolt line)



Try three ⅞-inch A325 bolts; use 45 mm edge and end distances.

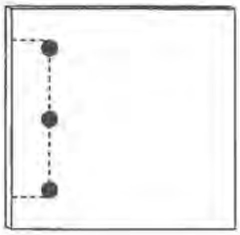
It is assumed that the bolt holes will be punched. The bolt hole size is taken as $22.2 + 2 + 2 \approx 26$ mm for net area calculations (Clause 12.3.2). See also Table 3-47.

$A_{ne} = [240 - (3 \times 26)]\, 12 = 1940 \text{ mm}^2$

$T_r = \phi_u A_{ne} F_u = 0.75 \times 1940 \times 440$ Clause 13.2(a)(iii)

$= 640$ kN > 525 kN

Block shear failure



Bolt gauge = $(240 - 2 \times 45) / 2 = 75$ mm

Net area in tension:

$$A_n = (2 \times 75 - 2 \times 26)\, 12 = 1180 \text{ mm}^2$$

Gross area in shear:

$$A_{gv} = 2 \times 45 \times 12 = 1080 \text{ mm}^2$$

$$T_r = \phi_u [U_t A_n F_u + 0.60 A_{gv} (F_y + F_u) / 2] \qquad \text{Clauses 13.2(a)(ii), 13.11}$$

$$= 0.75\,[1.0 \times 1180 \times 440 + 0.60 \times 1080\,(300 + 440) / 2]$$

$$= 569 \text{ kN} > 525 \text{ kN}$$

Bolt shear

Bolt: $V_r = 108$ kN (Table 3-4, threads intercepted)

Connection bolts (double shear):

$$V_r = 3 \times 2 \times 108$$

$$= 648 \text{ kN} > 525 \text{ kN}$$

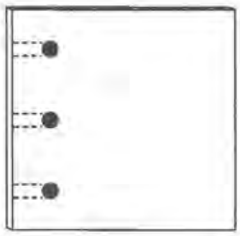
Bearing resistance at bolt holes

$$B_r = 3\,\phi_{br}\, n\, t\, d\, F_u \qquad \text{Clause 13.12.1.2(a)}$$

$$= 3 \times 0.80 \times 3 \times 12 \times 22.2 \times 440$$

$$= 844 \text{ kN} > 525 \text{ kN}$$

Plate shear failure (by bolts pulling out the end of the plate)



End distance (from centre of bolt hole): $e = 45$ mm

According to Clause 22.3.4, the minimum end distance from the centre of the bolt to the end of the member is 1.5 bolt diameters. The failure mode involves two parallel planes adjacent to each bolt hole, as shown on the previous figure.

Gross shear area:

$$A_{gv} = 3\,(2 \times 45)\,12 = 3240 \text{ mm}^2$$

$$T_r = \phi_u [0.60\, A_{gv} (F_y + F_u)/2] \qquad \text{Clause 13.11}$$

$$= 0.75\,[0.60 \times 3240\,(300 + 440)/2]$$

$$= 539 \text{ kN} > 525 \text{ kN}$$

Shear lag for the plate

Clause 12.3.3.3(b) of S16-14 provides for shear lag in plates that are connected by a pair of longitudinal welds along two edges parallel to the load. The effective net area is reduced if the length of the welds is less than $2w$.

Distance between welds, $w = 152$ mm

Try a weld length, $L_w = 150 \text{ mm} \approx w$

Plate area between the welds, $L_w < w$:

$$A_{n2} = 0.75\, L_w t \qquad \text{Clause 12.3.3.3(b)(iii)}$$

$$= 0.75 \times 150 \times 12 = 1350 \text{ mm}^2$$

Plate area on either side of the welds, connected by a single longitudinal weld:

$w = (240 - 152)/2 = 44$ mm. Therefore, $L_w > w$.

$$A_{n3} = \left(1 - \frac{\bar{x}}{L_w}\right) wt = 2\left(1 - \frac{44/2}{150}\right) 44 \times 12 = 901 \text{ mm}^2 \qquad \text{Clause 12.3.3.3(c)(i)}$$

Total effective net area:

$$A_{ne} = A_{n2} + A_{n3} = 1350 + 901 = 2250 \text{ mm}^2$$

Tensile resistance of the plate

$$T_r = \phi_u A_{ne} F_u \qquad \text{Clause 13.2(a)(iii)}$$

$$= 0.75 \times 2250 \times 440$$

$$= 743 \text{ kN} > 525 \text{ kN}$$

Welds

Factored unit weld resistance:

$$0.67\, \phi_w\, 0.707\, X_u = 0.67 \times 0.67 \times 0.707 \times 490 = 0.156 \text{ kN/mm}^2$$

For a weld length, $L_w = 150$ mm, the fillet weld size is:

$$525 \text{ kN} / (4 \times 150 \times 0.156) = 5.61 \text{ mm}$$

Use 6 mm

Tensile resistance of the HSS (gross-area yield)

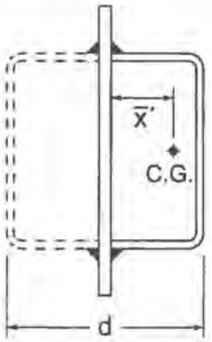
Cross-sectional area of the HSS152x152x6.4, $A_g = 3610 \text{ mm}^2$

$T_r = \phi A_g F_y$ Clause 13.2(a)(i)

$= 0.9 \times 3610 \times 350 = 1140 \text{ kN} > 525 \text{ kN}$

Shear lag for the HSS

Calculate $\bar{x}'$, the distance between the centre of gravity of half of the HSS cross-section and the edge of the connection plate:



It can be shown that, for a square HSS:

$$\bar{x}' \approx \left(\frac{3}{8}\right) d = \left(\frac{3}{8}\right) 152 = 57 \text{ mm}$$

Effective net area of the entire HSS section:

$$\frac{\bar{x}'}{L_w} = \frac{57}{150} = 0.380 > 0.1$$ Clause 12.3.3.4

Net area of the HSS, taking into account the slots and a 3 mm fit-up gap:

$$A_n = 3610 - 2(12+3)6.35 = 3420 \text{ mm}^2$$

$$A_{ne} = A_n\left(1.1 - \frac{\bar{x}'}{L_w}\right) = 3420\,(1.1 - 0.380) = 2460 \text{ mm}^2$$

$A_{ne} < 0.8 A_n = 0.8 \times 3420 = 2740 \text{ mm}^2$. Therefore, $A_{ne} = 2740 \text{ mm}^2$

$T_r = \phi_u A_{ne} F_u$ Clause 13.2(a)(iii)

$= 0.75 \times 2740 \times 450$

$= 925 \text{ kN} > 525 \text{ kN}$

Block shear around fillet welds

Failure modes involving block shear in the plate and HSS walls around the fillet welds do not govern for this example (calculations not shown).

Therefore, a PL12×240×250 mm long, slotted 150 mm into the HSS, is adequate.

HOLE DIAMETERS — Table 3-47

Specified and for Net Area

Bolt Size	Standard Hole Diameters, mm			Oversize Hole Diameters, mm		
	Specified#	Net Area Calculation		Specified	Net Area Calculation	
in.		Drilled Holes†	Other Than Drilled		Drilled Holes†	Other Than Drilled
½	14	14	16	–	–	–
⅝	17	17	19	20	20	22
¾	21	21	23	23	23	25
	or 22 *	22	24			
⅞	24	24	26	27 **	27	29
1	27	27	29	32 **	32	34
1⅛	30	30	32	37	37	39
1¼	33	33	35	40	40	42
1½	40	40	42	46	46	48

Notes:

For slotted hole dimensions, see Table 3-52.

All figures have been rounded to nearest millimetre, U.N.O.

\# Rounded down to nearest millimetre.

* Also permitted. See S16-14 Clause 22.3.5.1.

† Net area calculation may be based on the specified hole diameter if the holes are drilled (Clause 12.3.2).

** Undefined in S16; value adopted from RCSC Specification for Structural Joints Using High-Strength Bolts, 2014.

TENSION-CONTROL BOLT ASSEMBLIES AND INDICATORS — Table 3-48

Assembly/Indicator	ASTM Standard [1]	Remarks
ASTM F1852 [(2)]	ASTM F1852-11 Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength	F_u = 725 / 825 MPa
ASTM F2280 [(2)]	ASTM F2280-12 Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 150 ksi Minimum Tensile Strength	F_u = 1035 MPa
ASTM F959	ASTM F959-13 Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners	Types 325 and 490 available

Note: For slip-critical connections, see CSA S16-14 Table 3.

[1] As referenced by CSA S16-14

[2] Maximum bolt diameter for F1852 and F2280 in ASTM F3125-15 is 1¼ in. Prior to F3125-15, the limit was 1⅛ in.

STAGGERED HOLES IN TENSION MEMBERS — Table 3-49

Values of $s^2/4g$

Pitch "s" (mm)	Gauge "g" (mm)															
	25	30	35	40	45	50	60	70	80	100	120	160	200	240	280	320
25					3.5	3.1	2.6	2.2	2.0	1.6	1.3	1.0	0.8	0.7	0.6	0.5
30				5.6	5.0	4.5	3.8	3.2	2.8	2.3	1.9	1.4	1.1	0.9	0.8	0.7
35			8.8	7.7	6.8	6.1	5.1	4.4	3.8	3.1	2.6	1.9	1.5	1.3	1.1	1.0
40		13.3	11.4	10.0	8.9	8.0	6.7	5.7	5.0	4.0	3.3	2.5	2.0	1.7	1.4	1.3
45	20.3	16.9	14.5	12.7	11.3	10.1	8.4	7.2	6.3	5.1	4.2	3.2	2.5	2.1	1.8	1.6
50	25.0	20.8	17.9	15.6	13.9	12.5	10.4	8.9	7.8	6.3	5.2	3.9	3.1	2.6	2.2	2.0
55	30.3	25.2	21.6	18.9	16.8	15.1	12.6	10.8	9.5	7.6	6.3	4.7	3.8	3.2	2.7	2.4
60	36.0	30.0	25.7	22.5	20.0	18.0	15.0	12.9	11.3	9.0	7.5	5.6	4.5	3.8	3.2	2.8
65	42.3	35.2	30.2	26.4	23.5	21.1	17.6	15.1	13.2	10.6	8.8	6.6	5.3	4.4	3.8	3.3
70	49.0	40.8	35.0	30.6	27.2	24.5	20.4	17.5	15.3	12.3	10.2	7.7	6.1	5.1	4.4	3.8
75		46.9	40.2	35.2	31.3	28.1	23.4	20.1	17.6	14.1	11.7	8.8	7.0	5.9	5.0	4.4
80			45.7	40.0	35.6	32.0	26.7	22.9	20.0	16.0	13.3	10.0	8.0	6.7	5.7	5.0
90				50.6	45.0	40.5	33.8	28.9	25.3	20.3	16.9	12.7	10.1	8.4	7.2	6.3
100						50.0	41.7	35.7	31.3	25.0	20.8	15.6	12.5	10.4	8.9	7.8
110							50.4	43.2	37.8	30.3	25.2	18.9	15.1	12.6	10.8	9.5
120									45.0	36.0	30.0	22.5	18.0	15.0	12.9	11.3
130										42.3	35.2	26.4	21.1	17.6	15.1	13.2
140										49.0	40.8	30.6	24.5	20.4	17.5	15.3
150											46.9	35.2	28.1	23.4	20.1	17.6
160												40.0	32.0	26.7	22.9	20.0
170												45.2	36.1	30.1	25.8	22.6
180												50.6	40.5	33.8	28.9	25.3
190													45.1	37.6	32.2	28.2
200													50.0	41.7	35.7	31.3
210														45.9	39.4	34.5
220														50.4	43.2	37.8
230															47.2	41.3
240																45.0

SHEAR LAG
Values of $1-\bar{x}/L$

Table 3-50

L (mm)	$1-\bar{x}/L$															
	Distance $\bar{x}$ (mm)															
	10	15	20	25	30	35	40	45	50	55	60	65	70	80	90	100
40	0.75	0.63	0.50	0.38	0.25	0.13										
80	0.88	0.81	0.75	0.69	0.63	0.56	0.50	0.44	0.38	0.31	0.25	0.19	0.13			
120	0.92	0.88	0.83	0.79	0.75	0.71	0.67	0.63	0.58	0.54	0.50	0.46	0.42	0.33	0.25	0.17
160	0.94	0.91	0.88	0.84	0.81	0.78	0.75	0.72	0.69	0.66	0.63	0.59	0.56	0.50	0.44	0.38
200	0.95	0.93	0.90	0.88	0.85	0.83	0.80	0.78	0.75	0.73	0.70	0.68	0.65	0.60	0.55	0.50
240	0.96	0.94	0.92	0.90	0.88	0.85	0.83	0.81	0.79	0.77	0.75	0.73	0.71	0.67	0.63	0.58
280	0.96	0.95	0.93	0.91	0.89	0.88	0.86	0.84	0.82	0.80	0.79	0.77	0.75	0.71	0.68	0.64
320	0.97	0.95	0.94	0.92	0.91	0.89	0.88	0.86	0.84	0.83	0.81	0.80	0.78	0.75	0.72	0.69
360	0.97	0.96	0.94	0.93	0.92	0.90	0.89	0.88	0.86	0.85	0.83	0.82	0.81	0.78	0.75	0.72
400	0.98	0.96	0.95	0.94	0.93	0.91	0.90	0.89	0.88	0.86	0.85	0.84	0.83	0.80	0.78	0.75
440	0.98	0.97	0.95	0.94	0.93	0.92	0.91	0.90	0.89	0.88	0.86	0.85	0.84	0.82	0.80	0.77
460	0.98	0.97	0.96	0.95	0.94	0.93	0.92	0.91	0.90	0.89	0.88	0.86	0.85	0.83	0.81	0.79

See CSA S16-14 Clause 12.3.3.3(c)

SHEAR LAG
Values of $1.1-\bar{x}'/L_w$

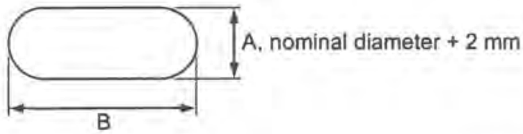
Table 3-51

L_w (mm)	$1.1-\bar{x}'/L_w$															
	Distance $\bar{x}'$ (mm)															
	10	15	20	25	30	35	40	45	50	55	60	70	80	90	100	120
40	0.85															
60	0.93	0.85														
80	0.98	0.91	0.85													
100		0.95	0.90	0.85	0.80											
120		0.98	0.93	0.89	0.85	0.81										
140		0.99	0.96	0.92	0.89	0.85	0.81									
160			0.98	0.94	0.91	0.88	0.85	0.82								
180			0.99	0.96	0.93	0.91	0.88	0.85	0.82							
200				0.98	0.95	0.93	0.90	0.88	0.85	0.83	0.80					
220				0.99	0.96	0.94	0.92	0.90	0.87	0.85	0.83					
240					0.98	0.95	0.93	0.91	0.89	0.87	0.85	0.81				
260					0.98	0.97	0.95	0.93	0.91	0.89	0.87	0.83				
280					0.99	0.98	0.96	0.94	0.92	0.90	0.89	0.85	0.81			
300						0.98	0.97	0.95	0.93	0.92	0.90	0.87	0.83	0.80		
320						0.99	0.98	0.96	0.94	0.93	0.91	0.88	0.85	0.82		
340							0.98	0.97	0.95	0.94	0.92	0.89	0.86	0.84	0.81	
360							0.99	0.98	0.96	0.95	0.93	0.91	0.88	0.85	0.82	
380							0.99	0.98	0.97	0.96	0.94	0.92	0.89	0.86	0.84	
400								0.99	0.98	0.96	0.95	0.93	0.90	0.88	0.85	0.80
420								0.99	0.98	0.97	0.96	0.93	0.91	0.89	0.86	0.81
440									0.99	0.98	0.96	0.94	0.92	0.90	0.87	0.83
460									0.99	0.98	0.97	0.95	0.93	0.90	0.88	0.84
480										0.99	0.98	0.95	0.93	0.91	0.89	0.85
500										0.99	0.98	0.96	0.94	0.92	0.90	0.86

See CSA S16-14 Clause 12.3.3.4

SLOTTED HOLE DIMENSIONS

Table 3-52



SHORT SLOT DIMENSIONS

Nominal Bolt Diameter	Slot Dimensions *	
	Width, A	Length, B
in.	mm	mm
5/8	18	A < B ≤ 22
3/4	21	22 < B ≤ 25
7/8	24	A < B ≤ 29 **
1	27	A < B ≤ 33 **
1 1/8	31	A < B ≤ 39
1 1/4	34	A < B ≤ 42
1 1/2	40	A < B ≤ 48

LONG SLOT DIMENSIONS

Nominal Bolt Diameter	Slot Dimensions *	
	Width, A	Length, B
in.	mm	mm
5/8	18	22 < B ≤ 40
3/4	21	25 < B ≤ 48
7/8	24	29 < B ≤ 56
1	27	33 < B ≤ 64
1 1/8	31	39 < B ≤ 71
1 1/4	34	42 < B ≤ 79
1 1/2	40	48 < B ≤ 95

See S16-14 Clause 22.3.5.2 for further information.

* Dimensions have been rounded to the nearest millimetre.

** Undefined in S16; value adopted from RCSC Specifications for Structural Joints Using High-Strength Bolts, 2014.

PART FOUR
COMPRESSION MEMBERS

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Unit Factored Compressive Resistances for Compression Members 4-6

Table 4-3 – Unit Factored Compressive Resistances, n = 1.34 4-7

Table 4-4 – Unit Factored Compressive Resistances, n = 2.24 4-11

Euler Buckling Load per Unit of Area 4-12

Table 4-5 – C_e / A 4-12

Factored Axial Compressive Resistances of Columns 4-13

W-Shapes

ASTM A992, A572 Grade 50 4-17

ASTM A913 Grade 65 4-28

Hollow Structural Sections – CSA G40.21-350W - Class C

Square Hollow Sections 4-32

Rectangular Hollow Sections 4-41

Round Hollow Sections 4-50

Hollow Structural Sections – CSA G40.21-350W - Class H

Square Hollow Sections 4-56

Rectangular Hollow Sections 4-65

Round Hollow Sections 4-74

Hollow Structural Sections – ASTM A500 Grade C

Square Hollow Sections 4-80

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Beam-Columns 4-101

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GENERAL INFORMATION

Width-to-Thickness Ratios

Limits on width-to-thickness ratios for various steel grades are listed in Table 4-1 on page 4-4. Width-to-thickness ratios for elements in axial compression are given in Table 4-2 on page 4-5.

Unit Factored Compressive Resistances for Compression Members

Tables 4-3 and 4-4, on pages 4-7 to 4-11, provide tables of unit factored compressive resistances, C_r/A, for slenderness ratios from 1 to 200 for various yield stresses of steel and values of n of 1.34 and 2.24. See page 4-6 for more information.

Euler Buckling Load per Unit of Area

Table 4-5, page 4-12, lists values of C_e/A for KL/r ratios varying from 1 to 200.

Factored Axial Compressive Resistances of Columns

These are the tables often referred to as "column load tables". See page 4-13 for a description of the contents and examples of use.

Beam-Columns

Width-to-thickness ratios for elements in flexural compression are given in Table 4-6 on page 4-102, and the Class of sections in combined axial compression and major-axis bending is given in Table 4-7 on page 4-103. Table 4-8, page 4-106, lists values of ω_1 for various ratios of factored end bending moments. For a general description of the design tables for beam-columns, see page 4-101.

Values of the amplification factor, U, corresponding to various values of C_f/C_e are listed in Table 4-9, page 4-107. For sections not listed in the *Beam Selection Table* in Part 5, factored moment resistances for various unbraced lengths can be found on page 4-108. Illustrative examples are given on page 4-111.

Factored Axial Compressive Resistances — Angle Struts

See page 4-115 for a description of the contents and design examples for single-angle and double-angle struts.

Column Base Plates

See page 4-153 for information and design examples for column base plates.

Anchor Rods

See page 4-158 for data on anchor rods, hole sizes, washers, and mechanical properties.

Bracing Assemblies

A design example is given on page 4-160.

LIMITS ON WIDTH-TO-THICKNESS RATIOS

Table 4-1 below lists the particular width-to-thickness (b_{el}/t, h/w or D/t) ratio limits for various material yield strengths, for each general value given in Tables 4-2 and 4-6.

Table 4-2, which is taken from Clause 11 of CSA S16-14, lists the width-to-thickness ratios for Class 1, 2 and 3 sections for various elements in axial compression. All sections not meeting these requirements are Class 4.

WIDTH-TO-THICKNESS LIMITS — Table 4-1

General Value	F_y (MPa)													
	250	260	280	300	317	320	**345**	**350**	380	400	450	480	485	550
$145/\sqrt{F_y}$	9.17	8.99	8.67	8.37		8.11	**7.81**	**7.75**	7.44	7.25	6.84	6.62	6.58	6.18
$170/\sqrt{F_y}$	10.8	10.5	10.2	9.81		9.50	**9.15**	**9.09**	8.72	8.50	8.01	7.76	7.72	7.25
$200/\sqrt{F_y}$	12.6	12.4	12.0	11.5		11.2	**10.8**	**10.7**	10.3	10.0	9.43	9.13	9.08	8.53
$250/\sqrt{F_y}$	15.8			14.4			13.5	**13.4**	12.8	12.5	11.8	11.4	11.4	
$340/\sqrt{F_y}$	21.5	21.1	20.3	19.6		19.0	**18.3**	18.2	17.4	17.0	16.0	15.5	15.4	14.5
$420/\sqrt{F_y}$	26.6	26.0	25.1	24.2		23.5	**22.6**	**22.4**	21.5	21.0	19.8	19.2	19.1	17.9
$525/\sqrt{F_y}$	33.2	32.6	31.4	30.3		29.3	**28.3**	**28.1**	26.9	26.3	24.7	24.0	23.8	22.4
$670/\sqrt{F_y}$ *	42.4	41.6	40.0	38.7		37.5	**36.1**	**35.8**	34.4	33.5	31.6	30.6	30.4	28.6
$840/\sqrt{F_y}$	53.1	52.1	50.2	48.5		47.0	45.2	44.9	43.1	42.0	39.6	38.3	38.1	35.8
$1100/\sqrt{F_y}$	69.6	68.2	65.7	63.5		61.5	**59.2**	**58.8**	56.4	55.0	51.9	50.2	49.9	46.9
$1700/\sqrt{F_y}$	108	105	102	98.1		95.0	**91.5**	**90.9**	87.2	85.0	80.1	77.6	77.2	72.5
$1900/\sqrt{F_y}$	120	118	114	110		106	**102**	102	97.5	95.0	89.6	86.7	86.3	81.0
$13\,000/F_y$				43.3	**41.0**		37.7	**37.1**	34.2	32.5	28.9	27.1		23.6
$18\,000/F_y$				60.0	**56.8**		52.2	**51.4**	47.4	45.0	40.0	37.5		32.7
$23\,000/F_y$				76.7	**72.6**		66.7	**65.7**	60.5	57.5	51.1	47.9		41.8
$66\,000/F_y$				220	**208**		191	**189**	174	165	147	138		120

* h/w limit for webs in pure compression, $C_f/(\phi C_y) = 1.0$

WIDTH-TO-THICKNESS RATIOS

Table 4-2

Elements in Axial Compression

Description of Element	Maximum Width-to-Thickness Ratios
Flanges of I-sections, T-sections and channels; plate-girder stiffeners	$\frac{b_{el}}{t} \leq \frac{200}{\sqrt{F_y}}$
Legs of angles	$\frac{b_{el}}{t} \leq \frac{250}{\sqrt{F_y}}$
Stems of T-sections	$\frac{b_{el}}{t} \leq \frac{340}{\sqrt{F_y}}$
Flanges of rectangular hollow sections; flange cover plates and diaphragm plates between lines of fasteners or welds; web of I-shape sections; web supported on both edges	$\frac{b_{el}}{t} \leq \frac{670}{\sqrt{F_y}}$
Perforated cover plates	$\frac{b_{el}}{t} \leq \frac{840}{\sqrt{F_y}}$
Circular hollow sections	$\frac{D}{t} \leq \frac{23\,000}{F_y}$

See CSA S16-14 Clause 11.

UNIT FACTORED COMPRESSIVE RESISTANCES FOR COMPRESSION MEMBERS, C_r/A

General

Table 4-3 on the following pages lists the unit factored compressive resistance, C_r/A (in MPa), calculated in accordance with the requirements of CSA S16-14 Clause 13.3.1 for members with F_y varying from 250 to 700 MPa, for values of KL/r from 1 to 200 with $n = 1.34$. The tabulated resistances apply to hot-rolled, fabricated structural sections and hollow structural sections manufactured according to CSA G40.20, Class C (cold-formed non-stress-relieved), ASTM A500 and A1085.

Table 4-4 lists the unit factored compressive resistance, C_r/A, for compression members consisting of doubly-symmetric welded three-plate members with oxy-flame-cut flange plates and $F_y = 350$ MPa, and HSS manufactured according to G40.20 Class H (hot-formed or cold-formed stress-relieved) with $F_y = 350$ MPa. Resistances have been calculated for values of KL/r from 1 to 200, in accordance with the requirements of Clause 13.3.1 with $n = 2.24$. Table 4-4 may also be used for A1085 HSS specified as Supplement S1, provided an adjustment is made for the small difference in F_y values (345 MPa vs. 350 MPa).

Use

To obtain the factored compressive resistance C_r for doubly symmetric Class 1, 2 or 3 sections, multiply the unit factored compressive resistance C_r/A for the appropriate F_y and KL/r ratio, by the cross-sectional area A of the column section. For Class 4 sections, see Clause 13.3.5.

Examples

1. Given:

   Find the factored compressive resistance of a W250x131 column of ASTM A992 steel ($F_y = 345$ MPa) for a KL/r ratio of 89.

   Solution:

   From the tables of properties and dimensions in Part 6, for W250x131, $A = 16\,700$ mm^2
   From Table 4-3, with $KL/r = 89$ and $F_y = 345$ MPa, $C_r/A = 155$ MPa
   Therefore, $C_r = 155 \text{ MPa} \times 16\,700 \text{ mm}^2 = 2\,590 \text{ kN}$

2. Given:

   Find the factored compressive resistance of an HSS 254x152x13 Class H column of CSA G40.21 Grade 350W ($F_y = 350$ MPa) and $KL/r = 89$.

   Solution:

   From the tables of properties and dimensions in Part 6 for HSS 254x152x13,
   $A = 9\,260$ mm^2
   From Table 4-4, with $KL/r = 89$ and $F_y = 350$ MPa, $C_r/A = 189$ MPa
   Therefore, $C_r = 189 \text{ MPa} \times 9\,260 \text{ mm}^2 = 1\,750 \text{ kN}$

Note:

For heavy and built-up sections, see Clause 13.3 of the CISC Commentary in Part 2 of this Handbook for more information on compressive resistance.

UNIT FACTORED COMPRESSIVE RESISTANCES, C_r/A (MPa)

Table 4-3

Compression members for which n = 1.34 applies

$\phi = 0.90$

$\frac{KL}{r} = 1$ to 50

$\frac{KL}{r}$	F_y (MPa)													
	250	260	280	300	317	**345**	**350**	380	400	450	480	485	550	700
1	225	234	252	270	285	**310**	**315**	342	360	405	432	436	495	630
2	225	234	252	270	285	**310**	**315**	342	360	405	432	436	495	630
3	225	234	252	270	285	**310**	**315**	342	360	405	432	436	495	630
4	225	234	252	270	285	**310**	**315**	342	360	405	432	436	495	630
5	225	234	252	270	285	**310**	**315**	342	360	405	432	436	495	629
6	225	234	252	270	285	**310**	**315**	342	360	405	431	436	494	629
7	225	234	252	270	285	**310**	**315**	342	359	404	431	436	494	628
8	225	234	252	270	285	**310**	**314**	341	359	404	431	435	493	627
9	225	234	252	269	285	**310**	**314**	341	359	404	430	435	493	626
10	225	233	251	269	284	**309**	**314**	341	359	403	430	434	492	625
11	224	233	251	269	284	**309**	**314**	340	358	403	429	434	491	623
12	224	233	251	269	284	**309**	**313**	340	358	402	428	433	490	621
13	224	233	251	269	284	**308**	**313**	339	357	401	428	432	489	619
14	224	233	250	268	283	**308**	**312**	339	356	400	427	431	488	617
15	224	232	250	268	283	**308**	**312**	338	356	399	426	430	486	615
16	223	232	250	267	282	**307**	**311**	338	355	398	424	429	485	612
17	223	232	249	267	282	**306**	**311**	337	354	397	423	427	483	609
18	223	231	249	266	281	**306**	**310**	336	353	396	422	426	481	605
19	222	231	249	266	281	**305**	**309**	335	352	395	420	424	479	602
20	222	231	248	265	280	**304**	**308**	334	351	393	418	422	476	598
21	222	230	248	265	279	**303**	**307**	333	350	392	416	421	474	594
22	221	230	247	264	279	**302**	**307**	332	348	390	415	419	471	589
23	221	229	246	263	278	**301**	**305**	331	347	388	412	416	468	584
24	220	229	246	263	277	**300**	**304**	329	346	386	410	414	465	579
25	220	228	245	262	276	**299**	**303**	328	344	384	408	412	462	574
26	219	227	244	261	275	**298**	**302**	326	342	382	405	409	459	569
27	218	227	243	260	274	**297**	**301**	325	341	380	403	407	455	563
28	218	226	243	259	273	**295**	**299**	323	339	377	400	404	452	557
29	217	225	242	258	272	**294**	**298**	321	337	375	397	401	448	551
30	216	224	241	257	270	**292**	**296**	320	335	372	394	398	444	544
31	216	224	240	256	269	**291**	**295**	318	333	370	391	395	440	538
32	215	223	239	254	268	**289**	**293**	316	330	367	388	391	436	531
33	214	222	238	253	266	**288**	**291**	314	328	364	385	388	431	524
34	213	221	237	252	265	**286**	**290**	311	326	361	381	385	427	517
35	212	220	235	251	263	**284**	**288**	309	323	358	378	381	422	510
36	211	219	234	249	262	**282**	**286**	307	321	355	374	377	418	503
37	210	218	233	248	260	**280**	**284**	305	318	351	370	374	413	495
38	209	217	232	246	259	**278**	**282**	302	316	348	367	370	408	488
39	208	216	230	245	257	**276**	**280**	300	313	345	363	366	403	481
40	207	214	229	243	255	**274**	**278**	297	310	341	359	362	398	473
41	206	213	228	242	253	**272**	**275**	295	307	338	355	358	393	466
42	205	212	226	240	251	**270**	**273**	292	304	334	351	354	388	458
43	204	211	225	238	250	**268**	**271**	289	301	330	347	349	383	450
44	202	209	223	237	248	**265**	**269**	287	298	327	343	345	378	443
45	201	208	222	235	246	**263**	**266**	284	295	323	338	341	372	435
46	200	207	220	233	244	**261**	**264**	281	292	319	334	337	367	428
47	199	205	218	231	242	**258**	**261**	278	289	315	330	332	362	420
48	197	204	217	229	240	**256**	**259**	275	286	311	326	328	357	413
49	196	202	215	227	237	**253**	**256**	273	283	308	321	324	351	405
50	195	201	213	225	235	**251**	**254**	270	280	304	317	319	346	398

Note: Values of C_r/A were calculated in accordance with S16-14 Clause 13.3.1 using n = 1.34, and apply to hot-rolled, fabricated structural sections and HSS produced to G40.20 Class C, ASTM A500 and A1085. For Class H Hollow Structural Sections, see Table 4-4.

UNIT FACTORED COMPRESSIVE RESISTANCES, C_r/A (MPa) — Table 4-3

Compression members for which n = 1.34 applies — $\frac{KL}{r}$ = 51 to 100

ϕ = 0.90

KL/r	Fy (MPa)													
	250	260	280	300	317	**345**	**350**	380	400	450	480	485	550	700
51	193	200	212	223	233	**249**	**251**	267	277	300	313	315	341	391
52	192	198	210	222	231	**246**	**249**	264	273	296	308	311	335	383
53	190	196	208	220	229	**243**	**246**	261	270	292	304	306	330	376
54	189	195	206	218	227	**241**	**243**	258	267	288	300	302	325	369
55	188	193	205	215	224	**238**	**241**	255	264	284	296	297	320	362
56	186	192	203	213	222	**236**	**238**	252	260	280	291	293	315	355
57	185	190	201	211	220	**233**	**235**	249	257	276	287	289	310	349
58	183	189	199	209	218	**230**	**233**	246	254	272	283	284	305	342
59	181	187	197	207	215	**228**	**230**	243	250	269	279	280	300	336
60	180	185	195	205	213	**225**	**227**	240	247	265	274	276	295	329
61	178	184	193	203	211	**223**	**225**	236	244	261	270	272	290	323
62	177	182	192	201	208	**220**	**222**	233	241	257	266	268	285	317
63	175	180	190	199	206	**217**	**219**	230	237	253	262	263	280	310
64	174	178	188	197	204	**215**	**217**	227	234	250	258	259	275	305
65	172	177	186	194	201	**212**	**214**	224	231	246	254	255	271	299
66	170	175	184	192	199	**210**	**211**	221	228	242	250	251	266	293
67	169	173	182	190	197	**207**	**209**	219	225	239	246	247	262	287
68	167	172	180	188	194	**204**	**206**	216	222	235	242	243	257	282
69	166	170	178	186	192	**202**	**203**	213	218	231	238	240	253	276
70	164	168	176	184	190	**199**	**201**	210	215	228	235	236	249	271
71	162	167	174	182	188	**197**	**198**	207	212	224	231	232	244	266
72	161	165	172	180	185	**194**	**196**	204	209	221	227	228	240	261
73	159	163	171	178	183	**192**	**193**	201	206	218	224	225	236	256
74	158	161	169	175	181	**189**	**191**	198	203	214	220	221	232	251
75	156	160	167	173	179	**187**	**188**	196	200	211	217	218	228	246
76	154	158	165	171	176	**184**	**186**	193	198	208	213	214	224	242
77	153	156	163	169	174	**182**	**183**	190	195	205	210	211	220	237
78	151	155	161	167	172	**179**	**181**	188	192	201	206	207	217	233
79	149	153	159	165	170	**177**	**178**	185	189	198	203	204	213	228
80	148	151	157	163	168	**175**	**176**	182	186	195	200	201	209	224
81	146	150	156	161	166	**172**	**173**	180	184	192	197	197	206	220
82	145	148	154	159	164	**170**	**171**	177	181	189	194	194	202	216
83	143	146	152	157	161	**168**	**169**	175	178	186	190	191	199	212
84	142	145	150	155	159	**165**	**166**	172	176	183	187	188	195	208
85	140	143	148	153	157	**163**	**164**	170	173	181	184	185	192	204
86	138	141	147	151	155	**161**	**162**	167	171	178	182	182	189	200
87	137	140	145	150	153	**159**	**160**	165	168	175	179	179	186	197
88	135	138	143	148	151	**157**	**158**	163	166	172	176	176	183	193
89	134	137	141	146	149	**155**	**155**	160	163	170	173	174	180	190
90	132	135	140	144	147	**152**	**153**	158	161	167	170	171	177	186
91	131	133	138	142	146	**150**	**151**	156	159	165	168	168	174	183
92	129	132	136	140	144	**148**	**149**	154	156	162	165	166	171	180
93	128	130	135	139	142	**146**	**147**	151	154	160	162	163	168	177
94	127	129	133	137	140	**144**	**145**	149	152	157	160	160	165	174
95	125	127	131	135	138	**142**	**143**	147	150	155	157	158	163	171
96	124	126	130	133	136	**140**	**141**	145	147	152	155	155	160	168
97	122	124	128	132	135	**139**	**139**	143	145	150	153	153	158	165
98	121	123	127	130	133	**137**	**137**	141	143	148	150	151	155	162
99	119	121	125	128	131	**135**	**135**	139	141	146	148	148	153	159
100	118	120	124	127	129	**133**	**134**	137	139	143	146	146	150	157

Note: Values of C_r/A were calculated in accordance with S16-14 Clause 13.3.1 using n = 1.34, and apply to hot-rolled, fabricated structural sections and HSS produced to G40.20 Class C, ASTM A500 and A1085. For Class H Hollow Structural Sections, see Table 4-4.

UNIT FACTORED COMPRESSIVE RESISTANCES, C_r/A (MPa)

Table 4-3

Compression members for which n = 1.34 applies

$\phi = 0.90$

$\frac{KL}{r} = 101$ to 150

KL/r	Fy (MPa)													
	250	260	280	300	317	345	350	380	400	450	480	485	550	700
101	117	119	122	125	128	131	132	135	137	141	144	144	148	154
102	115	117	121	124	126	129	130	133	135	139	141	142	145	151
103	114	116	119	122	124	128	128	131	133	137	139	140	143	149
104	113	114	118	121	123	126	127	130	131	135	137	137	141	147
105	111	113	116	119	121	124	125	128	129	133	135	135	139	144
106	110	112	115	118	120	123	123	126	128	131	133	133	137	142
107	109	110	113	116	118	121	122	124	126	129	131	131	134	140
108	108	109	112	115	117	119	120	123	124	127	129	129	132	137
109	106	108	111	113	115	118	118	121	122	126	127	128	130	135
110	105	107	109	112	114	116	117	119	121	124	125	126	128	133
111	104	105	108	110	112	115	115	118	119	122	124	124	127	131
112	103	104	107	109	111	113	114	116	117	120	122	122	125	129
113	102	103	105	108	109	112	112	114	116	119	120	120	123	127
114	100	102	104	106	108	110	111	113	114	117	118	118	121	125
115	99.2	100	103	105	107	109	109	111	113	115	117	117	119	123
116	98.1	99.3	102	104	105	107	108	110	111	114	115	115	117	121
117	96.9	98.2	100	102	104	106	106	108	110	112	113	113	116	119
118	95.8	97.0	99.2	101	103	105	105	107	108	110	112	112	114	117
119	94.7	95.9	98.0	99.8	101	103	104	106	107	109	110	110	112	116
120	93.6	94.8	96.8	98.6	100	102	102	104	105	107	109	109	111	114
121	92.6	93.7	95.6	97.4	98.7	101	101	103	104	106	107	107	109	112
122	91.5	92.6	94.5	96.2	97.5	99.4	99.7	101	102	105	106	106	108	111
123	90.5	91.5	93.4	95.0	96.3	98.1	98.4	100	101	103	104	104	106	109
124	89.4	90.4	92.3	93.9	95.1	96.9	97.2	98.8	99.7	102	103	103	105	107
125	88.4	89.4	91.2	92.7	93.9	95.7	96.0	97.5	98.4	100	101	101	103	106
126	87.4	88.4	90.1	91.6	92.8	94.5	94.7	96.2	97.1	99.0	99.9	100	102	104
127	86.4	87.4	89.0	90.5	91.6	93.3	93.5	95.0	95.9	97.7	98.6	98.7	100	103
128	85.4	86.4	88.0	89.4	90.5	92.1	92.4	93.8	94.6	96.4	97.3	97.4	98.9	101
129	84.5	85.4	87.0	88.4	89.4	91.0	91.2	92.6	93.4	95.1	96.0	96.1	97.6	99.9
130	83.5	84.4	85.9	87.3	88.3	89.8	90.1	91.4	92.2	93.9	94.7	94.8	96.2	98.5
131	82.6	83.4	84.9	86.3	87.3	88.7	89.0	90.3	91.0	92.6	93.4	93.5	94.9	97.1
132	81.7	82.5	84.0	85.2	86.2	87.6	87.9	89.1	89.9	91.4	92.2	92.3	93.7	95.8
133	80.8	81.5	83.0	84.2	85.2	86.6	86.8	88.0	88.7	90.2	91.0	91.1	92.4	94.5
134	79.9	80.6	82.0	83.2	84.2	85.5	85.7	86.9	87.6	89.1	89.8	89.9	91.2	93.2
135	79.0	79.7	81.1	82.3	83.2	84.5	84.7	85.8	86.5	87.9	88.6	88.7	90.0	91.9
136	78.1	78.8	80.1	81.3	82.2	83.4	83.6	84.8	85.4	86.8	87.5	87.6	88.8	90.7
137	77.2	77.9	79.2	80.4	81.2	82.4	82.6	83.7	84.4	85.7	86.4	86.5	87.6	89.4
138	76.4	77.1	78.3	79.4	80.2	81.4	81.6	82.7	83.3	84.6	85.3	85.4	86.5	88.2
139	75.5	76.2	77.4	78.5	79.3	80.5	80.7	81.7	82.3	83.5	84.2	84.3	85.4	87.1
140	74.7	75.4	76.6	77.6	78.4	79.5	79.7	80.7	81.3	82.5	83.1	83.2	84.3	85.9
141	73.9	74.5	75.7	76.7	77.5	78.6	78.7	79.7	80.3	81.5	82.1	82.1	83.2	84.8
142	73.1	73.7	74.8	75.8	76.6	77.6	77.8	78.7	79.3	80.4	81.0	81.1	82.1	83.7
143	72.3	72.9	74.0	74.9	75.7	76.7	76.9	77.8	78.3	79.5	80.0	80.1	81.1	82.6
144	71.5	72.1	73.2	74.1	74.8	75.8	76.0	76.9	77.4	78.5	79.0	79.1	80.0	81.5
145	70.7	71.3	72.3	73.3	73.9	74.9	75.1	76.0	76.5	77.5	78.0	78.1	79.0	80.5
146	70.0	70.5	71.5	72.4	73.1	74.1	74.2	75.0	75.5	76.6	77.1	77.2	78.1	79.4
147	69.2	69.8	70.7	71.6	72.3	73.2	73.3	74.2	74.6	75.6	76.1	76.2	77.1	78.4
148	68.5	69.0	70.0	70.8	71.4	72.3	72.5	73.3	73.8	74.7	75.2	75.3	76.1	77.4
149	67.7	68.3	69.2	70.0	70.6	71.5	71.7	72.4	72.9	73.8	74.3	74.4	75.2	76.5
150	67.0	67.5	68.4	69.2	69.8	70.7	70.8	71.6	72.0	73.0	73.4	73.5	74.3	75.5

Note: Values of C_r/A were calculated in accordance with S16-14 Clause 13.3.1 using n = 1.34, and apply to hot-rolled, fabricated structural sections and HSS produced to G40.20 Class C, ASTM A500 and A1085. For Class H Hollow Structural Sections, see Table 4-4.

UNIT FACTORED COMPRESSIVE RESISTANCES, C_r/A (MPa)

Table 4-3

Compression members for which n = 1.34 applies

$\phi = 0.90$

$\frac{KL}{r} = 151$ to 200

$\frac{KL}{r}$	F_y (MPa)													
	250	260	280	300	317	**345**	**350**	380	400	450	480	485	550	700
151	66.3	66.8	67.7	68.5	69.1	**69.9**	**70.0**	70.8	71.2	72.1	72.5	72.6	73.4	74.6
152	65.6	66.1	67.0	67.7	68.3	**69.1**	**69.2**	69.9	70.4	71.2	71.7	71.7	72.5	73.6
153	64.9	65.4	66.2	67.0	67.5	**68.3**	**68.4**	69.1	69.5	70.4	70.8	70.9	71.6	72.7
154	64.2	64.7	65.5	66.2	66.8	**67.5**	**67.7**	68.3	68.7	69.6	70.0	70.0	70.7	71.8
155	63.5	64.0	64.8	65.5	66.0	**66.8**	**66.9**	67.6	68.0	68.8	69.2	69.2	69.9	71.0
156	62.9	63.3	64.1	64.8	65.3	**66.0**	**66.2**	66.8	67.2	68.0	68.3	68.4	69.1	70.1
157	62.2	62.7	63.4	64.1	64.6	**65.3**	**65.4**	66.0	66.4	67.2	67.5	67.6	68.3	69.3
158	61.6	62.0	62.7	63.4	63.9	**64.6**	**64.7**	65.3	65.7	66.4	66.8	66.8	67.5	68.4
159	60.9	61.4	62.1	62.7	63.2	**63.9**	**64.0**	64.6	64.9	65.6	66.0	66.1	66.7	67.6
160	60.3	60.7	61.4	62.0	62.5	**63.2**	**63.3**	63.9	64.2	64.9	65.2	65.3	65.9	66.8
161	59.7	60.1	60.8	61.4	61.8	**62.5**	**62.6**	63.1	63.5	64.2	64.5	64.5	65.1	66.0
162	59.1	59.5	60.1	60.7	61.2	**61.8**	**61.9**	62.4	62.8	63.4	63.8	63.8	64.4	65.3
163	58.5	58.8	59.5	60.1	60.5	**61.1**	**61.2**	61.8	62.1	62.7	63.0	63.1	63.6	64.5
164	57.9	58.2	58.9	59.4	59.9	**60.5**	**60.6**	61.1	61.4	62.0	62.3	62.4	62.9	63.7
165	57.3	57.6	58.3	58.8	59.2	**59.8**	**59.9**	60.4	60.7	61.3	61.6	61.7	62.2	63.0
166	56.7	57.1	57.7	58.2	58.6	**59.2**	**59.3**	59.8	60.1	60.7	60.9	61.0	61.5	62.3
167	56.1	56.5	57.1	57.6	58.0	**58.5**	**58.6**	59.1	59.4	60.0	60.3	60.3	60.8	61.6
168	55.6	55.9	56.5	57.0	57.4	**57.9**	**58.0**	58.5	58.8	59.3	59.6	59.7	60.1	60.9
169	55.0	55.3	55.9	56.4	56.8	**57.3**	**57.4**	57.9	58.1	58.7	59.0	59.0	59.5	60.2
170	54.5	54.8	55.3	55.8	56.2	**56.7**	**56.8**	57.2	57.5	58.0	58.3	58.4	58.8	59.5
171	53.9	54.2	54.8	55.3	55.6	**56.1**	**56.2**	56.6	56.9	57.4	57.7	57.7	58.2	58.9
172	53.4	53.7	54.2	54.7	55.0	**55.5**	**55.6**	56.0	56.3	56.8	57.1	57.1	57.5	58.2
173	52.9	53.2	53.7	54.1	54.5	**55.0**	**55.0**	55.4	55.7	56.2	56.4	56.5	56.9	57.6
174	52.4	52.6	53.1	53.6	53.9	**54.4**	**54.5**	54.9	55.1	55.6	55.8	55.9	56.3	56.9
175	51.8	52.1	52.6	53.0	53.4	**53.8**	**53.9**	54.3	54.5	55.0	55.2	55.3	55.7	56.3
176	51.3	51.6	52.1	52.5	52.8	**53.3**	**53.3**	53.7	54.0	54.4	54.6	54.7	55.1	55.7
177	50.8	51.1	51.6	52.0	52.3	**52.7**	**52.8**	53.2	53.4	53.8	54.1	54.1	54.5	55.1
178	50.3	50.6	51.1	51.5	51.8	**52.2**	**52.3**	52.6	52.8	53.3	53.5	53.5	53.9	54.5
179	49.9	50.1	50.6	51.0	51.2	**51.7**	**51.7**	52.1	52.3	52.7	52.9	53.0	53.3	53.9
180	49.4	49.6	50.1	50.4	50.7	**51.1**	**51.2**	51.6	51.8	52.2	52.4	52.4	52.8	53.3
181	48.9	49.1	49.6	50.0	50.2	**50.6**	**50.7**	51.0	51.2	51.6	51.8	51.9	52.2	52.8
182	48.4	48.7	49.1	49.5	49.7	**50.1**	**50.2**	50.5	50.7	51.1	51.3	51.3	51.7	52.2
183	48.0	48.2	48.6	49.0	49.2	**49.6**	**49.7**	50.0	50.2	50.6	50.8	50.8	51.1	51.7
184	47.5	47.7	48.1	48.5	48.8	**49.1**	**49.2**	49.5	49.7	50.1	50.3	50.3	50.6	51.1
185	47.1	47.3	47.7	48.0	48.3	**48.6**	**48.7**	49.0	49.2	49.6	49.8	49.8	50.1	50.6
186	46.6	46.8	47.2	47.6	47.8	**48.2**	**48.2**	48.5	48.7	49.1	49.2	49.3	49.6	50.1
187	46.2	46.4	46.8	47.1	47.3	**47.7**	**47.7**	48.0	48.2	48.6	48.8	48.8	49.1	49.5
188	45.8	46.0	46.3	46.7	46.9	**47.2**	**47.3**	47.6	47.7	48.1	48.3	48.3	48.6	49.0
189	45.3	45.5	45.9	46.2	46.4	**46.8**	**46.8**	47.1	47.3	47.6	47.8	47.8	48.1	48.5
190	44.9	45.1	45.5	45.8	46.0	**46.3**	**46.4**	46.6	46.8	47.1	47.3	47.3	47.6	48.0
191	44.5	44.7	45.0	45.3	45.6	**45.9**	**45.9**	46.2	46.3	46.7	46.8	46.9	47.1	47.6
192	44.1	44.3	44.6	44.9	45.1	**45.4**	**45.5**	45.7	45.9	46.2	46.4	46.4	46.7	47.1
193	43.7	43.9	44.2	44.5	44.7	**45.0**	**45.0**	45.3	45.5	45.8	45.9	45.9	46.2	46.6
194	43.3	43.5	43.8	44.1	44.3	**44.6**	**44.6**	44.9	45.0	45.3	45.5	45.5	45.7	46.1
195	42.9	43.1	43.4	43.7	43.9	**44.1**	**44.2**	44.4	44.6	44.9	45.0	45.0	45.3	45.7
196	42.5	42.7	43.0	43.2	43.4	**43.7**	**43.8**	44.0	44.2	44.4	44.6	44.6	44.9	45.2
197	42.1	42.3	42.6	42.8	43.0	**43.3**	**43.4**	43.6	43.7	44.0	44.2	44.2	44.4	44.8
198	41.7	41.9	42.2	42.4	42.6	**42.9**	**43.0**	43.2	43.3	43.6	43.7	43.8	44.0	44.3
199	41.4	41.5	41.8	42.1	42.2	**42.5**	**42.6**	42.8	42.9	43.2	43.3	43.3	43.6	43.9
200	41.0	41.1	41.4	41.7	41.9	**42.1**	**42.2**	42.4	42.5	42.8	42.9	42.9	43.1	43.5

Note: Values of C_r/A were calculated in accordance with S16-14 Clause 13.3.1 using n = 1.34, and apply to hot-rolled, fabricated structural sections and HSS produced to G40.20 Class C, ASTM A500 and A1085. For Class H Hollow Structural Sections, see Table 4-4.

UNIT FACTORED COMPRESSIVE RESISTANCES, C_r/A (MPa)

Table 4-4

HSS Class H and other sections for which n = 2.24 applies

$\phi = 0.90$

$\frac{KL}{r}$	F_y (MPa) 350	$\frac{KL}{r}$	F_y (MPa) 350	$\frac{KL}{r}$	F_y (MPa) 350	$\frac{KL}{r}$	F_y (MPa) 350
1	315	51	293	101	157	151	76.4
2	315	52	291	102	154	152	75.5
3	315	53	289	103	152	153	74.5
4	315	54	287	104	150	154	73.6
5	315	55	285	105	147	155	72.7
6	315	56	283	106	145	156	71.8
7	315	57	281	107	143	157	70.9
8	315	58	279	108	141	158	70.1
9	315	59	276	109	138	159	69.2
10	315	60	274	110	136	160	68.4
11	315	61	272	111	134	161	67.6
12	315	62	269	112	132	162	66.7
13	315	63	266	113	130	163	66.0
14	315	64	264	114	128	164	65.2
15	315	65	261	115	126	165	64.4
16	315	66	258	116	124	166	63.7
17	315	67	255	117	123	167	62.9
18	315	68	253	118	121	168	62.2
19	315	69	250	119	119	169	61.5
20	315	70	247	120	117	170	60.8
21	315	71	244	121	115	171	60.1
22	314	72	241	122	114	172	59.4
23	314	73	238	123	112	173	58.7
24	314	74	235	124	110	174	58.1
25	314	75	231	125	109	175	57.4
26	314	76	228	126	107	176	56.8
27	314	77	225	127	106	177	56.2
28	313	78	222	128	104	178	55.6
29	313	79	219	129	103	179	54.9
30	313	80	216	130	101	180	54.4
31	312	81	213	131	99.9	181	53.8
32	312	82	210	132	98.5	182	53.2
33	312	83	207	133	97.1	183	52.6
34	311	84	204	134	95.8	184	52.1
35	311	85	201	135	94.5	185	51.5
36	310	86	198	136	93.2	186	51.0
37	309	87	195	137	91.9	187	50.4
38	309	88	192	138	90.7	188	49.9
39	308	89	189	139	89.5	189	49.4
40	307	90	186	140	88.3	190	48.9
41	306	91	183	141	87.1	191	48.4
42	305	92	180	142	85.9	192	47.9
43	304	93	178	143	84.8	193	47.4
44	303	94	175	144	83.7	194	46.9
45	302	95	172	145	82.6	195	46.4
46	301	96	170	146	81.5	196	46.0
47	299	97	167	147	80.5	197	45.5
48	298	98	164	148	79.4	198	45.1
49	296	99	162	149	78.4	199	44.6
50	295	100	159	150	77.4	200	44.2

Note: Values of C_r/A were calculated in accordance with S16-14 Clause 13.3.1 using n = 2.24, and apply to welded 3-plate sections with oxy-flame-cut flanges and HSS produced to G40.20 Class H.
For Class C Hollow Structural Sections, see Table 4-3.

C_e/A EULER BUCKLING LOAD

Table 4-5

Per Unit of Area, MPa

KL/r	C_e/A MPa	KL/r	C_e/A MPa	KL/r	C_e/A MPa	KL/r	C_e/A MPa	KL/r	C_e/A MPa
1	1 970 000	41	1 170	81	301	121	135	161	76.2
2	493 000	42	1 120	82	294	122	133	162	75.2
3	219 000	43	1 070	83	287	123	130	163	74.3
4	123 000	44	1 020	84	280	124	128	164	73.4
5	79 000	45	975	85	273	125	126	165	72.5
6	54 800	46	933	86	267	126	124	166	71.6
7	40 300	47	894	87	261	127	122	167	70.8
8	30 800	48	857	88	255	128	120	168	69.9
9	24 400	49	822	89	249	129	119	169	69.1
10	19 700	50	790	90	244	130	117	170	68.3
11	16 300	51	759	91	238	131	115	171	67.5
12	13 700	52	730	92	233	132	113	172	66.7
13	11 700	53	703	93	228	133	112	173	66.0
14	10 100	54	677	94	223	134	110	174	65.2
15	8 770	55	653	95	219	135	108	175	64.5
16	7 710	56	629	96	214	136	107	176	63.7
17	6 830	57	608	97	210	137	105	177	63.0
18	6 090	58	587	98	206	138	104	178	62.3
19	5 470	59	567	99	201	139	102	179	61.6
20	4 930	60	548	100	197	140	101	180	60.9
21	4 480	61	530	101	194	141	99.3	181	60.3
22	4 080	62	514	102	190	142	97.9	182	59.6
23	3 730	63	497	103	186	143	96.5	183	58.9
24	3 430	64	482	104	183	144	95.2	184	58.3
25	3 160	65	467	105	179	145	93.9	185	57.7
26	2 920	66	453	106	176	146	92.6	186	57.1
27	2 710	67	440	107	172	147	91.3	187	56.4
28	2 520	68	427	108	169	148	90.1	188	55.8
29	2 350	69	415	109	166	149	88.9	189	55.3
30	2 190	70	403	110	163	150	87.7	190	54.7
31	2 050	71	392	111	160	151	86.6	191	54.1
32	1 930	72	381	112	157	152	85.4	192	53.5
33	1 810	73	370	113	155	153	84.3	193	53.0
34	1 710	74	360	114	152	154	83.2	194	52.4
35	1 610	75	351	115	149	155	82.2	195	51.9
36	1 520	76	342	116	147	156	81.1	196	51.4
37	1 440	77	333	117	144	157	80.1	197	50.9
38	1 370	78	324	118	142	158	79.1	198	50.3
39	1 300	79	316	119	139	159	78.1	199	49.8
40	1 230	80	308	120	137	160	77.1	200	49.3

To obtain C_e, in kN, multiply the tabular value by the cross-sectional area, A, in mm^2, and divide by 1000.

FACTORED AXIAL COMPRESSIVE RESISTANCES OF COLUMNS

Tables

The tables on the following pages list the factored axial compressive resistances, C_r, in kilonewtons (kN) for W-shapes and Hollow Structural Sections (HSS) produced to the requirements of CSA Standard G40.21 (Class C and Class H) and ASTM A500 Grade C. The resistances have been computed for effective lengths with respect to the least radius of gyration varying from 0 mm up to 20 000 mm in accordance with the requirements of Clauses 13.3.1 and 13.3.5, CSA S16-14 with $n = 1.34$ for W-shapes and for HSS produced to G40.21 Class C and ASTM A500, and with $n = 2.24$ for HSS produced to G40.21 Class H.

Data for Welded Wide-Flange columns is no longer provided in this Handbook.

In all, 5 sets of tables are provided:

Set 1 — W-shapes conforming to ASTM A992 and ASTM A572 Grade 50 ($F_y = 345$ MPa, $n = 1.34$). Note: CSA S16-14 Update No. 1 (December 2016) led to significant changes in the 3rd printing of this (11th edition) Handbook.

Set 2 — W-shapes conforming to ASTM A913 Grade 65 ($F_y = 450$ MPa, $n = 1.34$)

Set 3 — HSS conforming to CSA G40.21-350W, Class C ($F_y = 350$ MPa, $n = 1.34$)

Set 4 — HSS conforming to CSA G40.21-350W, Class H ($F_y = 350$ MPa, $n = 2.24$)

Set 5 — HSS conforming to ASTM A500 Grade C ($F_y = 345$ MPa for rectangular and square, $F_y = 317$ MPa for round, $n = 1.34$)

The applicable steel grade (and class, as applicable) is listed at the top of each table, and the metric section size and mass are given at the top of the columns, while the equivalent imperial size and weight are listed at the bottom of the tables.

In Set 1, the minimum specified yield stress has been taken as $F_y = 345$ MPa, corresponding to steel grades ASTM A992 and A572 grade 50. Tabulated values may also be used for column sections produced to CSA G40.21-350W, although that steel grade is not shown in the table headings. Section sizes that are commonly used and generally readily available are highlighted in yellow colour. In Sets 3 to 5, a number of sizes are identified with an asterisk (*), denoting imported sections, which may be subject to a cost premium.

Class 4 Sections in Axial Compression

Sections that are Class 4 in axial compression only are identified. The factored axial compressive resistances for them have been computed in accordance with the requirements of Clause 13.3.5 of S16-14. The C_r values for W-shapes correspond to the greater of the resistances based on the effective area method according to Clause 13.3.5(a) and the effective yield stress method according to Clause 13.3.5(b).

For HSS, factored axial compressive resistances for Class 4 sections were calculated in accordance with Clause 13.3.5(a), except for a small number of square sections identified by a symbol (#), for which C_r was calculated in accordance with Clause 13.3.5(b).

Properties and design data are included at the bottom of the tables as follows:

Area	=	Total cross-sectional area, mm^2
$(b_{el}/t)\sqrt{345}$	=	Ratio of compression element width to flange thickness or design wall thickness, based on a yield stress, $F_y = 345$ MPa, for use in conjunction with S16-14 Tables 1 and 2.
$(b_{el}/t)\sqrt{350}$	=	Ratio of compression element width to flange thickness or design wall thickness, based on a yield stress, $F_y = 350$ MPa, when used as a seismic yielding member for which S16-14, Clause 27.1.7 applies and, for CSA products, also for use in conjunction with S16-14 Tables 1 and 2.
$(b_{el}/t)\sqrt{450}$	=	Ratio of compression element width to flange thickness or design wall thickness, based on a yield stress, $F_y = 450$ MPa, for use in conjunction with S16-14 Tables 1 and 2.
$(D/t)317$	=	Ratio of outside diameter of round HSS to design wall thickness, based on a yield stress, F_y = 317 MPa, for use in conjunction with S16-14 Tables 1 and 2.
$(D/t)350$	=	Ratio of outside diameter of round HSS to design wall thickness, based on a yield stress, $F_y = 350$ MPa, when used as a seismic yielding member for which S16-14, Clause 27.1.7 applies.
$(h/w)\sqrt{350}$	=	Ratio of clear depth of web to thickness, based on a yield stress, $F_y = 350$ MPa, when used as a seismic yielding member for which S16-14, Clause 27.1.7 applies and, for CSA products, also for use in conjunction with S16-14 Tables 1 and 2.
$(h/w)\sqrt{450}$	=	Ratio of clear depth of web to thickness, based on a yield stress, $F_y = 450$ MPa, for use in conjunction with S16-14 Tables 1 and 2.
L_u	=	Maximum unsupported length of compression flange for which no reduction in M_r is required, mm
M_{rx}	=	Factored moment resistance for bending about the X-X axis, computed considering $L \leq L_u$, for Class 1 and Class 2 sections ($\phi Z_x F_y$); for Class 3 sections ($\phi S_x F_y$); and for Class 4 sections ($\phi S_e F_y$) in accordance with S16-14 Clause 13.5(c)(iii), kN·m. See *Bending Resistances* below.
M_{ry}	=	Factored moment resistance for bending about the Y-Y axis, for Class 1 and Class 2 sections ($\phi Z_y F_y$); for Class 3 sections ($\phi S_y F_y$); and for Class 4 sections ($\phi S_e F_y$) in accordance with S16-14 Clause 13.5(c), kN·m. See *Bending Resistances* below.
r_x	=	Radius of gyration about the major, X-X, axis, mm
r_x / r_y	=	Ratio of radius of gyration of X-X axis to that of Y-Y axis
r_y	=	Radius of gyration about the minor, Y-Y, axis, mm
t	=	Flange thickness, mm
$\phi S_x F_y, \phi Z_x F_y$	=	Factored moment resistance for bending about the X-X axis, for Class 3 sections, and for Class 1 and Class 2 sections, respectively, in accordance with S16-14 Clause 13.5, kN·m. In the tables below, if either $\phi S_x F_y$ or $\phi Z_x F_y$ is not applicable, it is left blank.

$\phi S_y F_y, \phi Z_y F_y$ = Factored moment resistance for bending about the Y-Y axis, for Class 3 sections, and for Class 1 and Class 2 sections, respectively, in accordance with S16-14 Clause 13.5, kN·m. In the tables below, if either $\phi S_y F_y$ or $\phi Z_y F_y$ is not applicable, it is left blank.

Bending Resistances

For W-shape members, tabulated bending resistances about the X-X axis cannot be used for lateral-torsional buckling when the laterally unsupported length exceeds L_u.

The section Class is based on combined uniaxial bending and axial compression. *For members subject to bi-axial bending, bending resistances should be checked for compliance with S16-14 Table 2.*

For members in combined compression and bending, when the section Class with respect to the web slenderness (*h/w* ratio) is sensitive to the magnitude of axial compression, the factored bending resistances tabulated have been calculated with these assumptions:

- W-shape members for which bending resistances about either axis are functions of C_f are identified by the symbol (^) in the lower portion of the tables. Values of C_f at which the section Class changes from 2 to 3 are underlined, and the bending axes affected by the change are indicated by superscripts (^{x}) and (^{y}). Values of C_f at which the section Class changes from 3 to 4 are shown in boldface, but no superscript is shown since both bending axes are affected simultaneously. Bending resistances, $\phi S_{xe} F_y$ and $\phi S_{ye} F_y$, are not provided.
- Class 4 sections in pure compression are identified by the symbol (‡) next to the mass in kg/m. When flange slenderness (b_{el}/t ratio) renders the section a Class 4 section in pure compression, it is also a Class 4 section in combined bending and compression. In particular, the W150x22 section is also a Class 4 in bending about the X-X axis, and the value of $\phi S_x F_y$ was taken equal to $\phi S_{xe} F_y$ and preceded by the symbol (‡).
- For rectangular HSS identified as Class 4 in axial compression, the M_{rx} values tabulated are only valid for C_f values below the C_r value shown in bold face. Otherwise, the user must calculate M_{rx} as a Class 4 section.

Design of Axially Loaded Columns

The design of axially loaded columns (columns theoretically not subjected to combined bending and compression) involves the determination of the governing effective length and the selection of a section with the required resistance at that effective length. Factored axial compressive resistance tables for columns enable a designer to select a suitable section directly, without following a trial-and-error procedure.

Since the factored axial compressive resistances (C_r) tabulated have been computed on the basis of the least radius of gyration (r_y) for each section, the tables apply directly only to columns unbraced about the Y-Y axis. In certain cases, however, it is necessary to investigate the capacity of a column with reference to both the X-X axis and the Y-Y axis, or with reference only to the X-X axis. The ratio r_x / r_y included in the table of properties at the bottom of each resistance table provides a convenient means of investigating the strength of a column with respect to the X-X axis.

In general, a column having an effective length $K_x L_x$ with respect to the X-X axis will be able to carry a factored load equal to the tabulated factored axial compressive resistance based on the effective length $K_y L_y$ with respect to the Y-Y axis if $K_x L_x < K_y L_y (r_x / r_y)$.

Resistances of HSS Columns Produced to ASTM A500

The tables of resistances for HSS used as columns were computed in accordance with CSA Standard S16-14, with properties and dimensions based on a design wall thickness equal to 90% of the nominal thickness, and a value of F_y = 345 MPa for square and rectangular HSS and F_y = 317 for circular HSS MPa, as specified in ASTM A500 for grade C.

For HSS used as columns, the value of n = 1.34 (for the basic column curve, Clause 13.3, S16-14) was used in determining the factored axial compressive resistance, as HSS produced to ASTM A500 grade C are generally cold-formed non-stress-relieved sections. For more information on HSS produced to ASTM A500, see *Hollow Structural Sections* in Part 6.

Resistances of HSS Columns Produced to ASTM A1085

Since the manufacturing method for A1085 HSS is also permitted for the manufacturing of Class C CSA G40.21 HSS, the factored axial compressive resistance of an A1085 HSS column may be determined in accordance with S16-14 Clause 13.3.1 with the value of n taken as 1.34. The factored axial compressive resistance tables for Class C G40.21 HSS columns may be used, provided an adjustment for the small difference in F_y values (345 MPa vs. 350 MPa) is accounted for.

Examples

1. Given:

   A W310 column is required to carry a factored axial load of 3 600 kN. The effective lengths $K_y L_y$ and $K_x L_x$ are 4 500 mm and 7 600 mm, respectively. Use ASTM A992 steel.

Solution:

With $K_y L_y$ = 4 500, the lightest W310 section with sufficient factored axial compressive resistance is W310x129. C_r = 3 820 kN; r_x/r_y = 1.76.

$K_x L_x$ = 7 600 mm (required)

$K_y L_y (r_x/r_y)$ = 4 500 × 1.76 = 7 920 mm > 7 600 mm

The W310x129 has a factored compressive resistance of 3 820 kN with an effective length of $K_x L_x$ = 7 920 mm, and hence the section is adequate. Use W310x129.

2. Given:

   Same as example 1, except $K_x L_x$ = 9 500 mm

Solution:

$K_y L_y$ = 4 500 mm, $K_x L_x$ = 9 500 mm

Equivalent $K_y L_y$, for $K_x L_x$ of 9 500 mm = $K_x L_x / (r_x/r_y)$

Assuming that a heavy W310 section will be adequate, r_x/r_y = 1.76.

Equivalent $K_y L_y$ = 9 500 / 1.76 = 5 400 mm > 4 500 mm

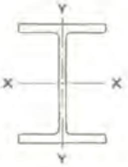
Therefore, $K_x L_x$ governs, and the effective $K_y L_y$ is 5 400 mm.

With $K_y L_y$ = 5 400 mm, a W310x143 is the lightest W310 that has a factored axial compressive resistance greater than the factored axial load of 3 600 kN (C_r for 5 500 mm = 3 630 kN; r_x/r_y = 1.76)

Use W310x143.

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)



ASTM A992, A572 Grade 50

F_y = 345 MPa

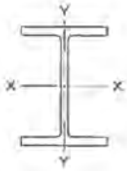
ϕ = 0.90

Designation		W360					
Mass (kg/m)		1086	990	900	818	744	677
Effective length (KL) in millimetres with respect to the least radius of gyration	0	43 000	39 200	35 700	32 400	29 400	26 800
	2 500	42 000	38 200	34 800	31 500	28 600	26 100
	3 000	41 400	37 700	34 300	31 000	28 100	25 600
	3 500	40 600	36 900	33 600	30 400	27 500	25 000
	4 000	39 700	36 000	32 700	29 600	26 800	24 400
	4 500	38 600	35 000	31 800	28 700	26 000	23 600
	5 000	37 400	33 900	30 700	27 700	25 000	22 700
	5 500	36 100	32 600	29 600	26 600	24 000	21 800
	6 000	34 600	31 300	28 300	25 500	22 900	20 800
	6 500	33 200	29 900	27 100	24 300	21 800	19 800
	7 000	31 600	28 500	25 800	23 100	20 700	18 800
	7 500	30 100	27 100	24 500	21 900	19 600	17 700
	8 000	28 600	25 700	23 200	20 700	18 500	16 700
	8 500	27 100	24 300	21 900	19 600	17 500	15 800
	9 000	25 700	23 000	20 700	18 500	16 500	14 900
	9 500	24 300	21 700	19 600	17 400	15 500	14 000
	10 000	22 900	20 500	18 500	16 400	14 600	13 100
	10 500	21 600	19 300	17 400	15 500	13 700	12 400
	11 000	20 400	18 200	16 400	14 600	12 900	11 600
	11 500	19 300	17 200	15 500	13 700	12 200	10 900
	12 000	18 200	16 200	14 600	12 900	11 500	10 300
	12 500	17 200	15 300	13 800	12 200	10 800	9 700
	13 000	16 300	14 500	13 000	11 500	10 200	9 140
	13 500	15 400	13 700	12 300	10 900	9 600	8 620
	14 000	14 600	12 900	11 600	10 300	9 070	8 140
	15 000	13 100	11 600	10 400	9 190	8 110	7 280
	16 000	11 800	10 400	9 370	8 260	7 280	6 530
	17 000	10 600	9 430	8 460	7 450	6 570	5 890
	18 000	9 660	8 540	7 660	6 750	5 940	5 330
PROPERTIES AND DESIGN DATA							
Area (mm^2)		139 000	126 000	115 000	105 000	94 800	86 500
t (mm)		125	115	106	97.0	88.9	81.5
r_x (mm)		207	203	198	194	190	186
r_y (mm)		119	117	116	114	112	111
r_x / r_y		1.74	1.74	1.71	1.70	1.70	1.68
$\phi S_x F_y$ (kN·m)							
$\phi Z_x F_y$ (kN·m)		8 450	7 550	6 710	5 990	5 340	4 750
L_u (mm)		21 200	19 600	18 300	16 900	15 700	14 600
$\phi S_y F_y$ (kN·m)							
$\phi Z_y F_y$ (kN·m)		4 160	3 730	3 320	2 970	2 650	2 380
§ $(b_{el}/t)\sqrt{350}$		34.0	36.4	39.0	42.1	45.5	49.1
§ $(h/w)\sqrt{350}$		76.5	83.3	90.6	99.0	108	117
IMPERIAL SIZE AND WEIGHT							
Weight (lb/ft)		730	665	605	550	500	455
Depth x Width (in.)		22⅜ x 17⅞	21⅝ x 17⅝	20⅞ x 17⅜	20¼ x 17¼	19⅝ x 17	19 x 16⅞

§ See S16-14 Clause 27.1.7 for seismic applications.

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)



ASTM A992, A572 Grade 50

F_y = 345 MPa

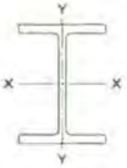
ϕ = 0.90

Designation		W360						
Mass (kg/m)		634	592	551	509	463	421	382
Effective length (KL) in millimetres with respect to the least radius of gyration	0	25 100	23 400	21 800	20 100	18 300	16 700	15 100
	2 500	24 400	22 700	21 100	19 500	17 700	16 100	14 600
	3 000	23 900	22 300	20 700	19 200	17 400	15 800	14 300
	3 500	23 400	21 800	20 200	18 700	17 000	15 400	14 000
	4 000	22 700	21 200	19 700	18 200	16 500	15 000	13 500
	4 500	22 000	20 500	19 000	17 600	15 900	14 400	13 100
	5 000	21 200	19 700	18 200	16 900	15 300	13 900	12 500
	5 500	20 300	18 900	17 400	16 100	14 600	13 200	11 900
	6 000	19 300	18 000	16 600	15 400	13 900	12 600	11 300
	6 500	18 400	17 100	15 800	14 600	13 200	11 900	10 700
	7 000	17 400	16 200	14 900	13 800	12 400	11 200	10 100
	7 500	16 500	15 300	14 100	13 000	11 700	10 600	9 530
	8 000	15 500	14 400	13 300	12 300	11 000	9 960	8 950
	8 500	14 600	13 500	12 500	11 500	10 400	9 360	8 400
	9 000	13 800	12 700	11 700	10 800	9 740	8 780	7 870
	9 500	12 900	12 000	11 000	10 200	9 140	8 230	7 380
	10 000	12 200	11 200	10 300	9 550	8 570	7 720	6 910
	10 500	11 400	10 600	9 690	8 970	8 050	7 240	6 480
	11 000	10 700	9 920	9 100	8 420	7 550	6 790	6 080
	11 500	10 100	9 320	8 550	7 910	7 090	6 370	5 700
	12 000	9 510	8 770	8 040	7 440	6 660	5 990	5 350
	12 500	8 950	8 250	7 560	7 000	6 270	5 630	5 030
	13 000	8 440	7 770	7 120	6 590	5 900	5 300	4 730
	13 500	7 960	7 330	6 710	6 210	5 560	4 990	4 450
	14 000	7 510	6 920	6 330	5 860	5 240	4 700	4 200
	15 000	6 710	6 180	5 650	5 230	4 680	4 190	3 740
	16 000	6 020	5 540	5 070	4 690	4 190	3 760	3 350
	17 000	5 420	4 990	4 560	4 220	3 770	3 380	3 010
	18 000	4 900	4 510	4 120	3 810	3 410	3 050	2 720
PROPERTIES AND DESIGN DATA								
Area (mm^2)		80 600	75 500	70 300	65 200	59 000	53 700	48 800
t (mm)		77.1	72.3	67.6	62.7	57.4	52.6	48.0
r_x (mm)		184	182	180	178	175	172	170
r_y (mm)		110	109	108	108	107	106	105
r_x / r_y		1.67	1.67	1.67	1.65	1.64	1.62	1.62
$\phi S_x F_y$ (kN·m)								
$\phi Z_x F_y$ (kN·m)		4 410	4 070	3 760	3 420	3 070	2 760	2 470
L_u (mm)		13 900	13 200	12 400	11 700	10 800	10 100	9 440
$\phi S_y F_y$ (kN·m)								
$\phi Z_y F_y$ (kN·m)		2 210	2 040	1 880	1 720	1 550	1 390	1 250
§ $(b_{el}/t)\sqrt{350}$		51.4	54.5	57.8	62.1	67.1	72.7	79.1
§ $(h/w)\sqrt{350}$		126	133	142	153	167	182	201
IMPERIAL SIZE AND WEIGHT								
Weight (lb/ft)		426	398	370	342	311	283	257
Depth x Width (in.)		18⅝ x 16¾	18¼ x 16⅝	17⅞ x 16½	17½ x 16⅜	17⅛ x 16¼	16¾ x 16⅛	16⅜ x 16

§ See S16-14 Clause 27.1.7 for seismic applications.

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)



ASTM A992, A572 Grade 50

F_y = 345 MPa

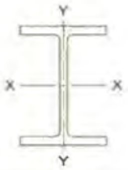
ϕ = 0.90

Designation		W360					
Mass (kg/m)		347	314	287	262	237	216
Effective length (KL) in millimetres with respect to the least radius of gyration	0	13 700	12 400	11 400	10 400	9 340	8 550
	2 500	13 300	12 000	11 000	10 000	9 020	8 250
	3 000	13 000	11 700	10 800	9 810	8 820	8 070
	3 500	12 700	11 400	10 500	9 550	8 580	7 840
	4 000	12 300	11 000	10 100	9 230	8 300	7 580
	4 500	11 800	10 600	9 750	8 870	7 980	7 280
	5 000	11 300	10 200	9 330	8 480	7 630	6 950
	5 500	10 800	9 680	8 880	8 070	7 250	6 600
	6 000	10 200	9 170	8 420	7 640	6 870	6 250
	6 500	9 660	8 670	7 950	7 210	6 480	5 890
	7 000	9 110	8 160	7 490	6 780	6 100	5 540
	7 500	8 570	7 670	7 040	6 370	5 730	5 190
	8 000	8 040	7 190	6 600	5 970	5 370	4 860
	8 500	7 540	6 740	6 180	5 590	5 020	4 550
	9 000	7 060	6 310	5 790	5 230	4 700	4 250
	9 500	6 620	5 900	5 420	4 890	4 400	3 970
	10 000	6 200	5 530	5 070	4 570	4 110	3 720
	10 500	5 800	5 170	4 750	4 280	3 850	3 470
	11 000	5 440	4 850	4 450	4 000	3 600	3 250
	11 500	5 100	4 540	4 170	3 750	3 370	3 040
	12 000	4 790	4 260	3 910	3 520	3 160	2 850
	12 500	4 500	4 000	3 670	3 300	2 970	2 680
	13 000	4 230	3 760	3 450	3 100	2 790	2 520
	13 500	3 980	3 540	3 250	2 920	2 630	2 370
	14 000	3 750	3 330	3 060	2 750	2 470	2 230
	15 000	3 340	2 970	2 720	2 450	2 200	1 980
	16 000	2 990	2 660	2 440	2 190	1 970	1 770
	17 000	2 690	2 390	2 190	1 970	1 770	1 590
	18 000	2 430	2 150	1 980	1 770	1 600	1 430
PROPERTIES AND DESIGN DATA							
Area (mm^2)		44 200	40 000	36 600	33 400	30 100	27 500
t (mm)		43.7	39.6	36.6	33.3	30.2	27.7
r_x (mm)		168	166	165	163	162	161
r_y (mm)		104	103	103	102	102	101
r_x / r_y		1.62	1.61	1.60	1.60	1.59	1.59
$\phi S_x F_y$ (kN·m)							
$\phi Z_x F_y$ (kN·m)		2 220	1 980	1 800	1 630	1 460	1 320
L_u (mm)		8 860	8 290	7 890	7 490	7 120	6 880
$\phi S_y F_y$ (kN·m)							
$\phi Z_y F_y$ (kN·m)		1 130	1 010	919	832	742	677
§ $(b_{el}/t)\sqrt{350}$		86.5	94.7	102	112	122	133
§ $(h/w)\sqrt{350}$		220	240	265	284	316	346
IMPERIAL SIZE AND WEIGHT							
Weight (lb/ft)		233	211	193	176	159	145
Depth x Width (in.)		16 x 15⅞	15¾ x 15¾	15½ x 15¾	15¼ x 15⅝	15 x 15⅝	14¾ x 15½

§ See S16-14 Clause 27.1.7 for seismic applications.

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)



ASTM A992, A572 Grade 50

F_y = 345 MPa

ϕ = 0.90

Designation	W360					W360	
Mass (kg/m)	196	179	162	†† 147	†† 134	122	110
Effective length (KL) in millimetres with respect to the least radius of gyration							
0	7 770	7 090	6 410	5 830	5 300	4 810	4 360
500	7 770	7 080	6 400	5 830	5 290	4 800	4 350
1 000	7 740	7 060	6 380	5 810	5 280	4 760	4 310
1 500	7 690	7 010	6 330	5 770	5 240	4 660	4 220
2 000	7 590	6 920	6 250	5 690	5 170	4 490	4 060
2 500	7 450	6 790	6 140	5 580	5 070	4 260	3 860
3 000	7 270	6 620	5 980	5 440	4 940	3 980	3 600
3 500	7 030	6 410	5 790	5 260	4 770	3 670	3 320
4 000	6 760	6 160	5 560	5 050	4 580	3 350	3 030
4 500	6 460	5 880	5 310	4 820	4 370	3 030	2 740
5 000	6 140	5 580	5 040	4 570	4 150	2 730	2 470
5 500	5 800	5 270	4 760	4 310	3 910	2 450	2 220
6 000	5 460	4 960	4 470	4 050	3 670	2 200	1 990
6 500	5 120	4 650	4 190	3 790	3 440	1 980	1 790
7 000	4 780	4 340	3 910	3 540	3 210	1 780	1 610
7 500	4 460	4 050	3 650	3 300	2 990	1 600	1 450
8 000	4 160	3 780	3 400	3 070	2 780	1 450	1 310
8 500	3 870	3 510	3 160	2 860	2 590	1 310	1 190
9 000	3 610	3 270	2 940	2 660	2 410	1 190	1 080
9 500	3 360	3 050	2 740	2 470	2 240	1 090	983
10 000	3 130	2 840	2 550	2 300	2 080	993	899
10 500	2 920	2 640	2 380	2 150	1 940	911	824
11 000	2 720	2 470	2 220	2 000	1 810	838	758
11 500	2 540	2 300	2 070	1 870	1 690	773	699
12 000	2 380	2 150	1 940	1 750	1 580	715	647
12 500	2 230	2 020	1 810	1 640	1 480	663	600
13 000	2 090	1 890	1 700	1 530	1 380		
13 500	1 960	1 780	1 600	1 440	1 300		
14 000	1 840	1 670	1 500	1 350	1 220		
PROPERTIES AND DESIGN DATA							
Area (mm^2)	25 000	22 800	20 600	18 800	17 100	15 500	14 100
t (mm)	26.2	23.9	21.8	19.8	18.0	21.7	19.9
r_x (mm)	159	159	158	157	156	154	154
r_y (mm)	95.6	95.2	94.9	94.3	94.0	63.0	63.0
r_x / r_y	1.66	1.67	1.66	1.66	1.66	2.44	2.44
$\phi S_x F_y$ (kN·m)				798	723		
$\phi Z_x F_y$ (kN·m)	1 190	1 080	975			705	640
L_u (mm)	6 370	6 170	5 980	6 190	6 030	4 040	3 940
$\phi S_y F_y$ (kN·m)				281	254		
$\phi Z_y F_y$ (kN·m)	578	522	472			227	206
§ $(b_{el}/t)\sqrt{350}$	134	146	159	175	192	111	120
§ $(h/w)\sqrt{350}$	365	399	451	487	535	460	525
IMPERIAL SIZE AND WEIGHT							
Weight (lb/ft)	132	120	109	99	90	82	74
Depth x Width (in.)	14⅝ x 14¾	14½ x 14⅝	14⅜ x 14⅝	14⅛ x 14⅝	14 x 14½	14¼ x 10⅛	14⅛ x 10⅛

§ See S16-14 Clause 27.1.7 for seismic applications.

Sections highlighted in yellow are generally readily available.

†† Class 3 in bending about either axis due to flange

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)

ASTM A992, A572 Grade 50

F_y = 345 MPa

ϕ = 0.90

Designation	W360		W360			W310		
Mass (kg/m)	101	91	79	‡ 72	‡ 64	500	454	415
Effective length (KL) in millimetres with respect to the least radius of gyration								
0	4 000	3 590	3 130	2 800	2 430	19 800	18 000	16 400
500	4 000	3 580	3 120	**2 790**	2 420	19 800	17 900	16 400
1 000	3 960	3 550	3 070	2 740	**2 370**	19 700	17 900	16 300
1 500	3 870	3 470	2 940	2 620	2 270 ^{x}	19 500	17 700	16 200
2 000	3 730	3 340	2 750	2 450	2 120	19 200	17 400	15 900
2 500	3 540	3 170	2 510	2 230	1 920	18 800	17 000	15 500
3 000	3 300	2 960	2 240	1 990	1 710	18 200	16 500	15 000
3 500	3 040	2 720	1 970	1 750	1 500	17 500	15 800	14 400
4 000	2 780	2 480	1 720	1 520	1 310	16 700	15 100	13 700
4 500	2 510	2 240	1 500	1 320	1 130	15 800	14 200	13 000
5 000	2 260	2 010	1 300	1 150	985	14 900	13 400	12 200
5 500	2 030	1 810	1 140 ^{y}	1 000 ^{y}	858 ^{y}	13 900	12 500	11 400
6 000	1 820	1 620	994	877	750	13 000	11 700	10 600
6 500	1 630 ^{y}	1 450 ^{y}	874	771	659	12 100	10 800	9 800
7 000	1 470	1 300	773	681	582	11 200	10 000	9 060
7 500	1 320	1 170	687	606	516	10 400	9 260	8 380
8 000	1 190	1 060	613	541	463	9 600	8 560	7 740
8 500	1 080	960	550	486	417	8 880	7 910	7 150
9 000	983	872	496	438	377	8 220	7 320	6 610
9 500	896	795	449	397	342	7 620	6 780	6 110
10 000	819	726				7 070	6 280	5 660
10 500	751	666				6 560	5 830	5 250
11 000	691	613				6 100	5 410	4 880
11 500	637	565				5 680	5 040	4 540
12 000	589	522				5 300	4 700	4 230
12 500	546					4 950	4 380	3 950
13 000						4 630	4 100	3 690
13 500						4 330	3 840	3 450
14 000						4 070	3 600	3 240
PROPERTIES AND DESIGN DATA								
Area (mm^2)	12 900	11 500	10 100	9 100	8 130	63 700	57 800	52 800
t (mm)	18.3	16.4	16.8	15.1	13.5	75.1	68.7	62.7
r_x (mm)	153	152	150	149	148	163	160	157
r_y (mm)	62.7	62.3	48.9	48.5	48.1	88.0	86.8	86.0
r_x / r_y	2.44	2.44	3.07	3.07	3.08	1.85	1.84	1.83
$\phi S_x F_y$ (kN·m)				^ 357	^ 320			
$\phi Z_x F_y$ (kN·m)	584	522	444	^ 397	^ 354	3 070	2 740	2 450
L_u (mm)	3 860	3 760	3 010	2 940	2 870	12 100	11 100	10 400
$\phi S_y F_y$ (kN·m)	^ 123	^ 110	^ 73.3	^ 65.2	^ 57.8			
$\phi Z_y F_y$ (kN·m)	^ 188	^ 167	^ 112	^ 100	^ 88.2	1 390	1 240	1 120
$^{§}(b_{el}/t)\sqrt{350}$	130	145	114	126	141	42.3	45.7	49.8
$^{§}(h/w)\sqrt{350}$	571	631	638	696	777	115	126	134
IMPERIAL SIZE AND WEIGHT								
Weight (lb/ft)	68	61	53	48	43	336	305	279
Depth x Width (in.)	14 x 10	13⅞ x 10	13⅞ x 8	13¾ x 8	13⅝ x 8	16⅞ x 13⅜	16⅜ x 13¼	15⅞ x 13⅛

§ See S16-14 Clause 27.1.7 for seismic applications.

Sections highlighted in yellow are generally readily available.

^, x, y See "Bending Resistances" in the previous section.

‡ Class 4

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)

ASTM A992, A572 Grade 50

F_y = 345 MPa

ϕ = 0.90

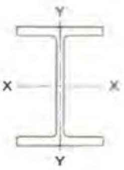
Designation		W310							
Mass (kg/m)		375	342	313	283	253	226	202	179
Effective length (KL) in millimetres with respect to the least radius of gyration	0	14 800	13 600	12 400	11 200	10 000	8 970	8 010	7 070
	500	14 800	13 600	12 400	11 200	9 990	8 960	8 000	7 060
	1 000	14 700	13 500	12 300	11 100	9 950	8 920	7 960	7 030
	1 500	14 600	13 400	12 200	11 000	9 840	8 820	7 870	6 940
	2 000	14 400	13 100	12 000	10 800	9 650	8 650	7 720	6 800
	2 500	14 000	12 800	11 700	10 500	9 390	8 410	7 500	6 610
	3 000	13 500	12 400	11 300	10 100	9 040	8 090	7 210	6 350
	3 500	13 000	11 800	10 800	9 700	8 630	7 720	6 870	6 040
	4 000	12 300	11 200	10 200	9 190	8 160	7 290	6 480	5 700
	4 500	11 600	10 600	9 610	8 640	7 660	6 840	6 070	5 330
	5 000	10 900	9 920	8 990	8 070	7 150	6 370	5 650	4 950
	5 500	10 100	9 240	8 360	7 500	6 630	5 910	5 230	4 580
	6 000	9 420	8 570	7 750	6 940	6 130	5 460	4 830	4 220
	6 500	8 720	7 930	7 160	6 410	5 650	5 030	4 440	3 880
	7 000	8 060	7 320	6 600	5 900	5 200	4 620	4 080	3 560
	7 500	7 440	6 750	6 080	5 440	4 790	4 250	3 750	3 270
	8 000	6 860	6 230	5 600	5 000	4 400	3 910	3 440	3 000
	8 500	6 330	5 740	5 160	4 610	4 050	3 590	3 170	2 760
	9 000	5 850	5 300	4 760	4 250	3 730	3 310	2 910	2 540
	9 500	5 400	4 900	4 400	3 920	3 440	3 050	2 690	2 340
	10 000	5 000	4 530	4 070	3 630	3 180	2 820	2 480	2 160
	10 500	4 640	4 200	3 770	3 360	2 940	2 610	2 290	1 990
	11 000	4 300	3 900	3 500	3 110	2 730	2 420	2 120	1 850
	11 500	4 000	3 620	3 250	2 890	2 530	2 240	1 970	1 710
	12 000	3 730	3 370	3 020	2 690	2 360	2 090	1 830	1 590
	12 500	3 480	3 150	2 820	2 510	2 200	1 940	1 710	1 480
	13 000	3 250	2 940	2 630	2 340	2 050	1 810	1 590	1 380
	13 500	3 040	2 750	2 460	2 190	1 920	1 700	1 490	1 290
	14 000	2 850	2 580	2 310	2 050	1 800	1 590	1 400	1 210
PROPERTIES AND DESIGN DATA									
Area (mm^2)		47 800	43 700	39 900	36 000	32 300	28 800	25 700	22 800
t (mm)		57.2	52.6	48.3	44.1	39.6	35.6	31.8	28.1
r_x (mm)		154	152	150	148	146	144	142	140
r_y (mm)		84.8	84.2	83.3	82.6	81.6	81.0	80.2	79.5
r_x / r_y		1.82	1.81	1.80	1.79	1.79	1.78	1.77	1.76
$\phi S_x F_y$ (kN·m)									
$\phi Z_x F_y$ (kN·m)		2 170	1 970	1 780	1 580	1 390	1 230	1 090	947
L_u (mm)		9 620	8 980	8 350	7 760	7 180	6 690	6 220	5 820
$\phi S_y F_y$ (kN·m)									
$\phi Z_y F_y$ (kN·m)		997	904	814	727	640	568	500	435
§ $(b_{el}/t)\sqrt{350}$		54.0	58.3	62.9	68.3	75.4	83.3	92.7	104
§ $(h/w)\sqrt{350}$		146	159	173	193	212	234	258	288
IMPERIAL SIZE AND WEIGHT									
Weight (lb/ft)		252	230	210	190	170	152	136	120
Depth x Width (in.)		15⅜ x 13	15 x 12⅞	14¾ x 12¾	14⅜ x 12⅝	14 x 12⅝	13¾ x 12½	13⅜ x 12⅜	13⅛ x 12⅜

§ See S16-14 Clause 27.1.7 for seismic applications.

Sections highlighted in yellow are generally readily available.

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)



ASTM A992, A572 Grade 50

F_y = 345 MPa

ϕ = 0.90

Designation		W310						W310	
Mass (kg/m)		158	143	129	118	107	†† 97	86	79
Effective length (KL) in millimetres with respect to the least radius of gyration	0	6 220	5 660	5 130	4 650	4 230	3 830	3 410	3 120
	500	6 220	5 650	5 120	4 640	4 220	3 820	3 410	3 110
	1 000	6 190	5 620	5 090	4 620	4 200	3 800	3 380	3 080
	1 500	6 110	5 560	5 030	4 560	4 150	3 760	3 310	3 020
	2 000	5 990	5 440	4 920	4 460	4 060	3 670	3 190	2 910
	2 500	5 810	5 280	4 770	4 320	3 930	3 560	3 030	2 760
	3 000	5 580	5 070	4 580	4 150	3 770	3 410	2 840	2 580
	3 500	5 300	4 810	4 350	3 930	3 570	3 230	2 620	2 380
	4 000	5 000	4 530	4 090	3 700	3 360	3 030	2 390	2 170
	4 500	4 670	4 240	3 820	3 450	3 130	2 830	2 170	1 960
	5 000	4 340	3 930	3 540	3 200	2 900	2 620	1 960	1 770
	5 500	4 010	3 630	3 270	2 950	2 670	2 410	1 760	1 590
	6 000	3 690	3 340	3 000	2 710	2 450	2 210	1 580	1 430
	6 500	3 390	3 070	2 760	2 480	2 250	2 030	1 420 y	1 280 y
	7 000	3 110	2 820	2 530	2 280	2 060	1 850	1 280	1 150
	7 500	2 850	2 580	2 310	2 080	1 880	1 700	1 150	1 040
	8 000	2 620	2 370	2 120	1 910	1 720	1 550	1 040	937
	8 500	2 400	2 170	1 950	1 750	1 580	1 420	944	849
	9 000	2 210	2 000	1 790	1 610	1 450	1 310	859	772
	9 500	2 030	1 840	1 650	1 480	1 340	1 200	783	704
	10 000	1 880	1 700	1 520	1 360	1 230	1 110	716	644
	10 500	1 730	1 570	1 400	1 260	1 140	1 020	657	590
	11 000	1 600	1 450	1 300	1 170	1 050	945	605	543
	11 500	1 490	1 340	1 200	1 080	974	876	558	501
	12 000	1 380	1 250	1 120	1 000	905	814	516	463
	12 500	1 290	1 160	1 040	934	842	757	478	429
	13 000	1 200	1 090	970	871	785	706		
	13 500	1 120	1 010	906	814	734	660		
	14 000	1 050	950	849	762	687	618		
PROPERTIES AND DESIGN DATA									
Area (mm²)		20 100	18 200	16 500	15 000	13 600	12 300	11 000	10 100
t (mm)		25.1	22.9	20.6	18.7	17.0	15.4	16.3	14.6
r_x (mm)		139	138	137	136	135	134	134	133
r_y (mm)		78.9	78.6	78.0	77.6	77.2	76.9	63.6	63.0
r_x / r_y		1.76	1.76	1.76	1.75	1.75	1.74	2.11	2.11
$\phi S_x F_y$ (kN·m)							447		
$\phi Z_x F_y$ (kN·m)		829	751	671	605	546		441	397
L_u (mm)		5 480	5 270	5 080	4 920	4 800	4 970	3 900	3 810
$\phi S_y F_y$ (kN·m)							148	^ 109	^ 97.5
$\phi Z_y F_y$ (kN·m)		379	345	308	277	250		^ 165	^ 148
§ $(b_{el}/t)\sqrt{350}$		116	126	140	154	168	185	146	163
§ $(h/w)\sqrt{350}$		334	370	395	435	475	524	570	588
IMPERIAL SIZE AND WEIGHT									
Weight (lb/ft)		106	96	87	79	72	65	58	53
Depth x Width (in.)		12⅞ x 12¼	12¾ x 12⅛	12½ x 12⅛	12⅜ x 12⅛	12¼ x 12	12⅛ x 12	12¼ x 10	12 x 10

§ See S16-14 Clause 27.1.7 for seismic applications.

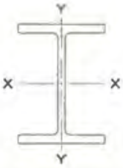
^, y See "Bending Resistances" in the previous section.

Sections highlighted in yellow are generally readily available.

†† Class 3 in bending about either axis due to flange

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)



ASTM A992, A572 Grade 50

F_y = 345 MPa

ϕ = 0.90

Designation		W310			W250				
Mass (kg/m)		74	67	‡ 60	167	149	131	115	101
Effective length (KL) in millimetres with respect to the least radius of gyration	0	2 930	2 620	2 320	6 620	5 890	5 190	4 540	4 000
	500	2 920	2 610	**2 320**	6 610	5 880	5 180	4 530	4 000
	1 000	2 870	2 570	2 270	6 560	5 840	5 140	4 500	3 960
	1 500	2 760	2 470	2 180	6 440	5 730	5 040	4 410	3 880
	2 000	2 580	2 310	2 040	6 250	5 560	4 890	4 270	3 760
	2 500	2 360	2 110	1 870	5 990	5 310	4 670	4 080	3 580
	3 000	2 120	1 890	1 670	5 660	5 010	4 400	3 840	3 370
	3 500	1 880	1 670	1 470	5 280	4 670	4 090	3 560	3 130
	4 000	1 640	1 460	1 290	4 870	4 310	3 770	3 280	2 870
	4 500	1 430	1 270	1 120	4 460	3 940	3 440	2 990	2 610
	5 000	1 250 [y]	1 110 [y]	977	4 060	3 580	3 120	2 710	2 370
	5 500	1 090	968	853 [y]	3 690	3 240	2 830	2 450	2 140
	6 000	959	848	747	3 340	2 930	2 550	2 210	1 930
	6 500	844	747	657	3 020	2 650	2 310	1 990	1 740
	7 000	747	660	581	2 730	2 400	2 080	1 800	1 570
	7 500	664	587	516	2 480	2 170	1 880	1 630	1 420
	8 000	594	524	462	2 250	1 970	1 710	1 480	1 280
	8 500	533	470	415	2 040	1 790	1 550	1 340	1 160
	9 000	480	424	374	1 860	1 630	1 420	1 220	1 060
	9 500	435	384	339	1 710	1 490	1 290	1 110	968
	10 000				1 560	1 370	1 180	1 020	886
	10 500				1 440	1 260	1 090	938	814
	11 000				1 320	1 160	1 000	864	749
	11 500				1 220	1 070	926	797	691
	12 000				1 130	990	857	738	640
	12 500				1 050	919	796	685	594
	13 000				979	855	740	637	552
	13 500				913				
	14 000								
PROPERTIES AND DESIGN DATA									
Area (mm^2)		9 480	8 520	7 610	21 200	19 000	16 700	14 600	12 900
t (mm)		16.3	14.6	13.1	31.8	28.4	25.1	22.1	19.6
r_x (mm)		132	131	130	119	117	115	114	113
r_y (mm)		49.9	49.5	49.3	68.1	67.4	66.8	66.2	65.6
r_x / r_y		2.65	2.65	2.64	1.75	1.74	1.72	1.72	1.72
$\phi S_x F_y$ (kN·m)				^ 261					
$\phi Z_x F_y$ (kN·m)		366	326	^ 290	755	661	574	497	435
L_u (mm)		3 100	3 020	2 960	5 900	5 480	5 080	4 740	4 470
$\phi S_y F_y$ (kN·m)		^ 71.1	^ 63.0	^ 55.9					
$\phi Z_y F_y$ (kN·m)		^ 109	^ 96.3	^ 85.4	354	311	270	234	204
§ $(b_{el}/t)\sqrt{350}$		118	131	145	78.0	86.6	97.3	110	123
§ $(h/w)\sqrt{350}$		552	609	690	220	244	273	312	353
IMPERIAL SIZE AND WEIGHT									
Weight (lb/ft)		50	45	40	112	100	88	77	68
Depth x Width (in.)		12¼ x 8⅛	12 x 8	12 x 8	11⅜ x 10⅜	11⅛ x 10⅜	10⅞ x 10¼	10⅝ x 10¼	10⅜ x 10⅛

§ See S16-14 Clause 27.1.7 for seismic applications.

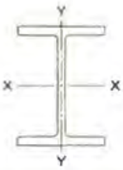
^, y See "Bending Resistances" in the previous section.

Sections highlighted in yellow are generally readily available.

‡ Class 4

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)



ASTM A992, A572 Grade 50

F_y = 345 MPa

ϕ = 0.90

Designation	W250			W250		
Mass (kg/m)	89	80	73	67	58	†† 49
Effective length (KL) in millimetres with respect to the least radius of gyration						
0	3 540	3 170	2 880	2 660	2 300	1 940
500	3 540	3 160	2 880	2 650	2 300	1 930
1 000	3 510	3 140	2 850	2 600	2 260	1 900
1 500	3 440	3 070	2 790	2 510	2 170	1 820
2 000	3 320	2 970	2 700	2 360	2 040	1 700
2 500	3 170	2 830	2 570	2 170	1 870	1 560
3 000	2 970	2 660	2 410	1 950	1 680	1 390
3 500	2 750	2 460	2 230	1 730	1 490	1 230
4 000	2 530	2 260	2 040	1 530	1 310	1 070
4 500	2 300	2 050	1 860	1 340	1 140	934
5 000	2 080	1 860	1 680	1 170	998	813
5 500	1 880	1 670	1 510	1 020	873	710
6 000	1 690	1 510	1 360	899	766	621
6 500	1 520	1 360	1 220	793	675	547
7 000	1 370	1 220	1 100	703	598	483
7 500	1 240	1 110	996	626	532	430
8 000	1 120	1 000	901	559	475	384
8 500	1 020	908	818	502	427	344
9 000	926	827	744	453	385	310
9 500	845	755	679	411	349	281
10 000	774	691	621	374	317	
10 500	710	634	570			
11 000	654	584	525			
11 500	604	539	484			
12 000	559	498	448			
12 500	518	462	416			
13 000	482	430				
13 500						
14 000						
PROPERTIES AND DESIGN DATA						
Area (mm^2)	11 400	10 200	9 290	8 580	7 420	6 260
t (mm)	17.3	15.6	14.2	15.7	13.5	11.0
r_x (mm)	112	111	110	110	108	106
r_y (mm)	65.1	65.0	64.6	51.0	50.4	49.2
r_x / r_y	1.72	1.71	1.70	2.16	2.14	2.15
$\phi S_x F_y$ (kN·m)						178
$\phi Z_x F_y$ (kN·m)	382	338	306	280	239	
L_u (mm)	4 260	4 130	4 010	3 260	3 130	3 160
$\phi S_y F_y$ (kN·m)						46.6
$\phi Z_y F_y$ (kN·m)	178	159	144	103	87.9	
§ $(b_{el}/t)\sqrt{350}$	138	153	167	122	141	172
§ $(h/w)\sqrt{350}$	394	447	489	474	526	569
IMPERIAL SIZE AND WEIGHT						
Weight (lb/ft)	60	54	49	45	39	33
Depth x Width (in.)	10¼ x 10⅛	10⅛ x 10	10 x 10	10⅛ x 8	9⅞ x 8	9¾ x 8

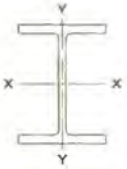
§ See S16-14 Clause 27.1.7 for seismic applications.

†† Class 3 in bending about either axis due to flange

Sections highlighted in yellow are generally readily available.

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)



ASTM A992, A572 Grade 50

F_y = 345 MPa

ϕ = 0.90

Designation		W200					
Mass (kg/m)		100	86	71	59	52	†† 46
Effective length (KL) in millimetres with respect to the least radius of gyration	0	3 930	3 430	2 830	2 350	2 070	1 820
	500	3 920	3 420	2 820	2 340	2 060	1 810
	1 000	3 870	3 370	2 780	2 300	2 030	1 780
	1 500	3 740	3 260	2 680	2 220	1 960	1 720
	2 000	3 550	3 080	2 540	2 100	1 840	1 620
	2 500	3 290	2 860	2 340	1 930	1 700	1 490
	3 000	2 990	2 600	2 130	1 750	1 540	1 340
	3 500	2 690	2 320	1 900	1 560	1 370	1 190
	4 000	2 390	2 060	1 680	1 380	1 210	1 050
	4 500	2 110	1 820	1 480	1 210	1 060	919
	5 000	1 860	1 600	1 300	1 060	928	804
	5 500	1 640	1 410	1 140	930	815	705
	6 000	1 440	1 240	1 010	819	717	619
	6 500	1 280	1 100	892	723	633	546
	7 000	1 140	976	792	642	561	484
	7 500	1 010	871	706	572	500	431
	8 000	909	780	632	512	447	386
	8 500	819	702	569	460	402	346
	9 000	740	634	514	415	363	313
	9 500	671	575	466	376	329	283
	10 000	611	524	424	342	299	258
	10 500	559	479	387			
	11 000						
	11 500						
	12 000						
	12 500						
	13 000						
	13 500						
	14 000						

PROPERTIES AND DESIGN DATA

Area (mm^2)	12 700	11 000	9 100	7 550	6 650	5 890
t (mm)	23.7	20.6	17.4	14.2	12.6	11.0
r_x (mm)	94.5	92.6	91.7	89.9	89.0	88.1
r_y (mm)	53.8	53.3	52.8	52.0	51.8	51.2
r_x / r_y	1.76	1.74	1.74	1.73	1.72	1.72
$\phi S_x F_y$ (kN·m)						139
$\phi Z_x F_y$ (kN·m)	357	305	249	203	177	
L_u (mm)	4 460	4 110	3 730	3 430	3 300	3 370
$\phi S_y F_y$ (kN·m)						46.9
$\phi Z_y F_y$ (kN·m)	165	142	116	94.1	82.6	
§ $(b_{el}/t)\sqrt{350}$	82.9	94.9	111	135	151	173
§ $(h/w)\sqrt{350}$	234	260	332	373	428	470

IMPERIAL SIZE AND WEIGHT

Weight (lb/ft)	67	58	48	40	35	31
Depth x Width (in.)	9 x 8¼	8¾ x 8¼	8½ x 8⅛	8¼ x 8⅛	8⅛ x 8	8 x 8

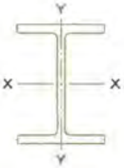
§ See S16-14 Clause 27.1.7 for seismic applications.

†† Class 3 in bending about either axis due to flange

Sections highlighted in yellow are generally readily available.

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)



ASTM A992, A572 Grade 50

F_y = 345 MPa

ϕ = 0.90

Designation	W200		W200		W150		
Mass (kg/m)	42	36	31	27	37	30	‡ 22
Effective length (KL) in millimetres with respect to the least radius of gyration							
0	1 650	1 420	1 240	1 050	1 470	1 180	842
500	1 640	1 410	1 230	1 040	1 460	1 170	836
1 000	1 590	1 370	1 160	979	1 410	1 130	804
1 500	1 490	1 280	1 030	866	1 310	1 050	740
2 000	1 350	1 160	874	726	1 170	930	652
2 500	1 190	1 020	716	591	1 010	801	556
3 000	1 020	872	$\underline{579}^{y}$	$\underline{475}^{y}$	854	677	465
3 500	867	740	469	383	717	567	387
4 000	735	$\underline{627}^{y}$	382	311	601	475	325
4 500	624	531	315	256	506	399	274
5 000	532	453	263	213	429	338	233
5 500	457	389	222	180	366	289	199
6 000	395	336	190	153	316	248	172
6 500	344	292			274	216	149
7 000	301	256			240	188	131
7 500	266	226			211	166	
8 000	236	201					
8 500							
9 000							
9 500							
10 000							
10 500							
11 000							
11 500							
12 000							
12 500							
13 000							
13 500							
14 000							
PROPERTIES AND DESIGN DATA							
Area (mm^2)	5 320	4 570	3 970	3 390	4 740	3 790	2 860
t (mm)	11.8	10.2	10.2	8.40	11.6	9.30	6.60
r_x (mm)	87.7	86.7	88.6	87.3	68.5	67.3	65.1
r_y (mm)	41.2	40.9	32.0	31.2	38.7	38.3	36.9
r_x / r_y	2.13	2.12	2.77	2.80	1.77	1.76	1.76
$\phi S_x F_y$ (kN·m)							‡ 46.2
$\phi Z_x F_y$ (kN·m)	138	118	104	86.6	96.3	75.8	
L_u (mm)	2 610	2 510	1 980	1 890	2 630	2 440	2 480
$\phi S_y F_y$ (kN·m)		^ 28.8	^ 19.0	^ 15.4			‡ 13.8
$\phi Z_y F_y$ (kN·m)	51.2	^ 43.8	^ 29.1	^ 23.6	43.5	34.5	
§ $(b_{el}/t)\sqrt{350}$	132	151	123	148	124	154	215
§ $(h/w)\sqrt{350}$	471	545	554	614	321	392	448
IMPERIAL SIZE AND WEIGHT							
Weight (lb/ft)	28	24	21	18	25	20	15
Depth x Width (in.)	8 x 6½	7⅞ x 6½	8¼ x 5¼	8⅛ x 5¼	6⅜ x 6⅛	6¼ x 6	6 x 6

§ See S16-14 Clause 27.1.7 for seismic applications.

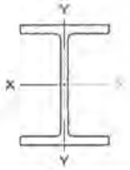
^, y See "Bending Resistances" in the previous section.

Sections highlighted in yellow are generally readily available.

‡ Class 4. See "Bending Resistances".

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)



ASTM A913 Grade 65

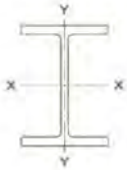
F_y = 450 MPa

ϕ = 0.90

Designation	W360					
Mass (kg/m)	1299	1202	1086	990	900	818
Effective length (KL) in millimetres with respect to the least radius of gyration: 0	67 000	62 000	56 100	51 100	46 500	42 300
2 500	65 000	60 100	54 300	49 400	44 900	40 700
3 000	63 800	58 900	53 200	48 300	43 900	39 800
3 500	62 300	57 500	51 800	47 000	42 700	38 600
4 000	60 600	55 800	50 200	45 500	41 300	37 300
4 500	58 500	53 800	48 300	43 700	39 700	35 800
5 000	56 300	51 700	46 300	41 800	37 900	34 100
5 500	53 800	49 400	44 100	39 800	36 000	32 400
6 000	51 300	47 000	41 900	37 700	34 100	30 600
6 500	48 700	44 500	39 600	35 600	32 200	28 800
7 000	46 100	42 100	37 300	33 500	30 300	27 100
7 500	43 500	39 700	35 100	31 500	28 400	25 400
8 000	41 000	37 300	33 000	29 500	26 600	23 700
8 500	38 600	35 100	30 900	27 600	24 900	22 200
9 000	36 300	32 900	29 000	25 900	23 300	20 700
9 500	34 100	30 900	27 100	24 200	21 800	19 300
10 000	32 000	29 000	25 400	22 600	20 400	18 100
10 500	30 000	27 200	23 800	21 200	19 100	16 900
11 000	28 200	25 500	22 300	19 800	17 800	15 800
11 500	26 500	24 000	20 900	18 600	16 700	14 800
12 000	24 900	22 500	19 600	17 400	15 700	13 800
12 500	23 500	21 200	18 400	16 400	14 700	13 000
13 000	22 100	19 900	17 300	15 400	13 800	12 200
13 500	20 800	18 800	16 300	14 500	13 000	11 500
14 000	19 600	17 700	15 400	13 600	12 200	10 800
15 000	17 500	15 800	13 700	12 100	10 900	9 590
16 000	15 700	14 100	12 300	10 900	9 730	8 570
17 000	14 200	12 700	11 000	9 750	8 740	7 690
18 000	12 800	11 500	9 960	8 800	7 890	6 940
19 000	11 600	10 400	9 030	7 970	7 140	6 280
20 000	10 600	9 500	8 220	7 250	6 500	5 710
PROPERTIES AND DESIGN DATA						
Area (mm^2)	165 000	153 000	139 000	126 000	115 000	105 000
t (mm)	140	130	125	115	106	97.0
r_x (mm)	214	208	207	203	198	194
r_y (mm)	124	122	119	117	116	114
r_x / r_y	1.73	1.70	1.74	1.74	1.71	1.70
M_{rx} (kN·m) ($L < L_u$)	13 400	12 200	11 000	9 840	8 750	7 820
L_u (mm)	18 900	17 900	16 300	15 100	14 100	13 100
M_{ry} (kN·m)	6 760	6 160	5 430	4 860	4 330	3 870
$(b_{el}/t)\sqrt{450}$	36.1	38.4	38.5	41.3	44.2	47.8
$(h/w)\sqrt{450}$	67.9	71.5	86.8	94.4	103	112
IMPERIAL SIZE AND WEIGHT						
Weight (lb/ft)	873	808	730	665	605	550
Depth x Width (in.)	23⅝ x 18¾	22⅞ x 18½	22⅜ x 17⅞	21⅝ x 17⅝	20⅞ x 17⅜	20¼ x 17¼

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)



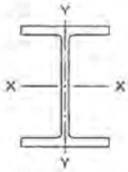
ASTM A913 Grade 65

F_y = 450 MPa

ϕ = 0.90

Designation	W360						
Mass (kg/m)	744	677	634	592	551	509	463
Effective length (KL) in millimetres with respect to the least radius of gyration							
0	38 400	35 000	32 700	30 600	28 400	26 300	23 900
2 500	36 900	33 600	31 400	29 300	27 200	25 200	22 800
3 000	36 000	32 800	30 600	28 600	26 500	24 500	22 200
3 500	35 000	31 800	29 700	27 700	25 600	23 700	21 500
4 000	33 700	30 600	28 600	26 600	24 600	22 800	20 600
4 500	32 300	29 300	27 300	25 400	23 500	21 700	19 700
5 000	30 700	27 900	25 900	24 100	22 300	20 600	18 600
5 500	29 100	26 400	24 500	22 800	21 000	19 500	17 600
6 000	27 500	24 900	23 100	21 400	19 800	18 300	16 500
6 500	25 800	23 300	21 700	20 100	18 500	17 100	15 400
7 000	24 200	21 900	20 300	18 800	17 300	16 000	14 400
7 500	22 600	20 400	18 900	17 500	16 100	14 900	13 400
8 000	21 100	19 100	17 700	16 300	15 000	13 900	12 500
8 500	19 700	17 800	16 500	15 200	14 000	12 900	11 600
9 000	18 400	16 600	15 300	14 200	13 000	12 000	10 800
9 500	17 200	15 400	14 300	13 200	12 100	11 200	10 000
10 000	16 000	14 400	13 300	12 300	11 300	10 400	9 340
10 500	15 000	13 400	12 400	11 400	10 500	9 710	8 700
11 000	14 000	12 600	11 600	10 700	9 790	9 060	8 110
11 500	13 100	11 700	10 800	9 980	9 140	8 460	7 570
12 000	12 200	11 000	10 100	9 330	8 540	7 910	7 080
12 500	11 500	10 300	9 490	8 740	8 000	7 400	6 620
13 000	10 800	9 660	8 900	8 190	7 500	6 940	6 210
13 500	10 100	9 070	8 360	7 690	7 040	6 510	5 820
14 000	9 510	8 530	7 860	7 230	6 610	6 120	5 470
15 000	8 450	7 580	6 980	6 420	5 870	5 430	4 850
16 000	7 550	6 760	6 230	5 720	5 230	4 840	4 320
17 000	6 770	6 070	5 580	5 130	4 690	4 340	3 880
18 000	6 100	5 470	5 030	4 620	4 220	3 910	3 490
19 000	5 520	4 950	4 550	4 180	3 820	3 530	3 160
20 000	5 020	4 500	4 140	3 800	3 470	3 210	2 870
PROPERTIES AND DESIGN DATA							
Area (mm^2)	94 800	86 500	80 600	75 500	70 300	65 200	59 000
t (mm)	88.9	81.5	77.1	72.3	67.6	62.7	57.4
r_x (mm)	190	186	184	182	180	178	175
r_y (mm)	112	111	110	109	108	108	107
r_x / r_y	1.70	1.68	1.67	1.67	1.67	1.65	1.64
M_{rx} (kN·m) ($L < L_u$)	6 970	6 200	5 750	5 310	4 900	4 460	4 000
L_u (mm)	12 100	11 300	10 800	10 200	9 630	9 140	8 520
M_{ry} (kN·m)	3 460	3 110	2 880	2 660	2 450	2 250	2 020
$(b_{el}/t)\sqrt{450}$	51.5	55.7	58.3	61.8	65.6	70.4	76.1
$(h/w)\sqrt{450}$	122	133	143	151	162	174	190
IMPERIAL SIZE AND WEIGHT							
Weight (lb/ft)	500	455	426	398	370	342	311
Depth x Width (in.)	19⅝ x 17	19 x 16⅞	18⅝ x 16¾	18¼ x 16⅝	17⅞ x 16½	17½ x 16⅜	17⅛ x 16¼

W COLUMNS
Factored Axial Compressive Resistances, C_r (kN)

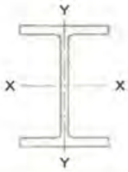


ASTM A913 Grade 65
F_y = 450 MPa
ϕ = 0.90

Designation		W360					
Mass (kg/m)		421	382	347	314	287	262
Effective length (KL) in millimetres with respect to the least radius of gyration	0	21 800	19 700	17 900	16 200	14 800	13 600
	2 500	20 800	18 800	17 100	15 400	14 100	12 900
	3 000	20 200	18 300	16 600	14 900	13 700	12 500
	3 500	19 500	17 700	16 000	14 400	13 200	12 000
	4 000	18 700	16 900	15 300	13 800	12 600	11 500
	4 500	17 800	16 100	14 600	13 100	12 000	10 900
	5 000	16 900	15 200	13 800	12 300	11 300	10 300
	5 500	15 900	14 300	12 900	11 600	10 600	9 650
	6 000	14 900	13 400	12 100	10 800	9 940	9 010
	6 500	14 000	12 500	11 300	10 100	9 270	8 390
	7 000	13 000	11 700	10 500	9 390	8 610	7 790
	7 500	12 100	10 900	9 750	8 710	8 000	7 220
	8 000	11 300	10 100	9 050	8 080	7 410	6 690
	8 500	10 500	9 370	8 400	7 490	6 870	6 200
	9 000	9 710	8 700	7 790	6 950	6 370	5 750
	9 500	9 030	8 080	7 230	6 440	5 910	5 330
	10 000	8 400	7 510	6 720	5 980	5 490	4 940
	10 500	7 820	6 990	6 250	5 560	5 100	4 590
	11 000	7 280	6 510	5 820	5 180	4 750	4 270
	11 500	6 800	6 070	5 430	4 830	4 430	3 980
	12 000	6 350	5 670	5 070	4 500	4 130	3 720
	12 500	5 940	5 300	4 740	4 210	3 860	3 470
	13 000	5 570	4 970	4 430	3 940	3 620	3 250
	13 500	5 220	4 660	4 160	3 690	3 390	3 040
	14 000	4 910	4 380	3 910	3 470	3 180	2 860
	15 000	4 350	3 880	3 460	3 070	2 820	2 530
	16 000	3 870	3 450	3 080	2 730	2 510	2 250
	17 000	3 470	3 090	2 760	2 450	2 250	2 010
	18 000	3 120	2 780	2 480	2 200	2 020	1 810
	19 000	2 820	2 520	2 240	1 990	1 830	1 640
	20 000	2 570	2 290	2 040	1 810	1 660	1 490
PROPERTIES AND DESIGN DATA							
Area (mm^2)		53 700	48 800	44 200	40 000	36 600	33 400
t (mm)		52.6	48.0	43.7	39.6	36.6	33.3
r_x (mm)		172	170	168	166	165	163
r_y (mm)		106	105	104	103	103	102
r_x / r_y		1.62	1.62	1.62	1.61	1.60	1.60
M_{rx} (kN·m) ($L < L_u$)		3 600	3 220	2 890	2 580	2 350	2 130
L_u (mm)		8 010	7 520	7 110	6 710	6 430	6 160
M_{ry} (kN·m)		1 820	1 630	1 470	1 310	1 200	1 090
$(b_{el}/t)\sqrt{450}$		82.5	89.7	98.1	107	116	127
$(h/w)\sqrt{450}$		207	228	249	272	300	322
IMPERIAL SIZE AND WEIGHT							
Weight (lb/ft)		283	257	233	211	193	176
Depth x Width (in.)		16¾ x 16⅛	16⅜ x 16	16 x 15⅞	15¾ x 15¾	15½ x 15¾	15¼ x 15⅝

W COLUMNS

Factored Axial Compressive Resistances, C_r (kN)



ASTM A913 Grade 65

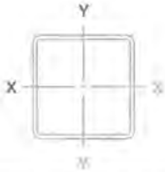
F_y = 450 MPa

ϕ = 0.90

Designation		W360		W360			
Mass (kg/m)		237	216	196	179	†† 162	†† 147
Effective length (KL) in millimetres with respect to the least radius of gyration	0	12 200	11 200	10 100	9 240	8 360	7 610
	500	12 200	11 100	10 100	9 240	8 350	7 600
	1 000	12 100	11 100	10 100	9 200	8 310	7 570
	1 500	12 000	11 000	9 980	9 100	8 220	7 490
	2 000	11 800	10 800	9 810	8 940	8 080	7 350
	2 500	11 600	10 600	9 550	8 710	7 860	7 150
	3 000	11 200	10 300	9 220	8 400	7 590	6 900
	3 500	10 800	9 880	8 830	8 040	7 260	6 590
	4 000	10 300	9 430	8 370	7 620	6 880	6 240
	4 500	9 810	8 940	7 880	7 170	6 470	5 870
	5 000	9 250	8 420	7 380	6 710	6 050	5 480
	5 500	8 680	7 890	6 870	6 240	5 620	5 100
	6 000	8 100	7 360	6 370	5 780	5 210	4 720
	6 500	7 540	6 840	5 880	5 340	4 810	4 350
	7 000	7 010	6 350	5 430	4 930	4 440	4 010
	7 500	6 500	5 880	5 010	4 540	4 090	3 690
	8 000	6 020	5 440	4 610	4 180	3 770	3 400
	8 500	5 580	5 040	4 250	3 860	3 470	3 130
	9 000	5 170	4 670	3 930	3 560	3 200	2 890
	9 500	4 790	4 330	3 630	3 290	2 960	2 670
	10 000	4 450	4 010	3 350	3 040	2 730	2 460
	10 500	4 130	3 730	3 110	2 810	2 530	2 280
	11 000	3 840	3 460	2 880	2 610	2 350	2 120
	11 500	3 580	3 230	2 680	2 430	2 180	1 970
	12 000	3 340	3 010	2 490	2 260	2 030	1 830
	12 500	3 120	2 810	2 330	2 110	1 890	1 700
	13 000	2 920	2 630	2 170	1 970	1 770	1 590
	13 500	2 740	2 460	2 030	1 840	1 650	1 490
	14 000	2 570	2 310	1 910	1 730	1 550	1 400
	15 000	2 270	2 050	1 680	1 520	1 370	1 230
	16 000	2 020	1 820	1 500	1 350	1 220	1 090
PROPERTIES AND DESIGN DATA							
Area (mm^2)		30 100	27 500	25 000	22 800	20 600	18 800
t (mm)		30.2	27.7	26.2	23.9	21.8	19.8
r_x (mm)		162	161	159	159	158	157
r_y (mm)		102	101	95.6	95.2	94.9	94.3
r_x / r_y		1.59	1.59	1.66	1.67	1.66	1.66
M_{rx} (kN·m) ($L < L_u$)		1 900	1 730	1 560	1 410	1 150	1 040
L_u (mm)		5 900	5 740	5 340	5 200	5 400	5 260
M_{ry} (kN·m)		968	883	753	680	405	366
$(b_{el}/t)\sqrt{450}$		139	151	151	166	181	198
$(h/w)\sqrt{450}$		359	392	413	453	511	553
IMPERIAL SIZE AND WEIGHT							
Weight (lb/ft)		159	145	132	120	109	99
Depth x Width (in.)		15 x 15⅝	14¾ x 15½	14⅝ x 14¾	14½ x 14⅝	14⅜ x 14⅝	14⅛ x 14⅝

†† Class 3 in bending about both axes

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS C
$\phi = 0.90$

Section (mm x mm x mm)	HSS 559 x 559	HSS 508 x 508			
	19*	22*	19*	16*	‡ 13* #
Mass (kg/m)	316	329	285	240	194
Effective length (KL) in millimetres with respect to the least radius of gyration					
0	12 700	13 200	11 400	9 640	7 700
500	12 700	13 200	11 400	9 640	7 700
1 000	12 700	13 200	11 400	9 630	7 700
1 500	12 600	13 200	11 400	9 620	7 690
2 000	12 600	13 200	11 400	9 610	7 670
2 500	12 600	13 100	11 400	9 580	7 650
3 000	12 600	13 100	11 300	9 540	7 630
3 500	12 500	13 000	11 300	9 500	7 590
4 000	12 500	12 900	11 200	9 440	7 540
4 500	12 400	12 800	11 100	9 360	7 490
5 000	12 300	12 700	11 000	9 280	7 420
5 500	12 200	12 500	10 900	9 180	7 340
6 000	12 100	12 400	10 700	9 070	7 250
6 500	11 900	12 200	10 600	8 940	7 160
7 000	11 800	12 000	10 400	8 800	7 050
7 500	11 600	11 800	10 200	8 650	6 930
8 000	11 400	11 600	10 000	8 490	6 810
8 500	11 300	11 300	9 840	8 320	6 670
9 000	11 100	11 100	9 620	8 140	6 530
9 500	10 900	10 800	9 400	7 960	6 390
10 000	10 600	10 500	9 160	7 760	6 230
10 500	10 400	10 300	8 930	7 570	6 080
11 000	10 200	9 990	8 680	7 370	5 920
11 500	9 940	9 710	8 440	7 160	5 760
12 000	9 700	9 420	8 190	6 960	5 600
12 500	9 450	9 140	7 950	6 750	5 440
13 000	9 210	8 860	7 700	6 550	5 270
14 000	8 720	8 300	7 220	6 140	4 950
15 000	8 230	7 760	6 760	5 750	4 640
16 000	7 760	7 250	6 310	5 380	4 340
17 000	7 300	6 760	5 890	5 030	4 060
18 000	6 860	6 300	5 500	4 690	3 800
PROPERTIES AND DESIGN DATA					
Area (mm^2)	40 200	41 900	36 300	30 600	24 700
r (mm)	219	197	198	200	201
M_r (kN·m)	2 540	2 380	2 080	1 770	1 240
$(b_{el}/t)\sqrt{350}$	474	353	424	524	673
IMPERIAL SIZE AND WEIGHT					
Weight (lb./ft.)	212	221	192	162	131
Thickness (in.)	0.750	0.875	0.750	0.625	0.500
Size (in.)	22 x 22	20 x 20			

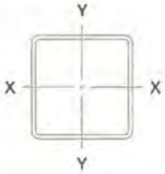
* Imported section

‡ Class 4

\# C_r calculated according to S16-14 Clause 13.3.5(b)

SQUARE HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W CLASS C $\phi = 0.90$

Section (mm x mm x mm)	HSS 457 x 457				HSS 406 x 406				
	22*	19*	16*	† 13*	22*	19*	16*	13*	‡ 9.5*
Mass (kg/m)	294	255	215	174	258	224	190	154	117
Effective length (KL) in millimetres with respect to the least radius of gyration									
0	11 800	10 200	8 630	6 990	10 400	9 010	7 620	6 170	4 370
500	11 800	10 200	8 630	6 990	10 400	9 010	7 620	6 170	4 370
1 000	11 800	10 200	8 620	6 990	10 400	9 000	7 620	6 170	4 370
1 500	11 800	10 200	8 610	6 980	10 300	8 980	7 600	6 160	4 360
2 000	11 700	10 200	8 590	6 960	10 300	8 950	7 580	6 140	4 340
2 500	11 700	10 200	8 560	6 940	10 200	8 910	7 540	6 110	4 320
3 000	11 600	10 100	8 520	6 900	10 200	8 840	7 480	6 060	4 290
3 500	11 500	10 000	8 460	6 860	10 100	8 760	7 410	6 010	4 260
4 000	11 400	9 950	8 390	6 800	9 940	8 660	7 330	5 940	4 210
4 500	11 300	9 840	8 300	6 730	9 800	8 530	7 230	5 860	4 150
5 000	11 200	9 720	8 200	6 660	9 630	8 390	7 110	5 770	4 090
5 500	11 000	9 580	8 090	6 560	9 430	8 230	6 970	5 660	4 010
6 000	10 800	9 430	7 960	6 460	9 220	8 050	6 820	5 540	3 930
6 500	10 600	9 250	7 810	6 350	8 990	7 850	6 660	5 410	3 840
7 000	10 400	9 070	7 660	6 220	8 740	7 640	6 480	5 270	3 740
7 500	10 200	8 870	7 490	6 090	8 480	7 420	6 290	5 130	3 640
8 000	9 910	8 650	7 310	5 950	8 210	7 190	6 100	4 970	3 530
8 500	9 650	8 430	7 130	5 800	7 930	6 950	5 900	4 810	3 420
9 000	9 380	8 200	6 930	5 650	7 650	6 710	5 700	4 650	3 310
9 500	9 100	7 960	6 730	5 490	7 370	6 460	5 490	4 490	3 190
10 000	8 820	7 720	6 530	5 330	7 080	6 220	5 290	4 320	3 080
10 500	8 540	7 480	6 330	5 170	6 800	5 980	5 080	4 160	2 960
11 000	8 250	7 230	6 120	5 000	6 520	5 740	4 880	4 000	2 850
11 500	7 970	6 990	5 920	4 840	6 250	5 500	4 690	3 840	2 730
12 000	7 690	6 750	5 710	4 670	5 990	5 280	4 490	3 690	2 620
12 500	7 410	6 510	5 510	4 510	5 730	5 050	4 310	3 530	2 520
13 000	7 140	6 270	5 320	4 350	5 490	4 840	4 120	3 390	2 410
14 000	6 620	5 820	4 940	4 050	5 020	4 440	3 780	3 110	2 220
15 000	6 120	5 390	4 570	3 750	4 600	4 060	3 470	2 850	2 040
16 000	5 660	4 990	4 240	3 480	4 210	3 720	3 180	2 620	1 870
17 000	5 240	4 620	3 920	3 230	3 860	3 420	2 920	2 410	1 720
18 000	4 850	4 280	3 630	2 990	3 540	3 140	2 680	2 210	1 580
PROPERTIES AND DESIGN DATA									
Area (mm^2)	37 400	32 500	27 400	22 200	32 900	28 600	24 200	19 600	14 900
r (mm)	176	178	179	181	155	157	158	160	161
M_r (kN·m)	1 900	1 660	1 410	995	1 470	1 290	1 100	904	570
$(b_{el}/t)\sqrt{350}$	310	374	464	599	267	324	404	524	723
IMPERIAL SIZE AND WEIGHT									
Weight (lb./ft.)	197	171	144	117	174	151	127	103	78.6
Thickness (in.)	0.875	0.750	0.625	0.500	0.875	0.750	0.625	0.500	0.375
Size (in.)	18 x 18				16 x 16				

* Imported section

† Class 3

‡ Class 4

SQUARE HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W CLASS C $\phi = 0.90$

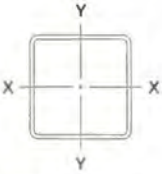
Section (mm x mm x mm)	HSS 356 x 356				HSS 305 x 305				
	16*	13*	† 9.5*	‡ 7.9*	16	13	9.5	† 7.9	‡ 6.4
Mass (kg/m)	164	133	102	85.4	139	113	86.5	72.7	58.7
Effective length (KL) in millimetres with respect to the least radius of gyration									
0	6 580	5 360	4 100	3 040	5 580	4 540	3 470	2 920	1 940
500	6 580	5 350	4 090	3 040	5 570	4 530	3 460	2 920	1 940
1 000	6 570	5 350	4 090	3 030	5 560	4 530	3 460	2 910	1 940
1 500	6 560	5 330	4 080	3 030	5 540	4 510	3 440	2 900	1 930
2 000	6 520	5 310	4 060	3 010	5 500	4 470	3 420	2 880	1 920
2 500	6 480	5 270	4 030	2 990	5 440	4 430	3 380	2 850	1 900
3 000	6 410	5 220	3 990	2 960	5 350	4 360	3 340	2 810	1 870
3 500	6 330	5 150	3 940	2 930	5 250	4 270	3 270	2 760	1 840
4 000	6 220	5 070	3 880	2 880	5 120	4 170	3 200	2 700	1 790
4 500	6 100	4 970	3 810	2 830	4 970	4 050	3 110	2 630	1 750
5 000	5 960	4 860	3 730	2 770	4 800	3 920	3 010	2 550	1 690
5 500	5 810	4 730	3 640	2 700	4 620	3 780	2 910	2 460	1 630
6 000	5 640	4 600	3 530	2 620	4 430	3 620	2 790	2 360	1 570
6 500	5 460	4 450	3 430	2 540	4 230	3 460	2 670	2 260	1 500
7 000	5 260	4 300	3 310	2 460	4 030	3 300	2 550	2 160	1 440
7 500	5 070	4 140	3 190	2 370	3 830	3 140	2 430	2 060	1 370
8 000	4 870	3 980	3 070	2 280	3 630	2 970	2 300	1 960	1 300
8 500	4 660	3 810	2 950	2 190	3 430	2 820	2 190	1 860	1 230
9 000	4 460	3 650	2 820	2 090	3 240	2 660	2 070	1 760	1 170
9 500	4 260	3 490	2 700	2 000	3 060	2 510	1 960	1 660	1 100
10 000	4 060	3 330	2 580	1 910	2 890	2 370	1 850	1 570	1 040
10 500	3 870	3 170	2 460	1 830	2 720	2 240	1 750	1 490	987
11 000	3 690	3 020	2 350	1 740	2 570	2 110	1 650	1 400	933
11 500	3 510	2 880	2 240	1 660	2 420	1 990	1 560	1 330	881
12 000	3 340	2 740	2 130	1 580	2 280	1 880	1 470	1 250	833
12 500	3 180	2 610	2 030	1 510	2 160	1 780	1 390	1 180	787
13 000	3 020	2 480	1 930	1 430	2 040	1 680	1 310	1 120	744
14 000	2 740	2 250	1 750	1 300	1 820	1 500	1 180	1 000	667
15 000	2 480	2 040	1 590	1 180	1 630	1 350	1 060	901	599
16 000	2 260	1 850	1 450	1 080	1 470	1 210	951	812	540
17 000	2 050	1 690	1 320	980	1 320	1 090	859	734	488
18 000	1 870	1 540	1 210	895	1 200	991	779	666	443
PROPERTIES AND DESIGN DATA									
Area (mm^2)	20 900	17 000	13 000	10 900	17 700	14 400	11 000	9 270	7 480
r (mm)	138	139	141	141	117	118	120	121	121
M_r (kN·m)	832	684	454	353	595	491	381	279	197
$(b_{el}/t)\sqrt{350}$	344	449	623	763	284	374	524	643	823
IMPERIAL SIZE AND WEIGHT									
Weight (lb./ft.)	110	89.7	68.4	57.4	93.4	76.1	58.1	48.9	39.4
Thickness (in.)	0.625	0.500	0.375	0.313	0.625	0.500	0.375	0.313	0.250
Size (in.)	14 x 14				12 x 12				

* Imported section

† Class 3

‡ Class 4

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



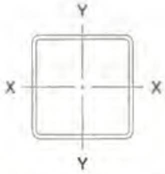
G40.21 350W
CLASS C
$\phi = 0.90$

Section (mm x mm x mm)	HSS 254 x 254					
	16	13	9.5	7.9	‡ 6.4 #	‡ 4.8
Mass (kg/m)	114	93.0	71.3	60.1	48.6	36.9
Effective length (KL) in millimetres with respect to the least radius of gyration						
0	4 570	3 720	2 860	2 410	1 930	1 100
500	4 560	3 710	2 860	2 410	1 930	1 100
1 000	4 550	3 700	2 850	2 400	1 920	1 100
1 500	4 520	3 680	2 830	2 390	1 910	1 090
2 000	4 460	3 630	2 800	2 360	1 890	1 080
2 500	4 380	3 570	2 750	2 320	1 860	1 060
3 000	4 270	3 480	2 690	2 270	1 820	1 040
3 500	4 130	3 380	2 610	2 200	1 770	1 010
4 000	3 970	3 250	2 520	2 120	1 710	972
4 500	3 790	3 110	2 410	2 040	1 640	933
5 000	3 600	2 960	2 300	1 940	1 570	891
5 500	3 400	2 800	2 180	1 840	1 490	846
6 000	3 200	2 640	2 050	1 740	1 410	800
6 500	3 000	2 470	1 930	1 640	1 330	753
7 000	2 810	2 320	1 810	1 530	1 250	708
7 500	2 620	2 160	1 690	1 440	1 170	663
8 000	2 440	2 020	1 580	1 340	1 090	621
8 500	2 270	1 880	1 480	1 250	1 020	580
9 000	2 110	1 750	1 380	1 170	957	542
9 500	1 970	1 640	1 290	1 090	894	506
10 000	1 830	1 530	1 200	1 020	836	473
10 500	1 710	1 420	1 120	954	781	442
11 000	1 600	1 330	1 050	892	731	414
11 500	1 490	1 240	979	834	685	387
12 000	1 390	1 160	917	782	642	363
12 500	1 310	1 090	860	733	602	340
13 000	1 220	1 020	807	688	566	320
14 000	1 080	902	713	609	501	283
15 000	958	801	634	541	445	252
16 000	855	715	566	483	398	225
17 000	766	641	508	434	357	202
18 000	690	578	458	391	322	182
PROPERTIES AND DESIGN DATA						
Area (mm^2)	14 500	11 800	9 090	7 650	6 190	4 710
r (mm)	96.1	97.6	99.1	99.9	101	101
M_r (kN·m)	400	334	260	221	155	96.6
$(b_{el}/t)\sqrt{350}$	224	299	424	524	673	919
IMPERIAL SIZE AND WEIGHT						
Weight (lb./ft.)	76.4	62.5	47.9	40.4	32.6	24.8
Thickness (in.)	0.625	0.500	0.375	0.313	0.250	0.188
Size (in.)	10 x 10					

‡ Class 4

C_r calculated according to S16-14 Clause 13.3.5(b)

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)

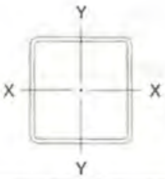


G40.21 350W
CLASS C
$\phi = 0.90$

Section (mm x mm x mm)		HSS 203 x 203					
		16	13	9.5	7.9	6.4	‡ 4.8
Mass (kg/m)		88.3	72.7	56.1	47.4	38.4	29.3
Effective length (KL) in millimetres with respect to the least radius of gyration	0	3 530	2 920	2 250	1 900	1 540	1 100
	500	3 520	2 910	2 250	1 900	1 540	1 100
	1 000	3 500	2 900	2 240	1 890	1 530	1 090
	1 500	3 450	2 860	2 210	1 870	1 520	1 080
	2 000	3 370	2 800	2 160	1 830	1 490	1 060
	2 500	3 260	2 710	2 100	1 770	1 440	1 030
	3 000	3 110	2 590	2 010	1 700	1 390	987
	3 500	2 940	2 450	1 910	1 620	1 320	940
	4 000	2 750	2 300	1 800	1 520	1 240	888
	4 500	2 550	2 140	1 680	1 420	1 160	831
	5 000	2 350	1 980	1 550	1 320	1 080	774
	5 500	2 160	1 820	1 430	1 220	999	716
	6 000	1 980	1 670	1 320	1 120	920	661
	6 500	1 810	1 530	1 210	1 030	846	608
	7 000	1 650	1 400	1 110	947	776	559
	7 500	1 510	1 280	1 010	868	712	513
	8 000	1 380	1 170	929	796	654	471
	8 500	1 260	1 070	853	731	601	433
	9 000	1 150	985	783	672	552	398
	9 500	1 060	905	721	618	509	367
	10 000	974	834	664	570	469	339
	10 500	898	769	613	527	434	313
	11 000	830	712	568	488	402	290
	11 500	769	659	526	452	373	269
	12 000	714	612	489	420	346	250
	12 500	664	570	455	392	323	233
	13 000	619	531	425	365	301	218
	14 000	540	464	371	320	263	191
	15 000	476	409	327	282	232	168
	16 000						149
	17 000						
	18 000						
PROPERTIES AND DESIGN DATA							
Area (mm^2)		11 200	9 260	7 150	6 040	4 900	3 730
r (mm)		75.3	76.9	78.4	79.2	79.9	80.7
M_r (kN·m)		244	205	162	138	113	71.6
$(b_{el}/t)\sqrt{350}$		165	224	324	404	524	720
IMPERIAL SIZE AND WEIGHT							
Weight (lb./ft.)		59.3	48.9	37.7	31.9	25.8	19.7
Thickness (in.)		0.625	0.500	0.375	0.313	0.250	0.188
Size (in.)		8 x 8					

‡ Class 4

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)

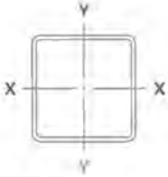


G40.21 350W
CLASS C
$\phi = 0.90$

Section (mm x mm x mm)	HSS 178 x 178						HSS 152 x 152				
	16	13	9.5	7.9	6.4	† 4.8	13	9.5	7.9	6.4	4.8
Mass (kg/m)	75.6	62.6	48.5	41.1	33.4	25.5	52.4	40.9	34.7	28.3	21.7
Effective length (KL) in millimetres with respect to the least radius of gyration											
0	3 040	2 510	1 950	1 650	1 340	1 020	2 100	1 640	1 400	1 140	869
500	3 030	2 510	1 940	1 650	1 340	1 020	2 100	1 640	1 390	1 130	868
1 000	3 000	2 490	1 930	1 630	1 330	1 010	2 070	1 620	1 380	1 120	858
1 500	2 940	2 440	1 890	1 600	1 300	998	2 010	1 570	1 340	1 090	837
2 000	2 840	2 360	1 840	1 560	1 270	971	1 910	1 500	1 280	1 050	802
2 500	2 710	2 250	1 760	1 490	1 220	932	1 780	1 410	1 200	984	755
3 000	2 540	2 120	1 660	1 410	1 150	883	1 630	1 290	1 110	910	700
3 500	2 350	1 970	1 540	1 320	1 080	826	1 480	1 170	1 010	829	639
4 000	2 150	1 810	1 420	1 220	995	766	1 320	1 050	907	748	578
4 500	1 950	1 650	1 300	1 110	913	704	1 170	939	810	670	518
5 000	1 760	1 490	1 180	1 010	833	643	1 040	835	721	597	463
5 500	1 590	1 350	1 070	920	757	585	917	741	641	532	413
6 000	1 430	1 220	971	833	686	531	812	658	570	474	368
6 500	1 290	1 100	877	754	622	482	721	586	508	423	329
7 000	1 160	992	794	683	564	437	643	523	454	378	294
7 500	1 050	897	719	619	511	397	575	468	407	339	264
8 000	946	812	652	562	465	361	516	421	366	305	238
8 500	858	738	593	511	423	329	465	380	330	276	215
9 000	781	672	541	466	386	300	421	344	299	250	195
9 500	712	614	494	427	353	275	382	312	272	227	177
10 000	652	562	453	391	324	252	348	285	248	207	162
10 500	598	516	416	360	298	232	318	261	227	190	148
11 000	550	475	384	331	275	214	292	239	209	174	136
11 500	508	439	354	306	254	198		220	192	161	126
12 000	470	406	328	284	235	183					
12 500	436	377	305	263	219	170					
13 000		351	283	245	203	158					
14 000						138					
15 000											
16 000											
17 000											
18 000											
PROPERTIES AND DESIGN DATA											
Area (mm²)	9 640	7 970	6 180	5 230	4 250	3 250	6 680	5 210	4 430	3 610	2 760
r (mm)	64.9	66.5	68.0	68.8	69.6	70.3	56.1	57.6	58.4	59.2	59.9
M_r (kN·m)	180	152	121	104	85.4	57.0	108	86.6	74.7	61.7	47.9
$(b_{el}/t)\sqrt{350}$	135	187	274	344	449	621	150	224	284	374	522
IMPERIAL SIZE AND WEIGHT											
Weight (lb./ft.)	50.8	42.1	32.6	27.6	22.4	17.1	35.2	27.5	23.3	19.0	14.6
Thickness (in.)	0.625	0.500	0.375	0.313	0.250	0.188	0.500	0.375	0.313	0.250	0.188
Size (in.)	7 x 7						6 x 6				

† Class 3

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



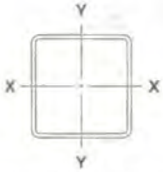
G40.21 350W
CLASS C
$\phi = 0.90$

Section (mm x mm x mm)	HSS 127 x 127					
	13	9.5	7.9	6.4	4.8	‡ 3.2 #
Mass (kg/m)	42.3	33.3	28.4	23.2	17.9	12.2
Effective length (KL) in millimetres with respect to the least radius of gyration						
0	1 700	1 340	1 140	932	718	485
500	1 690	1 330	1 140	929	716	483
1 000	1 650	1 300	1 110	912	703	475
1 500	1 570	1 240	1 070	874	675	457
2 000	1 450	1 160	991	815	631	428
2 500	1 300	1 040	899	742	576	392
3 000	1 140	925	799	661	515	352
3 500	992	808	700	581	454	312
4 000	855	700	608	507	397	273
4 500	736	606	527	440	346	239
5 000	634	524	457	383	301	208
5 500	549	455	398	333	263	182
6 000	478	397	347	292	230	160
6 500	418	349	305	256	203	141
7 000	368	307	269	226	179	125
7 500	326	273	239	201	159	111
8 000	290	243	213	179	142	99
8 500	260	218	191	161	128	89
9 000	234	196	172	145	115	80
9 500			156	131	104	73
10 000						66
10 500						
11 000						
11 500						
12 000						
12 500						
13 000						
14 000						
15 000						
16 000						
17 000						
18 000						
PROPERTIES AND DESIGN DATA						
Area (mm^2)	5 390	4 240	3 620	2 960	2 280	1 550
r (mm)	45.7	47.3	48.0	48.8	49.6	50.3
M_r (kN·m)	70.9	57.6	50.1	41.6	32.4	19.4
$(b_{el}/t)\sqrt{350}$	112	174	224	299	422	672
IMPERIAL SIZE AND WEIGHT						
Weight (lb./ft.)	28.4	22.4	19.1	15.6	12.0	8.17
Thickness (in.)	0.500	0.375	0.313	0.250	0.188	0.125
Size (in.)	5 x 5					

‡ Class 4

C_r calculated according to S16-14 Clause 13.3.5(b)

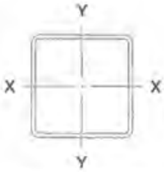
SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS C
$\phi = 0.90$

Section (mm x mm x mm)	HSS 102 x 102						HSS 89 x 89			
	13	9.5	7.9	6.4	4.8	3.2	9.5	7.9	6.4	4.8
Mass (kg/m)	32.2	25.7	22.1	18.2	14.1	9.62	21.9	18.9	15.6	12.2
Effective length (KL) in millimetres with respect to the least radius of gyration										
0	1 290	1 030	885	731	564	387	879	759	627	488
500	1 280	1 030	879	726	560	385	869	751	621	484
1 000	1 230	986	847	700	542	373	820	711	589	461
1 500	1 120	906	781	649	503	348	727	635	529	416
2 000	969	796	691	576	450	312	611	538	451	357
2 500	815	678	591	496	389	271	498	442	373	297
3 000	673	567	497	418	330	231	401	358	304	243
3 500	554	470	414	350	277	195	324	290	247	199
4 000	458	391	346	293	233	164	264	237	202	164
4 500	381	327	290	246	196	139	217	196	167	136
5 000	320	276	245	208	166	118	181	163	140	114
5 500	272	235	209	178	142	101	153	138	118	96
6 000	233	202	179	153	122	87	130	118	101	82
6 500	201	175	156	133	106	76		102	87	71
7 000	176	153	136	116	93	66				
7 500			120	102	82	58				
8 000						52				
8 500										
9 000										
9 500										
10 000										
10 500										
11 000										
11 500										
12 000										
12 500										
13 000										
14 000										
15 000										
16 000										
17 000										
18 000										
PROPERTIES AND DESIGN DATA										
Area (mm^2)	4 100	3 280	2 810	2 320	1 790	1 230	2 790	2 410	1 990	1 550
r (mm)	35.3	36.9	37.7	38.4	39.2	40.0	31.7	32.5	33.2	34.0
M_r (kN·m)	41.3	34.7	30.5	25.6	20.3	14.1	25.4	22.5	19.1	15.2
$(b_{el}/t)\sqrt{350}$	74.8	125	165	224	323	523	99.7	135	187	273
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)	21.6	17.3	14.8	12.2	9.45	6.47	14.7	12.7	10.5	8.17
Thickness (in.)	0.500	0.375	0.313	0.250	0.188	0.125	0.375	0.313	0.250	0.188
Size (in.)	4 x 4						3½ x 3½			

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS C
ϕ = 0.90

Section (mm x mm x mm)	HSS 76 x 76					HSS 64 x 64			HSS 51 x 51		
	9.5	7.9	6.4	4.8	3.2	6.4	4.8	3.2	6.4	4.8	3.2
Mass (kg/m)	18.1	15.7	13.1	10.3	7.09	10.6	8.35	5.82	8.05	6.45	4.55
Effective length (KL) in millimetres with respect to the least radius of gyration											
0	728	633	526	413	284	425	334	233	324	259	183
500	715	623	518	407	281	414	326	228	308	247	175
1 000	652	572	478	378	262	363	289	204	243	199	144
1 500	546	484	408	325	228	286	231	166	169	141	105
2 000	432	387	329	265	187	214	175	127	115	98	73
2 500	334	302	259	210	149	158	131	96	80	69	52
3 000	258	235	203	165	118	119	99	73	58	50	38
3 500	203	185	160	131	95	92	77	57	44	38	29
4 000	162	148	129	106	76	72	61	45			
4 500	131	121	105	86	62	58	49	36			
5 000	108	100	87	72	52						
5 500			73	60	44						
6 000											
6 500											
7 000											
7 500											
8 000											
8 500											
9 000											
9 500											
10 000											
10 500											
11 000											
11 500											
12 000											
12 500											
13 000											
14 000											
15 000											
16 000											
17 000											
18 000											
PROPERTIES AND DESIGN DATA											
Area (mm^2)	2 310	2 010	1 670	1 310	903	1 350	1 060	741	1 030	821	580
r (mm)	26.5	27.3	28.0	28.8	29.6	22.8	23.6	24.4	17.6	18.4	19.2
M_r (kN·m)	17.5	15.7	13.5	10.8	7.72	8.85	7.25	5.23	5.17	4.35	3.21
$(b_{el}/t)\sqrt{350}$	74.8	105	150	223	373	112	174	299	74.8	124	224
IMPERIAL SIZE AND WEIGHT											
Weight (lb./ft.)	12.2	10.6	8.81	6.89	4.76	7.11	5.61	3.91	5.41	4.33	3.06
Thickness (in.)	0.375	0.313	0.250	0.188	0.125	0.250	0.188	0.125	0.250	0.188	0.125
Size (in.)	3 x 3					2½ x 2½			2 x 2		

RECTANGULAR HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W CLASS C $\phi = 0.90$

Section (mm x mm x mm)	HSS 305 x 203					HSS 305 x 152				
	16	13	9.5	† 7.9	‡ 6.4	16	13	9.5	† 7.9	‡ 6.4
Mass (kg/m)	114	93.0	71.3	60.1	48.6	101	82.8	63.7	53.7	43.5
Effective length (KL) in millimetres with respect to the least radius of gyration										
0	4 570	3 720	2 860	2 410	1 740	4 060	3 340	2 560	2 160	1 540
500	4 560	3 710	2 860	2 410	1 740	4 060	3 330	2 550	2 150	1 530
1 000	4 540	3 700	2 850	2 400	1 730	4 010	3 300	2 530	2 130	1 520
1 500	4 490	3 650	2 820	2 370	1 710	3 910	3 220	2 470	2 090	1 490
2 000	4 400	3 580	2 760	2 330	1 680	3 750	3 100	2 380	2 010	**1 440**
2 500	4 270	3 480	2 690	2 270	**1 640**	3 530	2 930	2 260	1 910	1 370
3 000	4 100	3 350	2 590	2 190	1 580	3 280	2 720	2 110	1 790	1 280
3 500	3 900	3 190	2 480	2 090	1 520	2 990	2 500	1 940	1 650	1 180
4 000	3 670	3 020	2 340	1 980	1 440	2 710	2 270	1 770	1 510	1 080
4 500	3 440	2 830	2 200	1 860	1 350	2 430	2 040	1 600	1 360	981
5 000	3 190	2 630	2 060	1 740	1 270	2 170	1 830	1 440	1 230	886
5 500	2 950	2 440	1 910	1 620	1 180	1 940	1 640	1 290	1 100	797
6 000	2 720	2 250	1 770	1 500	1 090	1 730	1 460	1 160	991	716
6 500	2 500	2 070	1 630	1 380	1 010	1 540	1 310	1 040	890	644
7 000	2 290	1 910	1 500	1 270	930	1 380	1 180	932	801	580
7 500	2 100	1 750	1 380	1 170	857	1 240	1 060	839	722	523
8 000	1 930	1 610	1 270	1 080	790	1 120	954	758	652	473
8 500	1 770	1 480	1 170	996	728	1 010	863	686	591	429
9 000	1 630	1 360	1 080	918	672	916	783	624	537	390
9 500	1 500	1 260	995	848	621	834	713	568	490	356
10 000	1 390	1 160	919	784	574	761	652	520	448	325
10 500	1 280	1 070	851	726	532	697	597	476	411	299
11 000	1 190	993	789	673	493	641	549	438	378	275
11 500	1 100	922	733	625	458	590	506	404	349	254
12 000	1 020	858	682	582	427	545	468	374	323	235
12 500	952	799	635	543	398			346	299	217
13 000	889	746	593	507	372					
14 000	778	653	520	444	326					
15 000	686	576	459	392	288					
16 000		511	408	348	256					
PROPERTIES AND DESIGN DATA										
Area (mm^2)	14 500	11 800	9 090	7 650	6 190	12 900	10 600	8 120	6 850	5 540
r_x (mm)	110	111	113	114	115	105	106	108	109	110
r_y (mm)	79.8	81.2	82.7	83.4	84.1	60.1	61.5	62.9	63.7	64.4
r_x / r_y	1.38	1.37	1.37	1.37	1.37	1.75	1.72	1.72	1.71	1.71
M_{rx} (kN·m)	450	375	292	248	^ 202	375	315	247	210	^ 171
M_{ry} (kN·m)	340	283	221	165	116	230	193	152	115	80.5
$(b_{el}/t)\sqrt{350}$	284	374	524	643	823	284	374	524	643	823
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)	76.4	62.5	47.9	40.4	32.6	67.8	55.7	42.8	36.1	29.2
Thickness (in.)	0.625	0.500	0.375	0.313	0.250	0.625	0.500	0.375	0.313	0.250
Size (in.)	12 x 8					12 x 6				

† Class 3 in bending about Y-Y axis

‡ Class 4

^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.

RECTANGULAR HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W
CLASS C
$\phi = 0.90$

Section (mm x mm x mm)		HSS 254 x 203					HSS 254 x 152					
		16	13	9.5	7.9	‡ 6.4	16	13	9.5	7.9	‡ 6.4	‡ 4.8
Mass (kg/m)		101	82.8	63.7	53.7	43.5	88.3	72.7	56.1	47.4	38.4	29.3
Effective length (KL) in millimetres with respect to the least radius of gyration	0	4 060	3 340	2 560	2 160	1 740	3 530	2 920	2 250	1 900	1 540	983
	500	4 060	3 340	2 560	2 160	1 740	3 520	2 910	2 250	1 900	1 540	981
	1 000	4 040	3 320	2 540	2 150	1 730	3 480	2 880	2 230	1 880	**1 520**	972
	1 500	3 990	3 280	2 510	2 120	**1 710**	3 390	2 810	2 170	1 840	1 490	952
	2 000	3 900	3 210	2 460	2 080	1 680	3 240	2 690	2 090	1 770	1 430	918
	2 500	3 780	3 110	2 390	2 020	1 630	3 050	2 540	1 980	1 680	1 360	**872**
	3 000	3 620	2 990	2 300	1 950	1 570	2 810	2 360	1 840	1 560	1 270	815
	3 500	3 440	2 840	2 190	1 860	1 500	2 560	2 150	1 690	1 440	1 170	752
	4 000	3 230	2 680	2 070	1 760	1 420	2 310	1 950	1 530	1 310	1 070	687
	4 500	3 010	2 500	1 940	1 650	1 330	2 060	1 750	1 380	1 180	964	622
	5 000	2 790	2 320	1 810	1 530	1 240	1 840	1 560	1 240	1 060	868	561
	5 500	2 570	2 150	1 670	1 420	1 160	1 640	1 400	1 110	950	779	504
	6 000	2 360	1 980	1 540	1 310	1 070	1 460	1 250	992	851	698	453
	6 500	2 160	1 810	1 420	1 210	985	1 300	1 110	889	763	627	407
	7 000	1 980	1 660	1 300	1 110	907	1 160	997	797	685	563	366
	7 500	1 810	1 530	1 200	1 020	834	1 040	896	717	617	507	330
	8 000	1 660	1 400	1 100	940	767	936	807	647	557	458	298
	8 500	1 520	1 290	1 010	865	706	845	729	585	504	415	270
	9 000	1 400	1 180	931	797	651	766	662	531	458	377	246
	9 500	1 290	1 090	858	734	600	697	602	484	417	344	224
	10 000	1 190	1 000	792	678	554	636	550	442	381	314	205
	10 500	1 100	927	732	627	513	582	504	405	349	288	188
	11 000	1 010	858	678	581	476	534	463	373	321	265	173
	11 500	939	796	629	540	442	492	426	343	296	245	159
	12 000	872	739	585	502	411		394	317	274	226	147
	12 500	812	689	545	468	383					210	137
	13 000	757	642	509	437	358						
	14 000	662	562	446	383	313						
	15 000	584	496	393	338	277						
	16 000			349	300	246						
PROPERTIES AND DESIGN DATA												
Area (mm^2)		12 900	10 600	8 120	6 850	5 540	11 200	9 260	7 150	6 040	4 900	3 730
r_x (mm)		92.8	94.4	96.0	96.8	97.6	88.3	90.1	91.9	92.8	93.6	94.5
r_y (mm)		77.9	79.3	80.8	81.6	82.3	58.8	60.3	61.7	62.4	63.1	63.8
r_x / r_y		1.19	1.19	1.19	1.19	1.19	1.50	1.49	1.49	1.49	1.48	1.48
M_{rx} (kN·m)		340	284	223	190	^ 155	280	235	186	158	^ 129	^ 99.9
M_{ry} (kN·m)		291	244	191	163	116	195	164	130	111	80.3	49.1
$(b_{el} / t)\sqrt{350}$		224	299	424	524	673	224	299	424	524	673	919
IMPERIAL SIZE AND WEIGHT												
Weight (lb./ft.)		67.8	55.7	42.8	36.1	29.2	59.3	48.9	37.7	31.9	25.8	19.7
Thickness (in.)		0.625	0.500	0.375	0.313	0.250	0.625	0.500	0.375	0.313	0.250	0.188
Size (in.)		10 x 8					10 x 6					

‡ Class 4

^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.

RECTANGULAR HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W
CLASS C
$\phi = 0.90$

Section (mm x mm x mm)	HSS 203 x 152						HSS 203 x 102				
	16	13	9.5	7.9	6.4	‡ 4.8	13	9.5	7.9	6.4	‡ 4.8
Mass (kg/m)	75.6	62.6	48.5	41.1	33.4	25.5	52.4	40.9	34.7	28.3	21.7
Effective length (KL) in millimetres with respect to the least radius of gyration											
0	3 040	2 510	1 950	1 650	1 340	985	2 100	1 640	1 400	1 140	831
500	3 030	2 510	1 940	1 640	1 340	983	2 090	1 630	1 390	1 130	826
1 000	2 990	2 480	1 920	1 630	1 320	973	2 020	1 580	1 350	1 100	**804**
1 500	2 910	2 410	1 870	1 590	1 290	**951**	1 880	1 480	1 260	1 030	758
2 000	2 770	2 310	1 800	1 520	1 240	915	1 680	1 330	1 140	938	690
2 500	2 590	2 160	1 690	1 440	1 170	867	1 450	1 160	1 000	825	610
3 000	2 380	2 000	1 570	1 340	1 090	807	1 230	992	859	712	528
3 500	2 160	1 820	1 430	1 220	1 000	742	1 030	839	730	607	451
4 000	1 930	1 640	1 290	1 110	909	675	865	708	618	515	384
4 500	1 720	1 460	1 160	996	819	609	729	599	524	438	328
5 000	1 530	1 300	1 040	892	734	547	618	510	447	374	280
5 500	1 360	1 160	926	797	657	490	528	437	384	321	241
6 000	1 200	1 030	826	712	587	439	455	377	332	278	209
6 500	1 070	919	738	636	526	393	395	328	289	242	182
7 000	954	821	660	570	472	353	345	287	253	212	160
7 500	854	736	593	513	424	318	304	253	223	188	141
8 000	767	662	534	462	382	287		225	198	167	125
8 500	692	598	483	418	346	259					112
9 000	626	542	438	379	314	236					
9 500	569	493	398	345	286	215					
10 000	519	450	364	315	261	196					
10 500	475	412	333	289	240	180					
11 000	436	378	306	265	220	165					
11 500		348	282	245	203	152					
12 000			261	226	188	141					
12 500											
13 000											
14 000											
15 000											
16 000											
PROPERTIES AND DESIGN DATA											
Area (mm^2)	9 640	7 970	6 180	5 230	4 250	3 250	6 680	5 210	4 430	3 610	2 760
r_x (mm)	71.7	73.4	75.1	75.9	76.7	77.5	68.4	70.3	71.2	72.2	73.1
r_y (mm)	57.1	58.6	60.0	60.8	61.5	62.2	39.1	40.5	41.3	42.0	42.7
r_x / r_y	1.26	1.25	1.25	1.25	1.25	1.25	1.75	1.74	1.72	1.72	1.71
M_{rx} (kN·m)	196	166	132	113	92.9	^ 71.8	128	103	88.5	73.1	^ 56.7
M_{ry} (kN·m)	160	136	108	92.9	76.5	49.3	77.5	62.7	54.2	45.0	29.4
$(b_{el}/t)\sqrt{350}$	165	224	324	404	524	720	224	324	404	524	720
IMPERIAL SIZE AND WEIGHT											
Weight (lb./ft.)	50.8	42.1	32.6	27.6	22.4	17.1	35.2	27.5	23.3	19.0	14.6
Thickness (in.)	0.625	0.500	0.375	0.313	0.250	0.188	0.500	0.375	0.313	0.250	0.188
Size (in.)	8 x 6						8 x 4				

‡ Class 4

^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.

RECTANGULAR HOLLOW SECTIONS

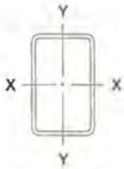
Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W
CLASS C
ϕ = 0.90

Section (mm x mm x mm)	HSS 178 x 127					HSS 152 x 102					
	13	9.5	7.9	6.4	† 4.8	13	9.5	7.9	6.4	4.8	‡ 3.2
Mass (kg/m)	52.4	40.9	34.7	28.3	21.7	42.3	33.3	28.4	23.2	17.9	12.2
Effective length (KL) in millimetres with respect to the least radius of gyration											
0	2 100	1 640	1 400	1 140	869	1 700	1 340	1 140	932	718	437
500	2 100	1 640	1 390	1 130	867	1 690	1 330	1 130	927	714	434
1 000	2 060	1 610	1 370	1 110	853	1 620	1 280	1 100	899	693	422
1 500	1 970	1 540	1 310	1 070	822	1 500	1 190	1 020	840	650	**397**
2 000	1 830	1 440	1 230	1 010	774	1 330	1 060	917	757	588	360
2 500	1 660	1 320	1 130	926	712	1 130	921	797	660	515	317
3 000	1 480	1 180	1 010	833	643	953	781	679	565	442	274
3 500	1 290	1 040	894	739	572	795	657	573	478	376	233
4 000	1 120	908	784	650	504	663	551	482	404	318	198
4 500	975	791	684	568	442	556	464	407	342	270	169
5 000	846	689	597	497	387	470	394	346	291	230	144
5 500	736	601	522	435	339	400	337	296	249	197	124
6 000	643	526	458	382	298	344	290	255	215	171	107
6 500	565	463	403	337	263	298	252	222	187	149	93
7 000	498	410	357	298	233	261	220	194	164	130	82
7 500	442	364	317	265	208	229	194	171	145	115	72
8 000	395	325	283	237	186				128	102	64
8 500	354	292	254	213	167						
9 000	319	263	229	192	151						
9 500	288	238	208	174	137						
10 000			189	158	124						
10 500											
11 000											
11 500											
12 000											
12 500											
13 000											
14 000											
15 000											
16 000											
PROPERTIES AND DESIGN DATA											
Area (mm^2)	6 680	5 210	4 430	3 610	2 760	5 390	4 240	3 620	2 960	2 280	1 550
r_x (mm)	62.9	64.6	65.4	66.2	67.1	52.2	54.0	54.9	55.7	56.5	57.4
r_y (mm)	48.1	49.6	50.3	51.1	51.8	37.7	39.2	39.9	40.6	41.3	42.1
r_x / r_y	1.31	1.30	1.30	1.30	1.30	1.38	1.38	1.38	1.37	1.37	1.36
M_{rx} (kN·m)	119	95.4	82.2	68.0	52.9	79.4	64.9	56.1	46.6	36.5	^ 25.3
M_{ry} (kN·m)	93.9	75.6	65.2	53.9	36.9	59.5	48.8	42.2	35.3	27.7	14.5
$(b_{el}/t)\sqrt{350}$	187	274	344	449	621	150	224	284	374	522	822
IMPERIAL SIZE AND WEIGHT											
Weight (lb./ft.)	35.2	27.5	23.3	19.0	14.6	28.4	22.4	19.1	15.6	12.0	8.17
Thickness (in.)	0.500	0.375	0.313	0.250	0.188	0.500	0.375	0.313	0.250	0.188	0.125
Size (in.)	7 x 5					6 x 4					

† Class 3 in bending about Y-Y axis
^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.
‡ Class 4

RECTANGULAR HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS C
$\phi = 0.90$

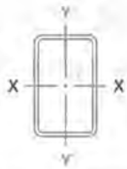
Section (mm x mm x mm)		HSS 152 x 76					
		13	9.5	7.9	6.4	4.8	‡ 3.2
Mass (kg/m)		37.3	29.5	25.2	20.7	16.0	10.9
Effective length (KL) in millimetres with respect to the least radius of gyration	0	1 500	1 180	1 010	832	643	386
	500	1 470	1 170	1 000	822	635	382
	1 000	1 360	1 090	937	772	599	**361**
	1 500	1 160	944	818	679	530	322
	2 000	936	774	677	565	445	272
	2 500	735	617	543	457	362	223
	3 000	576	489	432	366	291	180
	3 500	456	390	346	294	235	146
	4 000	366	314	280	238	191	119
	4 500	298	257	229	196	157	98
	5 000	247	213	191	163	131	82
	5 500	207	179	160	137	110	69
	6 000			137	117	94	59
	6 500						
	7 000						
	7 500						
	8 000						
	8 500						
	9 000						
	9 500						
	10 000						
	10 500						
	11 000						
	11 500						
	12 000						
	12 500						
	13 000						
	14 000						
	15 000						
	16 000						
PROPERTIES AND DESIGN DATA							
Area (mm^2)		4 750	3 760	3 220	2 640	2 040	1 390
r_x (mm)		49.3	51.3	52.3	53.2	54.1	55.0
r_y (mm)		28.0	29.4	30.1	30.8	31.5	32.2
r_x / r_y		1.76	1.74	1.74	1.73	1.72	1.71
M_{rx} (kN·m)		65.2	53.9	46.9	39.4	30.9	^ 21.5
M_{ry} (kN·m)		39.1	32.8	28.7	24.1	19.1	10.1
$(b_{el} / t)\sqrt{350}$		150	224	284	374	522	822
IMPERIAL SIZE AND WEIGHT							
Weight (lb./ft.)		25.0	19.8	17.0	13.9	10.7	7.32
Thickness (in.)		0.500	0.375	0.313	0.250	0.188	0.125
Size (in.)		6 x 3					

‡ Class 4

^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.

RECTANGULAR HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS C
$\phi = 0.90$

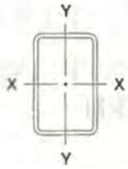
Section (mm x mm x mm)	HSS 127 x 76					
	13	9.5	7.9	6.4	4.8	‡ 3.2
Mass (kg/m)	32.2	25.7	22.1	18.2	14.1	9.62
Effective length (KL) in millimetres with respect to the least radius of gyration						
0	1 290	1 030	885	731	564	387
500	1 270	1 020	873	721	557	**382**
1 000	1 170	945	814	675	523	360
1 500	987	813	706	590	460	319
2 000	789	661	579	488	383	268
2 500	615	523	461	391	310	218
3 000	479	412	365	311	248	176
3 500	378	327	291	249	199	142
4 000	302	263	235	202	162	115
4 500	246	215	192	165	133	95
5 000	203	178	159	137	110	79
5 500		150	134	116	93	67
6 000				98	79	57
6 500						
7 000						
7 500						
8 000						
8 500						
9 000						
9 500						
10 000						
10 500						
11 000						
11 500						
12 000						
12 500						
13 000						
14 000						
15 000						
16 000						
PROPERTIES AND DESIGN DATA						
Area (mm^2)	4 100	3 280	2 810	2 320	1 790	1 230
r_x (mm)	41.4	43.3	44.2	45.1	45.9	46.8
r_y (mm)	27.3	28.7	29.4	30.1	30.8	31.6
r_x / r_y	1.52	1.51	1.50	1.50	1.49	1.48
M_{rx} (kN·m)	47.6	39.7	35.0	29.4	23.2	^ 16.2
M_{ry} (kN·m)	32.8	27.7	24.3	20.6	16.3	10.1
$(b_{el}/t)\sqrt{350}$	112	174	224	299	422	672
IMPERIAL SIZE AND WEIGHT						
Weight (lb./ft.)	21.6	17.3	14.8	12.2	9.45	6.47
Thickness (in.)	0.500	0.375	0.313	0.250	0.188	0.125
Size (in.)	5 x 3					

‡ Class 4

^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.

RECTANGULAR HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)

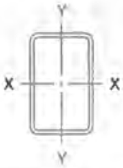


G40.21 350W CLASS C ϕ = 0.90

Section (mm x mm x mm)	HSS 102 x 76					HSS 102 x 51				
	9.5	7.9	6.4	4.8	3.2	9.5	7.9	6.4	4.8	3.2
Mass (kg/m)	21.9	18.9	15.6	12.2	8.35	18.1	15.7	13.1	10.3	7.09
Effective length (KL) in millimetres with respect to the least radius of gyration										
0	879	759	627	488	334	728	633	526	413	284
500	865	748	618	482	330	693	606	505	398	275
1 000	797	693	576	451	310	556	495	419	335	235
1 500	679	595	499	393	272	393	356	308	250	178
2 000	546	483	409	325	226	270	248	217	179	129
2 500	428	381	325	260	183	190	176	155	129	94
3 000	335	300	257	207	146	139	129	114	95	70
3 500	265	238	205	166	117	105	98	87	73	53
4 000	212	191	165	134	95				57	42
4 500	173	156	135	110	78					
5 000	143	129	112	91	65					
5 500	120	109	94	77	55					
6 000				65	47					
6 500										
7 000										
7 500										
8 000										
8 500										
9 000										
9 500										
10 000										
10 500										
11 000										
11 500										
12 000										
12 500										
13 000										
14 000										
15 000										
16 000										
PROPERTIES AND DESIGN DATA										
Area (mm^2)	2 790	2 410	1 990	1 550	1 060	2 310	2 010	1 670	1 310	903
r_x (mm)	35.0	35.9	36.7	37.5	38.4	32.2	33.2	34.2	35.1	36.1
r_y (mm)	27.8	28.5	29.3	30.0	30.7	18.2	18.9	19.6	20.3	21.0
r_x / r_y	1.26	1.26	1.25	1.25	1.25	1.77	1.76	1.74	1.73	1.72
M_{rx} (kN·m)	27.7	24.5	20.8	16.6	11.7	20.7	18.6	16.0	12.9	9.14
M_{ry} (kN·m)	22.6	20.0	17.0	13.6	9.58	12.3	11.2	9.70	7.88	5.64
$(b_{el}/t)\sqrt{350}$	125	165	224	323	523	125	165	224	323	523
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)	14.7	12.7	10.5	8.17	5.61	12.2	10.6	8.81	6.89	4.76
Thickness (in.)	0.375	0.313	0.250	0.188	0.125	0.375	0.313	0.250	0.188	0.125
Size (in.)	4 x 3					4 x 2				

RECTANGULAR HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)

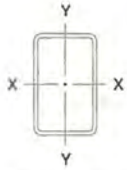


G40.21 350W
CLASS C
$\phi = 0.90$

Section (mm x mm x mm)	HSS 89 x 64		HSS 76 x 51			
	6.4	4.8	7.9	6.4	4.8	3.2
Mass (kg/m)	13.1	10.3	12.6	10.6	8.35	5.82
Effective length (KL) in millimetres with respect to the least radius of gyration						
0	526	413	504	425	334	233
500	514	404	480	407	321	225
1 000	458	363	384	332	266	189
1 500	370	296	271	239	195	141
2 000	282	228	186	167	138	101
2 500	212	173	130	118	98	73
3 000	161	132	95	87	72	54
3 500	125	103	72	66	55	41
4 000	99	82				32
4 500	80	66				
5 000						
5 500						
6 000						
6 500						
7 000						
7 500						
8 000						
8 500						
9 000						
9 500						
10 000						
10 500						
11 000						
11 500						
12 000						
12 500						
13 000						
14 000						
15 000						
16 000						
PROPERTIES AND DESIGN DATA						
Area (mm^2)	1 670	1 310	1 600	1 350	1 060	741
r_x (mm)	31.4	32.3	25.2	26.1	27.0	27.8
r_y (mm)	24.1	24.8	18.1	18.9	19.6	20.3
r_x / r_y	1.30	1.30	1.39	1.38	1.38	1.37
M_{rx} (kN·m)	14.9	12.0	11.3	9.92	8.13	5.86
M_{ry} (kN·m)	11.7	9.48	8.47	7.43	6.11	4.41
$(b_{el} / t)\sqrt{350}$	187	273	105	150	223	373
IMPERIAL SIZE AND WEIGHT						
Weight (lb./ft.)	8.81	6.89	8.45	7.11	5.61	3.91
Thickness (in.)	0.250	0.188	0.313	0.250	0.188	0.125
Size (in.)	3½ x 2½		3 x 2			

RECTANGULAR HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS C
$\phi = 0.90$

Section (mm x mm x mm)	HSS 76 x 38			HSS 64 x 38			HSS 51 x 25	
	6.4	4.8	3.2	6.4	4.8	3.2	4.8	3.2
Mass (kg/m)	9.31	7.40	5.18	8.05	6.45	4.55	4.54	3.28
Effective length (KL) in millimetres with respect to the least radius of gyration								
0	375	297	208	324	259	183	182	132
500	341	273	193	293	236	169	139	105
1 000	235	194	141	197	165	122	67	54
1 500	144	122	91	120	103	78	35	28
2 000	92	79	60	75	66	51		
2 500	62	53	41	51	44	35		
3 000			29			25		
3 500								
4 000								
4 500								
5 000								
5 500								
6 000								
6 500								
7 000								
7 500								
8 000								
8 500								
9 000								
9 500								
10 000								
10 500								
11 000								
11 500								
12 000								
12 500								
13 000								
14 000								
15 000								
16 000								
PROPERTIES AND DESIGN DATA								
Area (mm^2)	1 190	942	660	1 030	821	580	578	418
r_x (mm)	24.7	25.6	26.6	20.7	21.6	22.5	16.1	17.1
r_y (mm)	14.0	14.7	15.4	13.6	14.3	15.1	9.08	9.78
r_x / r_y	1.76	1.74	1.73	1.52	1.51	1.49	1.77	1.75
M_{rx} (kN·m)	8.16	6.74	4.91	5.92	4.98	3.69	2.59	2.00
M_{ry} (kN·m)	4.91	4.10	3.02	4.10	3.47	2.57	1.55	1.21
$(b_{el}/t)\sqrt{350}$	150	223	373	112	174	299	124	224
IMPERIAL SIZE AND WEIGHT								
Weight (lb./ft.)	6.26	4.97	3.48	5.41	4.33	3.06	3.05	2.21
Thickness (in.)	0.250	0.188	0.125	0.250	0.188	0.125	0.188	0.125
Size (in.)	3 x 1½			2½ x 1½			2 x 1	

ROUND HOLLOW SECTIONS

G40.21 350W
CLASS C
$\phi = 0.90$

Factored Axial Compressive Resistances, C_r (kN)

Section (mm x mm)	HSS 406				HSS 356			HSS 324		
	16	13	9.5	† 6.4	13	9.5	† 6.4	13	9.5	6.4
Mass (kg/m)	153	123	93.3	62.6	107	81.3	54.7	97.5	73.9	49.7
Effective length (KL) in millimetres with respect to the least radius of gyration										
0	6 140	4 950	3 750	2 510	4 320	3 280	2 200	3 910	2 960	1 990
500	6 140	4 940	3 750	2 510	4 310	3 270	2 190	3 900	2 960	1 990
1 000	6 130	4 940	3 740	2 510	4 310	3 270	2 190	3 900	2 960	1 990
1 500	6 120	4 930	3 730	2 500	4 290	3 260	2 180	3 880	2 940	1 980
2 000	6 090	4 900	3 720	2 490	4 260	3 240	2 170	3 840	2 920	1 960
2 500	6 040	4 870	3 690	2 480	4 220	3 200	2 150	3 790	2 880	1 940
3 000	5 980	4 820	3 650	2 450	4 160	3 160	2 120	3 720	2 830	1 900
3 500	5 900	4 760	3 610	2 420	4 080	3 100	2 080	3 640	2 760	1 860
4 000	5 810	4 680	3 550	2 380	3 990	3 030	2 040	3 530	2 690	1 810
4 500	5 690	4 590	3 480	2 340	3 880	2 950	1 980	3 420	2 600	1 750
5 000	5 560	4 490	3 410	2 290	3 760	2 860	1 920	3 290	2 500	1 690
5 500	5 420	4 370	3 320	2 230	3 630	2 770	1 860	3 150	2 400	1 620
6 000	5 260	4 250	3 230	2 170	3 490	2 660	1 790	3 000	2 290	1 550
6 500	5 090	4 110	3 130	2 100	3 340	2 550	1 720	2 850	2 180	1 470
7 000	4 910	3 970	3 020	2 030	3 190	2 440	1 640	2 700	2 060	1 400
7 500	4 730	3 820	2 910	1 960	3 040	2 320	1 570	2 550	1 950	1 320
8 000	4 540	3 670	2 800	1 880	2 890	2 210	1 490	2 400	1 840	1 250
8 500	4 350	3 520	2 680	1 810	2 740	2 100	1 420	2 260	1 730	1 180
9 000	4 160	3 370	2 570	1 730	2 600	1 990	1 340	2 130	1 630	1 110
9 500	3 970	3 220	2 460	1 660	2 460	1 880	1 270	2 000	1 530	1 040
10 000	3 790	3 070	2 350	1 580	2 320	1 780	1 200	1 880	1 440	980
10 500	3 610	2 930	2 240	1 510	2 200	1 680	1 140	1 760	1 350	922
11 000	3 440	2 790	2 130	1 440	2 070	1 590	1 080	1 660	1 270	867
11 500	3 280	2 660	2 030	1 370	1 960	1 500	1 020	1 560	1 200	816
12 000	3 120	2 530	1 940	1 310	1 850	1 420	963	1 470	1 130	768
12 500	2 970	2 410	1 840	1 250	1 750	1 340	911	1 380	1 060	723
13 000	2 820	2 290	1 750	1 190	1 660	1 270	862	1 300	1 000	682
14 000	2 560	2 080	1 590	1 080	1 480	1 140	773	1 160	890	607
15 000	2 320	1 880	1 440	978	1 330	1 020	695	1 030	795	543
16 000	2 100	1 710	1 310	890	1 200	924	627	926	714	487
17 000	1 910	1 560	1 200	811	1 090	835	568	834	643	439
18 000	1 750	1 420	1 090	740	984	758	515	754	581	397
PROPERTIES AND DESIGN DATA										
Area (mm^2)	19 500	15 700	11 900	7 980	13 700	10 400	6 970	12 400	9 410	6 330
r (mm)	138	139	140	141	121	122	123	110	111	112
M_r (kN·m)	762	621	473	248	469	359	188	387	297	202
(D / t) 350	8 960	11 200	14 900	22 400	9 800	13 100	19 600	8 930	11 900	17 900
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)	103	82.9	62.7	42.1	72.2	54.7	36.8	65.5	49.6	33.4
Thickness (in.)	0.625	0.500	0.375	0.250	0.500	0.375	0.250	0.500	0.375	0.250
Size (in.)	16 OD				14 OD			12.75 OD		

† Class 3

ROUND HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W
CLASS C
ϕ = 0.90

Section (mm x mm)	HSS 273					HSS 245	
	13	9.5	7.9	6.4	† 4.8	9.5	6.4
Mass (kg/m)	81.6	61.9	51.9	41.8	31.6	55.2	37.3
Effective length (KL) in millimetres with respect to the least radius of gyration							
0	3 280	2 490	2 080	1 680	1 270	2 210	1 500
500	3 270	2 480	2 080	1 670	1 270	2 210	1 500
1 000	3 260	2 480	2 070	1 670	1 260	2 200	1 490
1 500	3 240	2 460	2 060	1 660	1 260	2 180	1 470
2 000	3 190	2 420	2 030	1 630	1 240	2 140	1 450
2 500	3 130	2 370	1 990	1 600	1 220	2 080	1 410
3 000	3 040	2 310	1 940	1 560	1 180	2 010	1 360
3 500	2 930	2 230	1 870	1 510	1 140	1 920	1 300
4 000	2 810	2 140	1 800	1 450	1 100	1 820	1 240
4 500	2 670	2 040	1 710	1 380	1 050	1 710	1 160
5 000	2 520	1 930	1 620	1 310	995	1 600	1 090
5 500	2 370	1 810	1 530	1 230	938	1 480	1 010
6 000	2 220	1 700	1 430	1 160	881	1 370	938
6 500	2 070	1 590	1 340	1 080	825	1 270	867
7 000	1 930	1 480	1 250	1 010	770	1 170	800
7 500	1 800	1 380	1 160	941	718	1 070	738
8 000	1 670	1 280	1 080	876	668	989	680
8 500	1 550	1 190	1 000	814	622	911	627
9 000	1 440	1 110	934	757	578	840	578
9 500	1 330	1 030	868	704	538	775	534
10 000	1 240	955	808	655	501	716	494
10 500	1 150	889	752	610	466	663	458
11 000	1 070	828	701	569	435	615	424
11 500	1 000	773	654	531	406	571	395
12 000	935	722	611	496	380	532	367
12 500	874	675	572	464	355	496	343
13 000	819	633	536	435	333	463	320
14 000	721	557	472	384	294	406	281
15 000	639	494	419	340	261	358	248
16 000	569	440	373	303	232	318	220
17 000	510	394	334	272	208		
18 000	459	355	301	245	187		
PROPERTIES AND DESIGN DATA							
Area (mm^2)	10 400	7 890	6 610	5 320	4 030	7 030	4 750
r (mm)	92.2	93.2	93.8	94.3	94.9	83.1	84.2
M_r (kN·m)	272	209	176	142	83.8	166	113
(D / t) 350	7 530	10 000	12 000	15 100	20 000	8 980	13 500
IMPERIAL SIZE AND WEIGHT							
Weight (lb./ft.)	54.8	41.6	34.9	28.1	21.3	37.1	25.1
Thickness (in.)	0.500	0.375	0.313	0.250	0.188	0.375	0.250
Size (in.)	10.75 OD					9.625 OD	

† Class 3

ROUND HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W
CLASS C
$\phi = 0.90$

Section (mm x mm)	HSS 219				HSS 178		HSS 168			
	13	9.5	6.4	4.8	13	9.5	13	9.5	6.4	4.8
Mass (kg/m)	64.6	49.3	33.3	25.3	51.7	39.5	48.7	37.3	25.4	19.3
Effective length (KL) in millimetres with respect to the least radius of gyration										
0	2 590	1 980	1 340	1 010	2 080	1 590	1 960	1 500	1 020	775
500	2 590	1 970	1 330	1 010	2 070	1 580	1 950	1 490	1 020	773
1 000	2 570	1 960	1 330	1 010	2 050	1 570	1 920	1 470	1 000	764
1 500	2 530	1 930	1 310	994	1 990	1 530	1 870	1 430	975	743
2 000	2 470	1 890	1 280	971	1 910	1 460	1 770	1 360	930	710
2 500	2 380	1 820	1 230	938	1 790	1 380	1 650	1 270	870	665
3 000	2 270	1 730	1 180	897	1 650	1 270	1 510	1 160	800	612
3 500	2 130	1 640	1 110	848	1 500	1 160	1 360	1 050	725	556
4 000	1 990	1 530	1 040	794	1 350	1 050	1 210	939	650	499
4 500	1 840	1 410	967	738	1 210	941	1 070	834	579	445
5 000	1 690	1 300	891	681	1 080	840	945	739	514	396
5 500	1 540	1 190	818	625	956	749	834	654	456	351
6 000	1 410	1 090	749	573	851	667	738	579	405	312
6 500	1 280	994	684	523	758	596	654	514	360	278
7 000	1 170	906	624	478	677	533	582	458	321	248
7 500	1 060	826	570	437	607	478	520	410	288	222
8 000	968	754	521	399	546	431	467	368	259	200
8 500	884	689	476	365	493	389	420	332	233	180
9 000	809	631	437	335	447	353	380	300	211	163
9 500	742	579	401	308	406	321	345	273	192	148
10 000	682	532	369	283	371	293	314	248	175	135
10 500	628	491	340	261	339	268	287	227	160	124
11 000	580	453	314	241	311	247	264	208	147	114
11 500	536	420	291	224	287	227				105
12 000	498	389	270	208						
12 500	462	362	251	193						
13 000	431	337	234	180						
14 000	376	294	205	157						
15 000			180	138						
16 000										
17 000										
18 000										
PROPERTIES AND DESIGN DATA										
Area (mm^2)	8 230	6 270	4 240	3 220	6 590	5 040	6 210	4 750	3 230	2 460
r (mm)	73.1	74.2	75.3	75.8	58.5	59.6	55.2	56.2	57.3	57.8
M_r (kN·m)	171	132	90.7	69.3	109	85.1	97.0	75.9	52.6	40.3
(D / t) 350	6 040	8 050	12 100	16 000	4 900	6 530	4 640	6 180	9 280	12 300
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)	43.4	33.1	22.4	17.0	34.7	26.6	32.7	25.1	17.0	13.0
Thickness (in.)	0.500	0.375	0.250	0.188	0.500	0.375	0.500	0.375	0.250	0.188
Size (in.)	8.625 OD				7 OD		6.625 OD			

ROUND HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W
CLASS C
$\phi = 0.90$

Section (mm x mm)	HSS 141			HSS 127	
	13	9.5	6.4	9.5	6.4
Mass (kg/m)	40.3	31.0	21.1	27.6	18.9
Effective length (KL) in millimetres with respect to the least radius of gyration					
0	1 620	1 240	847	1 110	759
500	1 610	1 240	844	1 100	755
1 000	1 570	1 210	827	1 070	735
1 500	1 500	1 160	791	1 010	693
2 000	1 380	1 070	736	911	631
2 500	1 240	966	667	801	557
3 000	1 090	853	592	689	482
3 500	944	743	518	587	413
4 000	814	643	450	497	351
4 500	700	555	390	423	299
5 000	604	480	338	361	256
5 500	523	416	294	310	220
6 000	455	363	256	268	191
6 500	398	318	225	233	166
7 000	350	280	199	204	146
7 500	310	248	176	180	129
8 000	276	221	157	160	115
8 500	247	198	141		102
9 000	223	178	127		
9 500			115		
10 000					
10 500					
11 000					
11 500					
12 000					
12 500					
13 000					
14 000					
15 000					
16 000					
17 000					
18 000					
PROPERTIES AND DESIGN DATA					
Area (mm^2)	5 130	3 950	2 690	3 520	2 410
r (mm)	45.7	46.7	47.8	41.7	42.7
M_r (kN·m)	66.5	52.3	36.5	41.6	29.1
(D / t) 350	3 890	5 190	7 790	4 660	7 000
IMPERIAL SIZE AND WEIGHT					
Weight (lb./ft.)	27.1	20.8	14.2	18.6	12.7
Thickness (in.)	0.500	0.375	0.250	0.375	0.250
Size (in.)	5.563 OD			5 OD	

ROUND HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W CLASS C $\phi = 0.90$

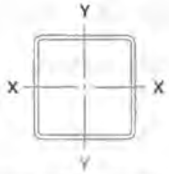
Section (mm x mm)		HSS 89			HSS 76			HSS 73		
		6.4	4.8	3.2	6.4	4.8	3.2	6.4	4.8	3.2
Mass (kg/m)		12.9	9.92	6.72	10.9	8.42	5.73	10.4	8.04	5.48
Effective length (KL) in millimetres with respect to the least radius of gyration	0	520	397	270	438	337	230	419	321	220
	500	513	392	266	428	330	225	409	314	215
	1 000	477	366	249	385	298	204	363	280	193
	1 500	414	319	218	314	245	169	291	226	157
	2 000	339	263	181	242	191	133	220	173	121
	2 500	270	210	145	184	145	102	165	130	92
	3 000	213	167	116	140	112	78	125	99	70
	3 500	170	133	93	109	87	61	97	77	54
	4 000	137	108	75	87	69	49	77	61	43
	4 500	112	88	62	70	56	40	62	49	35
	5 000	93	73	51		46	33			
	5 500	78	62	43						
	6 000			37						
	6 500									
	7 000									
	7 500									
	8 000									
	8 500									
	9 000									
	9 500									
	10 000									
	10 500									
	11 000									
	11 500									
	12 000									
	12 500									
	13 000									
	14 000									
	15 000									
	16 000									
	17 000									
	18 000									
PROPERTIES AND DESIGN DATA										
Area (mm^2)		1 650	1 260	856	1 390	1 070	729	1 330	1 020	698
r (mm)		29.3	29.8	30.3	24.8	25.3	25.8	23.7	24.2	24.7
M_r (kN·m)		13.7	10.7	7.37	9.80	7.69	5.36	8.91	7.02	4.88
(D / t) 350		4 900	6 510	9 780	4 200	5 580	8 390	4 020	5 350	8 030
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)		8.69	6.66	4.52	7.35	5.66	3.85	7.01	5.40	3.68
Thickness (in.)		0.250	0.188	0.125	0.250	0.188	0.125	0.250	0.188	0.125
Size (in.)		3.5 OD			3 OD			2.875 OD		

ROUND HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W
CLASS C
$\phi = 0.90$

Section (mm x mm)	HSS 64			HSS 60			HSS 48	
	6.4	4.8	3.2	6.4	4.8	3.2	4.8	3.2
Mass (kg/m)	8.95	6.92	4.73	8.45	6.54	4.48	5.13	3.54
Effective length (KL) in millimetres with respect to the least radius of gyration								
0	359	278	190	340	263	180	206	142
500	346	268	184	326	252	173	191	133
1 000	291	228	158	268	210	146	141	100
1 500	218	172	121	195	154	108	91	66
2 000	155	124	88	136	109	77	60	43
2 500	112	90	64	97	78	56	41	30
3 000	83	67	48	71	58	41	29	21
3 500	63	51	37	54	44	31		
4 000	49	40	29			25		
4 500								
5 000								
5 500								
6 000								
6 500								
7 000								
7 500								
8 000								
8 500								
9 000								
9 500								
10 000								
10 500								
11 000								
11 500								
12 000								
12 500								
13 000								
14 000								
15 000								
16 000								
17 000								
18 000								
PROPERTIES AND DESIGN DATA								
Area (mm^2)	1 140	882	603	1 080	834	571	654	451
r (mm)	20.3	20.8	21.4	19.2	19.7	20.2	15.5	16.0
M_r (kN·m)	6.55	5.20	3.65	5.86	4.66	3.28	2.86	2.04
(D / t) 350	3 500	4 650	6 990	3 320	4 420	6 640	3 540	5 320
IMPERIAL SIZE AND WEIGHT								
Weight (lb./ft.)	6.01	4.65	3.18	5.68	4.40	3.01	3.45	2.38
Thickness (in.)	0.250	0.188	0.125	0.250	0.188	0.125	0.188	0.125
Size (in.)	2.5 OD			2.375 OD			1.9 OD	

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS H
ϕ = 0.90

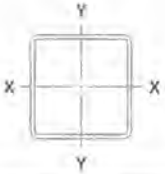
Section (mm x mm x mm)	HSS 559 x 559	HSS 508 x 508			
	19*	22*	19*	16*	‡ 13* #
Mass (kg/m)	316	329	285	240	194
Effective length (KL) in millimetres with respect to the least radius of gyration					
0	12 700	13 200	11 400	9 640	7 700
500	12 700	13 200	11 400	9 640	7 700
1 000	12 700	13 200	11 400	9 640	7 700
1 500	12 700	13 200	11 400	9 640	7 700
2 000	12 700	13 200	11 400	9 640	7 700
2 500	12 700	13 200	11 400	9 640	7 700
3 000	12 700	13 200	11 400	9 640	7 700
3 500	12 700	13 200	11 400	9 630	7 700
4 000	12 700	13 200	11 400	9 630	7 690
4 500	12 600	13 200	11 400	9 620	7 690
5 000	12 600	13 200	11 400	9 610	7 680
5 500	12 600	13 100	11 400	9 590	7 660
6 000	12 600	13 100	11 300	9 570	7 650
6 500	12 600	13 100	11 300	9 540	7 620
7 000	12 500	13 000	11 300	9 500	7 590
7 500	12 500	12 900	11 200	9 450	7 560
8 000	12 400	12 800	11 100	9 390	7 510
8 500	12 400	12 700	11 000	9 320	7 460
9 000	12 300	12 600	10 900	9 230	7 390
9 500	12 200	12 500	10 800	9 130	7 310
10 000	12 100	12 300	10 700	9 020	7 220
10 500	12 000	12 100	10 500	8 880	7 120
11 000	11 800	11 900	10 300	8 730	7 000
11 500	11 700	11 600	10 100	8 570	6 880
12 000	11 500	11 400	9 890	8 390	6 740
12 500	11 300	11 100	9 660	8 190	6 580
13 000	11 100	10 800	9 410	7 990	6 420
14 000	10 600	10 200	8 880	7 550	6 080
15 000	10 100	9 550	8 320	7 080	5 720
16 000	9 540	8 890	7 750	6 610	5 340
17 000	8 980	8 250	7 200	6 140	4 970
18 000	8 410	7 640	6 660	5 700	4 610
PROPERTIES AND DESIGN DATA					
Area (mm^2)	40 200	41 900	36 300	30 600	24 700
r (mm)	219	197	198	200	201
M_r (kN·m)	2 540	2 380	2 080	1 770	1 240
$(b_{el}/t)\sqrt{350}$	474	353	424	524	673
IMPERIAL SIZE AND WEIGHT					
Weight (lb./ft.)	212	221	192	162	131
Thickness (in.)	0.750	0.875	0.750	0.625	0.500
Size (in.)	22 x 22	20 x 20			

* Imported section

‡ Class 4

\# C_r calculated according to S16-14 Clause 13.3.5(b)

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS H
$\phi = 0.90$

Section (mm x mm x mm)		HSS 457 x 457				HSS 406 x 406				
		22*	19*	16*	† 13*	22*	19*	16*	13*	‡ 9.5*
Mass (kg/m)		294	255	215	174	258	224	190	154	117
Effective length (KL) in millimetres with respect to the least radius of gyration	0	11 800	10 200	8 630	6 990	10 400	9 010	7 620	6 170	4 370
	500	11 800	10 200	8 630	6 990	10 400	9 010	7 620	6 170	4 370
	1 000	11 800	10 200	8 630	6 990	10 400	9 010	7 620	6 170	4 370
	1 500	11 800	10 200	8 630	6 990	10 400	9 010	7 620	6 170	4 370
	2 000	11 800	10 200	8 630	6 990	10 400	9 010	7 620	6 170	4 370
	2 500	11 800	10 200	8 630	6 990	10 400	9 010	7 620	6 170	4 370
	3 000	11 800	10 200	8 630	6 990	10 400	9 000	7 620	6 170	4 370
	3 500	11 800	10 200	8 620	6 990	10 300	8 990	7 610	6 160	4 360
	4 000	11 800	10 200	8 610	6 980	10 300	8 980	7 600	6 150	4 360
	4 500	11 700	10 200	8 600	6 970	10 300	8 960	7 580	6 140	4 350
	5 000	11 700	10 200	8 590	6 960	10 300	8 920	7 550	6 120	4 330
	5 500	11 700	10 200	8 560	6 940	10 200	8 880	7 520	6 090	4 310
	6 000	11 600	10 100	8 530	6 910	10 100	8 820	7 470	6 060	4 290
	6 500	11 600	10 100	8 490	6 880	10 000	8 740	7 400	6 010	4 250
	7 000	11 500	10 000	8 430	6 840	9 920	8 650	7 320	5 940	4 210
	7 500	11 400	9 910	8 360	6 790	9 780	8 520	7 220	5 870	4 160
	8 000	11 300	9 810	8 280	6 720	9 600	8 380	7 100	5 770	4 090
	8 500	11 100	9 690	8 180	6 640	9 400	8 210	6 960	5 670	4 020
	9 000	10 900	9 540	8 060	6 550	9 170	8 020	6 800	5 540	3 930
	9 500	10 700	9 380	7 920	6 440	8 910	7 800	6 630	5 400	3 840
	10 000	10 500	9 190	7 770	6 320	8 630	7 570	6 430	5 250	3 730
	10 500	10 300	8 990	7 600	6 190	8 330	7 310	6 220	5 080	3 610
	11 000	10 000	8 760	7 410	6 040	8 020	7 050	6 000	4 910	3 490
	11 500	9 730	8 520	7 210	5 880	7 700	6 770	5 770	4 720	3 360
	12 000	9 420	8 260	6 990	5 710	7 370	6 490	5 530	4 540	3 230
	12 500	9 110	7 990	6 770	5 540	7 040	6 210	5 290	4 350	3 100
	13 000	8 790	7 720	6 540	5 350	6 720	5 930	5 060	4 160	2 960
	14 000	8 130	7 160	6 070	4 980	6 090	5 390	4 600	3 790	2 700
	15 000	7 480	6 600	5 600	4 600	5 510	4 880	4 170	3 440	2 460
	16 000	6 860	6 060	5 150	4 230	4 980	4 420	3 780	3 120	2 230
	17 000	6 280	5 550	4 720	3 890	4 510	4 000	3 420	2 830	2 020
	18 000	5 750	5 090	4 330	3 570	4 080	3 630	3 110	2 570	1 840
PROPERTIES AND DESIGN DATA										
Area (mm^2)		37 400	32 500	27 400	22 200	32 900	28 600	24 200	19 600	14 900
r (mm)		176	178	179	181	155	157	158	160	161
M_r (kN·m)		1 900	1 660	1 410	995	1 470	1 290	1 100	904	570
$(b_{el}/t)\sqrt{350}$		310	374	464	599	267	324	404	524	723
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)		197	171	144	117	174	151	127	103	78.6
Thickness (in.)		0.875	0.750	0.625	0.500	0.875	0.750	0.625	0.500	0.375
Size (in.)		18 x 18				16 x 16				

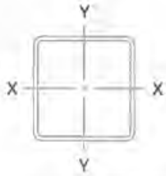
* Imported section

‡ Class 4

† Class 3

SQUARE HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W CLASS H $\phi = 0.90$

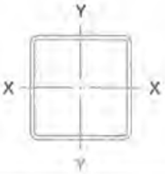
Section (mm x mm x mm)	HSS 356 x 356				HSS 305 x 305				
	16*	13*	† 9.5*	‡ 7.9*	16	13	9.5	† 7.9	‡ 6.4
Mass (kg/m)	164	133	102	85.4	139	113	86.5	72.7	58.7
Effective length (KL) in millimetres with respect to the least radius of gyration									
0	6 580	5 360	4 100	3 040	5 580	4 540	3 470	2 920	1 940
500	6 580	5 350	4 090	3 040	5 580	4 540	3 460	2 920	1 940
1 000	6 580	5 350	4 090	3 040	5 580	4 540	3 460	2 920	1 940
1 500	6 580	5 350	4 090	3 040	5 570	4 540	3 460	2 920	1 940
2 000	6 580	5 350	4 090	3 040	5 570	4 530	3 460	2 920	1 940
2 500	6 580	5 350	4 090	3 040	5 570	4 530	3 460	2 920	1 940
3 000	6 570	5 350	4 090	3 030	5 560	4 520	3 450	2 910	1 930
3 500	6 560	5 340	4 080	3 030	5 540	4 500	3 440	2 900	1 930
4 000	6 540	5 320	4 070	3 020	5 500	4 480	3 430	2 890	1 920
4 500	6 510	5 300	4 060	3 010	5 460	4 440	3 400	2 870	1 900
5 000	6 470	5 270	4 030	2 990	5 390	4 390	3 360	2 830	1 880
5 500	6 420	5 220	4 000	2 970	5 290	4 320	3 310	2 790	1 860
6 000	6 340	5 170	3 960	2 940	5 180	4 220	3 240	2 740	1 820
6 500	6 250	5 090	3 900	2 900	5 030	4 110	3 160	2 670	1 770
7 000	6 130	5 000	3 840	2 850	4 860	3 970	3 060	2 590	1 720
7 500	5 990	4 890	3 760	2 790	4 660	3 820	2 950	2 490	1 660
8 000	5 830	4 760	3 660	2 720	4 450	3 640	2 820	2 390	1 590
8 500	5 640	4 610	3 550	2 640	4 220	3 460	2 690	2 280	1 510
9 000	5 440	4 450	3 430	2 550	3 990	3 280	2 550	2 160	1 440
9 500	5 230	4 280	3 310	2 450	3 760	3 090	2 400	2 050	1 360
10 000	5 000	4 090	3 170	2 350	3 530	2 900	2 260	1 930	1 280
10 500	4 770	3 910	3 030	2 250	3 310	2 720	2 130	1 810	1 200
11 000	4 530	3 720	2 890	2 140	3 100	2 550	2 000	1 700	1 130
11 500	4 300	3 530	2 750	2 040	2 900	2 390	1 870	1 600	1 060
12 000	4 070	3 350	2 610	1 930	2 710	2 230	1 750	1 500	994
12 500	3 860	3 170	2 470	1 830	2 530	2 090	1 640	1 400	932
13 000	3 640	3 000	2 340	1 740	2 370	1 960	1 540	1 310	874
14 000	3 250	2 680	2 090	1 550	2 080	1 720	1 350	1 160	770
15 000	2 910	2 390	1 870	1 390	1 840	1 520	1 200	1 020	681
16 000	2 600	2 140	1 680	1 250	1 630	1 350	1 060	910	605
17 000	2 340	1 930	1 510	1 120	1 460	1 200	949	813	540
18 000	2 100	1 740	1 360	1 010	1 310	1 080	852	729	484
PROPERTIES AND DESIGN DATA									
Area (mm²)	20 900	17 000	13 000	10 900	17 700	14 400	11 000	9 270	7 480
r (mm)	138	139	141	141	117	118	120	121	121
M_r (kN·m)	832	684	454	353	595	491	381	279	197
$(b_{el}/t)\sqrt{350}$	344	449	623	763	284	374	524	643	823
IMPERIAL SIZE AND WEIGHT									
Weight (lb./ft.)	110	89.7	68.4	57.4	93.4	76.1	58.1	48.9	39.4
Thickness (in.)	0.625	0.500	0.375	0.313	0.625	0.500	0.375	0.313	0.250
Size (in.)	14 x 14				12 x 12				

* Imported section

‡ Class 4

† Class 3

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



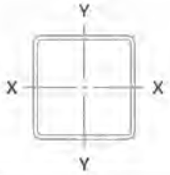
G40.21 350W CLASS H $\phi = 0.90$

Section (mm x mm x mm)		HSS 254 x 254					
		16	13	9.5	7.9	‡ 6.4 #	‡ 4.8
Mass (kg/m)		114	93.0	71.3	60.1	48.6	36.9
Effective length (KL) in millimetres with respect to the least radius of gyration	0	4 570	3 720	2 860	2 410	1 930	1 100
	500	4 570	3 720	2 860	2 410	1 930	1 100
	1 000	4 570	3 720	2 860	2 410	1 930	1 100
	1 500	4 570	3 720	2 860	2 410	1 930	1 100
	2 000	4 560	3 710	2 860	2 410	1 930	1 100
	2 500	4 550	3 700	2 850	2 400	1 920	1 100
	3 000	4 530	3 690	2 840	2 390	1 920	1 090
	3 500	4 490	3 660	2 820	2 380	1 900	1 090
	4 000	4 430	3 610	2 790	2 350	1 880	1 070
	4 500	4 340	3 540	2 740	2 310	1 850	1 060
	5 000	4 220	3 450	2 670	2 250	1 810	1 030
	5 500	4 070	3 330	2 580	2 180	1 760	1 000
	6 000	3 880	3 190	2 480	2 100	1 690	962
	6 500	3 680	3 030	2 360	2 000	1 620	918
	7 000	3 450	2 850	2 220	1 890	1 530	868
	7 500	3 220	2 660	2 080	1 770	1 440	816
	8 000	2 990	2 480	1 940	1 650	1 350	763
	8 500	2 760	2 300	1 800	1 530	1 250	710
	9 000	2 550	2 120	1 670	1 420	1 160	659
	9 500	2 350	1 960	1 540	1 320	1 080	610
	10 000	2 170	1 810	1 430	1 220	999	565
	10 500	2 000	1 670	1 320	1 120	925	523
	11 000	1 850	1 540	1 220	1 040	857	484
	11 500	1 710	1 430	1 130	964	794	449
	12 000	1 580	1 320	1 050	895	737	416
	12 500	1 470	1 230	974	831	685	387
	13 000	1 360	1 140	906	774	638	360
	14 000	1 190	995	789	674	556	314
	15 000	1 040	872	692	591	488	276
	16 000	918	770	611	522	431	243
	17 000	815	684	543	464	383	216
	18 000	729	611	485	415	343	194
PROPERTIES AND DESIGN DATA							
Area (mm^2)		14 500	11 800	9 090	7 650	6 190	4 710
r (mm)		96.1	97.6	99.1	99.9	101	101
M_r (kN·m)		400	334	260	221	155	96.6
$(b_{el}/t)\sqrt{350}$		224	299	424	524	673	919
IMPERIAL SIZE AND WEIGHT							
Weight (lb./ft.)		76.4	62.5	47.9	40.4	32.6	24.8
Thickness (in.)		0.625	0.500	0.375	0.313	0.250	0.188
Size (in.)		10 x 10					

‡ Class 4

C_r calculated according to S16-14 Clause 13.3.5(b)

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)

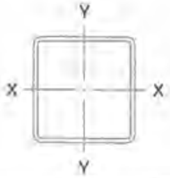


G40.21 350W
CLASS H
$\phi = 0.90$

Section (mm x mm x mm)	HSS 203 x 203					
	16	13	9.5	7.9	6.4	‡ 4.8
Mass (kg/m)	88.3	72.7	56.1	47.4	38.4	29.3
Effective length (KL) in millimetres with respect to the least radius of gyration						
0	3 530	2 920	2 250	1 900	1 540	1 100
500	3 530	2 920	2 250	1 900	1 540	1 100
1 000	3 530	2 920	2 250	1 900	1 540	1 100
1 500	3 520	2 910	2 250	1 900	1 540	1 100
2 000	3 510	2 910	2 240	1 900	1 540	1 090
2 500	3 490	2 890	2 230	1 890	1 530	1 090
3 000	3 440	2 850	2 200	1 860	1 510	1 080
3 500	3 360	2 790	2 160	1 830	1 490	1 060
4 000	3 240	2 700	2 090	1 770	1 440	1 030
4 500	3 080	2 570	2 000	1 700	1 380	989
5 000	2 880	2 420	1 890	1 610	1 310	938
5 500	2 660	2 240	1 760	1 500	1 230	878
6 000	2 430	2 060	1 620	1 380	1 130	813
6 500	2 200	1 870	1 480	1 270	1 040	747
7 000	1 990	1 700	1 350	1 150	946	681
7 500	1 790	1 530	1 220	1 050	859	620
8 000	1 620	1 380	1 100	947	779	562
8 500	1 460	1 250	999	858	706	510
9 000	1 320	1 130	906	779	641	464
9 500	1 200	1 030	823	708	584	422
10 000	1 090	938	750	646	532	385
10 500	996	857	686	590	487	352
11 000	912	785	628	541	446	323
11 500	838	721	578	498	410	297
12 000	772	664	533	459	378	274
12 500	713	614	492	424	350	254
13 000	661	569	456	393	324	235
14 000	571	492	395	340	281	204
15 000	499	430	345	297	245	178
16 000						157
17 000						
18 000						
PROPERTIES AND DESIGN DATA						
Area (mm^2)	11 200	9 260	7 150	6 040	4 900	3 730
r (mm)	75.3	76.9	78.4	79.2	79.9	80.7
M_r (kN·m)	244	205	162	138	113	71.6
$(b_{el}/t)\sqrt{350}$	165	224	324	404	524	720
IMPERIAL SIZE AND WEIGHT						
Weight (lb./ft.)	59.3	48.9	37.7	31.9	25.8	19.7
Thickness (in.)	0.625	0.500	0.375	0.313	0.250	0.188
Size (in.)	8 x 8					

‡ Class 4

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS H
$\phi = 0.90$

Section (mm x mm x mm)	HSS 178 x 178						HSS 152 x 152				
	16	13	9.5	7.9	6.4	† 4.8	13	9.5	7.9	6.4	4.8
Mass (kg/m)	75.6	62.6	48.5	41.1	33.4	25.5	52.4	40.9	34.7	28.3	21.7
Effective length (KL) in millimetres with respect to the least radius of gyration											
0	3 040	2 510	1 950	1 650	1 340	1 020	2 100	1 640	1 400	1 140	869
500	3 040	2 510	1 950	1 650	1 340	1 020	2 100	1 640	1 400	1 140	869
1 000	3 040	2 510	1 950	1 650	1 340	1 020	2 100	1 640	1 390	1 140	869
1 500	3 030	2 510	1 940	1 640	1 340	1 020	2 100	1 630	1 390	1 130	867
2 000	3 010	2 490	1 930	1 640	1 330	1 020	2 070	1 620	1 380	1 120	859
2 500	2 970	2 460	1 910	1 620	1 320	1 010	2 020	1 580	1 350	1 100	843
3 000	2 890	2 400	1 870	1 590	1 290	989	1 930	1 520	1 290	1 060	813
3 500	2 770	2 310	1 810	1 530	1 250	959	1 790	1 420	1 210	997	767
4 000	2 600	2 180	1 710	1 460	1 190	914	1 620	1 290	1 110	916	707
4 500	2 400	2 020	1 590	1 360	1 110	857	1 440	1 160	997	824	638
5 000	2 170	1 840	1 460	1 250	1 020	791	1 260	1 020	881	731	567
5 500	1 940	1 660	1 320	1 130	931	720	1 100	891	773	642	499
6 000	1 730	1 480	1 180	1 020	838	649	955	777	676	563	438
6 500	1 530	1 320	1 050	908	751	582	833	680	592	494	385
7 000	1 360	1 170	940	811	671	521	730	597	520	434	339
7 500	1 210	1 040	839	724	600	466	643	526	459	384	300
8 000	1 080	930	750	648	537	418	569	467	407	340	266
8 500	964	833	673	582	483	376	507	416	363	304	237
9 000	866	750	606	524	435	339	454	373	326	272	213
9 500	782	677	548	474	393	307	409	336	293	245	192
10 000	709	614	497	430	357	278	370	304	265	222	174
10 500	645	559	453	392	325	254	336	276	241	202	158
11 000	589	511	414	358	298	232	307	252	220	184	144
11 500	540	469	379	329	273	213		231	202	169	132
12 000	497	431	349	302	251	196					
12 500	459	398	322	279	232	181					
13 000		368	298	258	215	168					
14 000						145					
15 000											
16 000											
17 000											
18 000											

PROPERTIES AND DESIGN DATA

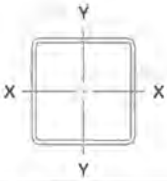
Area (mm^2)	9 640	7 970	6 180	5 230	4 250	3 250	6 680	5 210	4 430	3 610	2 760
r (mm)	64.9	66.5	68.0	68.8	69.6	70.3	56.1	57.6	58.4	59.2	59.9
M_r (kN·m)	180	152	121	104	85.4	57.0	108	86.6	74.7	61.7	47.9
$(b_{el}/t)\sqrt{350}$	135	187	274	344	449	621	150	224	284	374	522

IMPERIAL SIZE AND WEIGHT

Weight (lb./ft.)	50.8	42.1	32.6	27.6	22.4	17.1	35.2	27.5	23.3	19.0	14.6
Thickness (in.)	0.625	0.500	0.375	0.313	0.250	0.188	0.500	0.375	0.313	0.250	0.188
Size (in.)	7 x 7						6 x 6				

† Class 3

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



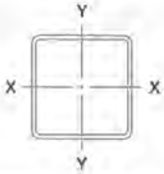
G40.21 350W
CLASS H
$\phi = 0.90$

Section (mm x mm x mm)	HSS 127 x 127					
	13	9.5	7.9	6.4	4.8	‡ 3.2 #
Mass (kg/m)	42.3	33.3	28.4	23.2	17.9	12.2
Effective length (KL) in millimetres with respect to the least radius of gyration						
0	1 700	1 340	1 140	932	718	485
500	1 700	1 340	1 140	932	718	485
1 000	1 690	1 330	1 140	931	717	484
1 500	1 680	1 320	1 130	925	713	482
2 000	1 630	1 290	1 110	906	699	473
2 500	1 540	1 230	1 050	866	670	455
3 000	1 400	1 120	969	800	622	424
3 500	1 220	995	861	715	558	383
4 000	1 040	857	745	622	488	336
4 500	879	729	636	532	420	290
5 000	741	618	540	454	359	249
5 500	628	525	460	387	307	213
6 000	536	449	394	332	263	183
6 500	462	388	340	287	228	159
7 000	401	337	296	250	198	138
7 500	351	295	259	219	174	121
8 000	309	260	229	193	153	107
8 500	275	231	203	172	136	95
9 000	245	207	182	153	122	85
9 500			163	138	110	77
10 000						69
10 500						
11 000						
11 500						
12 000						
12 500						
13 000						
14 000						
15 000						
16 000						
17 000						
18 000						
PROPERTIES AND DESIGN DATA						
Area (mm^2)	5 390	4 240	3 620	2 960	2 280	1 550
r (mm)	45.7	47.3	48.0	48.8	49.6	50.3
M_r (kN·m)	70.9	57.6	50.1	41.6	32.4	19.4
$(b_{el}/t)\sqrt{350}$	112	174	224	299	422	672
IMPERIAL SIZE AND WEIGHT						
Weight (lb./ft.)	28.4	22.4	19.1	15.6	12.0	8.17
Thickness (in.)	0.500	0.375	0.313	0.250	0.188	0.125
Size (in.)	5 x 5					

‡ Class 4

C_r calculated according to S16-14 Clause 13.3.5(b)

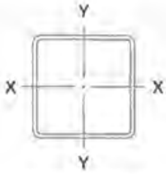
SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS H
$\phi = 0.90$

Section (mm x mm x mm)	HSS 102 x 102						HSS 89 x 89			
	13	9.5	7.9	6.4	4.8	3.2	9.5	7.9	6.4	4.8
Mass (kg/m)	32.2	25.7	22.1	18.2	14.1	9.62	21.9	18.9	15.6	12.2
Effective length (KL) in millimetres with respect to the least radius of gyration										
0	1 290	1 030	885	731	564	387	879	759	627	488
500	1 290	1 030	885	731	564	387	878	759	627	488
1 000	1 280	1 030	881	728	562	386	871	753	622	485
1 500	1 250	1 010	863	714	552	380	833	724	600	469
2 000	1 160	941	813	675	524	362	743	651	544	429
2 500	1 000	831	723	605	473	329	613	544	459	366
3 000	823	695	611	515	406	285	483	433	368	296
3 500	662	566	501	424	337	238	378	340	291	236
4 000	531	458	407	346	276	196	299	270	232	188
4 500	431	373	332	283	227	161	240	217	187	152
5 000	354	308	274	234	188	134	196	178	153	125
5 500	295	257	229	196	157	112	163	148	127	104
6 000	249	217	194	166	133	95	137	125	107	88
6 500	213	186	166	142	114	82		106	92	75
7 000	184	161	144	123	99	71				
7 500			125	107	86	62				
8 000						54				
8 500										
9 000										
9 500										
10 000										
10 500										
11 000										
11 500										
12 000										
12 500										
13 000										
14 000										
15 000										
16 000										
17 000										
18 000										
PROPERTIES AND DESIGN DATA										
Area (mm^2)	4 100	3 280	2 810	2 320	1 790	1 230	2 790	2 410	1 990	1 550
r (mm)	35.3	36.9	37.7	38.4	39.2	40.0	31.7	32.5	33.2	34.0
M_r (kN·m)	41.3	34.7	30.5	25.6	20.3	14.1	25.4	22.5	19.1	15.2
$(b_{el}/t)\sqrt{350}$	74.8	125	165	224	323	523	99.7	135	187	273
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)	21.6	17.3	14.8	12.2	9.45	6.47	14.7	12.7	10.5	8.17
Thickness (in.)	0.500	0.375	0.313	0.250	0.188	0.125	0.375	0.313	0.250	0.188
Size (in.)	4 x 4						3½ x 3½			

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W CLASS H $\phi = 0.90$

Section (mm x mm x mm)		HSS 76 x 76					HSS 64 x 64			HSS 51 x 51		
		9.5	7.9	6.4	4.8	3.2	6.4	4.8	3.2	6.4	4.8	3.2
Mass (kg/m)		18.1	15.7	13.1	10.3	7.09	10.6	8.35	5.82	8.05	6.45	4.55
Effective length (KL) in millimetres with respect to the least radius of gyration	0	728	633	526	413	284	425	334	233	324	259	183
	500	727	633	526	412	284	424	333	233	323	257	182
	1 000	713	622	518	407	281	409	323	227	290	235	169
	1 500	651	574	481	381	265	349	281	200	206	174	129
	2 000	531	476	405	325	229	260	214	156	133	114	87
	2 500	402	365	314	256	183	185	154	114	88	77	59
	3 000	300	274	238	195	141	134	112	83	62	54	42
	3 500	227	209	182	150	109	100	84	62	46	40	31
	4 000	177	163	142	117	85	77	65	48			
	4 500	141	130	113	94	68	61	51	38			
	5 000	114	106	92	76	56						
	5 500			76	63	46						
	6 000											
	6 500											
	7 000											
	7 500											
	8 000											
	8 500											
	9 000											
	9 500											
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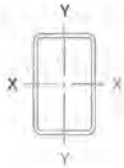
	11 500											
	12 000											
	12 500											
	13 000											
	14 000											
	15 000											
	16 000											
	17 000											
	18 000											
PROPERTIES AND DESIGN DATA												
Area (mm^2)		2 310	2 010	1 670	1 310	903	1 350	1 060	741	1 030	821	580
r (mm)		26.5	27.3	28.0	28.8	29.6	22.8	23.6	24.4	17.6	18.4	19.2
M_r (kN·m)		17.5	15.7	13.5	10.8	7.72	8.85	7.25	5.23	5.17	4.35	3.21
$(b_{el}/t)\sqrt{350}$		74.8	105	150	223	373	112	174	299	74.8	124	224
IMPERIAL SIZE AND WEIGHT												
Weight (lb./ft.)		12.2	10.6	8.81	6.89	4.76	7.11	5.61	3.91	5.41	4.33	3.06
Thickness (in.)		0.375	0.313	0.250	0.188	0.125	0.250	0.188	0.125	0.250	0.188	0.125
Size (in.)		3 x 3					2½ x 2½			2 x 2		

Section (mm x mm x mm)		HSS 305 x 203					HSS 305 x 152				
		16	13	9.5	† 7.9	‡ 6.4	16	13	9.5	† 7.9	‡ 6.4
Mass (kg/m)		114	93.0	71.3	60.1	48.6	101	82.8	63.7	53.7	43.5
Effective length (KL) in millimetres with respect to the least radius of gyration	0	4 570	3 720	2 860	2 410	1 740	4 060	3 340	2 560	2 160	1 540
	500	4 570	3 720	2 860	2 410	1 740	4 060	3 340	2 560	2 160	1 540
	1 000	4 570	3 720	2 860	2 410	1 740	4 060	3 340	2 560	2 160	1 540
	1 500	4 560	3 710	2 860	2 410	1 740	4 050	3 330	2 550	2 150	1 530
	2 000	4 550	3 710	2 860	2 400	1 740	4 020	3 300	2 530	2 140	1 520
	2 500	4 530	3 690	2 840	2 390	1 730	3 940	3 250	2 490	2 110	1 500
	3 000	4 480	3 650	2 820	2 370	1 710	3 800	3 140	2 420	2 050	**1 460**
	3 500	4 400	3 590	2 770	2 330	1 690	3 590	2 980	2 310	1 960	1 400
	4 000	4 270	3 490	2 700	2 280	1 650	3 310	2 770	2 150	1 830	1 310
	4 500	4 100	3 360	2 600	2 200	**1 590**	2 990	2 510	1 970	1 680	1 210
	5 000	3 880	3 190	2 480	2 100	1 520	2 660	2 250	1 770	1 510	1 090
	5 500	3 620	2 990	2 330	1 980	1 440	2 350	1 990	1 570	1 350	974
	6 000	3 350	2 770	2 170	1 840	1 340	2 060	1 750	1 390	1 190	865
	6 500	3 070	2 550	2 000	1 700	1 240	1 810	1 540	1 230	1 060	766
	7 000	2 790	2 330	1 840	1 560	1 140	1 590	1 360	1 090	935	679
	7 500	2 540	2 120	1 670	1 430	1 040	1 410	1 210	962	830	603
	8 000	2 300	1 920	1 520	1 300	951	1 250	1 070	857	739	537
	8 500	2 090	1 750	1 390	1 180	866	1 120	958	766	661	481
	9 000	1 890	1 590	1 260	1 080	789	1 000	860	688	594	432
	9 500	1 720	1 450	1 150	982	720	903	776	620	536	390
	10 000	1 570	1 320	1 050	898	659	818	702	562	486	354
	10 500	1 440	1 210	962	822	603	743	639	511	442	322
	11 000	1 320	1 110	883	755	554	679	583	467	404	294
	11 500	1 210	1 020	813	695	510	622	535	428	370	269
	12 000	1 120	940	750	641	471	572	492	394	340	248
	12 500	1 030	869	694	593	436			363	314	229
	13 000	957	806	643	550	404					
	14 000	829	698	557	477	350					
	15 000	724	610	487	416	306					
	16 000		537	429	367	270					
PROPERTIES AND DESIGN DATA											
Area (mm^2)		14 500	11 800	9 090	7 650	6 190	12 900	10 600	8 120	6 850	5 540
r_x (mm)		110	111	113	114	115	105	106	108	109	110
r_y (mm)		79.8	81.2	82.7	83.4	84.1	60.1	61.5	62.9	63.7	64.4
r_x / r_y		1.38	1.37	1.37	1.37	1.37	1.75	1.72	1.72	1.71	1.71
M_{rx} (kN·m)		450	375	292	248	^ 202	375	315	247	210	^ 171
M_{ry} (kN·m)		340	283	221	165	116	230	193	152	115	80.5
$(b_{el}/t)\sqrt{350}$		284	374	524	643	823	284	374	524	643	823
IMPERIAL SIZE AND WEIGHT											
Weight (lb./ft.)		76.4	62.5	47.9	40.4	32.6	67.8	55.7	42.8	36.1	29.2
Thickness (in.)		0.625	0.500	0.375	0.313	0.250	0.625	0.500	0.375	0.313	0.250
Size (in.)		12 x 8					12 x 6				

† Class 3 in bending about Y-Y axis
^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.
‡ Class 4

RECTANGULAR HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)



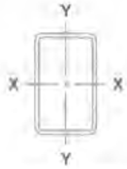
G40.21 350W CLASS H $\phi = 0.90$

Section (mm x mm x mm)	HSS 254 x 203					HSS 254 x 152					
	16	13	9.5	7.9	‡6.4	16	13	9.5	7.9	‡6.4	‡4.8
Mass (kg/m)	101	82.8	63.7	53.7	43.5	88.3	72.7	56.1	47.4	38.4	29.3
Effective length (KL) in millimetres with respect to the least radius of gyration: 0	4 060	3 340	2 560	2 160	1 740	3 530	2 920	2 250	1 900	1 540	983
500	4 060	3 340	2 560	2 160	1 740	3 530	2 920	2 250	1 900	1 540	983
1 000	4 060	3 340	2 560	2 160	1 740	3 530	2 920	2 250	1 900	1 540	983
1 500	4 060	3 340	2 560	2 160	1 740	3 520	2 910	2 250	1 900	1 530	981
2 000	4 050	3 330	2 550	2 150	1 740	3 480	2 880	2 230	1 880	**1 520**	975
2 500	4 020	3 310	2 540	2 140	1 730	3 410	2 830	2 190	1 850	1 500	960
3 000	3 980	3 270	2 510	2 120	**1 710**	3 280	2 730	2 120	1 800	1 460	934
3 500	3 890	3 210	2 470	2 080	1 680	3 080	2 580	2 010	1 710	1 390	892
4 000	3 770	3 120	2 400	2 030	1 640	2 830	2 380	1 870	1 590	1 300	**834**
4 500	3 600	2 990	2 310	1 950	1 580	2 540	2 150	1 700	1 450	1 190	765
5 000	3 400	2 820	2 190	1 860	1 500	2 250	1 920	1 520	1 300	1 070	690
5 500	3 160	2 630	2 050	1 740	1 410	1 970	1 690	1 350	1 160	949	615
6 000	2 910	2 430	1 900	1 620	1 310	1 730	1 490	1 190	1 020	840	546
6 500	2 650	2 230	1 740	1 490	1 210	1 510	1 310	1 050	901	742	483
7 000	2 410	2 030	1 590	1 360	1 110	1 330	1 150	924	796	656	427
7 500	2 180	1 840	1 450	1 240	1 010	1 180	1 020	819	706	582	379
8 000	1 970	1 670	1 310	1 130	919	1 040	904	728	628	518	338
8 500	1 780	1 510	1 190	1 020	836	930	807	650	561	463	302
9 000	1 620	1 370	1 080	930	761	834	724	584	504	416	272
9 500	1 470	1 250	986	847	693	752	652	526	454	376	245
10 000	1 340	1 140	900	773	633	680	591	477	412	340	222
10 500	1 220	1 040	823	707	580	618	537	434	374	310	202
11 000	1 120	952	755	649	532	564	490	396	342	283	185
11 500	1 030	875	695	597	489	517	449	363	313	259	169
12 000	949	807	641	551	452		413	334	288	238	156
12 500	877	746	592	509	418					220	144
13 000	813	691	549	472	387						
14 000	703	598	476	409	335						
15 000	614	523	415	357	293						
16 000			366	315	258						
PROPERTIES AND DESIGN DATA											
Area (mm^2)	12 900	10 600	8 120	6 850	5 540	11 200	9 260	7 150	6 040	4 900	3 730
r_x (mm)	92.8	94.4	96.0	96.8	97.6	88.3	90.1	91.9	92.8	93.6	94.5
r_y (mm)	77.9	79.3	80.8	81.6	82.3	58.8	60.3	61.7	62.4	63.1	63.8
r_x / r_y	1.19	1.19	1.19	1.19	1.19	1.50	1.49	1.49	1.49	1.48	1.48
M_{rx} (kN·m)	340	284	223	190	^ 155	280	235	186	158	^ 129	^ 99.9
M_{ry} (kN·m)	291	244	191	163	116	195	164	130	111	80.3	49.1
$(b_{el}/t)\sqrt{350}$	224	299	424	524	673	224	299	424	524	673	919
IMPERIAL SIZE AND WEIGHT											
Weight (lb./ft.)	67.8	55.7	42.8	36.1	29.2	59.3	48.9	37.7	31.9	25.8	19.7
Thickness (in.)	0.625	0.500	0.375	0.313	0.250	0.625	0.500	0.375	0.313	0.250	0.188
Size (in.)	10 x 8					10 x 6					

‡ Class 4

^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.

RECTANGULAR HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS H
$\phi = 0.90$

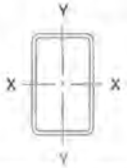
Section (mm x mm x mm)	HSS 203 x 152						HSS 203 x 102				
	16	13	9.5	7.9	6.4	‡ 4.8	13	9.5	7.9	6.4	‡ 4.8
Mass (kg/m)	75.6	62.6	48.5	41.1	33.4	25.5	52.4	40.9	34.7	28.3	21.7
Effective length (KL) in millimetres with respect to the least radius of gyration											
0	3 040	2 510	1 950	1 650	1 340	985	2 100	1 640	1 400	1 140	831
500	3 040	2 510	1 950	1 650	1 340	985	2 100	1 640	1 400	1 140	830
1 000	3 030	2 510	1 950	1 650	1 340	984	2 100	1 640	1 390	1 130	829
1 500	3 020	2 500	1 940	1 640	1 330	982	2 060	1 610	1 370	1 120	**819**
2 000	2 990	2 480	1 920	1 630	1 320	975	1 960	1 540	1 320	1 080	789
2 500	2 920	2 430	1 890	1 600	1 300	**959**	1 760	1 410	1 210	994	732
3 000	2 800	2 330	1 820	1 550	1 260	930	1 510	1 220	1 060	875	648
3 500	2 610	2 190	1 720	1 460	1 200	884	1 250	1 020	893	743	554
4 000	2 380	2 010	1 580	1 350	1 110	822	1 030	846	741	619	464
4 500	2 120	1 800	1 430	1 230	1 010	749	842	698	614	514	386
5 000	1 860	1 590	1 270	1 090	901	672	698	580	511	429	323
5 500	1 630	1 400	1 120	967	798	596	584	487	429	361	272
6 000	1 420	1 220	984	850	703	526	495	413	365	307	231
6 500	1 240	1 070	864	748	619	464	424	354	313	263	198
7 000	1 090	941	761	659	546	410	367	307	271	228	172
7 500	958	831	673	583	484	363	320	268	237	199	150
8 000	850	737	598	518	430	323		236	208	176	132
8 500	757	658	533	463	384	289					118
9 000	679	590	478	415	345	259					
9 500	611	531	431	374	311	234					
10 000	553	481	390	339	282	212					
10 500	502	437	355	308	256	193					
11 000	458	399	324	281	234	176					
11 500		365	297	258	214	161					
12 000			273	237	197	148					
12 500											
13 000											
14 000											
15 000											
16 000											
PROPERTIES AND DESIGN DATA											
Area (mm^2)	9 640	7 970	6 180	5 230	4 250	3 250	6 680	5 210	4 430	3 610	2 760
r_x (mm)	71.7	73.4	75.1	75.9	76.7	77.5	68.4	70.3	71.2	72.2	73.1
r_y (mm)	57.1	58.6	60.0	60.8	61.5	62.2	39.1	40.5	41.3	42.0	42.7
r_x / r_y	1.26	1.25	1.25	1.25	1.25	1.25	1.75	1.74	1.72	1.72	1.71
M_{rx} (kN·m)	196	166	132	113	92.9	^ 71.8	128	103	88.5	73.1	^ 56.7
M_{ry} (kN·m)	160	136	108	92.9	76.5	49.3	77.5	62.7	54.2	45.0	29.4
$(b_{el} / t)\sqrt{350}$	165	224	324	404	524	720	224	324	404	524	720
IMPERIAL SIZE AND WEIGHT											
Weight (lb./ft.)	50.8	42.1	32.6	27.6	22.4	17.1	35.2	27.5	23.3	19.0	14.6
Thickness (in.)	0.625	0.500	0.375	0.313	0.250	0.188	0.500	0.375	0.313	0.250	0.188
Size (in.)	8 x 6						8 x 4				

‡ Class 4

^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.

RECTANGULAR HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)



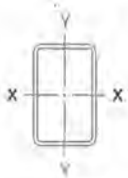
G40.21 350W CLASS H $\phi = 0.90$

Section (mm x mm x mm)	HSS 178 x 127					HSS 152 x 102					
	13	9.5	7.9	6.4	† 4.8	13	9.5	7.9	6.4	4.8	‡ 3.2
Mass (kg/m)	52.4	40.9	34.7	28.3	21.7	42.3	33.3	28.4	23.2	17.9	12.2
Effective length (KL) in millimetres with respect to the least radius of gyration											
0	2 100	1 640	1 400	1 140	869	1 700	1 340	1 140	932	718	437
500	2 100	1 640	1 400	1 140	869	1 700	1 340	1 140	932	718	437
1 000	2 100	1 640	1 390	1 140	869	1 690	1 330	1 140	930	716	435
1 500	2 090	1 630	1 390	1 130	864	1 660	1 310	1 120	916	706	430
2 000	2 040	1 600	1 360	1 110	850	1 560	1 240	1 070	876	677	414
2 500	1 950	1 530	1 310	1 070	821	1 390	1 120	968	800	622	**382**
3 000	1 790	1 420	1 220	1 000	770	1 170	961	836	695	544	337
3 500	1 590	1 280	1 100	906	700	960	798	698	584	460	286
4 000	1 380	1 120	964	799	620	780	654	574	482	381	239
4 500	1 180	959	831	692	539	638	537	473	398	316	198
5 000	1 000	819	712	595	464	526	445	392	331	263	165
5 500	852	701	610	510	399	440	373	329	278	221	139
6 000	730	602	525	440	344	372	316	279	236	188	118
6 500	630	520	454	381	298	319	271	239	202	161	102
7 000	548	453	395	332	260	276	234	207	175	139	88
7 500	480	397	347	291	229	241	204	181	153	122	77
8 000	424	351	306	257	202				135	107	68
8 500	376	312	272	229	180						
9 000	336	279	244	205	161						
9 500	302	251	219	184	145						
10 000			198	166	131						
10 500											
11 000											
11 500											
12 000											
12 500											
13 000											
14 000											
15 000											
16 000											
PROPERTIES AND DESIGN DATA											
Area (mm^2)	6 680	5 210	4 430	3 610	2 760	5 390	4 240	3 620	2 960	2 280	1 550
r_x (mm)	62.9	64.6	65.4	66.2	67.1	52.2	54.0	54.9	55.7	56.5	57.4
r_y (mm)	48.1	49.6	50.3	51.1	51.8	37.7	39.2	39.9	40.6	41.3	42.1
r_x / r_y	1.31	1.30	1.30	1.30	1.30	1.38	1.38	1.38	1.37	1.37	1.36
M_{rx} (kN·m)	119	95.4	82.2	68.0	52.9	79.4	64.9	56.1	46.6	36.5	^ 25.3
M_{ry} (kN·m)	93.9	75.6	65.2	53.9	36.9	59.5	48.8	42.2	35.3	27.7	14.5
$(b_{el}/t)\sqrt{350}$	187	274	344	449	621	150	224	284	374	522	822
IMPERIAL SIZE AND WEIGHT											
Weight (lb./ft.)	35.2	27.5	23.3	19.0	14.6	28.4	22.4	19.1	15.6	12.0	8.17
Thickness (in.)	0.500	0.375	0.313	0.250	0.188	0.500	0.375	0.313	0.250	0.188	0.125
Size (in.)	7 x 5					6 x 4					

† Class 3 in bending about Y-Y axis ^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.

‡ Class 4

RECTANGULAR HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



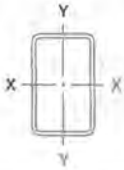
G40.21 350W
CLASS H
$\phi = 0.90$

Section (mm x mm x mm)	HSS 152 x 76					
	13	9.5	7.9	6.4	4.8	‡ 3.2
Mass (kg/m)	37.3	29.5	25.2	20.7	16.0	10.9
Effective length (KL) in millimetres with respect to the least radius of gyration						
0	1 500	1 180	1 010	832	643	386
500	1 500	1 180	1 010	831	642	386
1 000	1 470	1 170	1 000	823	637	383
1 500	1 370	1 100	950	783	609	**367**
2 000	1 150	949	827	690	541	329
2 500	894	755	665	561	445	274
3 000	677	580	516	438	350	217
3 500	517	447	399	340	273	171
4 000	404	350	313	268	216	135
4 500	322	280	251	215	173	109
5 000	262	228	205	176	142	89
5 500	217	189	170	146	118	74
6 000			143	123	99	62
6 500						
7 000						
7 500						
8 000						
8 500						
9 000						
9 500						
10 000						
10 500						
11 000						
11 500						
12 000						
12 500						
13 000						
14 000						
15 000						
16 000						
PROPERTIES AND DESIGN DATA						
Area (mm^2)	4 750	3 760	3 220	2 640	2 040	1 390
r_x (mm)	49.3	51.3	52.3	53.2	54.1	55.0
r_y (mm)	28.0	29.4	30.1	30.8	31.5	32.2
r_x / r_y	1.76	1.74	1.74	1.73	1.72	1.71
M_{rx} (kN·m)	65.2	53.9	46.9	39.4	30.9	^ 21.5
M_{ry} (kN·m)	39.1	32.8	28.7	24.1	19.1	10.1
$(b_{el}/t)\sqrt{350}$	150	224	284	374	522	822
IMPERIAL SIZE AND WEIGHT						
Weight (lb./ft.)	25.0	19.8	17.0	13.9	10.7	7.32
Thickness (in.)	0.500	0.375	0.313	0.250	0.188	0.125
Size (in.)	6 x 3					

‡ Class 4

^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.

RECTANGULAR HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS H
ϕ = 0.90

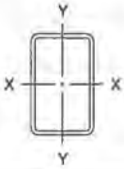
Section (mm x mm x mm)		HSS 127 x 76					
		13	9.5	7.9	6.4	4.8	‡ 3.2
Mass (kg/m)		32.2	25.7	22.1	18.2	14.1	9.62
Effective length (KL) in millimetres with respect to the least radius of gyration	0	1 290	1 030	885	731	564	387
	500	1 290	1 030	885	730	564	386
	1 000	1 270	1 020	874	723	558	**383**
	1 500	1 170	953	823	684	531	366
	2 000	971	812	709	596	468	326
	2 500	745	638	564	479	380	269
	3 000	560	487	433	371	297	212
	3 500	426	373	334	287	231	165
	4 000	332	292	262	226	182	131
	4 500	265	233	209	181	146	105
	5 000	215	190	171	148	119	86
	5 500		158	142	122	99	71
	6 000				103	83	60
	6 500						
	7 000						
	7 500						
	8 000						
	8 500						
	9 000						
	9 500						
	10 000						
	10 500						
	11 000						
	11 500						
	12 000						
	12 500						
	13 000						
	14 000						
	15 000						
	16 000						
PROPERTIES AND DESIGN DATA							
Area (mm^2)		4 100	3 280	2 810	2 320	1 790	1 230
r_x (mm)		41.4	43.3	44.2	45.1	45.9	46.8
r_y (mm)		27.3	28.7	29.4	30.1	30.8	31.6
r_x / r_y		1.52	1.51	1.50	1.50	1.49	1.48
M_{rx} (kN·m)		47.6	39.7	35.0	29.4	23.2	^ 16.2
M_{ry} (kN·m)		32.8	27.7	24.3	20.6	16.3	10.1
$(b_{el}/t)\sqrt{350}$		112	174	224	299	422	672
IMPERIAL SIZE AND WEIGHT							
Weight (lb./ft.)		21.6	17.3	14.8	12.2	9.45	6.47
Thickness (in.)		0.500	0.375	0.313	0.250	0.188	0.125
Size (in.)		5 x 3					

‡ Class 4

^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.

RECTANGULAR HOLLOW SECTIONS

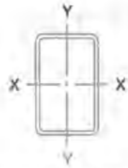
Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS H
$\phi = 0.90$

Section (mm x mm x mm)	HSS 102 x 76					HSS 102 x 51				
	9.5	7.9	6.4	4.8	3.2	9.5	7.9	6.4	4.8	3.2
Mass (kg/m)	21.9	18.9	15.6	12.2	8.35	18.1	15.7	13.1	10.3	7.09
Effective length (KL) in millimetres with respect to the least radius of gyration										
0	879	759	627	488	334	728	633	526	413	284
500	878	759	626	488	334	724	631	524	411	284
1 000	865	748	619	483	330	659	582	489	388	269
1 500	802	699	582	457	314	482	438	379	308	219
2 000	672	593	501	397	276	315	292	258	214	155
2 500	520	464	398	319	224	211	197	175	147	107
3 000	393	354	305	247	175	149	140	124	104	77
3 500	300	271	235	191	136	110	103	92	77	57
4 000	234	212	184	150	107				60	44
4 500	187	169	147	120	86					
5 000	152	138	120	98	70					
5 500	126	114	100	81	58					
6 000				68	49					
6 500										
7 000										
7 500										
8 000										
8 500										
9 000										
9 500										
10 000										
10 500										
11 000										
11 500										
12 000										
12 500										
13 000										
14 000										
15 000										
16 000										
PROPERTIES AND DESIGN DATA										
Area (mm^2)	2 790	2 410	1 990	1 550	1 060	2 310	2 010	1 670	1 310	903
r_x (mm)	35.0	35.9	36.7	37.5	38.4	32.2	33.2	34.2	35.1	36.1
r_y (mm)	27.8	28.5	29.3	30.0	30.7	18.2	18.9	19.6	20.3	21.0
r_x / r_y	1.26	1.26	1.25	1.25	1.25	1.77	1.76	1.74	1.73	1.72
M_{rx} (kN·m)	27.7	24.5	20.8	16.6	11.7	20.7	18.6	16.0	12.9	9.14
M_{ry} (kN·m)	22.6	20.0	17.0	13.6	9.58	12.3	11.2	9.70	7.88	5.64
$(b_{el}/t)\sqrt{350}$	125	165	224	323	523	125	165	224	323	523
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)	14.7	12.7	10.5	8.17	5.61	12.2	10.6	8.81	6.89	4.76
Thickness (in.)	0.375	0.313	0.250	0.188	0.125	0.375	0.313	0.250	0.188	0.125
Size (in.)	4 x 3					4 x 2				

RECTANGULAR HOLLOW SECTIONS



G40.21 350W

Factored Axial Compressive Resistances, C_r (kN)

CLASS H

$\phi = 0.90$

Section (mm x mm x mm)	HSS 89 x 64		HSS 76 x 51			
	6.4	4.8	7.9	6.4	4.8	3.2
Mass (kg/m)	13.1	10.3	12.6	10.6	8.35	5.82
Effective length (KL) in millimetres with respect to the least radius of gyration						
0	526	413	504	425	334	233
500	525	412	501	423	333	233
1 000	510	402	456	391	310	219
1 500	448	357	332	294	240	174
2 000	345	280	216	196	164	121
2 500	251	206	145	132	111	83
3 000	183	151	102	94	79	59
3 500	138	114	76	69	59	44
4 000	106	88				34
4 500	84	70				
5 000						
5 500						
6 000						
6 500						
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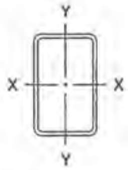
9 500						
10 000						
10 500						
11 000						
11 500						
12 000						
12 500						
13 000						
14 000						
15 000						
16 000						
PROPERTIES AND DESIGN DATA						
Area (mm^2)	1 670	1 310	1 600	1 350	1 060	741
r_x (mm)	31.4	32.3	25.2	26.1	27.0	27.8
r_y (mm)	24.1	24.8	18.1	18.9	19.6	20.3
r_x / r_y	1.30	1.30	1.39	1.38	1.38	1.37
M_{rx} (kN·m)	14.9	12.0	11.3	9.92	8.13	5.86
M_{ry} (kN·m)	11.7	9.48	8.47	7.43	6.11	4.41
$(b_{el}/t)\sqrt{350}$	187	273	105	150	223	373
IMPERIAL SIZE AND WEIGHT						
Weight (lb./ft.)	8.81	6.89	8.45	7.11	5.61	3.91
Thickness (in.)	0.250	0.188	0.313	0.250	0.188	0.125
Size (in.)	3½ x 2½		3 x 2			

RECTANGULAR HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W CLASS H $\phi = 0.90$

Section (mm x mm x mm)	HSS 76 x 38			HSS 64 x 38			HSS 51 x 25	
	6.4	4.8	3.2	6.4	4.8	3.2	4.8	3.2
Mass (kg/m)	9.31	7.40	5.18	8.05	6.45	4.55	4.54	3.28
Effective length (KL) in millimetres with respect to the least radius of gyration								
0	375	297	208	324	259	183	182	132
500	369	293	206	319	255	181	165	122
1 000	288	238	172	243	203	149	79	64
1 500	170	145	109	140	121	93	37	31
2 000	101	88	67	83	73	57		
2 500	66	57	44	54	47	37		
3 000			31			26		
3 500								
4 000								
4 500								
5 000								
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11 500								
12 000								
12 500								
13 000								
14 000								
15 000								
16 000								
PROPERTIES AND DESIGN DATA								
Area (mm^2)	1 190	942	660	1 030	821	580	578	418
r_x (mm)	24.7	25.6	26.6	20.7	21.6	22.5	16.1	17.1
r_y (mm)	14.0	14.7	15.4	13.6	14.3	15.1	9.08	9.78
r_x / r_y	1.76	1.74	1.73	1.52	1.51	1.49	1.77	1.75
M_{rx} (kN·m)	8.16	6.74	4.91	5.92	4.98	3.69	2.59	2.00
M_{ry} (kN·m)	4.91	4.10	3.02	4.10	3.47	2.57	1.55	1.21
$(b_{el} / t)\sqrt{350}$	150	223	373	112	174	299	124	224
IMPERIAL SIZE AND WEIGHT								
Weight (lb./ft.)	6.26	4.97	3.48	5.41	4.33	3.06	3.05	2.21
Thickness (in.)	0.250	0.188	0.125	0.250	0.188	0.125	0.188	0.125
Size (in.)	3 x 1½			2½ x 1½			2 x 1	

ROUND HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W CLASS H $\phi = 0.90$

Section (mm x mm)		HSS 406				HSS 356			HSS 324		
		16	13	9.5	† 6.4	13	9.5	† 6.4	13	9.5	6.4
Mass (kg/m)		153	123	93.3	62.6	107	81.3	54.7	97.5	73.9	49.7
Effective length (KL) in millimetres with respect to the least radius of gyration	0	6 140	4 950	3 750	2 510	4 320	3 280	2 200	3 910	2 960	1 990
	500	6 140	4 950	3 750	2 510	4 320	3 280	2 200	3 910	2 960	1 990
	1 000	6 140	4 950	3 750	2 510	4 320	3 280	2 200	3 910	2 960	1 990
	1 500	6 140	4 950	3 750	2 510	4 310	3 280	2 200	3 910	2 960	1 990
	2 000	6 140	4 940	3 750	2 510	4 310	3 270	2 190	3 900	2 960	1 990
	2 500	6 140	4 940	3 750	2 510	4 310	3 270	2 190	3 900	2 960	1 990
	3 000	6 130	4 940	3 740	2 510	4 300	3 270	2 190	3 890	2 950	1 990
	3 500	6 120	4 930	3 740	2 510	4 290	3 260	2 180	3 870	2 940	1 980
	4 000	6 100	4 920	3 730	2 500	4 270	3 240	2 170	3 840	2 920	1 960
	4 500	6 080	4 900	3 710	2 490	4 240	3 220	2 160	3 800	2 880	1 940
	5 000	6 040	4 870	3 690	2 480	4 190	3 180	2 140	3 740	2 840	1 910
	5 500	5 990	4 830	3 660	2 460	4 130	3 140	2 110	3 650	2 780	1 870
	6 000	5 920	4 770	3 620	2 430	4 050	3 080	2 070	3 550	2 700	1 820
	6 500	5 830	4 700	3 570	2 400	3 940	3 000	2 020	3 430	2 610	1 760
	7 000	5 720	4 620	3 510	2 360	3 820	2 910	1 960	3 280	2 510	1 690
	7 500	5 590	4 510	3 430	2 310	3 690	2 810	1 890	3 120	2 390	1 620
	8 000	5 440	4 390	3 340	2 250	3 530	2 700	1 820	2 960	2 260	1 530
	8 500	5 270	4 260	3 240	2 180	3 370	2 580	1 740	2 780	2 130	1 450
	9 000	5 080	4 110	3 130	2 110	3 200	2 450	1 650	2 610	2 000	1 360
	9 500	4 880	3 950	3 010	2 030	3 020	2 320	1 570	2 440	1 870	1 270
	10 000	4 660	3 780	2 880	1 950	2 850	2 180	1 480	2 280	1 750	1 190
	10 500	4 450	3 610	2 750	1 860	2 680	2 060	1 390	2 120	1 630	1 110
	11 000	4 230	3 430	2 620	1 770	2 510	1 930	1 310	1 970	1 520	1 040
	11 500	4 010	3 260	2 490	1 690	2 360	1 810	1 230	1 840	1 420	967
	12 000	3 800	3 090	2 360	1 600	2 210	1 700	1 150	1 710	1 320	902
	12 500	3 600	2 930	2 240	1 520	2 070	1 590	1 080	1 600	1 230	842
	13 000	3 400	2 770	2 120	1 440	1 940	1 500	1 020	1 490	1 150	787
	14 000	3 040	2 470	1 900	1 290	1 710	1 320	896	1 310	1 010	689
	15 000	2 710	2 210	1 700	1 150	1 510	1 170	793	1 150	887	607
	16 000	2 430	1 980	1 520	1 030	1 350	1 040	706	1 020	786	538
	17 000	2 180	1 780	1 370	927	1 200	926	630	907	700	479
	18 000	1 960	1 600	1 230	836	1 080	831	566	812	627	429
PROPERTIES AND DESIGN DATA											
Area (mm^2)		19 500	15 700	11 900	7 980	13 700	10 400	6 970	12 400	9 410	6 330
r (mm)		138	139	140	141	121	122	123	110	111	112
M_r (kN·m)		762	621	473	248	469	359	188	387	297	202
(D / t) 350		8 960	11 200	14 900	22 400	9 800	13 100	19 600	8 930	11 900	17 900
IMPERIAL SIZE AND WEIGHT											
Weight (lb./ft.)		103	82.9	62.7	42.1	72.2	54.7	36.8	65.5	49.6	33.4
Thickness (in.)		0.625	0.500	0.375	0.250	0.500	0.375	0.250	0.500	0.375	0.250
Size (in.)		16 OD				14 OD			12.75 OD		

† Class 3

ROUND HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W
CLASS H
$\phi = 0.90$

Section (mm x mm)	HSS 273					HSS 245	
	13	9.5	7.9	6.4	† 4.8	9.5	6.4
Mass (kg/m)	81.6	61.9	51.9	41.8	31.6	55.2	37.3
Effective length (KL) in millimetres with respect to the least radius of gyration							
0	3 280	2 490	2 080	1 680	1 270	2 210	1 500
500	3 280	2 490	2 080	1 680	1 270	2 210	1 500
1 000	3 280	2 490	2 080	1 680	1 270	2 210	1 500
1 500	3 270	2 480	2 080	1 680	1 270	2 210	1 500
2 000	3 270	2 480	2 080	1 670	1 270	2 210	1 490
2 500	3 260	2 470	2 070	1 670	1 260	2 200	1 490
3 000	3 240	2 460	2 060	1 660	1 260	2 180	1 470
3 500	3 210	2 440	2 040	1 640	1 250	2 140	1 450
4 000	3 160	2 400	2 010	1 620	1 230	2 090	1 420
4 500	3 080	2 350	1 970	1 590	1 200	2 020	1 370
5 000	2 980	2 270	1 910	1 540	1 170	1 920	1 310
5 500	2 860	2 180	1 830	1 480	1 120	1 810	1 240
6 000	2 710	2 070	1 740	1 410	1 070	1 690	1 150
6 500	2 550	1 950	1 640	1 330	1 010	1 560	1 070
7 000	2 380	1 820	1 540	1 240	948	1 430	981
7 500	2 200	1 690	1 430	1 160	883	1 300	897
8 000	2 030	1 560	1 320	1 070	818	1 190	818
8 500	1 870	1 440	1 220	988	755	1 080	745
9 000	1 720	1 330	1 120	910	696	984	679
9 500	1 580	1 220	1 030	838	641	897	620
10 000	1 450	1 120	950	771	590	820	567
10 500	1 340	1 030	875	710	544	751	519
11 000	1 230	952	807	655	502	689	477
11 500	1 140	879	745	606	464	634	439
12 000	1 050	814	690	561	430	585	405
12 500	975	755	640	520	399	541	375
13 000	906	701	595	483	371	502	348
14 000	787	609	517	420	322	435	301
15 000	689	533	452	368	282	380	263
16 000	607	470	399	325	249	335	232
17 000	539	418	354	288	221		
18 000	482	373	317	258	198		

PROPERTIES AND DESIGN DATA

Area (mm^2)	10 400	7 890	6 610	5 320	4 030	7 030	4 750
r (mm)	92.2	93.2	93.8	94.3	94.9	83.1	84.2
M_r (kN·m)	272	209	176	142	83.8	166	113
(D / t) 350	7 530	10 000	12 000	15 100	20 000	8 980	13 500

IMPERIAL SIZE AND WEIGHT

Weight (lb./ft.)	54.8	41.6	34.9	28.1	21.3	37.1	25.1
Thickness (in.)	0.500	0.375	0.313	0.250	0.188	0.375	0.250
Size (in.)	10.75 OD					9.625 OD	

† Class 3

ROUND HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W
CLASS H
$\phi = 0.90$

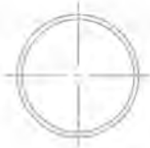
Section (mm x mm)	HSS 219				HSS 178		HSS 168			
	13	9.5	6.4	4.8	13	9.5	13	9.5	6.4	4.8
Mass (kg/m)	64.6	49.3	33.3	25.3	51.7	39.5	48.7	37.3	25.4	19.3
Effective length (KL) in millimetres with respect to the least radius of gyration										
0	2 590	1 980	1 340	1 010	2 080	1 590	1 960	1 500	1 020	775
500	2 590	1 980	1 340	1 010	2 080	1 590	1 960	1 500	1 020	775
1 000	2 590	1 970	1 340	1 010	2 070	1 590	1 950	1 500	1 020	774
1 500	2 590	1 970	1 330	1 010	2 070	1 580	1 950	1 490	1 010	772
2 000	2 580	1 970	1 330	1 010	2 050	1 570	1 920	1 470	1 000	764
2 500	2 560	1 950	1 320	1 000	2 010	1 540	1 870	1 440	980	747
3 000	2 520	1 920	1 300	990	1 930	1 480	1 780	1 370	938	717
3 500	2 450	1 870	1 270	967	1 810	1 400	1 650	1 270	877	671
4 000	2 350	1 800	1 230	933	1 660	1 290	1 490	1 150	798	613
4 500	2 220	1 710	1 160	887	1 490	1 160	1 310	1 030	712	548
5 000	2 070	1 590	1 090	832	1 310	1 030	1 150	898	627	483
5 500	1 900	1 470	1 010	769	1 150	906	995	782	548	423
6 000	1 730	1 340	920	704	1 010	794	864	681	478	369
6 500	1 560	1 210	835	639	883	697	752	594	418	323
7 000	1 400	1 090	754	578	776	613	659	520	367	284
7 500	1 260	982	680	521	685	542	580	459	323	250
8 000	1 130	884	613	470	608	481	513	406	287	222
8 500	1 020	797	553	425	542	430	457	362	255	198
9 000	921	721	500	384	486	385	409	324	229	177
9 500	834	653	454	349	438	347	368	292	206	160
10 000	759	594	413	318	396	314	333	264	187	144
10 500	692	542	377	290	360	286	303	240	170	131
11 000	633	496	345	266	329	261	276	219	155	120
11 500	581	456	317	244	301	239				110
12 000	535	420	292	225						
12 500	495	388	270	208						
13 000	458	359	250	192						
14 000	396	311	216	166						
15 000			189	145						
16 000										
17 000										
18 000										
PROPERTIES AND DESIGN DATA										
Area (mm^2)	8 230	6 270	4 240	3 220	6 590	5 040	6 210	4 750	3 230	2 460
r (mm)	73.1	74.2	75.3	75.8	58.5	59.6	55.2	56.2	57.3	57.8
M_r (kN·m)	171	132	90.7	69.3	109	85.1	97.0	75.9	52.6	40.3
(D / t) 350	6 040	8 050	12 100	16 000	4 900	6 530	4 640	6 180	9 280	12 300
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)	43.4	33.1	22.4	17.0	34.7	26.6	32.7	25.1	17.0	13.0
Thickness (in.)	0.500	0.375	0.250	0.188	0.500	0.375	0.500	0.375	0.250	0.188
Size (in.)	8.625 OD				7 OD		6.625 OD			

ROUND HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W CLASS H
ϕ = 0.90

Section (mm x mm)	HSS 141			HSS 127	
	13	9.5	6.4	9.5	6.4
Mass (kg/m)	40.3	31.0	21.1	27.6	18.9
Effective length (KL) in millimetres with respect to the least radius of gyration					
0	1 620	1 240	847	1 110	759
500	1 620	1 240	847	1 110	759
1 000	1 610	1 240	846	1 110	757
1 500	1 600	1 230	840	1 090	748
2 000	1 560	1 200	821	1 050	722
2 500	1 470	1 140	782	965	669
3 000	1 330	1 040	718	848	593
3 500	1 160	915	637	718	506
4 000	992	785	551	598	424
4 500	837	666	470	496	353
5 000	706	563	399	413	295
5 500	598	478	339	347	248
6 000	510	409	291	295	211
6 500	439	352	251	253	181
7 000	382	306	218	219	157
7 500	334	268	191	192	137
8 000	294	237	169	169	121
8 500	261	210	150		107
9 000	234	188	134		
9 500			120		
10 000					
10 500					
11 000					
11 500					
12 000					
12 500					
13 000					
14 000					
15 000					
16 000					
17 000					
18 000					
PROPERTIES AND DESIGN DATA					
Area (mm^2)	5 130	3 950	2 690	3 520	2 410
r (mm)	45.7	46.7	47.8	41.7	42.7
M_r (kN·m)	66.5	52.3	36.5	41.6	29.1
(D / t) 350	3 890	5 190	7 790	4 660	7 000
IMPERIAL SIZE AND WEIGHT					
Weight (lb./ft.)	27.1	20.8	14.2	18.6	12.7
Thickness (in.)	0.500	0.375	0.250	0.375	0.250
Size (in.)	5.563 OD			5 OD	

ROUND HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)



G40.21 350W
CLASS H
$\phi = 0.90$

Section (mm x mm)	HSS 89			HSS 76			HSS 73		
	6.4	4.8	3.2	6.4	4.8	3.2	6.4	4.8	3.2
Mass (kg/m)	12.9	9.92	6.72	10.9	8.42	5.73	10.4	8.04	5.48
Effective length (KL) in millimetres with respect to the least radius of gyration									
0	520	397	270	438	337	230	419	321	220
500	519	397	270	437	337	229	418	321	220
1 000	513	392	267	426	329	225	406	312	214
1 500	483	371	253	379	295	203	353	274	190
2 000	415	321	221	298	234	163	270	212	149
2 500	330	257	178	219	174	122	194	154	109
3 000	253	199	138	161	128	90	142	113	80
3 500	195	153	107	121	97	68	106	85	60
4 000	153	120	84	94	75	53	82	65	47
4 500	122	96	68	74	60	42	65	52	37
5 000	100	79	55		48	34			
5 500	83	65	46						
6 000			39						
6 500									
7 000									
7 500									
8 000									
8 500									
9 000									
9 500									
10 000									
10 500									
11 000									
11 500									
12 000									
12 500									
13 000									
14 000									
15 000									
16 000									
17 000									
18 000									
PROPERTIES AND DESIGN DATA									
Area (mm^2)	1 650	1 260	856	1 390	1 070	729	1 330	1 020	698
r (mm)	29.3	29.8	30.3	24.8	25.3	25.8	23.7	24.2	24.7
M_r (kN·m)	13.7	10.7	7.37	9.80	7.69	5.36	8.91	7.02	4.88
(D / t) 350	4 900	6 510	9 780	4 200	5 580	8 390	4 020	5 350	8 030
IMPERIAL SIZE AND WEIGHT									
Weight (lb./ft.)	8.69	6.66	4.52	7.35	5.66	3.85	7.01	5.40	3.68
Thickness (in.)	0.250	0.188	0.125	0.250	0.188	0.125	0.250	0.188	0.125
Size (in.)	3.5 OD			3 OD			2.875 OD		

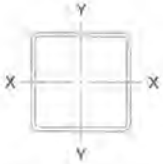
ROUND HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)

G40.21 350W
CLASS H
$\phi = 0.90$

Section (mm x mm)	HSS 64			HSS 60			HSS 48	
	6.4	4.8	3.2	6.4	4.8	3.2	4.8	3.2
Mass (kg/m)	8.95	6.92	4.73	8.45	6.54	4.48	5.13	3.54
Effective length (KL) in millimetres with respect to the least radius of gyration								
0	359	278	190	340	263	180	206	142
500	358	277	189	339	262	179	204	141
1 000	337	262	181	314	245	169	172	121
1 500	268	212	149	240	190	133	110	79
2 000	186	149	106	161	130	92	67	49
2 500	128	103	74	109	88	63	44	32
3 000	91	74	53	77	63	45	31	23
3 500	67	55	40	57	47	33		
4 000	52	42	30			26		
4 500								
5 000								
5 500								
6 000								
6 500								
7 000								
7 500								
8 000								
8 500								
9 000								
9 500								
10 000								
10 500								
11 000								
11 500								
12 000								
12 500								
13 000								
14 000								
15 000								
16 000								
17 000								
18 000								
PROPERTIES AND DESIGN DATA								
Area (mm^2)	1 140	882	603	1 080	834	571	654	451
r (mm)	20.3	20.8	21.4	19.2	19.7	20.2	15.5	16.0
M_r (kN·m)	6.55	5.20	3.65	5.86	4.66	3.28	2.86	2.04
(D / t) 350	3 500	4 650	6 990	3 320	4 420	6 640	3 540	5 320
IMPERIAL SIZE AND WEIGHT								
Weight (lb./ft.)	6.01	4.65	3.18	5.68	4.40	3.01	3.45	2.38
Thickness (in.)	0.250	0.188	0.125	0.250	0.188	0.125	0.188	0.125
Size (in.)	2.5 OD			2.375 OD			1.9 OD	

SQUARE HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$



ASTM A500 Grade C $F_y = 345$ MPa

Section (mm x mm x mm)	HSS 406 x 406		HSS 356 x 356			HSS 305 x 305			
	16*	† 13*	16*	13*	‡ 9.5*	16	13	† 9.5	‡ 7.9
Mass (kg/m)	190	154	164	133	102	139	113	86.5	72.7
Effective length (KL) in millimetres with respect to the least radius of gyration									
0	6 800	5 500	5 900	4 780	3 510	5 000	4 070	3 100	2 440
500	6 800	5 500	5 900	4 780	3 510	5 000	4 070	3 100	2 440
1 000	6 790	5 490	5 890	4 780	3 500	4 990	4 060	3 090	2 430
1 500	6 780	5 480	5 880	4 760	3 490	4 970	4 040	3 080	2 430
2 000	6 760	5 460	5 850	4 740	3 480	4 930	4 010	3 060	2 410
2 500	6 730	5 440	5 810	4 710	3 450	4 880	3 970	3 030	2 380
3 000	6 680	5 400	5 750	4 660	3 420	4 810	3 910	2 980	2 350
3 500	6 620	5 350	5 670	4 610	3 380	4 720	3 840	2 930	2 310
4 000	6 550	5 290	5 580	4 530	3 330	4 600	3 750	2 860	2 260
4 500	6 460	5 220	5 480	4 450	3 270	4 480	3 650	2 790	2 200
5 000	6 350	5 140	5 350	4 350	3 200	4 330	3 540	2 700	2 130
5 500	6 240	5 050	5 220	4 250	3 120	4 170	3 410	2 610	2 060
6 000	6 110	4 940	5 070	4 130	3 030	4 010	3 270	2 510	1 980
6 500	5 960	4 830	4 900	4 000	2 940	3 830	3 130	2 400	1 900
7 000	5 810	4 710	4 730	3 870	2 850	3 660	2 990	2 290	1 810
7 500	5 650	4 580	4 560	3 730	2 740	3 480	2 850	2 180	1 730
8 000	5 480	4 440	4 380	3 580	2 640	3 300	2 700	2 070	1 640
8 500	5 310	4 300	4 200	3 440	2 540	3 120	2 560	1 970	1 560
9 000	5 130	4 160	4 020	3 300	2 430	2 960	2 430	1 860	1 480
9 500	4 950	4 020	3 840	3 150	2 330	2 790	2 290	1 760	1 400
10 000	4 770	3 870	3 670	3 010	2 220	2 640	2 170	1 670	1 320
10 500	4 590	3 730	3 500	2 870	2 120	2 490	2 050	1 570	1 250
11 000	4 410	3 580	3 330	2 740	2 030	2 350	1 930	1 490	1 180
11 500	4 230	3 440	3 170	2 610	1 930	2 220	1 820	1 410	1 120
12 000	4 060	3 300	3 020	2 490	1 840	2 090	1 720	1 330	1 060
12 500	3 900	3 170	2 870	2 370	1 750	1 980	1 630	1 260	1 000
13 000	3 740	3 040	2 740	2 260	1 670	1 870	1 540	1 190	946
14 000	3 430	2 790	2 480	2 050	1 520	1 670	1 380	1 060	847
15 000	3 150	2 560	2 250	1 860	1 380	1 500	1 240	955	762
16 000	2 890	2 350	2 040	1 690	1 250	1 350	1 110	860	687
17 000	2 650	2 160	1 860	1 540	1 140	1 220	1 010	778	621
18 000	2 440	1 990	1 700	1 410	1 040	1 110	913	706	563
PROPERTIES AND DESIGN DATA									
Area (mm^2)	21 900	17 700	19 000	15 400	11 700	16 100	13 100	9 980	8 380
r (mm)	159	160	138	140	141	118	119	120	121
M_r (kN·m)	990	699	748	612	396	537	444	294	239
$(b_{el}/t)\sqrt{345}$	454	586	388	504	696	322	421	586	718
§ $(b_{el}/t)\sqrt{350}$	457	590	391	507	701	324	424	590	723
IMPERIAL SIZE AND WEIGHT									
Weight (lb./ft.)	127	103	110	89.7	68.4	93.4	76.1	58.1	48.9
Design Thick.(in.)	0.563	0.450	0.563	0.450	0.338	0.563	0.450	0.338	0.281
Size (in.)	16 x 16		14 x 14			12 x 12			

§ See S16-14 Clause 27.1.7 for seismic applications

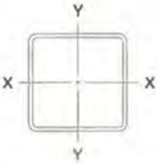
‡ Class 4

* Imported section

† Class 3

SQUARE HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$



ASTM A500 Grade C

$F_y = 345$ MPa

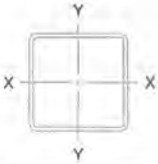
Section (mm x mm x mm)		HSS 254 x 254					HSS 203 x 203				
		16	13	9.5	† 7.9	‡ 6.4	16	13	9.5	7.9	† 6.4
Mass (kg/m)		114	93.0	71.3	60.1	48.6	88.3	72.7	56.1	47.4	38.4
Effective length (KL) in millimetres with respect to the least radius of gyration	0	4 100	3 350	2 560	2 150	1 560	3 200	2 620	2 020	1 700	1 380
	500	4 100	3 350	2 550	2 150	1 560	3 190	2 610	2 010	1 700	1 370
	1 000	4 080	3 340	2 550	2 140	1 560	3 180	2 600	2 000	1 690	1 370
	1 500	4 060	3 320	2 530	2 130	1 550	3 130	2 570	1 980	1 670	1 350
	2 000	4 010	3 280	2 500	2 110	1 530	3 060	2 510	1 940	1 640	1 330
	2 500	3 930	3 220	2 460	2 070	1 510	2 960	2 430	1 880	1 590	1 290
	3 000	3 840	3 150	2 400	2 030	1 470	2 840	2 330	1 810	1 530	1 240
	3 500	3 720	3 050	2 340	1 970	1 430	2 690	2 210	1 720	1 450	1 180
	4 000	3 580	2 940	2 250	1 900	1 380	2 520	2 080	1 620	1 370	1 110
	4 500	3 430	2 820	2 160	1 820	1 330	2 340	1 940	1 510	1 280	1 040
	5 000	3 260	2 690	2 060	1 740	1 270	2 170	1 800	1 400	1 190	971
	5 500	3 080	2 550	1 960	1 650	1 210	1 990	1 660	1 300	1 100	899
	6 000	2 900	2 400	1 850	1 560	1 140	1 830	1 520	1 190	1 020	829
	6 500	2 730	2 260	1 740	1 470	1 080	1 670	1 400	1 100	935	763
	7 000	2 550	2 120	1 630	1 380	1 010	1 530	1 280	1 010	858	701
	7 500	2 380	1 980	1 530	1 290	949	1 400	1 170	924	788	644
	8 000	2 220	1 850	1 430	1 210	888	1 280	1 070	847	723	591
	8 500	2 070	1 730	1 340	1 130	831	1 170	985	778	664	544
	9 000	1 930	1 610	1 250	1 060	777	1 070	904	715	611	500
	9 500	1 800	1 500	1 170	987	726	987	832	659	563	461
	10 000	1 680	1 400	1 090	922	679	909	767	607	519	426
	10 500	1 570	1 310	1 020	862	635	839	708	561	480	393
	11 000	1 460	1 220	952	806	594	776	655	520	444	364
	11 500	1 370	1 140	891	754	556	719	607	482	412	338
	12 000	1 280	1 070	834	707	521	668	564	448	383	315
	12 500	1 200	1 000	782	663	489	621	525	417	357	293
	13 000	1 120	942	735	623	459	579	490	389	333	273
	14 000	993	833	650	551	407	506	428	341	292	240
	15 000	882	740	578	490	362	446	377	300	257	211
	16 000	787	661	516	438	324					187
	17 000	706	593	463	393	291					
	18 000	636	534	418	354	262					
PROPERTIES AND DESIGN DATA											
Area (mm^2)		13 200	10 800	8 230	6 930	5 600	10 300	8 430	6 490	5 480	4 430
r (mm)		96.8	98.2	99.6	100	101	76.1	77.5	78.9	79.5	80.2
M_r (kN·m)		363	301	233	170	129	222	186	146	124	87.3
$(b_{el}/t)\sqrt{345}$		256	338	476	586	751	190	256	366	454	586
§ $(b_{el}/t)\sqrt{350}$		258	341	479	590	756	191	258	368	457	590
IMPERIAL SIZE AND WEIGHT											
Weight (lb./ft.)		76.4	62.5	47.9	40.4	32.6	59.3	48.9	37.7	31.9	25.8
Design Thick.(in.)		0.563	0.450	0.338	0.281	0.225	0.563	0.450	0.338	0.281	0.225
Size (in.)		10 x 10					8 x 8				

§ See S16-14 Clause 27.1.7 for seismic applications

† Class 3

‡ Class 4

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$



ASTM A500 Grade C $F_y = 345$ MPa

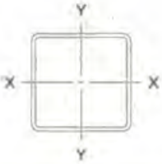
Section (mm x mm x mm)	HSS 178 x 178					HSS 152 x 152				
	16	13	9.5	7.9	6.4	13	9.5	7.9	6.4	† 4.8
Mass (kg/m)	75.6	62.6	48.5	41.1	33.4	52.4	40.9	34.7	28.3	21.7
Effective length (KL) in millimetres with respect to the least radius of gyration										
0	2 740	2 260	1 750	1 470	1 200	1 900	1 470	1 250	1 020	776
500	2 730	2 250	1 740	1 470	1 190	1 890	1 470	1 250	1 010	775
1 000	2 710	2 240	1 730	1 460	1 190	1 870	1 450	1 230	1 000	766
1 500	2 660	2 200	1 700	1 440	1 170	1 820	1 420	1 200	977	748
2 000	2 570	2 130	1 650	1 400	1 130	1 730	1 350	1 150	937	718
2 500	2 450	2 030	1 580	1 340	1 090	1 620	1 270	1 080	882	677
3 000	2 310	1 920	1 490	1 270	1 030	1 490	1 170	999	817	629
3 500	2 140	1 790	1 400	1 190	966	1 350	1 070	910	747	575
4 000	1 970	1 640	1 290	1 100	895	1 210	960	821	675	521
4 500	1 790	1 500	1 180	1 010	823	1 080	858	735	605	468
5 000	1 620	1 370	1 080	919	752	955	764	656	541	419
5 500	1 470	1 240	978	835	684	847	680	584	482	374
6 000	1 320	1 120	886	758	621	751	605	520	430	334
6 500	1 190	1 010	802	686	564	668	539	464	384	299
7 000	1 070	912	726	622	511	596	481	415	344	268
7 500	971	826	658	564	464	534	432	372	309	240
8 000	879	749	598	513	422	479	388	335	278	217
8 500	799	681	544	467	385	432	351	303	251	196
9 000	727	621	496	426	351	391	318	274	228	178
9 500	664	567	454	390	322	356	289	250	207	162
10 000	608	520	416	358	295	324	263	228	189	148
10 500	558	478	383	329	272	297	241	209	173	135
11 000	514	440	353	304	250	272	221	192	159	124
11 500	475	406	326	281	231		204	176	147	115
12 000	439	376	302	260	215					106
12 500	407	349	280	241	199					
13 000	379	325	261	225	185					
14 000										
15 000										
16 000										
17 000										
18 000										
PROPERTIES AND DESIGN DATA										
Area (mm^2)	8 820	7 270	5 620	4 750	3 850	6 110	4 750	4 020	3 270	2 500
r (mm)	65.7	67.1	68.5	69.2	69.9	56.7	58.1	58.8	59.5	60.2
M_r (kN·m)	164	138	109	93.5	76.7	98.4	78.2	67.4	55.3	36.9
$(b_{el}/t)\sqrt{345}$	157	215	311	388	503	173	256	322	421	584
§ $(b_{el}/t)\sqrt{350}$	158	216	313	390	507	175	257	324	424	588
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)	50.8	42.1	32.6	27.6	22.4	35.2	27.5	23.3	19.0	14.6
Design Thick.(in.)	0.563	0.450	0.338	0.281	0.225	0.450	0.338	0.281	0.225	0.169
Size (in.)	7 x 7					6 x 6				

§ See S16-14 Clause 27.1.7 for seismic applications

† Class 3

SQUARE HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$



ASTM A500 Grade C $F_y = 345$ MPa

Section (mm x mm x mm)	HSS 127 x 127					HSS 102 x 102					
	13	9.5	7.9	6.4	4.8	13	9.5	7.9	6.4	4.8	† 3.2
Mass (kg/m)	42.3	33.3	28.4	23.2	17.9	32.2	25.7	22.1	18.2	14.1	9.62
Effective length (KL) in millimetres with respect to the least radius of gyration											
0	1 540	1 200	1 020	835	640	1 180	932	798	655	506	345
500	1 530	1 200	1 020	832	638	1 170	925	793	651	503	343
1 000	1 500	1 170	1 000	817	626	1 120	891	765	629	487	332
1 500	1 430	1 120	960	784	602	1 020	821	708	584	453	310
2 000	1 320	1 050	896	733	564	896	726	628	521	406	279
2 500	1 190	948	815	669	516	759	621	540	450	352	243
3 000	1 050	842	726	598	463	631	521	455	381	300	208
3 500	917	738	638	527	409	522	434	381	320	253	176
4 000	793	642	556	461	358	433	362	318	268	212	148
4 500	684	556	483	401	313	361	303	268	226	179	125
5 000	591	482	420	349	273	304	256	226	191	152	107
5 500	513	420	366	305	238	258	218	193	164	130	91
6 000	447	367	320	267	209	222	188	166	141	112	79
6 500	392	322	281	235	184	192	163	144	122	98	69
7 000	345	284	248	207	163	167	142	126	107	85	60
7 500	306	252	220	184	145			111	94	75	53
8 000	273	225	197	165	129						47
8 500	244	202	176	148	116						
9 000	220	182	159	133	105						
9 500		164	144	120	95						
10 000											
10 500											
11 000											
11 500											
12 000											
12 500											
13 000											
14 000											
15 000											
16 000											
17 000											
18 000											

PROPERTIES AND DESIGN DATA

Area (mm^2)	4 950	3 870	3 300	2 690	2 060	3 790	3 000	2 570	2 110	1 630	1 110
r (mm)	46.3	47.7	48.4	49.1	49.8	35.9	37.3	38.0	38.7	39.4	40.1
M_r (kN·m)	64.9	52.5	45.3	37.6	29.2	38.5	31.7	27.7	23.2	18.2	10.9
$(b_{el}/t)\sqrt{345}$	132	201	256	338	474	90.8	146	190	256	365	586
§ $(b_{el}/t)\sqrt{350}$	133	202	257	341	478	91.5	147	191	257	367	590

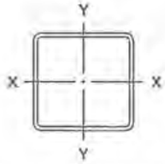
IMPERIAL SIZE AND WEIGHT

Weight (lb./ft.)	28.4	22.4	19.1	15.6	12.0	21.6	17.3	14.8	12.2	9.45	6.47
Design Thick.(in.)	0.450	0.338	0.281	0.225	0.169	0.450	0.338	0.281	0.225	0.169	0.113
Size (in.)	5 x 5					4 x 4					

§ See S16-14 Clause 27.1.7 for seismic applications

† Class 3

SQUARE HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$



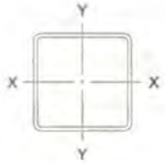
ASTM A500 Grade C
$F_y = 345$ MPa

Section (mm x mm x mm)	HSS 89 x 89				HSS 76 x 76				
	9.5	7.9	6.4	4.8	9.5	7.9	6.4	4.8	3.2
Mass (kg/m)	21.9	18.9	15.6	12.2	18.1	15.7	13.1	10.3	7.09
Effective length (KL) in millimetres with respect to the least radius of gyration									
0	798	686	565	438	661	571	475	369	254
500	789	679	560	434	650	562	468	364	251
1 000	747	645	533	414	596	518	434	339	234
1 500	666	578	480	375	503	442	373	294	204
2 000	563	492	411	323	401	355	303	240	169
2 500	462	406	341	270	312	278	240	191	135
3 000	374	330	279	222	243	218	189	151	107
3 500	303	269	228	182	191	172	150	120	86
4 000	247	220	187	150	153	138	120	97	69
4 500	204	182	155	124	124	112	98	79	57
5 000	170	152	130	104	103	93	81	66	47
5 500	144	128	110	88		78	68	55	40
6 000	123	110	94	76					
6 500		95	81	65					
7 000									
7 500									
8 000									
8 500									
9 000									
9 500									
10 000									
10 500									
11 000									
11 500									
12 000									
12 500									
13 000									
14 000									
15 000									
16 000									
17 000									
18 000									
PROPERTIES AND DESIGN DATA									
Area (mm^2)	2 570	2 210	1 820	1 410	2 130	1 840	1 530	1 190	818
r (mm)	32.1	32.8	33.5	34.2	26.9	27.6	28.4	29.0	29.7
M_r (kN·m)	23.3	20.6	17.3	13.7	16.2	14.4	12.3	9.81	6.92
$(b_{el}/t)\sqrt{345}$	118	157	214	310	90.7	124	173	255	421
§ $(b_{el}/t)\sqrt{350}$	119	158	216	312	91.3	125	174	257	424
IMPERIAL SIZE AND WEIGHT									
Weight (lb./ft.)	14.7	12.7	10.5	8.17	12.2	10.6	8.81	6.89	4.76
Design Thick.(in.)	0.338	0.281	0.225	0.169	0.338	0.281	0.225	0.169	0.113
Size (in.)	3½ x 3½				3 x 3				

§ See S16-14 Clause 27.1.7 for seismic applications

SQUARE HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$



ASTM A500 Grade C $F_y = 345$ MPa

Section (mm x mm x mm)	HSS 64 x 64			HSS 51 x 51			HSS 38 x 38	
	6.4	4.8	3.2	6.4	4.8	3.2	4.8	3.2
Mass (kg/m)	10.6	8.35	5.82	8.05	6.45	4.55	4.54	3.28
Effective length (KL) in millimetres with respect to the least radius of gyration								
0	385	301	209	294	233	164	166	119
500	375	295	204	280	223	157	150	108
1 000	332	262	184	224	182	130	101	76
1 500	264	212	150	158	131	95	61	47
2 000	199	161	115	109	91	67	39	30
2 500	149	121	88	76	65	48	26	20
3 000	112	92	67	56	47	35		
3 500	87	72	52	42	36	27		
4 000	69	57	41					
4 500	55	46	33					
5 000								
5 500								
6 000								
6 500								
7 000								
7 500								
8 000								
8 500								
9 000								
9 500								
10 000								
10 500								
11 000								
11 500								
12 000								
12 500								
13 000								
14 000								
15 000								
16 000								
17 000								
18 000								
PROPERTIES AND DESIGN DATA								
Area (mm^2)	1 240	971	673	947	752	527	534	382
r (mm)	23.2	23.9	24.6	18.0	18.7	19.4	13.5	14.2
M_r (kN·m)	8.14	6.58	4.69	4.81	3.97	2.90	2.03	1.54
$(b_{el}/t)\sqrt{345}$	132	200	338	90.7	145	256	90.3	173
§ $(b_{el}/t)\sqrt{350}$	133	201	341	91.3	146	257	90.9	174
IMPERIAL SIZE AND WEIGHT								
Weight (lb./ft.)	7.11	5.61	3.91	5.41	4.33	3.06	3.05	2.21
Design Thick.(in.)	0.225	0.169	0.113	0.225	0.169	0.113	0.169	0.113
Size (in.)	2½ x 2½			2 x 2			1½ x 1½	

§ See S16-14 Clause 27.1.7 for seismic applications

RECTANGULAR HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$

ASTM A500 Grade C $F_y = 345$ MPa

Section (mm x mm x mm)	HSS 305 x 203				HSS 305 x 152			
	16	13	† 9.5	‡ 7.9	16	13	† 9.5	‡ 7.9
Mass (kg/m)	114	93.0	71.3	60.1	101	82.8	63.7	53.7
Effective length (KL) in millimetres with respect to the least radius of gyration								
0	4 100	3 350	2 560	2 070	3 630	2 980	2 290	1 840
500	4 090	3 350	2 550	2 070	3 630	2 970	2 280	1 840
1 000	4 070	3 330	2 540	2 060	3 590	2 940	2 260	**1 820**
1 500	4 030	3 300	2 520	2 040	3 500	2 880	2 210	1 790
2 000	3 950	3 240	2 470	**2 000**	3 370	2 770	2 130	1 720
2 500	3 840	3 150	2 410	1 950	3 180	2 620	2 030	1 640
3 000	3 690	3 030	2 320	1 880	2 960	2 450	1 900	1 540
3 500	3 520	2 900	2 220	1 800	2 710	2 250	1 750	1 420
4 000	3 320	2 740	2 100	1 710	2 460	2 050	1 600	1 300
4 500	3 110	2 570	1 980	1 610	2 210	1 850	1 450	1 180
5 000	2 900	2 400	1 850	1 510	1 980	1 660	1 310	1 060
5 500	2 690	2 230	1 720	1 400	1 770	1 490	1 170	957
6 000	2 480	2 060	1 590	1 300	1 580	1 340	1 050	860
6 500	2 280	1 900	1 470	1 200	1 420	1 200	946	774
7 000	2 100	1 750	1 360	1 110	1 270	1 080	851	696
7 500	1 930	1 610	1 250	1 020	1 140	968	767	628
8 000	1 770	1 480	1 150	944	1 030	874	694	568
8 500	1 630	1 360	1 060	870	932	792	629	515
9 000	1 500	1 260	979	803	846	719	572	469
9 500	1 380	1 160	904	742	771	655	521	427
10 000	1 280	1 070	836	686	704	599	477	391
10 500	1 180	990	774	636	645	549	437	359
11 000	1 090	918	718	590	593	505	402	330
11 500	1 010	853	667	548	546	466	371	305
12 000	943	793	621	510	505	431	343	282
12 500	879	739	579	476			318	261
13 000	820	690	541	445				
14 000	718	605	474	390				
15 000	634	534	419	344				
16 000	562	474	372	306				
PROPERTIES AND DESIGN DATA								
Area (mm^2)	13 200	10 800	8 230	6 930	11 700	9 590	7 360	6 200
r_x (mm)	111	112	114	114	105	107	109	110
r_y (mm)	80.5	81.8	83.1	83.8	60.8	62.1	63.4	64.0
r_x / r_y	1.38	1.37	1.37	1.36	1.73	1.72	1.72	1.72
M_{rx} (kN·m)	407	338	262	^ 222	342	284	222	^ 188
M_{ry} (kN·m)	307	255	174	141	209	175	120	97.9
$(b_{el}/t)\sqrt{345}$	322	421	586	718	322	421	586	718
§ $(b_{el}/t)\sqrt{350}$	324	424	590	723	324	424	590	723
IMPERIAL SIZE AND WEIGHT								
Weight (lb./ft.)	76.4	62.5	47.9	40.4	67.8	55.7	42.8	36.1
Design Thick.(in.)	0.563	0.450	0.338	0.281	0.563	0.450	0.338	0.281
Size (in.)	12 x 8				12 x 6			

§ See S16-14 Clause 27.1.7 for seismic applications

^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.

† Class 3 in bending about Y-Y axis

‡ Class 4

RECTANGULAR HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$

ASTM A500 Grade C
$F_y = 345$ MPa

Section (mm x mm x mm)		HSS 254 x 203					HSS 254 x 152				
		16	13	9.5	† 7.9	‡ 6.4	16	13	9.5	† 7.9	‡ 6.4
Mass (kg/m)		101	82.8	63.7	53.7	43.5	88.3	72.7	56.1	47.4	38.4
Effective length (KL) in millimetres with respect to the least radius of gyration	0	3 630	2 980	2 290	1 930	1 470	3 200	2 620	2 020	1 700	1 290
	500	3 630	2 970	2 280	1 920	1 470	3 190	2 610	2 010	1 700	1 290
	1 000	3 610	2 960	2 270	1 910	1 460	3 160	2 590	1 990	1 680	1 270
	1 500	3 570	2 930	2 250	1 890	**1 450**	3 080	2 520	1 950	1 650	**1 250**
	2 000	3 490	2 870	2 200	1 860	1 420	2 950	2 430	1 870	1 590	1 200
	2 500	3 390	2 780	2 140	1 810	1 380	2 780	2 290	1 780	1 500	1 140
	3 000	3 250	2 680	2 060	1 740	1 330	2 580	2 130	1 660	1 410	1 070
	3 500	3 090	2 550	1 970	1 660	1 270	2 350	1 950	1 520	1 300	986
	4 000	2 910	2 410	1 860	1 570	1 210	2 130	1 770	1 390	1 180	900
	4 500	2 720	2 250	1 750	1 480	1 140	1 910	1 590	1 250	1 070	816
	5 000	2 520	2 100	1 630	1 380	1 060	1 710	1 430	1 130	962	735
	5 500	2 330	1 940	1 510	1 280	985	1 520	1 280	1 010	864	661
	6 000	2 150	1 790	1 400	1 180	912	1 360	1 140	904	775	593
	6 500	1 970	1 640	1 290	1 090	842	1 210	1 020	811	695	533
	7 000	1 810	1 510	1 180	1 010	776	1 090	916	728	625	480
	7 500	1 660	1 390	1 090	925	714	975	824	655	563	432
	8 000	1 520	1 270	1 000	851	658	878	743	592	509	391
	8 500	1 390	1 170	921	784	606	794	672	536	461	354
	9 000	1 280	1 080	848	722	559	720	610	487	419	322
	9 500	1 180	992	782	666	516	655	555	443	382	294
	10 000	1 090	916	722	616	477	598	507	405	349	269
	10 500	1 010	847	668	570	441	548	465	372	320	246
	11 000	931	784	619	528	409	503	427	342	294	227
	11 500	863	728	575	490	380	464	394	315	272	209
	12 000	802	677	535	456	354		364	291	251	193
	12 500	747	630	499	425	330				233	179
	13 000	697	588	465	397	308					
	14 000	610	515	408	348	270					
	15 000	537	454	360	307	238					
	16 000			319	273	212					
PROPERTIES AND DESIGN DATA											
Area (mm^2)		11 700	9 590	7 360	6 200	5 020	10 300	8 430	6 490	5 480	4 430
r_x (mm)		93.6	95.1	96.5	97.2	97.9	89.2	90.8	92.4	93.2	94.0
r_y (mm)		78.6	79.9	81.3	81.9	82.6	59.6	60.8	62.1	62.8	63.4
r_x / r_y		1.19	1.19	1.19	1.19	1.19	1.50	1.49	1.49	1.48	1.48
M_{rx} (kN·m)		309	257	200	170	^ 118	255	213	167	142	^ 116
M_{ry} (kN·m)		265	220	172	127	96.1	178	149	117	87.9	66.3
$(b_{el}/t)\sqrt{345}$		256	338	476	586	751	256	338	476	586	751
§ $(b_{el}/t)\sqrt{350}$		258	341	479	590	756	258	341	479	590	756
IMPERIAL SIZE AND WEIGHT											
Weight (lb./ft.)		67.8	55.7	42.8	36.1	29.2	59.3	48.9	37.7	31.9	25.8
Design Thick.(in.)		0.563	0.450	0.338	0.281	0.225	0.563	0.450	0.338	0.281	0.225
Size (in.)		10 x 8					10 x 6				

§ See S16-14 Clause 27.1.7 for seismic applications

^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.

† Class 3 in bending about Y-Y axis

‡ Class 4

RECTANGULAR HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$

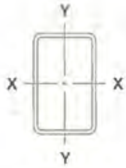
ASTM A500 Grade C $F_y = 345$ MPa

Section (mm x mm x mm)	HSS 203 x 152					HSS 203 x 102			
	16	13	9.5	7.9	† 6.4	13	9.5	7.9	† 6.4
Mass (kg/m)	75.6	62.6	48.5	41.1	33.4	52.4	40.9	34.7	28.3
Effective length (KL) in millimetres with respect to the least radius of gyration									
0	2 740	2 260	1 750	1 470	1 200	1 900	1 470	1 250	1 020
500	2 730	2 250	1 740	1 470	1 190	1 890	1 470	1 240	1 010
1 000	2 700	2 230	1 720	1 460	1 180	1 830	1 420	1 210	983
1 500	2 630	2 170	1 680	1 420	1 150	1 700	1 340	1 130	926
2 000	2 510	2 080	1 620	1 370	1 110	1 530	1 210	1 030	842
2 500	2 360	1 960	1 530	1 290	1 050	1 330	1 060	904	743
3 000	2 170	1 810	1 420	1 200	980	1 130	908	779	642
3 500	1 970	1 650	1 300	1 100	901	955	772	664	549
4 000	1 780	1 490	1 180	1 000	819	805	653	564	467
4 500	1 590	1 340	1 060	903	740	680	554	479	398
5 000	1 410	1 190	947	809	664	578	472	409	340
5 500	1 250	1 060	846	724	595	494	405	351	293
6 000	1 110	947	756	648	533	426	350	304	253
6 500	993	846	676	580	478	371	305	265	221
7 000	887	757	606	520	429	324	267	232	194
7 500	795	679	545	468	386	286	236	205	171
8 000	715	611	491	422	348		209	182	152
8 500	645	552	444	382	315				
9 000	584	501	403	346	286				
9 500	531	455	367	316	261				
10 000	485	416	335	288	238				
10 500	444	381	307	264	219				
11 000	407	350	282	243	201				
11 500	375	322	260	224	185				
12 000			240	207	171				
12 500									
13 000									
14 000									
15 000									
16 000									
PROPERTIES AND DESIGN DATA									
Area (mm^2)	8 820	7 270	5 620	4 750	3 850	6 110	4 750	4 020	3 270
r_x (mm)	72.6	74.1	75.6	76.3	77.1	69.2	70.9	71.7	72.5
r_y (mm)	57.8	59.1	60.5	61.1	61.8	39.7	41.0	41.6	42.2
r_x / r_y	1.26	1.25	1.25	1.25	1.25	1.74	1.73	1.72	1.72
M_{rx} (kN·m)	179	151	119	102	83.5	116	92.8	79.8	65.5
M_{ry} (kN·m)	147	124	97.8	83.5	59.9	70.8	56.8	49.1	35.7
$(b_{el}/t)\sqrt{345}$	190	256	366	454	586	256	366	454	586
§ $(b_{el}/t)\sqrt{350}$	191	258	368	457	590	258	368	457	590
IMPERIAL SIZE AND WEIGHT									
Weight (lb./ft.)	50.8	42.1	32.6	27.6	22.4	35.2	27.5	23.3	19.0
Design Thick.(in.)	0.563	0.450	0.338	0.281	0.225	0.450	0.338	0.281	0.225
Size (in.)	8 x 6					8 x 4			

§ See S16-14 Clause 27.1.7 for seismic applications

† Class 3 in bending about Y-Y axis

RECTANGULAR HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$



ASTM A500 Grade C
$F_y = 345$ MPa

Section (mm x mm x mm)	HSS 178 x 127					HSS 152 x 102				
	13	9.5	7.9	6.4	‡ 4.8	13	9.5	7.9	6.4	† 4.8
Mass (kg/m)	52.4	40.9	34.7	28.3	21.7	42.3	33.3	28.4	23.2	17.9
Effective length (KL) in millimetres with respect to the least radius of gyration										
0	1 900	1 470	1 250	1 020	762	1 540	1 200	1 020	835	640
500	1 890	1 470	1 240	1 010	759	1 530	1 190	1 020	831	636
1 000	1 860	1 440	1 220	996	**747**	1 470	1 160	988	806	618
1 500	1 780	1 390	1 180	960	721	1 370	1 080	923	756	581
2 000	1 660	1 300	1 110	904	680	1 210	967	831	683	527
2 500	1 510	1 190	1 020	832	627	1 050	840	726	598	463
3 000	1 350	1 070	914	751	567	884	715	621	513	399
3 500	1 190	946	811	667	506	741	603	525	436	340
4 000	1 040	830	713	588	446	620	508	444	369	289
4 500	902	724	623	516	392	522	429	375	313	245
5 000	784	632	545	451	344	442	364	319	267	210
5 500	683	552	477	396	302	377	312	274	229	180
6 000	598	485	419	348	266	324	269	236	198	156
6 500	526	427	369	307	235	282	234	206	172	136
7 000	465	378	327	272	208	246	205	180	151	119
7 500	413	336	291	242	185	217	180	159	133	105
8 000	368	300	260	217	166			141	118	93
8 500	330	269	234	195	149					
9 000	298	243	211	176	135					
9 500	270	220	191	159	122					
10 000		200	174	145	111					
10 500										
11 000										
11 500										
12 000										
12 500										
13 000										
14 000										
15 000										
16 000										
PROPERTIES AND DESIGN DATA										
Area (mm²)	6 110	4 750	4 020	3 270	2 500	4 950	3 870	3 300	2 690	2 060
r_x (mm)	63.6	65.1	65.8	66.6	67.3	52.9	54.5	55.3	56.0	56.8
r_y (mm)	48.7	50.0	50.7	51.4	52.0	38.3	39.6	40.3	40.9	41.6
r_x / r_y	1.31	1.30	1.30	1.30	1.29	1.38	1.38	1.37	1.37	1.37
M_{rx} (kN·m)	109	86.6	74.2	61.2	^ 47.2	73.0	59.0	50.9	42.2	32.9
M_{ry} (kN·m)	85.7	68.6	59.0	48.4	32.2	54.6	44.4	38.5	32.0	21.8
$(b_{el}/t)\sqrt{345}$	215	311	388	503	694	173	256	322	421	584
§ $(b_{el}/t)\sqrt{350}$	216	313	390	507	699	175	257	324	424	588
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)	35.2	27.5	23.3	19.0	14.6	28.4	22.4	19.1	15.6	12.0
Design Thick.(in.)	0.450	0.338	0.281	0.225	0.169	0.450	0.338	0.281	0.225	0.169
Size (in.)	7 x 5					6 x 4				

§ See S16-14 Clause 27.1.7 for seismic applications

^ M_{rx} decreases for C_r values above the number in bold. Check the class of section.

† Class 3 in bending about Y-Y axis

‡ Class 4

RECTANGULAR HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$

ASTM A500 Grade C $F_y = 345$ MPa

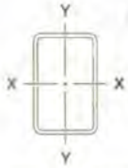
Y X X Y

Section (mm x mm x mm)	HSS 152 x 76					HSS 127 x 76				
	13	9.5	7.9	6.4	† 4.8	13	9.5	7.9	6.4	4.8
Mass (kg/m)	37.3	29.5	25.2	20.7	16.0	32.2	25.7	22.1	18.2	14.1
Effective length (KL) in millimetres with respect to the least radius of gyration										
0	1 360	1 070	910	745	571	1 180	932	798	655	506
500	1 340	1 050	899	736	565	1 160	919	788	647	500
1 000	1 240	986	843	693	534	1 070	856	737	607	471
1 500	1 070	861	740	612	474	913	741	643	533	416
2 000	869	711	616	512	400	736	607	531	443	349
2 500	687	570	497	416	326	578	484	426	358	283
3 000	541	454	397	334	263	453	383	339	286	228
3 500	430	363	318	269	213	358	305	271	229	183
4 000	346	293	258	218	173	288	246	219	186	149
4 500	283	240	212	179	143	235	201	180	153	123
5 000	234	200	176	149	119	194	167	149	127	102
5 500	197	168	148	126	101	163	140	126	107	86
6 000			126	107	86				91	73
6 500										
7 000										
7 500										
8 000										
8 500										
9 000										
9 500										
10 000										
10 500										
11 000										
11 500										
12 000										
12 500										
13 000										
14 000										
15 000										
16 000										
PROPERTIES AND DESIGN DATA										
Area (mm^2)	4 370	3 440	2 930	2 400	1 840	3 790	3 000	2 570	2 110	1 630
r_x (mm)	50.1	51.9	52.7	53.6	54.4	42.1	43.8	44.6	45.4	46.2
r_y (mm)	28.5	29.8	30.4	31.0	31.7	27.8	29.1	29.8	30.4	31.1
r_x / r_y	1.76	1.74	1.73	1.73	1.72	1.51	1.51	1.50	1.49	1.49
M_{rx} (kN·m)	60.2	49.1	42.5	35.4	27.8	44.1	36.3	32.0	26.6	20.9
M_{ry} (kN·m)	36.3	30.0	26.1	21.8	15.1	30.5	25.4	22.2	18.7	14.7
$(b_{el}/t)\sqrt{345}$	173	256	322	421	584	132	201	256	338	474
§ $(b_{el}/t)\sqrt{350}$	175	257	324	424	588	133	202	257	341	478
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)	25.0	19.8	17.0	13.9	10.7	21.6	17.3	14.8	12.2	9.45
Design Thick.(in.)	0.450	0.338	0.281	0.225	0.169	0.450	0.338	0.281	0.225	0.169
Size (in.)	6 x 3					5 x 3				

§ See S16-14 Clause 27.1.7 for seismic applications

† Class 3 in bending about Y-Y axis

RECTANGULAR HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$



ASTM A500 Grade C $F_y = 345$ MPa

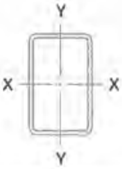
Section (mm x mm x mm)	HSS 102 x 76					HSS 102 x 51				
	9.5	7.9	6.4	4.8	† 3.2	9.5	7.9	6.4	4.8	† 3.2
Mass (kg/m)	21.9	18.9	15.6	12.2	8.35	18.1	15.7	13.1	10.3	7.09
Effective length (KL) in millimetres with respect to the least radius of gyration: 0	798	686	565	438	299	661	571	475	369	254
500	786	677	558	432	295	632	548	457	357	246
1 000	728	629	521	405	278	514	452	383	302	211
1 500	624	544	454	355	245	369	330	284	228	161
2 000	506	445	374	295	205	256	231	202	164	117
2 500	399	353	299	237	166	181	165	145	118	85
3 000	314	279	238	189	133	133	121	107	88	63
3 500	249	222	190	152	107	100	92	82	67	48
4 000	200	179	153	123	87				52	38
4 500	163	146	126	101	72					
5 000	135	121	104	84	60					
5 500	113	102	88	71	50					
6 000				60	43					
6 500										
7 000										
7 500										
8 000										
8 500										
9 000										
9 500										
10 000										
10 500										
11 000										
11 500										
12 000										
12 500										
13 000										
14 000										
15 000										
16 000										
PROPERTIES AND DESIGN DATA										
Area (mm^2)	2 570	2 210	1 820	1 410	963	2 130	1 840	1 530	1 190	818
r_x (mm)	35.5	36.3	37.0	37.8	38.5	32.8	33.7	34.6	35.4	36.3
r_y (mm)	28.2	28.9	29.6	30.2	30.9	18.6	19.2	19.9	20.5	21.1
r_x / r_y	1.26	1.26	1.25	1.25	1.25	1.76	1.76	1.74	1.73	1.72
M_{rx} (kN·m)	25.5	22.4	18.9	14.9	10.4	19.2	17.1	14.6	11.6	8.20
M_{ry} (kN·m)	20.8	18.3	15.5	12.3	7.48	11.5	10.3	8.88	7.14	4.47
$(b_{el} / t)\sqrt{345}$	146	190	256	365	586	146	190	256	365	586
§ $(b_{el} / t)\sqrt{350}$	147	191	257	367	590	147	191	257	367	590
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)	14.7	12.7	10.5	8.17	5.61	12.2	10.6	8.81	6.89	4.76
Design Thick.(in.)	0.338	0.281	0.225	0.169	0.113	0.338	0.281	0.225	0.169	0.113
Size (in.)	4 x 3					4 x 2				

§ See S16-14 Clause 27.1.7 for seismic applications

† Class 3 in bending about Y-Y axis

RECTANGULAR HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$



ASTM A500 Grade C $F_y = 345$ MPa

Section (mm x mm x mm)	HSS 89 x 64		HSS 76 x 51			
	6.4	4.8	7.9	6.4	4.8	3.2
Mass (kg/m)	13.1	10.3	12.6	10.6	8.35	5.82
Effective length (KL) in millimetres with respect to the least radius of gyration						
0	475	369	460	385	301	209
500	465	362	439	369	290	202
1 000	416	326	356	304	243	171
1 500	339	268	255	221	179	129
2 000	260	208	177	155	127	93
2 500	197	158	125	110	91	67
3 000	150	121	91	81	67	50
3 500	117	95	69	61	51	38
4 000	93	75				30
4 500	75	61				
5 000		50				
5 500						
6 000						
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10 500						
11 000						
11 500						
12 000						
12 500						
13 000						
14 000						
15 000						
16 000						

PROPERTIES AND DESIGN DATA

Area (mm^2)	1 530	1 190	1 480	1 240	971	673
r_x (mm)	31.8	32.5	25.7	26.5	27.2	28.0
r_y (mm)	24.4	25.0	18.5	19.1	19.8	20.5
r_x / r_y	1.30	1.30	1.39	1.39	1.37	1.37
M_{rx} (kN·m)	13.6	10.8	10.5	9.13	7.39	5.28
M_{ry} (kN·m)	10.7	8.57	7.89	6.83	5.56	3.97
$(b_{el}/t)\sqrt{345}$	214	310	124	173	255	421
§ $(b_{el}/t)\sqrt{350}$	216	312	125	174	257	424

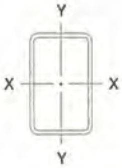
IMPERIAL SIZE AND WEIGHT

Weight (lb./ft.)	8.81	6.89	8.45	7.11	5.61	3.91
Design Thick.(in.)	0.225	0.169	0.281	0.225	0.169	0.113
Size (in.)	3½ x 2½		3 x 2			

§ See S16-14 Clause 27.1.7 for seismic applications

RECTANGULAR HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$



ASTM A500 Grade C $F_y = 345$ MPa

Section (mm x mm x mm)		HSS 76 x 38			HSS 64 x 38			HSS 51 x 25	
		6.4	4.8	3.2	6.4	4.8	3.2	4.8	3.2
Mass (kg/m)		9.31	7.40	5.18	8.05	6.45	4.55	4.54	3.28
Effective length (KL) in millimetres with respect to the least radius of gyration	0	338	267	186	294	233	164	166	119
	500	310	247	173	267	215	152	129	96
	1 000	217	178	128	184	153	111	64	50
	1 500	136	114	83	113	96	71	33	27
	2 000	87	73	55	72	62	46		
	2 500	59	50	37	48	42	32		
	3 000			27			23		
	3 500								
	4 000								
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	16 000								
PROPERTIES AND DESIGN DATA									
Area (mm^2)		1 090	861	600	947	752	527	534	382
r_x (mm)		25.1	25.9	26.8	21.1	21.9	22.7	16.4	17.3
r_y (mm)		14.3	14.9	15.5	13.9	14.6	15.2	9.29	9.93
r_x / r_y		1.76	1.74	1.73	1.52	1.50	1.49	1.77	1.74
M_{rx} (kN·m)		7.51	6.15	4.44	5.53	4.56	3.32	2.40	1.82
M_{ry} (kN·m)		4.53	3.76	2.73	3.82	3.17	2.33	1.44	1.11
$(b_{el}/t)\sqrt{345}$		173	255	421	132	200	338	145	256
§ $(b_{el}/t)\sqrt{350}$		174	257	424	133	201	341	146	257
IMPERIAL SIZE AND WEIGHT									
Weight (lb./ft.)		6.26	4.97	3.48	5.41	4.33	3.06	3.05	2.21
Design Thick.(in.)		0.225	0.169	0.113	0.225	0.169	0.113	0.169	0.113
Size (in.)		3 x 1½			2½ x 1½			2 x 1	

§ See S16-14 Clause 27.1.7 for seismic applications

ROUND HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$

ASTM A500 Grade C

F_y = 317 MPa

Section (mm x mm)	HSS 508		HSS 457		HSS 406			
	13*	† 9.5*	13*	9.5*	16	13	9.5	† 6.4
Mass (kg/m)	155	117	139	105	153	123	93.3	62.6
Effective length (KL) in millimetres with respect to the least radius of gyration								
0	5 080	3 850	4 560	3 450	5 020	4 050	3 050	2 050
500	5 080	3 850	4 560	3 450	5 020	4 050	3 050	2 050
1 000	5 080	3 850	4 560	3 450	5 020	4 050	3 050	2 050
1 500	5 070	3 840	4 550	3 440	5 000	4 040	3 040	2 050
2 000	5 060	3 840	4 540	3 430	4 980	4 020	3 030	2 040
2 500	5 040	3 820	4 520	3 420	4 950	4 000	3 010	2 030
3 000	5 020	3 810	4 490	3 400	4 910	3 960	2 990	2 010
3 500	4 990	3 780	4 450	3 370	4 850	3 920	2 950	1 990
4 000	4 950	3 750	4 410	3 340	4 780	3 860	2 910	1 960
4 500	4 900	3 720	4 350	3 300	4 700	3 800	2 870	1 930
5 000	4 850	3 680	4 290	3 250	4 610	3 720	2 810	1 890
5 500	4 780	3 630	4 220	3 190	4 500	3 640	2 750	1 850
6 000	4 710	3 580	4 140	3 130	4 390	3 550	2 680	1 810
6 500	4 630	3 520	4 050	3 070	4 260	3 450	2 600	1 760
7 000	4 550	3 450	3 950	3 000	4 130	3 340	2 530	1 710
7 500	4 460	3 380	3 850	2 920	3 990	3 230	2 440	1 650
8 000	4 360	3 310	3 740	2 840	3 850	3 120	2 360	1 590
8 500	4 250	3 230	3 630	2 760	3 700	3 000	2 270	1 540
9 000	4 150	3 150	3 520	2 670	3 550	2 880	2 180	1 480
9 500	4 030	3 070	3 400	2 580	3 410	2 770	2 100	1 420
10 000	3 920	2 980	3 290	2 500	3 260	2 650	2 010	1 360
10 500	3 800	2 900	3 170	2 410	3 120	2 540	1 920	1 300
11 000	3 690	2 810	3 050	2 320	2 980	2 430	1 840	1 250
11 500	3 570	2 720	2 940	2 230	2 850	2 320	1 760	1 190
12 000	3 450	2 630	2 820	2 150	2 720	2 210	1 680	1 140
12 500	3 340	2 540	2 710	2 070	2 600	2 110	1 610	1 090
13 000	3 220	2 460	2 610	1 980	2 480	2 020	1 530	1 040
14 000	3 000	2 290	2 400	1 830	2 260	1 840	1 400	949
15 000	2 790	2 130	2 210	1 690	2 050	1 670	1 270	865
16 000	2 590	1 980	2 040	1 550	1 870	1 530	1 160	790
17 000	2 410	1 840	1 870	1 430	1 710	1 390	1 060	722
18 000	2 230	1 710	1 730	1 320	1 560	1 280	973	662
PROPERTIES AND DESIGN DATA								
Area (mm^2)	17 800	13 500	16 000	12 100	17 600	14 200	10 700	7 200
r (mm)	176	177	158	159	139	140	141	142
M_r (kN·m)	805	471	648	494	628	508	388	203
(D / t) 317	14 100	18 800	12 700	16 900	9 020	11 300	15 000	22 500
§ (D / t) 350	15 600	20 700	14 000	18 700	9 950	12 400	16 600	24 900
IMPERIAL SIZE AND WEIGHT								
Weight (lb./ft.)	104	78.7	93.6	70.7	103	82.9	62.7	42.1
Design Thick. (in.)	0.450	0.338	0.450	0.338	0.563	0.450	0.338	0.225
Size (in.)	20 OD		18 OD		16 OD			

§ See S16-14 Clause 27.1.7 for seismic applications

* Imported section

† Class 3

ROUND HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$

ASTM A500 Grade C $F_y = 317$ MPa

Section (mm x mm)	HSS 356			HSS 324		
	13	9.5	† 6.4	13	9.5	6.4
Mass (kg/m)	107	81.3	54.7	97.5	73.9	49.7
Effective length (KL) in millimetres with respect to the least radius of gyration						
0	3 540	2 670	1 790	3 200	2 430	1 630
500	3 540	2 670	1 790	3 190	2 420	1 630
1 000	3 530	2 660	1 790	3 190	2 420	1 630
1 500	3 520	2 650	1 790	3 170	2 410	1 620
2 000	3 500	2 640	1 780	3 150	2 390	1 610
2 500	3 470	2 620	1 760	3 110	2 370	1 590
3 000	3 430	2 590	1 740	3 070	2 330	1 570
3 500	3 370	2 550	1 710	3 010	2 280	1 540
4 000	3 310	2 500	1 680	2 930	2 230	1 500
4 500	3 230	2 440	1 640	2 850	2 170	1 460
5 000	3 140	2 370	1 600	2 750	2 090	1 410
5 500	3 040	2 300	1 550	2 650	2 020	1 360
6 000	2 940	2 220	1 500	2 540	1 930	1 310
6 500	2 830	2 140	1 450	2 420	1 850	1 250
7 000	2 720	2 060	1 390	2 310	1 760	1 190
7 500	2 600	1 970	1 330	2 190	1 670	1 130
8 000	2 480	1 880	1 270	2 070	1 590	1 080
8 500	2 360	1 790	1 220	1 960	1 500	1 020
9 000	2 250	1 710	1 160	1 850	1 420	964
9 500	2 140	1 620	1 100	1 750	1 340	911
10 000	2 030	1 540	1 050	1 650	1 260	860
10 500	1 920	1 460	994	1 550	1 190	812
11 000	1 820	1 390	943	1 470	1 130	766
11 500	1 730	1 320	895	1 380	1 060	723
12 000	1 640	1 250	849	1 300	1 000	682
12 500	1 550	1 180	805	1 230	946	645
13 000	1 470	1 120	764	1 160	893	609
14 000	1 330	1 010	688	1 040	798	545
15 000	1 190	912	621	930	716	489
16 000	1 080	825	562	836	644	440
17 000	979	748	510	755	582	397
18 000	890	681	464	684	527	360
PROPERTIES AND DESIGN DATA						
Area (mm^2)	12 400	9 350	6 290	11 200	8 500	5 720
r (mm)	122	123	124	111	112	113
M_r (kN·m)	385	294	154	320	243	165
(D / t) 317	9 860	13 100	19 700	8 980	12 000	18 000
§ (D / t) 350	10 900	14 500	21 800	9 920	13 200	19 800
IMPERIAL SIZE AND WEIGHT						
Weight (lb./ft.)	72.2	54.7	36.8	65.5	49.6	33.4
Design Thick. (in.)	0.450	0.338	0.225	0.450	0.338	0.225
Size (in.)	14 OD			12.75 OD		

§ See S16-14 Clause 27.1.7 for seismic applications

† Class 3

ROUND HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$

ASTM A500 Grade C
$F_y = 317$ MPa

Section (mm x mm)	HSS 273					HSS 245	
	13	9.5	7.9	6.4	† 4.8	9.5	6.4
Mass (kg/m)	81.6	61.9	51.9	41.8	31.6	55.2	37.3
Effective length (KL) in millimetres with respect to the least radius of gyration							
0	2 680	2 030	1 700	1 370	1 040	1 810	1 220
500	2 680	2 030	1 700	1 370	1 040	1 810	1 220
1 000	2 670	2 030	1 700	1 360	1 030	1 810	1 220
1 500	2 650	2 010	1 690	1 360	1 030	1 790	1 210
2 000	2 620	1 990	1 670	1 340	1 010	1 760	1 190
2 500	2 570	1 950	1 640	1 320	997	1 720	1 160
3 000	2 510	1 910	1 600	1 290	974	1 670	1 130
3 500	2 430	1 850	1 550	1 250	945	1 600	1 080
4 000	2 340	1 780	1 500	1 200	912	1 520	1 030
4 500	2 240	1 710	1 430	1 150	874	1 440	979
5 000	2 130	1 620	1 360	1 100	833	1 360	921
5 500	2 010	1 540	1 290	1 040	791	1 270	862
6 000	1 900	1 450	1 220	984	747	1 180	804
6 500	1 780	1 360	1 150	926	702	1 100	747
7 000	1 670	1 280	1 070	868	659	1 020	693
7 500	1 560	1 190	1 010	813	617	940	642
8 000	1 450	1 110	939	759	577	869	594
8 500	1 350	1 040	876	709	539	804	550
9 000	1 260	970	817	662	503	743	509
9 500	1 180	904	763	617	470	688	471
10 000	1 100	844	712	576	438	638	437
10 500	1 020	787	664	538	410	592	406
11 000	954	736	621	503	383	550	377
11 500	892	688	581	471	358	512	351
12 000	835	644	544	441	336	477	328
12 500	782	603	510	413	315	446	306
13 000	733	566	478	388	295	417	286
14 000	648	500	423	343	261	366	252
15 000	575	444	376	305	232	324	223
16 000	513	397	335	272	207	288	198
17 000	460	356	301	244	186		
18 000	415	321	271	220	168		
PROPERTIES AND DESIGN DATA							
Area (mm^2)	9 400	7 130	5 970	4 800	3 630	6 360	4 290
r (mm)	92.6	93.6	94.1	94.6	95.0	83.5	84.4
M_r (kN·m)	223	171	144	117	68.5	136	93.0
(D / t) 317	7 570	10 100	12 100	15 100	20 100	9 030	13 600
§ (D / t) 350	8 360	11 100	13 400	16 700	22 200	9 970	15 000
IMPERIAL SIZE AND WEIGHT							
Weight (lb./ft.)	54.8	41.6	34.9	28.1	21.3	37.1	25.1
Design Thick. (in.)	0.450	0.338	0.281	0.225	0.169	0.338	0.225
Size (in.)	10.75 OD					9.625 OD	

§ See S16-14 Clause 27.1.7 for seismic applications

† Class 3

ROUND HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$

ASTM A500 Grade C

$F_y = 317$ MPa

Section (mm x mm)	HSS 219				HSS 178		HSS 168			
	13	9.5	6.4	4.8	13	9.5	13	9.5	6.4	4.8
Mass (kg/m)	64.6	49.3	33.3	25.3	51.7	39.5	48.7	37.3	25.4	19.3
Effective length (KL) in millimetres with respect to the least radius of gyration										
0	2 130	1 620	1 090	827	1 700	1 300	1 610	1 230	833	633
500	2 130	1 620	1 090	827	1 700	1 300	1 600	1 230	831	632
1 000	2 110	1 610	1 090	822	1 680	1 290	1 580	1 210	822	625
1 500	2 090	1 590	1 070	813	1 640	1 260	1 540	1 180	802	611
2 000	2 040	1 550	1 050	796	1 580	1 210	1 470	1 130	770	586
2 500	1 980	1 510	1 020	773	1 500	1 150	1 380	1 070	726	554
3 000	1 890	1 440	979	742	1 390	1 070	1 280	986	673	514
3 500	1 790	1 370	930	706	1 280	987	1 160	900	616	471
4 000	1 680	1 290	876	666	1 160	900	1 040	812	557	427
4 500	1 570	1 200	819	623	1 050	813	933	728	501	384
5 000	1 450	1 110	761	579	942	732	831	649	448	344
5 500	1 340	1 030	703	535	844	656	739	578	400	307
6 000	1 230	945	647	493	755	589	657	515	357	274
6 500	1 120	866	594	453	677	528	585	460	319	246
7 000	1 030	794	545	416	607	474	523	412	286	220
7 500	939	727	500	382	546	427	469	369	257	198
8 000	860	666	458	350	493	386	422	333	231	178
8 500	788	611	421	322	446	350	381	300	209	161
9 000	723	561	387	296	405	318	345	272	190	146
9 500	665	516	356	272	369	289	314	248	173	133
10 000	613	476	328	251	337	265	286	226	158	122
10 500	565	440	303	232	309	243	262	207	144	111
11 000	523	407	281	215	284	223	240	190	133	102
11 500	485	377	261	200	262	206			122	94
12 000	450	350	242	185						
12 500	419	326	226	173						
13 000	391	304	210	161						
14 000	342	266	184	141						
15 000			162	124						
16 000										
17 000										
18 000										
PROPERTIES AND DESIGN DATA										
Area (mm^2)	7 460	5 670	3 830	2 900	5 970	4 560	5 630	4 310	2 920	2 220
r (mm)	73.5	74.5	75.5	76.0	59.0	59.9	55.6	56.6	57.5	58.0
M_r (kN·m)	141	108	74.2	56.5	90.4	70.2	80.5	62.5	43.1	33.1
(D / t) 317	6 080	8 090	12 100	16 200	4 930	6 570	4 670	6 220	9 330	12 400
§ (D / t) 350	6 710	8 940	13 400	17 800	5 440	7 250	5 150	6 870	10 300	13 700
IMPERIAL SIZE AND WEIGHT										
Weight (lb./ft.)	43.4	33.1	22.4	17.0	34.7	26.6	32.7	25.1	17.0	13.0
Design Thick. (in.)	0.450	0.338	0.225	0.169	0.450	0.338	0.450	0.338	0.225	0.169
Size (in.)	8.625 OD				7 OD		6.625 OD			

§ See S16-14 Clause 27.1.7 for seismic applications

ROUND HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$

ASTM A500 Grade C
F_y = 317 MPa

Section (mm x mm)	HSS 141			HSS 127	
	13	9.5	6.4	9.5	6.4
Mass (kg/m)	40.3	31.0	21.1	27.6	18.9
Effective length (KL) in millimetres with respect to the least radius of gyration					
0	1 330	1 020	696	910	622
500	1 320	1 020	694	906	619
1 000	1 300	999	682	884	605
1 500	1 240	959	656	837	574
2 000	1 160	896	615	767	528
2 500	1 050	818	563	683	473
3 000	938	731	506	595	414
3 500	823	644	447	513	358
4 000	716	562	392	439	308
4 500	622	490	342	376	265
5 000	540	426	299	323	228
5 500	470	372	261	279	197
6 000	411	325	229	242	171
6 500	361	286	202	211	150
7 000	319	253	179	186	132
7 500	283	225	159	164	117
8 000	252	201	142	146	104
8 500	226	180	127		93
9 000	204	162	115		
9 500			104		
10 000					
10 500					
11 000					
11 500					
12 000					
12 500					
13 000					
14 000					
15 000					
16 000					
17 000					
18 000					
PROPERTIES AND DESIGN DATA					
Area (mm²)	4 660	3 580	2 440	3 190	2 180
r (mm)	46.1	47.0	48.0	42.0	42.9
M_r (kN·m)	55.1	43.1	30.0	34.5	24.0
(D / t) 317	3 920	5 220	7 830	4 690	7 040
§ (D / t) 350	4 330	5 760	8 650	5 180	7 770
IMPERIAL SIZE AND WEIGHT					
Weight (lb./ft.)	27.1	20.8	14.2	18.6	12.7
Design Thick. (in.)	0.450	0.338	0.225	0.338	0.225
Size (in.)	5.563 OD			5 OD	

§ See S16-14 Clause 27.1.7 for seismic applications

ROUND HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$

ASTM A500 Grade C $F_y = 317$ MPa

Section (mm x mm)	HSS 89			HSS 76			HSS 73		
	6.4	4.8	3.2	6.4	4.8	3.2	6.4	4.8	3.2
Mass (kg/m)	12.9	9.92	6.72	10.9	8.42	5.73	10.4	8.04	5.48
Effective length (KL) in millimetres with respect to the least radius of gyration									
0	425	325	221	362	277	188	345	265	180
500	420	321	218	356	272	185	338	259	176
1 000	395	303	206	324	249	170	305	235	160
1 500	348	268	183	270	209	143	250	194	133
2 000	290	225	154	213	166	115	194	151	105
2 500	235	183	126	164	129	89	147	116	81
3 000	188	147	102	127	100	69	113	89	62
3 500	152	118	82	100	79	55	88	69	49
4 000	123	96	67	79	63	44	70	55	39
4 500	101	79	55	64	51	36	57	45	32
5 000	84	66	46	53	42	29			
5 500	71	56	39						
6 000			33						
6 500									
7 000									
7 500									
8 000									
8 500									
9 000									
9 500									
10 000									
10 500									
11 000									
11 500									
12 000									
12 500									
13 000									
14 000									
15 000									
16 000									
17 000									
18 000									
PROPERTIES AND DESIGN DATA									
Area (mm^2)	1 490	1 140	773	1 270	971	659	1 210	928	630
r (mm)	29.5	29.9	30.4	25.0	25.5	25.9	23.9	24.3	24.8
M_r (kN·m)	11.3	8.79	6.05	8.13	6.36	4.39	7.42	5.79	4.02
(D / t) 317	4 930	6 550	9 850	4 220	5 620	8 450	4 050	5 380	8 090
§ (D / t) 350	5 440	7 240	10 900	4 660	6 200	9 330	4 470	5 940	8 930
IMPERIAL SIZE AND WEIGHT									
Weight (lb./ft.)	8.69	6.66	4.52	7.35	5.66	3.85	7.01	5.40	3.68
Design Thick. (in.)	0.225	0.169	0.113	0.225	0.169	0.113	0.225	0.169	0.113
Size (in.)	3.5 OD			3 OD			2.875 OD		

§ See S16-14 Clause 27.1.7 for seismic applications

ROUND HOLLOW SECTIONS

Factored Axial Compressive Resistances, C_r (kN) $\phi = 0.90$

ASTM A500
Grade C
F_y = 317 MPa

Section (mm x mm)	HSS 64			HSS 60			HSS 48	
	6.4	4.8	3.2	6.4	4.8	3.2	4.8	3.2
Mass (kg/m)	8.95	6.92	4.73	8.45	6.54	4.48	5.13	3.54
Effective length (KL) in millimetres with respect to the least radius of gyration								
0	297	228	155	280	216	147	169	116
500	288	222	151	270	208	143	159	110
1 000	247	192	132	228	178	122	121	85
1 500	190	149	104	170	134	93	81	58
2 000	139	110	77	122	97	68	54	39
2 500	101	81	57	88	70	50	37	27
3 000	76	61	43	65	52	37	27	19
3 500	58	47	33	50	40	28		
4 000	46	37	26			22		
4 500								
5 000								
5 500								
6 000								
6 500								
7 000								
7 500								
8 000								
8 500								
9 000								
9 500								
10 000								
10 500								
11 000								
11 500								
12 000								
12 500								
13 000								
14 000								
15 000								
16 000								
17 000								
18 000								
PROPERTIES AND DESIGN DATA								
Area (mm^2)	1 040	800	545	981	756	516	594	408
r (mm)	20.5	21.0	21.5	19.4	19.9	20.3	15.6	16.1
M_r (kN·m)	5.48	4.31	3.00	4.88	3.85	2.69	2.38	1.69
(D / t) 317	3 520	4 680	7 040	3 340	4 450	6 680	3 560	5 350
§ (D / t) 350	3 890	5 170	7 770	3 690	4 910	7 380	3 930	5 910
IMPERIAL SIZE AND WEIGHT								
Weight (lb./ft.)	6.01	4.65	3.18	5.68	4.40	3.01	3.45	2.38
Design Thick. (in.)	0.225	0.169	0.113	0.225	0.169	0.113	0.169	0.113
Size (in.)	2.5 OD			2.375 OD			1.9 OD	

§ See S16-14 Clause 27.1.7 for seismic applications

BEAM-COLUMNS

Table 4-6 on the next page provides the essence of CSA S16-14 Clause 11 and lists the width-to-thickness ratios for Class 1, 2 and 3 sections for various elements in flexural compression. All sections not meeting these requirements are Class 4. The class for webs in combined flexural and axial compression is a function of the ratio of the factored axial load to the axial compressive load at yield stress $C_f/(\phi\, C_y)$, in accordance with Clause 11.2.

Values of $C_f/(\phi\, C_y)$ at which the webs change class are tabulated in Table 4-7. The tables may be used for W-shapes produced to ASTM A992, A572 Grade 50 and A913 Grade 50. Some members with webs that are always Class 1 are controlled by flanges that are not Class 1. Therefore, these members and their flange classification are also included in the tables.

Table 4-8 lists values of the equivalent uniform bending coefficients, ω_1 for various ratios M_{f1}/M_{f2} of factored end bending moments applied to beam-columns. The values of ω_1 are computed in accordance with the requirements of Clause 13.8.5, S16-14.

Table 4-9 has been prepared to facilitate the design of beam-columns in accordance with the requirements of Clause 13.8, S16-14, which incorporates the variable U in the factor U_1. Values of the amplification factor U corresponding to various values of C_f/C_e are listed.

The tables on subsequent pages list the factored moment resistance for pure bending about the major axis, M_{rx}, for cases where the unsupported length of compression flange, L, is less than L_u, and the factored moment resistance, M'_{rx}, where L is greater than L_u. The first table includes W-shapes normally used as columns and produced to ASTM A992 and A572 Grade 50, while the second includes W-shapes produced to ASTM A913 Grade 65. Tabulated values for A992 and A572 Grade 50 may also be used for W-shape columns produced to CSA G40.21-350W. Sections are ordered as in Part 6 of this Handbook, with all of the sections of the same nominal dimensions listed together.

The M_{rx} and M'_{rx} values are based on the class of section in bending about the X-X axis, without axial load. However, the class of a section used as a beam-column is a function of the ratio $C_f/(\phi\, C_y)$ as mentioned above. For example, a W410x39 of ASTM A992 steel becomes a Class 3 section when $C_f/(\phi\, C_y)$ exceeds 0.572, based on the Class 2 limit for h/w of:

$$\frac{1700}{\sqrt{F_y}}\left(1-0.61\frac{C_f}{\phi C_y}\right)$$

Thus, sections whose loading causes a change from Class 2 to Class 3 need to have their tabulated values of M_{rx} and M'_{rx} adjusted. A conservative method is to multiply the listed values by the factor S_x/Z_x.

WIDTH-TO-THICKNESS RATIOS

Table 4-6

Elements in Flexural Compression[1]

Description of Element	Maximum Width-to-Thickness Ratios		
	Class 1	Class 2	Class 3
Flanges of I-sections or T-sections in bending about the major axis; plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact with an axis of symmetry in the plane of loading	$\frac{b_{el}}{t} \le \frac{145}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{170}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{200}{\sqrt{F_y}}$
Stems of T-sections; flanges of I-sections in bending about the minor axis	$\frac{b_{el}}{t} \le \frac{145}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{170}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{340}{\sqrt{F_y}}$
Flanges of rectangular hollow sections	$\frac{b_{el}}{t} \le \frac{420}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{670}{\sqrt{F_y}}$
Flanges of box sections; flange cover plates and diaphragm plates between lines of fasteners or welds; flange cover plates and diaphragm plates between lines of fasteners or welds	$\frac{b_{el}}{t} \le \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{670}{\sqrt{F_y}}$
Webs of I-sections in bending about the major axis	$\frac{h}{w} \le \frac{1100}{\sqrt{F_y}}\left(1-0.39\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1700}{\sqrt{F_y}}\left(1-0.61\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left(1-0.65\frac{C_f}{\phi C_y}\right)$
Webs of I-sections in bending about the minor axis: a) For $C_f > 0.4\phi C_y$	$\frac{h}{w} \le \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \le \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left(1-0.65\frac{C_f}{\phi C_y}\right)$
b) For $C_f \le 0.4\phi C_y$	$\frac{h}{w} \le \frac{1100}{\sqrt{F_y}}\left(1-1.31\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1700}{\sqrt{F_y}}\left(1-1.73\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left(1-0.65\frac{C_f}{\phi C_y}\right)$
Webs of I-sections in bi-axial bending[2], with $\frac{M_{fy}}{S_y} > \frac{0.9M_{fx}}{S_x}$	$\frac{h}{w} \le \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \le \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left(1-0.65\frac{C_f}{\phi C_y}\right)$
Circular hollow sections	$\frac{D}{t} \le \frac{13\,000}{F_y}$	$\frac{D}{t} \le \frac{18\,000}{F_y}$	$\frac{D}{t} \le \frac{66\,000}{F_y}$

1. See CSA S16-14 Clause 11.

2. If $\frac{M_{fy}}{S_y} \le \frac{0.9M_{fx}}{S_x}$, the limits for webs of I-sections subject to combined axial compression and bending about the major axis shall apply.

Designation	Web			Flange
	1 $C_f/\phi C_y \le$	2 $C_f/\phi C_y \le$	3 $C_f/\phi C_y \le$	
W1100x499	0.852	0.931	0.944	1
x433	0.541	0.802	0.836	1
x390	0.339	0.719	0.765	1
x343	0.091	0.616	0.680	1
W1000x976	1.0	—	—	1
x883	1.0	—	—	1
x748	1.0	—	—	1
x642	1.0	—	—	1
x591	1.0	—	—	1
x554	1.0	—	—	1
x539	1.0	—	—	1
x483	0.982	0.985	0.989	1
x443	0.861	0.935	0.947	1
x412	0.660	0.852	0.877	1
x371	0.450	0.765	0.804	1
x321	0.129	0.632	0.693	1
x296	0.130	0.632	0.693	1
W1000x584	1.0	—	—	1
x494	1.0	—	—	1
x486	1.0	—	—	1
x438	1.0	—	—	1
x415	1.0	—	—	1
x393	0.917	0.958	0.966	1
x350	0.660	0.852	0.877	1
x314	0.460	0.769	0.808	1
x272	0.129	0.632	0.693	1
x249	0.129	0.632	0.693	1
x222	0.053	0.601	0.666	1
W920x1377	1.0	—	—	1
x1269	1.0	—	—	1
x1194	1.0	—	—	1
x1077	1.0	—	—	1
x970	1.0	—	—	1
x787	1.0	—	—	1
x725	1.0	—	—	1
x656	1.0	—	—	1
x588	1.0	—	—	1
x537	1.0	—	—	1
x491	1.0	—	—	1
x449	1.0	—	—	1
x420	0.903	0.952	0.961	1
x390	0.810	0.914	0.929	1
x368	0.725	0.878	0.900	1
x344	0.628	0.838	0.866	1
W920x381	1.0	—	—	1
x345	0.873	0.940	0.951	1
x313	0.793	0.907	0.923	1
x289	0.638	0.843	0.869	1
x271	0.533	0.799	0.833	1
x253	0.404	0.746	0.788	1
x238	0.299	0.702	0.752	1
x223	0.214	0.667	0.722	1
x201	0.106	0.623	0.685	1

Designation	Web			Flange
	1 $C_f/\phi C_y \le$	2 $C_f/\phi C_y \le$	3 $C_f/\phi C_y \le$	
W840x576	1.0	—	—	1
x527	1.0	—	—	1
x473	1.0	—	—	1
x433	1.0	—	—	1
x392	1.0	—	—	1
x359	0.929	0.963	0.971	1
x329	0.812	0.915	0.930	1
x299	0.669	0.855	0.880	1
W840x251	0.534	0.800	0.833	1
x226	0.420	0.752	0.794	1
x210	0.323	0.712	0.760	1
x193	0.218	0.669	0.723	1
x176	0.098	0.619	0.682	1
W760x582	1.0	—	—	1
x531	1.0	—	—	1
x484	1.0	—	—	1
x434	1.0	—	—	1
x389	1.0	—	—	1
x350	1.0	—	—	1
x314	0.983	0.985	0.989	1
x284	0.835	0.924	0.938	1
x257	0.689	0.864	0.887	1
W760x220	0.677	0.859	0.883	1
x196	0.568	0.814	0.845	1
x185	0.475	0.775	0.813	1
x173	0.403	0.745	0.788	1
x161	0.307	0.706	0.754	1
x147	0.206	0.664	0.719	1
x134	—	0.557	0.630	2
W690x802	1.0	—	—	1
x548	1.0	—	—	1
x500	1.0	—	—	1
x457	1.0	—	—	1
x419	1.0	—	—	1
x384	1.0	—	—	1
x350	1.0	—	—	1
x323	1.0	—	—	1
x289	1.0	—	—	1
x265	1.0	—	—	1
x240	0.899	0.950	0.960	1
x217	0.750	0.889	0.908	1
W690x192	0.759	0.893	0.911	1
x170	0.636	0.842	0.869	1
x152	0.430	0.756	0.797	1
x140	0.308	0.706	0.755	1
x125	0.176	0.651	0.709	1

— Indicates web is never that class.
For seismic applications, see S16-14 Clause 27.1.7.

Designation	Web			Flange
	1 $C_f/\phi C_y \le$	2 $C_f/\phi C_y \le$	3 $C_f/\phi C_y \le$	
W610x551	1.0	—	—	1
x498	1.0	—	—	1
x455	1.0	—	—	1
x415	1.0	—	—	1
x372	1.0	—	—	1
x341	1.0	—	—	1
x307	1.0	—	—	1
x285	1.0	—	—	1
x262	1.0	—	—	1
x241	1.0	—	—	1
x217	1.0	—	—	1
x195	0.953	0.973	0.979	1
x174	0.793	0.907	0.923	1
x155	0.611	0.831	0.860	2
W610x153	0.791	0.906	0.923	1
x140	0.672	0.856	0.881	1
x125	0.480	0.777	0.815	1
x113	0.347	0.722	0.768	1
x101	0.201	0.662	0.717	1
W610x92	0.288	0.698	0.748	1
x82	0.081	0.612	0.676	1
W530x409	1.0	—	—	1
x369	1.0	—	—	1
x332	1.0	—	—	1
x300	1.0	—	—	1
x272	1.0	—	—	1
x248	1.0	—	—	1
x219	1.0	—	—	1
x196	1.0	—	—	1
x182	1.0	—	—	1
x165	1.0	—	—	1
x150	0.851	0.931	0.944	1
W530x138	1.0	—	—	1
x123	0.906	0.954	0.963	1
x109	0.693	0.865	0.888	1
x101	0.569	0.814	0.846	1
x92	0.434	0.758	0.799	1
x82	0.279	0.694	0.745	2
x72	0.148	0.640	0.699	3
W530x85	0.454	0.766	0.805	1
x74	0.324	0.713	0.760	1
x66	0.121	0.629	0.690	1

— Indicates web is never that class.
For seismic applications, see S16-14 Clause 27.1.7.

Designation	Web			Flange
	1 $C_f/\phi C_y \le$	2 $C_f/\phi C_y \le$	3 $C_f/\phi C_y \le$	
W460x464	1.0	—	—	1
x421	1.0	—	—	1
x384	1.0	—	—	1
x349	1.0	—	—	1
x315	1.0	—	—	1
x286	1.0	—	—	1
x260	1.0	—	—	1
x235	1.0	—	—	1
x213	1.0	—	—	1
x193	1.0	—	—	1
x177	1.0	—	—	1
x158	1.0	—	—	1
x144	1.0	—	—	1
x128	1.0	—	—	1
x113	0.847	0.929	0.942	2
W460x106	1.0	—	—	1
x97	0.939	0.967	0.974	1
x89	0.801	0.910	0.926	1
x82	0.692	0.865	0.888	1
x74	0.505	0.788	0.823	1
W460x68	0.527	0.797	0.831	1
x60	0.246	0.680	0.733	1
x52	0.124	0.630	0.691	1
W410x149	1.0	—	—	1
x132	1.0	—	—	1
x114	1.0	—	—	1
x100	0.914	0.957	0.965	1
W410x85	1.0	—	—	1
x74	0.863	0.936	0.948	1
x67	0.689	0.863	0.887	1
x60	0.420	0.752	0.794	1
x54	0.363	0.729	0.774	2
W410x46	0.210	0.665	0.721	1
x39	—	0.572	0.642	2
W360x1086	1.0	—	—	1
x990	1.0	—	—	1
x900	1.0	—	—	1
x818	1.0	—	—	1
x744	1.0	—	—	1
x677	1.0	—	—	1
x634	1.0	—	—	1
x592	1.0	—	—	1
x551	1.0	—	—	1
x509	1.0	—	—	1
x463	1.0	—	—	1
x421	1.0	—	—	1
x382	1.0	—	—	1
x347	1.0	—	—	1
x314	1.0	—	—	1
x287	1.0	—	—	1
x262	1.0	—	—	1
x237	1.0	—	—	1
x216	1.0	—	—	1

CLASS OF SECTIONS
Combined Axial Compression and Major-Axis Bending

Table 4-7
F_y = 345 MPa
ASTM A992, A572 Gr. 50, A913 Gr. 50

Designation	Web 1 $C_f/\phi C_y \le$	Web 2 $C_f/\phi C_y \le$	Web 3 $C_f/\phi C_y \le$	Flange
W360x196	1.0	—	—	1
x179	1.0	—	—	1
x162	1.0	—	—	2
x147	1.0	—	—	3
x134	1.0	—	—	3
W360x122	1.0	—	—	1
x110	1.0	—	—	1
x101	1.0	—	—	1
x91	1.0	—	—	1
W360x79	1.0	—	—	1
x72	0.954	0.973	0.979	1
x64	0.765	0.895	0.913	1
W360x57	0.746	0.887	0.907	1
x51	0.569	0.814	0.845	1
x45	0.478	0.776	0.814	2
W360x39	0.355	0.726	0.771	1
x33	0.086	0.614	0.678	1
W310x500	1.0	—	—	1
x454	1.0	—	—	1
x415	1.0	—	—	1
x375	1.0	—	—	1
x342	1.0	—	—	1
x313	1.0	—	—	1
x283	1.0	—	—	1
x253	1.0	—	—	1
x226	1.0	—	—	1
x202	1.0	—	—	1
x179	1.0	—	—	1
x158	1.0	—	—	1
x143	1.0	—	—	1
x129	1.0	—	—	1
x118	1.0	—	—	2
x107	1.0	—	—	2
x97	1.0	—	—	3
W310x86	1.0	—	—	1
x79	1.0	—	—	2
W310x74	1.0	—	—	1
x67	1.0	—	—	1
x60	0.966	0.978	0.983	1
W310x52	0.909	0.954	0.963	1
x45	0.658	0.851	0.876	1
x39	0.395	0.742	0.785	2
W310x33	0.652	0.849	0.874	1
x28	0.463	0.770	0.809	1
x24	0.310	0.707	0.755	1
x21	0.089	0.615	0.679	2

— Indicates web is never that class.
For seismic applications, see S16-14 Clause 27.1.7.

Designation	Web 1 $C_f/\phi C_y \le$	Web 2 $C_f/\phi C_y \le$	Web 3 $C_f/\phi C_y \le$	Flange
W250x167	1.0	—	—	1
x149	1.0	—	—	1
x131	1.0	—	—	1
x115	1.0	—	—	1
x101	1.0	—	—	1
x89	1.0	—	—	1
x80	1.0	—	—	2
x73	1.0	—	—	2
W250x67	1.0	—	—	1
x58	1.0	—	—	1
x49	1.0	—	—	3
W250x45	1.0	—	—	1
x39	1.0	—	—	1
x33	0.862	0.935	0.947	2
W250x28	0.940	0.968	0.974	1
x25	0.859	0.934	0.946	1
x22	0.771	0.898	0.916	1
x18	0.396	0.742	0.785	3
W200x100	1.0	—	—	1
x86	1.0	—	—	1
x71	1.0	—	—	1
x59	1.0	—	—	1
x52	1.0	—	—	2
x46	1.0	—	—	3
W200x42	1.0	—	—	1
x36	1.0	—	—	2
W200x31	1.0	—	—	1
x27	1.0	—	—	2
W200x22	1.0	—	—	1
x19	1.0	—	—	2
x15	0.655	0.850	0.875	3
W150x37	1.0	—	—	1
x30	1.0	—	—	2
x22	1.0	—	—	4
W150x24	1.0	—	—	1
x18	1.0	—	—	1
x14	1.0	—	—	2
x13	1.0	—	—	3
W130x28	1.0	—	—	1
x24	1.0	—	—	1
W100x19	1.0	—	—	1

VALUES OF ω_1 * Table 4-8

Single Curvature				Double Curvature			
$\frac{M_{f1}}{M_{f2}}$	ω_1	$\frac{M_{f1}}{M_{f2}}$	ω_1	$\frac{M_{f1}}{M_{f2}}$	ω_1	$\frac{M_{f1}}{M_{f2}}$	ω_1
1.00	1.00	0.50	0.80	0.00	0.60	0.55	0.40
0.95	0.98	0.45	0.78	0.05	0.58	0.60	0.40
0.90	0.96	0.40	0.76	0.10	0.56	0.65	0.40
0.85	0.94	0.35	0.74	0.15	0.54	0.70	0.40
0.80	0.92	0.30	0.72	0.20	0.52	0.75	0.40
0.75	0.90	0.25	0.70	0.25	0.50	0.80	0.40
0.70	0.88	0.20	0.68	0.30	0.48	0.85	0.40
0.65	0.86	0.15	0.66	0.35	0.46	0.90	0.40
0.60	0.84	0.10	0.64	0.40	0.44	0.95	0.40
0.55	0.82	0.05	0.62	0.45	0.42	1.00	0.40
		0.00	0.60	0.50	0.40		

* See Clause 13.8.5, CSA S16-14

The value of ω_1 is used to modify the bending term in the beam-column interaction expression to account for various end moment and transverse bending loading conditions of the columns.

For columns of a frame not subject to transverse loads between supports, use the values of ω_1 shown in Table 4-8.

For members subjected to distributed loads or a series of point loads between supports, $\omega_1 = 1.0$, and for members subjected to a concentrated load or moment between supports, $\omega_1 = 0.85$.

The values of ω_1 given in Table 4-8 are derived from:

$$\omega_1 = 0.6 - 0.4\,\kappa \geq 0.4$$

where:

κ = M_{f1} / M_{f2} for moments at opposite ends of the unbraced column length, positive for double curvature, and negative for single curvature in which,

M_{f1} = the smaller factored end moment, and

M_{f2} = the larger factored end moment.

AMPLIFICATION FACTOR *

Table 4-9

$$U = \frac{1}{1 - \dfrac{C_f}{C_e}}$$

$\frac{C_f}{C_e}$	U	$\frac{C_f}{C_e}$	U	$\frac{C_f}{C_e}$	U	$\frac{C_f}{C_e}$	U
0.01	1.01	0.26	1.35	0.51	2.04	0.76	4.17
0.02	1.02	0.27	1.37	0.52	2.08	0.77	4.35
0.03	1.03	0.28	1.39	0.53	2.13	0.78	4.55
0.04	1.04	0.29	1.41	0.54	2.17	0.79	4.76
0.05	1.05	0.30	1.43	0.55	2.22	0.80	5.00
0.06	1.06	0.31	1.45	0.56	2.27	0.81	5.26
0.07	1.08	0.32	1.47	0.57	2.33	0.82	5.56
0.08	1.09	0.33	1.49	0.58	2.38	0.83	5.88
0.09	1.10	0.34	1.52	0.59	2.44	0.84	6.25
0.10	1.11	0.35	1.54	0.60	2.50	0.85	6.67
0.11	1.12	0.36	1.56	0.61	2.56	0.86	7.14
0.12	1.14	0.37	1.59	0.62	2.63	0.87	7.69
0.13	1.15	0.38	1.61	0.63	2.70	0.88	8.33
0.14	1.16	0.39	1.64	0.64	2.78	0.89	9.09
0.15	1.18	0.40	1.67	0.65	2.86	0.90	10.0
0.16	1.19	0.41	1.69	0.66	2.94	0.91	11.1
0.17	1.20	0.42	1.72	0.67	3.03	0.92	12.5
0.18	1.22	0.43	1.75	0.68	3.13	0.93	14.3
0.19	1.23	0.44	1.79	0.69	3.23	0.94	16.7
0.20	1.25	0.45	1.82	0.70	3.33	0.95	20.0
0.21	1.27	0.46	1.85	0.71	3.45	0.96	25.0
0.22	1.28	0.47	1.89	0.72	3.57	0.97	33.3
0.23	1.30	0.48	1.92	0.73	3.70	0.98	50.0
0.24	1.32	0.49	1.96	0.74	3.85	0.99	100.0
0.25	1.33	0.50	2.00	0.75	4.00		

* See Clause 13.8.4 in CSA S16-14.

FACTORED MOMENT RESISTANCES
OF COLUMNS, M_{rx} and M'_{rx} (kN·m)
F_y = 345 MPa

W Shapes
ASTM A992
A572 Grade 50

Designation	M_{rx}	M'_{rx} for the following unsupported lengths in millimetres									
		6 000	8 000	10 000	12 000	14 000	16 000	18 000	20 000	24 000	28 000
W360x1086	8 450	—	—	—	—	—	—	—	—	8 270	8 030
W360x990	7 550	—	—	—	—	—	—	—	7 520	7 290	7 060
W360x900	6 710	—	—	—	—	—	—	—	6 610	6 390	6 170
W360x818	5 990	—	—	—	—	—	—	5 930	5 830	5 610	5 400
W360x744	5 340	—	—	—	—	—	5 320	5 220	5 120	4 910	4 700
W360x677	4 750	—	—	—	—	—	4 680	4 580	4 480	4 280	4 080
W360x634	4 410	—	—	—	—	4 400	4 310	4 210	4 110	3 920	3 720
W360x592	4 070	—	—	—	—	4 030	3 930	3 840	3 740	3 550	3 360
W360x551	3 760	—	—	—	—	3 680	3 580	3 490	3 390	3 200	3 010
W360x509	3 420	—	—	—	3 400	3 310	3 220	3 120	3 030	2 850	2 670
W360x463	3 070	—	—	—	3 010	2 920	2 830	2 740	2 650	2 470	2 290
W360x421	2 760	—	—	—	2 670	2 580	2 500	2 410	2 320	2 140	1 970
W360x382	2 470	—	—	2 450	2 360	2 270	2 180	2 100	2 010	1 840	1 670
W360x347	2 220	—	—	2 170	2 080	1 990	1 910	1 820	1 740	1 570	1 380
W360x314	1 980	—	—	1 900	1 820	1 730	1 650	1 570	1 480	1 320	1 130
W360x287	1 800	—	1 800	1 710	1 630	1 550	1 460	1 380	1 300	1 120	953
W360x262	1 630	—	1 610	1 530	1 440	1 360	1 280	1 190	1 110	927	790
W360x237	1 460	—	1 420	1 340	1 250	1 170	1 090	1 010	916	755	643
W360x216	1 320	—	1 280	1 190	1 110	1 030	951	867	773	637	542
W360x196	1 190	—	1 130	1 040	960	879	799	702	626	516	439
W360x179	1 080	—	1 010	924	843	762	671	588	524	430	366
W360x162	975	974	895	814	733	653	558	488	434	356	302
† W360x147	798	—	740	675	609	544	467	407	361	295	250
† W360x134	723	—	663	598	532	461	392	341	302	246	208
W310x500	3 070	—	—	—	—	2 990	2 910	2 840	2 760	2 600	2 450
W310x454	2 740	—	—	—	2 710	2 630	2 550	2 480	2 400	2 250	2 100
W310x415	2 450	—	—	—	2 390	2 320	2 250	2 170	2 100	1 960	1 810
W310x375	2 170	—	—	2 160	2 090	2 020	1 950	1 880	1 810	1 670	1 530
W310x342	1 970	—	—	1 930	1 860	1 790	1 720	1 650	1 580	1 450	1 310
W310x313	1 780	—	—	1 720	1 650	1 580	1 510	1 450	1 380	1 240	1 090
W310x283	1 580	—	1 580	1 510	1 440	1 370	1 310	1 240	1 180	1 040	890
W310x253	1 390	—	1 370	1 300	1 240	1 170	1 110	1 040	978	831	711
W310x226	1 230	—	1 190	1 120	1 060	997	934	871	803	667	570
W310x202	1 090	—	1 030	967	904	841	778	712	639	530	453
W310x179	947	941	877	814	752	691	629	555	497	412	352
W310x158	829	813	750	687	626	565	494	436	390	323	275
W310x143	751	729	666	604	543	476	411	363	324	268	229
W310x129	671	643	581	520	460	390	336	296	264	218	186
W310x118	605	574	513	452	388	324	279	245	219	180	154
W310x107	546	512	453	392	325	271	233	204	182	150	127
† W310x97	447	423	375	326	272	226	193	169	150	123	105

Note: Moment resistances are based on class of section for X-X axis of bending only, ω_2 = 1.0.

† Class 3

FACTORED MOMENT RESISTANCES
OF COLUMNS, M_{rx} and M'_{rx} (kN·m)
F_y = 345 MPa

W Shapes
ASTM A992
A572 Grade 50

Designation	M_{rx}	M'_{rx} for the following unsupported lengths in millimetres									
		4 000	5 000	6 000	7 000	8 000	9 000	10 000	12 000	14 000	16 000
W250x167	755	—	—	752	730	708	687	665	622	580	537
W250x149	661	—	—	650	628	607	586	565	523	481	439
W250x131	574	—	—	555	533	512	492	471	430	389	341
W250x115	497	—	491	470	449	429	408	388	348	301	262
W250x101	435	—	424	403	382	362	342	322	279	236	205
W250x89	382	—	367	346	326	306	285	265	220	186	161
W250x80	338	—	321	302	282	262	242	221	179	151	130
W250x73	306	—	287	268	248	228	209	185	149	126	108
W200x100	357	—	349	335	321	307	294	280	253	222	194
W200x86	305	—	292	279	265	252	238	225	197	168	147
W200x71	249	246	232	219	205	192	179	166	137	116	101
W200x59	203	195	182	169	155	142	128	114	93.2	79.1	68.8
W200x52	177	168	155	142	129	116	101	89.5	73.3	62.1	53.9
† W200x46	139	133	122	112	101	90.2	78.4	69.5	56.6	47.9	41.5
W150x37	96.3	85.4	77.6	69.8	61.5	53.2	46.9	42.0	34.7	29.6	25.8
W150x30	75.8	63.9	56.3	48.1	40.2	34.6	30.4	27.2	22.4	19.1	16.6
‡ W150x22	46.2	38.5	33.2	27.2	22.5	19.2	16.8	14.9	12.2	10.4	9.00

Note: Moment resistances are based on class of section for X-X axis of bending only, ω_2 = 1.0.

† Class 3

‡ Class 4

FACTORED MOMENT RESISTANCES OF COLUMNS, M_{rx} and M'_{rx} (kN·m)
F_y = 450 MPa

W Shapes
ASTM A913 Grade 65

Designation	M_{rx}	M'_{rx} for the following unsupported lengths in millimetres									
		6 000	8 000	10 000	12 000	14 000	16 000	18 000	20 000	24 000	28 000
W360x1299	13 400	—	—	—	—	—	—	—	13 300	12 900	12 500
W360x1202	12 200	—	—	—	—	—	—	12 100	11 900	11 500	11 100
W360x1086	11 000	—	—	—	—	—	—	10 800	10 600	10 200	9 810
W360x990	9 840	—	—	—	—	—	9 750	9 550	9 360	8 960	8 560
W360x900	8 750	—	—	—	—	—	8 570	8 380	8 190	7 810	7 430
W360x818	7 820	—	—	—	—	7 730	7 550	7 360	7 180	6 810	6 440
W360x744	6 970	—	—	—	—	6 800	6 620	6 440	6 270	5 910	5 560
W360x677	6 200	—	—	—	6 140	5 970	5 800	5 630	5 460	5 120	4 780
W360x634	5 750	—	—	—	5 650	5 480	5 310	5 150	4 980	4 650	4 320
W360x592	5 310	—	—	—	5 160	5 000	4 830	4 670	4 510	4 180	3 860
W360x551	4 900	—	—	4 870	4 710	4 540	4 380	4 220	4 060	3 740	3 410
W360x509	4 460	—	—	4 390	4 230	4 070	3 910	3 750	3 600	3 290	2 980
W360x463	4 000	—	—	3 880	3 730	3 570	3 420	3 260	3 110	2 810	2 450
W360x421	3 600	—	—	3 440	3 290	3 140	2 990	2 840	2 690	2 390	2 040
W360x382	3 220	—	3 190	3 030	2 880	2 730	2 590	2 440	2 290	1 960	1 680
W360x347	2 890	—	2 820	2 670	2 530	2 380	2 240	2 090	1 950	1 620	1 380
W360x314	2 580	—	2 480	2 340	2 190	2 050	1 900	1 760	1 590	1 320	1 130
W360x287	2 350	—	2 240	2 090	1 950	1 810	1 670	1 510	1 350	1 120	953
W360x262	2 130	—	2 000	1 850	1 710	1 570	1 430	1 260	1 120	927	790
W360x237	1 900	1 890	1 750	1 610	1 470	1 330	1 170	1 030	916	755	643
W360x216	1 730	1 710	1 570	1 430	1 290	1 150	988	867	773	637	542
W360x196	1 560	1 510	1 370	1 230	1 090	931	800	702	626	516	439
W360x179	1 410	1 360	1 220	1 080	941	783	671	588	524	430	366
† W360x162	1 150	1 110	1 010	893	781	653	558	488	434	356	302
† W360x147	1 040	1 000	896	784	664	548	467	407	361	295	250

Note: Moment resistances are based on class of section for X-X axis of bending only, ω_2 = 1.0.

† Class 3

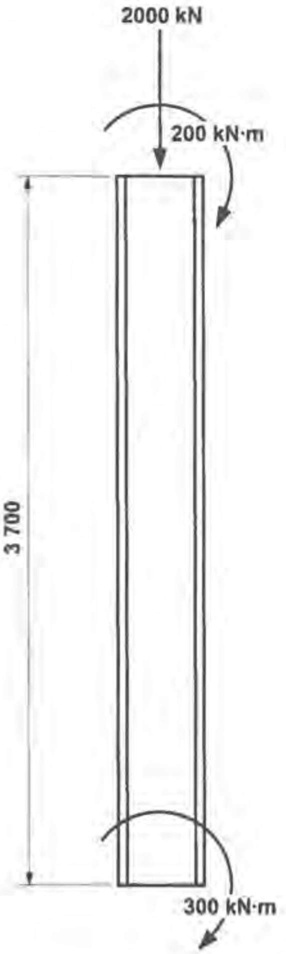
DESIGN OF BEAM-COLUMNS

Examples

1. Given:

Design a steel column in a braced frame for the factored loads shown. The moments cause bending about the X-X axis of the column. The P-Δ effects have been included in the analysis. The steel grade is ASTM A992 (F_y = 345 MPa).

Solution:



L = 3 700 mm M_{f1} = 200 kN·m

C_f = 2 000 kN M_{f2} = 300 kN·m

Try a W310x118 column.

Although Table 5-1, *Class of Sections in Bending*, lists the W310x118 as Class 2, the addition of axial load might change that class (according to Table 2 of S16-14 Clause 11.2). However, an examination of Table 4-7, *Class of Sections – Combined Axial Compression and Major-Axis Bending*, shows that the W310x118 is always a Class 2 section, and Clause 13.8.2 applies:

$$\frac{C_f}{C_r} + \frac{0.85 U_{1x} M_{fx}}{M_{rx}} + \frac{\beta U_{1y} M_{fy}}{M_{ry}} \le 1.0$$

i) Cross-sectional strength

C_f = 2 000 kN

M_{fx} = 300 kN·m

From the tables of *Factored Axial Compressive Resistances* in Part 4,

$C_r = C_{ro}$ = 4 650 kN, and M_{rx} = 605 kN·m

M_{f1} / M_{f2} = 200 / 300 = 0.67 (double curvature)

From Table 4-8, *Values of* ω_1, $\omega_{1x} = 0.40$

$KL_x / r_x = 1.0\,(3\,700 / 136) = 27.2$

From Table 4-5, *Euler Buckling Load*, C_e / A = 2 670 MPa (by interpolation)

$C_e = 2\,670 \times 15\,000$ mm^2 = 40 100 kN, $C_f / C_e = 2\,000 / 40\,100 = 0.0500$

From Table 4-9, *Amplification factor*, U = 1.05

$U_{1x} = \omega_{1x} U = 0.40 \times 1.05 = 0.42 < 1.0$ Therefore, $U_{1x} = 1.0$

$$\frac{2000}{4650} + \frac{0.85 \times 1.0 \times 300}{605} = 0.430 + 0.421 = 0.851 < 1.0$$

ii) Overall member strength

$KL_x/r_x = 27.2$ From Table 4-3, *Unit Factored Compressive Resistances*,

$C_r/A = 297$ MPa, for $F_y = 345$ MPa

$C_r = C_{rx} = 297 \times 15\,000/10^3 = 4\,460$ kN (uniaxial bending about axis X-X)

$U_{1x} = 0.4 \times 1.05 = 0.42$ (for braced frames)

$$\frac{2\,000}{4\,460} + \frac{0.85 \times 0.42 \times 300}{605} = 0.448 + 0.177 = 0.625 < 1.0$$

iii) Lateral-torsional buckling strength

$C_r = C_{ry} = C_{rL} = 3\,840$ kN

(by interpolation, from the tables of *Factored Axial Compressive resistances*)

$L = 3\,700$ mm $< L_u = 4\,920$ mm, $M_{rx} = 605$ kN·m

$U_{1x} = 1.0$

$$\frac{2\,000}{3\,840} + \frac{0.85 \times 1.0 \times 300}{605} = 0.521 + 0.421 = 0.942 < 1.0$$

The W310x118 column section is adequate.

Comments:

1. C_r could be more accurately determined by computing the KL/r values and entering the tables of *Unit Factored Compressive Resistances* (Table 4-3) for the larger KL/r and multiplying that value by the area of the column.
2. When $L > L_u$ the tables of *Factored Moment Resistances of Columns* on the preceding pages will be more useful. (Caution: if a column section changes from Class 2 to Class 3 on account of high axial loads, M_{rx} or M'_{rx} values need to be adjusted.)

2. Given:

Same as Example 1, except that the column is part of an unbraced (sway) frame. Additional moments of 50 kN·m at each end of the column cause bending about the Y-Y axis, such that double curvature is induced in the column.

Solution:

$L = 3\,700$ mm $M_{fx1} = 200$ kN·m $M_{fy1} = 50$ kN·m

$C_f = 2\,000$ kN $M_{fx2} = 300$ kN·m $M_{fy2} = 50$ kN·m

Try a W310x129 column.

This section is a heavier one of the same series as the W310x118 used in Example 1, but is a Class 1 section. S16-14 Clause 13.8.2 applies.

i) Cross-sectional strength

For members in unbraced frames, the cross-sectional strength check does not govern.

ii) Overall member strength

$C_f = 2\,000$ kN, $M_{fx} = 300$ kN·m, $M_{fy} = 50$ kN·m

$KL_x/r_x = 1.0\ (3\,700/137) = 27.0$, $KL_y/r_y = 1.0\ (3\,700/78.0) = 47.4$

From Table 4-3, *Unit Factored Compressive Resistances*,

$C_r/A = 257$ MPa (for $KL/r = 47.4$, by interpolation)

$C_r = 257 \times 16\,500/10^3 = 4\,240$ kN

From the tables of *Factored Axial Compressive Resistances* in Part 4,

$M_{rx} = 671$ kN·m and $M_{ry} = 308$ kN·m

$U_{1x} = 1.0$ and $U_{1y} = 1.0$ (unbraced frame)

$$\lambda_y = \frac{K\,L_y}{r_y}\sqrt{\frac{F_y}{\pi^2 E}} = 47.4\sqrt{\frac{345}{\pi^2 200000}} = 0.627$$

$\beta = 0.6 + 0.4\,\lambda_y = 0.6 + 0.4 \times 0.627 = 0.851 > 0.85$, Therefore, $\beta = 0.85$

$$\frac{2000}{4240} + \frac{0.85 \times 1.0 \times 300}{671} + \frac{0.85 \times 1.0 \times 50}{308} = 0.472 + 0.380 + 0.138 = 0.990 < 1.0$$

iii) Lateral-torsional buckling

$C_r = C_{ry} = C_{rL} = 4\,240$ kN (previously calculated)

$L = 3\,700$ mm $< L_u = 5\,080$ mm, $M_{rx} = 671$ kN·m

$U_{1x} = 1.0$ and $U_{1y} = 1.0$

$\beta = 0.85$ (previously calculated)

$$\frac{2000}{4240} + \frac{0.85 \times 1.0 \times 300}{671} + \frac{0.85 \times 1.0 \times 50}{308} = 0.472 + 0.380 + 0.138 = 0.990 < 1.0$$

The W310x129 column section is adequate.

iv) Biaxial bending interaction

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} = \frac{300}{671} + \frac{50}{308} = 0.447 + 0.162 = 0.609 < 1.0$$

Shear

Where beams with large end moments are connected to columns with thin webs, a check for shear capacity in the column web will be necessary.

Note

For further design examples of beam-columns, see "Limit States Design in Structural Steel", G.L. Kulak and G.Y. Grondin, CISC.

NOTES

FACTORED AXIAL COMPRESSIVE RESISTANCES

Angle Struts

Single-Angle Struts

The tables of factored axial compressive resistances for single-angle struts on the following pages are based on the provisions of CSA S16-14 Clause 13.3.3 for angles connected through one leg and meeting the requirements of Clause 13.3.3.1. For unequal-leg angles, tables are provided for both the long-leg connected and short-leg connected cases. Members that do not satisfy these conditions must be designed for combined compression and bending by taking into account the effects of eccentricity in accordance with Clauses 13.3.2 and 13.3.3.4 or by using a more rigorous procedure.

The design tables are intended for angles of CSA G40.21-350W grade steel that are individual members or members of planar trusses, and members of box or space trusses. The tables are to be used with hot-rolled angles and are not intended for diagonal members of braced frames.

For Class 4 angles, the resistances have been calculated in accordance with Clause 13.3.5 on the basis of effective area.

The value r_x appearing at the bottom of the tables is the radius of gyration about the axis parallel to the connected leg, while r_y is the radius of gyration about the perpendicular axis. r'_y is the radius of gyration about the minor principal axis. Consult the solved design example at the end of this section for more information.

Double-Angle Struts

The tables of factored axial compressive resistances for double-angle struts are based on the requirements of Clause 13.3.1, CSA S16-14 with n = 1.34 for axis X-X and Clause 13.3.2 for axis Y-Y. For Class 4 angles, the resistances are computed based on the requirements of Clause 13.3.5 using the effective area method.

Factored axial compressive resistances with respect to various effective lengths relative to both the X-X and Y-Y axes, and the U-U and V-V axes for starred angles, are listed for angles made from CSA G40.21-350W. The yield stress F_y for G40.21 350W steel angles is 350 MPa for all thicknesses listed.

The resistances listed in the tables for axis Y-Y are based on closely spaced interconnectors. The resistances for struts that lack the closely spaced interconnectors should be determined by taking into account the additional slenderness of the component angles between interconnectors. The actual number of interconnectors and method of interconnection should therefore be taken into account in accordance with Clause 19.2.4, S16-14. Consult the design example in Part 4. For starred angles, these requirements may be waived, provided that interconnectors are spaced no further than at the one-third points, in accordance with Clause 19.2.5.

The factored axial compressive resistances pertaining to effective lengths based on the Y-Y axis have been computed for angles spaced 10 mm back-to-back. Consult the design example to obtain factored compressive resistances for different spacings.

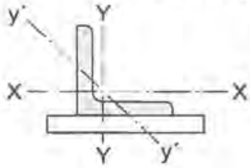
The value r_z appearing with the properties of double-angle struts is the minimum radius of gyration of a single angle about its minor principal axis. Values for r_x and r_y are those for a double-angle strut. See Part 6 for a more comprehensive list of angle properties.

SINGLE-ANGLE STRUTS

Equal-Leg Angles

in Compliance with S16-14 Clause 13.3.3.1

Factored Compressive Resistances, C_r (kN)



CSA G40.21-350W

Designation (mm x mm x mm)		L 152 x 152				L 127 x 127				
		19	16	13	‡ 9.5	19	16	13	9.5	‡ 7.9
Mass (kg/m)		42.7	36.0	29.2	22.2	35.1	29.8	24.1	18.3	15.3
INDIVIDUAL MEMBERS AND PLANAR TRUSSES										
Length of member (L) in millimetres	0	1 060	896	724	457	874	741	600	456	317
	500	951	805	651	412	766	651	527	401	280
	1 000	851	721	584	370	670	570	462	352	246
	1 500	761	646	523	332	586	499	405	309	216
	2 000	682	579	469	298	513	438	356	272	190
	2 500	612	520	422	268	451	385	314	240	168
	3 000	550	468	380	241	399	340	278	213	149
	3 500	496	422	343	218	330	284	233	180	126
	4 000	432	369	301	193	275	236	194	150	105
	4 500	369	316	258	165	231	199	164	126	88.9
	5 000	318	272	222	143	197	170	140	108	75.9
	5 500	276	236	193	124					
	6 000	242	207	170	109					
BOX AND SPACE TRUSSES										
Length of member (L) in millimetres	0	1 230	1 040	841	531	1 020	861	697	530	369
	500	1 110	937	757	479	891	757	614	467	326
	1 000	988	837	677	429	776	660	535	408	285
	1 500	878	745	604	383	672	573	465	355	248
	2 000	780	662	537	341	583	497	405	309	216
	2 500	693	589	478	304	506	432	352	270	189
	3 000	617	525	426	271	438	375	307	236	165
	3 500	550	469	381	243	372	319	261	201	141
	4 000	479	409	334	213	318	273	224	172	121
	4 500	420	359	293	187	275	236	194	149	105
	5 000	371	317	259	165	239	206	169	130	91.5
	5 500	329	281	230	147	210	181	148	114	80.4
	6 000	293	251	205	131			131	101	71.2
	6 500	263	225	184	118					
	7 000	237	203	166	106					
PROPERTIES OF SINGLE ANGLES										
Area (mm^2)		5 450	4 590	3 710	2 810	4 480	3 780	3 070	2 330	1 960
$r_x = r_y$ (mm)		46.3	46.7	47.1	47.6	38.3	38.7	39.1	39.5	39.8
r'_y (mm)		29.7	29.8	30.0	30.2	24.8	24.8	25.0	25.1	25.2
IMPERIAL SIZE AND WEIGHT										
Weight (lb/ft)		28.7	24.2	19.6	14.9	23.6	20.0	16.2	12.3	10.3
Thickness (in)		3/4	5/8	1/2	3/8	3/4	5/8	1/2	3/8	5/16
Size (in)		6 x 6				5 x 5				

This table is not intended for diagonal braces in braced frames.

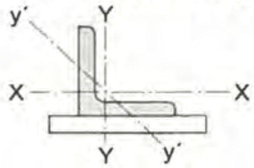
‡ Class 4

SINGLE-ANGLE STRUTS

Equal-Leg Angles

in Compliance with S16-14 Clause 13.3.3.1

Factored Compressive Resistances, C_r (kN)



CSA G40.21-350W

INDIVIDUAL MEMBERS AND PLANAR TRUSSES											
Designation (mm x mm x mm)		**L 102 x 102**						**L 89 x 89**			
		19	13	11	9.5	7.9	‡ 6.4	13	9.5	7.9	‡ 6.4
Mass (kg/m)		27.5	19.0	16.8	14.6	12.2	9.8	16.5	12.6	10.7	8.6
Length of member (L) in millimetres	0	688	475	419	363	305	203	410	314	264	203
	500	582	404	357	309	260	173	340	261	220	169
	1 000	491	342	302	262	221	147	281	216	182	140
	1 500	415	291	257	223	188	126	233	179	151	117
	2 000	353	248	219	191	161	108	194	150	127	98.1
	2 500	297	212	188	164	138	92.5	152	118	101	78.1
	3 000	233	167	149	130	110	73.9	117	91.7	77.9	60.6
	3 500	187	134	119	104	88.4	59.4	93.0	72.7	61.9	48.1
	4 000	152	110	97.6	85.3	72.4	48.7				
	4 500										
	5 000										
	5 500										
	6 000										
BOX AND SPACE TRUSSES											
Length of member (L) in millimetres	0	799	552	487	421	354	236	477	365	307	236
	500	677	470	415	359	302	202	395	303	255	197
	1 000	566	395	349	303	255	170	323	248	209	162
	1 500	472	331	293	254	214	143	263	203	172	133
	2 000	395	278	246	214	181	121	216	167	141	109
	2 500	327	232	206	179	152	102	171	133	113	87.6
	3 000	267	190	169	147	125	83.6	138	107	91.2	70.8
	3 500	221	158	140	122	104	69.6	113	88.1	74.8	58.1
	4 000	186	133	118	103	87.3	58.7	93.8	73.3	62.3	48.4
	4 500	158	113	100	87.7	74.4	50.0				
	5 000										
	5 500										
	6 000										
	6 500										
	7 000										
PROPERTIES OF SINGLE ANGLES											
Area (mm^2)		3 510	2 420	2 140	1 850	1 550	1 250	2 100	1 600	1 350	1 090
$r_x = r_y$ (mm)		30.3	31.1	31.3	31.5	31.7	31.9	26.9	27.3	27.5	27.7
r'_y (mm)		19.8	19.9	20.0	20.1	20.2	20.3	17.3	17.4	17.5	17.6
IMPERIAL SIZE AND WEIGHT											
Weight (lb/ft)		18.5	12.8	11.3	9.8	8.2	6.6	11.1	8.5	7.2	5.8
Thickness (in)		3/4	1/2	7/16	3/8	5/16	1/4	1/2	3/8	5/16	1/4
Size (in)		6 x 6						3 1/2 x 3 1/2			

This table is not intended for diagonal braces in braced frames.

‡ Class 4

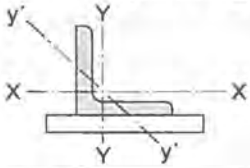
SINGLE-ANGLE STRUTS

Equal-Leg Angles

in Compliance with S16-14 Clause 13.3.3.1

Factored Compressive Resistances, C_r (kN)

CSA G40.21-350W



INDIVIDUAL MEMBERS AND PLANAR TRUSSES											
Designation (mm x mm x mm)		L 76 x 76					L 64 x 64				
		13	9.5	7.9	6.4	‡ 4.8	13	9.5	7.9	6.4	4.8
Mass (kg/m)		14.0	10.7	9.1	7.3	5.5	11.4	8.7	7.4	6.1	4.6
Length of member (L) in millimetres	0	347	266	224	181	114	284	219	185	150	114
	500	278	214	181	146	92.3	217	168	142	115	88.1
	1 000	222	172	145	118	74.4	165	129	109	88.9	68.0
	1 500	178	138	117	95.3	60.4	128	99.7	84.9	69.3	53.2
	2 000	139	109	92.7	75.8	48.4	87.0	68.6	58.8	48.4	37.5
	2 500	102	80.1	68.3	55.9	35.8	62.2	49.2	42.2	34.8	27.0
	3 000	77.3	61.0	52.0	42.6	27.3					
	3 500										
	4 000										
	4 500										
	5 000										
	5 500										
	6 000										
BOX AND SPACE TRUSSES											
Length of member (L) in millimetres	0	403	310	261	211	133	330	255	215	174	132
	500	322	248	210	170	107	251	194	165	134	102
	1 000	254	196	166	135	85.2	187	146	124	101	77.3
	1 500	200	155	132	107	67.9	140	109	93.4	76.5	59.0
	2 000	155	121	103	83.8	53.4	101	79.6	68.0	55.8	43.2
	2 500	119	93.4	79.6	65.0	41.5	75.9	59.8	51.2	42.1	32.6
	3 000	94.1	73.9	63.0	51.5	32.9				32.6	25.3
	3 500	75.8	59.7	50.9	41.6	26.7					
	4 000										
	4 500										
	5 000										
	5 500										
	6 000										
	6 500										
	7 000										
PROPERTIES OF SINGLE ANGLES											
Area (mm^2)		1 770	1 360	1 150	929	703	1 450	1 120	942	768	581
$r_x = r_y$ (mm)		22.8	23.2	23.4	23.6	23.9	18.8	19.1	19.3	19.5	19.8
r'_y (mm)		14.8	14.9	15.0	15.0	15.1	12.4	12.4	12.4	12.5	12.6
IMPERIAL SIZE AND WEIGHT											
Weight (lb/ft)		9.4	7.2	6.1	4.9	3.7	7.7	5.9	5.0	4.1	3.1
Thickness (in)		1/2	3/8	5/16	1/4	3/16	1/2	3/8	5/16	1/4	3/16
Size (in)		3 x 3					2 1/2 x 2 1/2				

This table is not intended for diagonal braces in braced frames.

‡ Class 4

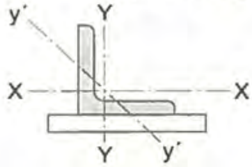
SINGLE-ANGLE STRUTS

Equal-Leg Angles

in Compliance with S16-14 Clause 13.3.3.1

Factored Compressive Resistances, C_r (kN)

CSA G40.21-350W



INDIVIDUAL MEMBERS AND PLANAR TRUSSES

Designation (mm x mm x mm)	L 51 x 51					L 44 x 44			L 38 x 38		
	9.5	7.9	6.4	4.8	‡ 3.2	6.4	4.8	‡ 3.2	6.4	4.8	3.2
Mass (kg/m)	7.0	5.8	4.7	3.6	2.4	4.1	3.1	2.1	3.4	2.7	1.8
Length of member (L) in millimetres											
0	172	145	118	90.2	50.9	103	78.4	50.9	86.8	66.5	45.4
500	122	104	85.2	65.2	36.9	70.2	54.1	35.3	55.5	42.9	29.5
1 000	87.9	75.1	61.6	47.3	26.9	48.5	37.6	24.7	34.7	27.2	19.0
1 500	57.9	50.0	41.4	32.1	18.4	29.2	23.0	15.3	19.3	15.2	10.7
2 000	37.8	32.7	27.2	21.1	12.2						
2 500											
3 000											
3 500											
4 000											
4 500											
5 000											
5 500											
6 000											

BOX AND SPACE TRUSSES

Length of member (L) in millimetres	L 51 x 51 9.5	L 51 x 51 7.9	L 51 x 51 6.4	L 51 x 51 4.8	L 51 x 51 ‡ 3.2	L 44 x 44 6.4	L 44 x 44 4.8	L 44 x 44 ‡ 3.2	L 38 x 38 6.4	L 38 x 38 4.8	L 38 x 38 3.2
0	199	169	138	105	59.1	119	91.2	59.1	101	77.3	52.8
500	141	120	98.2	75.2	42.6	80.6	62.2	40.6	63.4	49.0	33.7
1 000	98.4	84.2	69.1	53.1	30.2	53.9	41.9	27.5	38.6	30.2	21.0
1 500	66.4	57.1	47.2	36.5	20.9	34.4	27.0	17.8	23.5	18.5	12.9
2 000	46.1	39.8	32.9	25.5	14.7	23.4	18.4	12.2			
2 500											
3 000											
3 500											
4 000											
4 500											
5 000											
5 500											
6 000											
6 500											
7 000											

PROPERTIES OF SINGLE ANGLES

Area (mm^2)	877	742	605	461	312	525	401	272	444	340	232
$r_x = r_y$ (mm)	15.1	15.3	15.5	15.7	15.9	13.4	13.7	13.9	11.4	11.6	11.8
r'_y (mm)	9.89	9.90	9.93	10.0	10.1	8.68	8.73	8.82	7.42	7.45	7.52

IMPERIAL SIZE AND WEIGHT

Weight (lb/ft)	4.7	3.9	3.2	2.4	1.7	2.8	2.1	1.4	2.3	1.8	1.2
Thickness (in)	3/8	5/16	1/4	3/16	1/8	1/4	3/16	1/8	1/4	3/16	1/8
Size (in)	3 x 3					1 3/4 x 1 3/4			1 1/2 x 1 1/2		

This table is not intended for diagonal braces in braced frames.

‡ Class 4

SINGLE-ANGLE STRUTS

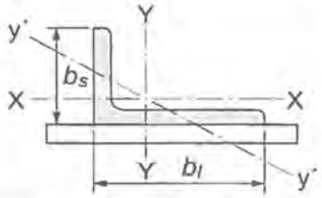
Unequal-Leg Angles

Long Leg Connected

In Compliance with S16-14 Clause 13.3.3.1

Factored Compressive Resistances, C_r (kN)

CSA G40.21-350W



Designation (mm x mm x mm)	L 152 x 102					L 127 x 89			L 127 x 76			
	19	16	13	‡ 9.5	‡ 7.9	9.5	‡ 7.9	‡ 6.4	13	9.5	‡ 7.9	‡ 6.4
Mass (kg/m)	35.0	29.6	24.0	18.2	15.3	15.4	12.9	10.4	19.0	14.5	12.1	9.8
INDIVIDUAL MEMBERS AND PLANAR TRUSSES												
Length of member (L) in mm: 0	874	741	600	410	311	385	291	203	473	361	271	192
500	732	622	505	346	263	317	240	168	372	285	214	152
1 000	611	520	423	291	221	260	197	138	292	225	169	121
1 500	512	437	356	245	187	214	162	114	231	179	135	96.3
2 000	431	368	301	208	158	178	135	94.8	170	133	101	72.8
2 500	350	301	248	173	133	136	104	73.2	124	97.1	73.8	53.1
3 000	273	235	194	136	104	105	79.9	56.5				
3 500	218	187	155	109	83.2		63.2	44.7				
4 000				88.5	67.8							
4 500												
BOX AND SPACE TRUSSES												
Length of member (L) in mm: 0	1 020	861	697	476	361	447	338	236	550	420	315	223
500	851	723	587	402	306	368	278	195	431	331	249	177
1 000	704	599	488	335	255	298	226	158	333	256	193	138
1 500	581	496	404	279	212	242	183	129	258	199	151	108
2 000	482	412	337	233	177	196	149	105	193	151	114	81.8
2 500	390	335	275	191	146	155	118	83.0	147	115	87.4	62.7
3 000	317	272	224	156	119	124	94.5	66.6	115	90.3	68.6	49.3
3 500	261	224	185	129	98.7	101	77.2	54.4				
4 000	218	187	155	108	82.8	83.8	64.0	45.2				
4 500			131	91.7	70.2							
5 000												
PROPERTIES OF SINGLE ANGLES												
Area (mm^2)	4 480	3 780	3 060	2 330	1 950	1 970	1 650	1 330	2 420	1 850	1 550	1 250
r_x (mm)	28.6	28.9	29.3	29.8	30.0	26.0	26.2	26.4	21.1	21.5	21.7	21.9
r_y (mm)	47.6	48.0	48.5	48.9	49.2	40.6	40.8	41.0	40.3	40.8	41.0	41.2
r'_y (mm)	21.9	22.0	22.2	22.4	22.5	19.3	19.4	19.6	16.5	16.6	16.7	16.8
IMPERIAL SIZE AND WEIGHT												
Weight (lb/ft)	23.6	20.0	16.2	12.3	10.3	10.4	8.7	7.0	12.8	9.8	8.2	6.6
Thickness (in)	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$
Size (in)	6 x 4					5 x 3 $\frac{1}{2}$			5 x 3			

This table is not intended for diagonal braces in braced frames.

‡ Class 4

SINGLE-ANGLE STRUTS

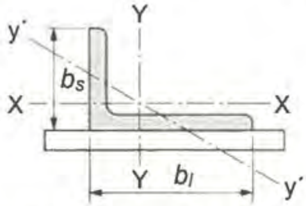
Unequal-Leg Angles

Long Leg Connected

In Compliance with S16-14 Clause 13.3.3.1

Factored Compressive Resistances, C_r (kN)

CSA G40.21-350W



Designation (mm x mm x mm)	L 102 x 89			L 102 x 76				L 89 x 76			
	9.5	7.9	‡ 6.4	13	9.5	7.9	‡ 6.4	13	9.5	7.9	‡ 6.4
Mass (kg/m)	13.5	11.4	9.2	16.4	12.6	10.7	8.6	15.1	11.7	9.8	8.0
INDIVIDUAL MEMBERS AND PLANAR TRUSSES											
Length of member (L) in mm: 0	338	284	203	411	314	264	192	379	290	244	192
500	280	236	169	326	251	211	154	302	232	196	154
1 000	231	195	140	258	199	168	123	240	185	157	124
1 500	191	162	116	206	159	135	98.5	192	149	126	99.9
2 000	160	135	97.1	156	122	104	76.3	148	116	98.6	78.5
2 500	124	106	76.6	114	89.3	76.1	56.0	108	85.0	72.5	57.8
3 000	96.2	82.2	59.3			57.7	42.5	82.0	64.6	55.1	44.0
3 500	76.2	65.2	47.1								
4 000											
4 500											
BOX AND SPACE TRUSSES											
Length of member (L) in mm: 0	393	330	236	478	365	307	223	440	337	284	223
500	325	274	196	378	291	245	178	350	269	227	179
1 000	266	224	161	295	227	192	140	274	212	179	142
1 500	216	183	131	230	178	151	110	215	167	142	112
2 000	178	151	108	175	137	116	85.1	165	129	110	87.2
2 500	141	120	86.2	134	105	89.3	65.6	127	99.6	84.7	67.4
3 000	113	96.4	69.5	105	82.7	70.4	51.8	100	78.6	67.0	53.3
3 500	92.6	79.0	57.0				41.7		63.4	54.0	43.1
4 000	76.9	65.7	47.4								
4 500											
5 000											
PROPERTIES OF SINGLE ANGLES											
Area (mm^2)	1 720	1 450	1 170	2 100	1 600	1 350	1 090	1 940	1 480	1 250	1 010
r_x (mm)	26.8	27.1	27.3	21.9	22.3	22.5	22.7	22.4	22.8	23.0	23.2
r_y (mm)	31.9	32.1	32.3	31.8	32.2	32.4	32.7	27.3	27.7	27.9	28.1
r'_y (mm)	18.5	18.6	18.7	16.2	16.4	16.5	16.6	15.8	15.9	15.9	16.0
IMPERIAL SIZE AND WEIGHT											
Weight (lb/ft)	9.1	7.7	6.2	11.1	8.5	7.2	5.8	10.2	7.9	6.6	5.4
Thickness (in)	3/8	5/16	1/4	1/2	3/8	5/16	1/4	1/2	3/8	5/16	1/4
Size (in)	4 x 3 1/2			4 x 3				3 1/2 x 3			

This table is not intended for diagonal braces in braced frames.

‡ Class 4

SINGLE-ANGLE STRUTS

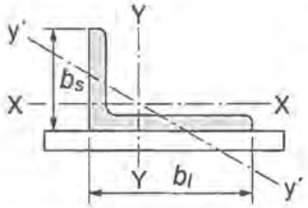
Unequal-Leg Angles

Long Leg Connected

In Compliance with S16-14 Clause 13.3.3.1

Factored Compressive Resistances, C_r (kN)

CSA G40.21-350W



Designation (mm x mm x mm)	L 89 x 64				L 76 x 64		L 76 x 51				
	13	9.5	7.9	‡ 6.4	9.5	7.9	13	9.5	7.9	6.4	‡ 4.8
Mass (kg/m)	13.9	10.7	9.0	7.3	9.8	8.3	11.5	8.8	7.4	6.1	4.6
INDIVIDUAL MEMBERS AND PLANAR TRUSSES											
Length of member (L) in mm											
0	347	266	224	176	243	205	284	219	185	150	102
500	261	202	170	134	185	156	197	153	130	106	72.3
1 000	197	153	129	102	141	119	138	108	91.8	75.1	51.6
1 500	147	116	99.1	79.0	109	92.5	85.2	67.8	58.4	48.2	33.5
2 000	99.0	78.5	67.2	53.6	73.8	63.2					
2 500				38.3	52.7	45.2					
3 000											
3 500											
4 000											
4 500											
BOX AND SPACE TRUSSES											
Length of member (L) in mm											
0	403	310	261	205	282	238	330	255	215	174	119
500	302	233	197	156	214	181	226	176	149	122	83.4
1 000	222	173	147	116	160	135	153	120	103	83.9	57.7
1 500	162	127	109	86.3	119	101	99.4	78.7	67.6	55.7	38.6
2 000	116	91.8	78.4	62.5	85.9	73.4	68.0	54.1	46.5	38.4	26.7
2 500	86.7	68.6	58.7	46.8	64.4	55.1					
3 000											
3 500											
4 000											
4 500											
5 000											
PROPERTIES OF SINGLE ANGLES											
Area (mm^2)	1 770	1 360	1 150	929	1 240	1 050	1 450	1 120	942	768	582
r_x (mm)	17.9	18.3	18.5	18.7	18.7	18.9	13.9	14.2	14.4	14.6	14.8
r_y (mm)	27.6	28.0	28.2	28.4	23.6	23.8	23.5	23.9	24.1	24.3	24.5
r'_y (mm)	13.6	13.6	13.7	13.8	13.3	13.3	10.9	10.9	11.0	11.0	11.1
IMPERIAL SIZE AND WEIGHT											
Weight (lb/ft)	9.4	7.2	6.1	4.9	6.6	5.6	7.7	5.9	5.0	4.1	3.1
Thickness (in)	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$
Size (in)	3 $\frac{1}{2}$ x 2 $\frac{1}{2}$				3 x 2 $\frac{1}{2}$		3 x 2				

This table is not intended for diagonal braces in braced frames.

‡ Class 4

SINGLE-ANGLE STRUTS

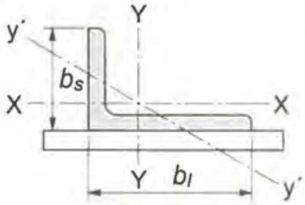
Unequal-Leg Angles

In Compliance with
S16-14 Clause 13.3.3.1

Long Leg Connected

Factored Compressive
Resistances, C_r (kN)

CSA G40.21-350W



INDIVIDUAL MEMBERS AND PLANAR TRUSSES

Designation (mm x mm x mm)		L 64 x 51				L 51 x 38		
		9.5	7.9	6.4	4.8	6.4	4.8	‡ 3.2
Mass (kg/m)		7.9	6.7	5.4	4.2	4.2	3.1	2.1
Length of member (L) in mm	0	195	165	134	102	103	78.4	48.2
	500	138	117	95.5	72.9	64.5	49.7	30.8
	1 000	97.8	83.5	68.4	52.4	39.1	30.6	19.2
	1 500	62.9	54.2	44.8	34.7		17.0	10.7
	2 000			29.3	22.7			
	2 500							
	3 000							
	3 500							
	4 000							
	4 500							

BOX AND SPACE TRUSSES

Length of member (L) in mm	0	227	192	156	119	119	91.1	56.0
	500	159	135	110	84.1	73.6	56.8	35.2
	1 000	109	93.4	76.6	58.8	43.9	34.2	21.4
	1 500	72.6	62.4	51.5	39.8	26.5	20.7	13.1
	2 000	50.1	43.2	35.7	27.6			
	2 500							
	3 000							
	3 500							
	4 000							
	4 500							
	5 000							

PROPERTIES OF SINGLE ANGLES

Area (mm^2)	1 000	845	684	522	525	401	272
r_x (mm)	14.6	14.8	15.0	15.2	11.0	11.2	11.4
r_y (mm)	19.5	19.7	19.9	20.1	15.8	16.0	16.3
r'_y (mm)	10.7	10.7	10.8	10.9	8.12	8.18	8.27

IMPERIAL SIZE AND WEIGHT

Weight (lb/ft)	5.3	4.5	3.6	2.8	2.8	2.1	1.4
Thickness (in)	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$
Size (in)	$2\frac{1}{2} \times 2$				$2 \times 1\frac{1}{2}$		

This table is not intended for diagonal braces in braced frames.

‡ Class 4

SINGLE-ANGLE STRUTS

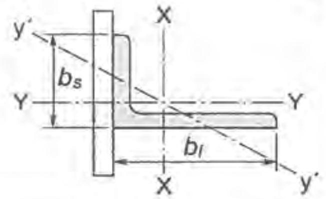
Unequal-Leg Angles

Short Leg Connected

In Compliance with
S16-14 Clause 13.3.3.1

Factored Compressive Resistances, C_r (kN)

CSA G40.21-350W



Designation (mm x mm x mm)	L 152 x 102					L 127 x 89			L 127 x 76			
	19	16	13	‡ 9.5	‡ 7.9	9.5	‡ 7.9	‡ 6.4	13	9.5	‡ 7.9	‡ 6.4
Mass (kg/m)	35.0	29.6	24.0	18.2	15.3	15.4	12.9	10.4	19.0	14.5	12.1	9.8
INDIVIDUAL MEMBERS AND PLANAR TRUSSES												
Length of member (L) in mm												
0	819	694	562	384	291	364	275	192	430	328	246	175
500	736	624	506	346	263	321	243	170	379	290	217	154
1 000	660	561	455	312	237	283	214	150	334	255	192	136
1 500	593	504	409	280	213	249	189	132	294	225	169	120
2 000	533	453	368	253	192	220	167	117	260	199	150	107
2 500	480	408	332	228	174	193	147	104	184	142	107	77.0
3 000	402	343	282	195	149	143	109	77.5	134	103	78.1	56.0
3 500	310	265	218	151	116	109	83.1	59.1			58.9	42.2
4 000	244	209	172	119	91.3	85.4	65.1	46.3				
4 500	197	168	139	96.3	73.7							
BOX AND SPACE TRUSSES												
Length of member (L) in mm												
0	928	787	637	435	330	414	313	219	482	367	276	196
500	831	706	572	391	297	364	275	192	422	322	242	172
1 000	742	630	511	350	266	319	241	169	368	282	212	150
1 500	661	562	457	313	238	278	211	147	321	246	185	132
2 000	589	501	408	280	213	243	184	129	281	216	162	115
2 500	526	448	365	250	191	213	162	113	234	181	137	97.9
3 000	471	401	327	225	171	183	139	98.9	173	133	101	72.5
3 500	397	339	279	193	148	141	108	76.6	132	102	77.1	55.3
4 000	317	271	223	155	118	112	85.2	60.6	103	79.6	60.4	43.3
4 500	258	220	181	126	96.3	90.1	68.7	48.9				
5 000	213	182	150	104	79.6							
PROPERTIES OF SINGLE ANGLES												
Area (mm^2)	4 480	3 780	3 060	2 330	1 950	1 970	1 650	1 330	2 420	1 850	1 550	1 250
r_x (mm)	47.6	48.0	48.5	48.9	49.2	40.6	40.8	41.0	40.3	40.8	41.0	41.2
r_y (mm)	28.6	28.9	29.3	29.8	30.0	26.0	26.2	26.4	21.1	21.5	21.7	21.9
r'_y (mm)	21.9	22.0	22.2	22.4	22.5	19.3	19.4	19.6	16.5	16.6	16.7	16.8
IMPERIAL SIZE AND WEIGHT												
Weight (lb/ft)	23.6	20.0	16.2	12.3	10.3	10.4	8.7	7.0	12.8	9.8	8.2	6.6
Thickness (in)	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$
Size (in)	6 x 4					5 x 3 $\frac{1}{2}$			5 x 3			

This table is not intended for diagonal braces in braced frames.

‡ Class 4

SINGLE-ANGLE STRUTS

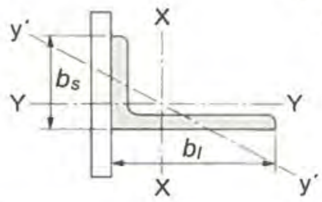
Unequal-Leg Angles

Short Leg Connected

In Compliance with S16-14 Clause 13.3.3.1

Factored Compressive Resistances, C_r (kN)

CSA G40.21-350W



Designation (mm x mm x mm)	L 102 x 89			L 102 x 76				L 89 x 76			
	9.5	7.9	‡ 6.4	13	9.5	7.9	‡ 6.4	13	9.5	7.9	‡ 6.4
Mass (kg/m)	13.5	11.4	9.2	16.4	12.6	10.7	8.6	15.1	11.7	9.8	8.0
INDIVIDUAL MEMBERS AND PLANAR TRUSSES											
Length of member (L) in mm: 0	333	280	200	394	302	254	184	372	285	240	189
500	284	239	171	336	257	217	158	308	237	200	157
1 000	241	203	145	286	219	185	135	255	197	166	131
1 500	206	173	124	244	187	158	115	212	164	139	109
2 000	176	149	107	209	161	136	99.2	178	138	116	92.1
2 500	152	128	91.9	155	121	103	75.5	137	106	89.0	70.9
3 000	117	99.2	71.5	112	87.7	74.6	54.8	98.7	76.5	64.4	51.3
3 500	88.7	75.3	54.3								
4 000											
4 500											
BOX AND SPACE TRUSSES											
Length of member (L) in mm: 0	384	323	231	451	345	290	211	429	328	276	218
500	327	275	197	383	294	247	180	355	273	230	181
1 000	276	233	167	323	248	209	152	291	224	189	149
1 500	233	196	140	272	209	176	129	238	184	155	123
2 000	196	166	119	229	177	149	109	196	152	128	102
2 500	165	140	100	193	150	127	92.8	157	122	103	82.0
3 000	136	115	82.9	146	114	96.8	71.1	127	98.7	83.8	66.5
3 500	114	96.2	69.2	111	86.6	73.7	54.1	97.5	75.5	63.6	50.6
4 000	91.0	77.2	55.7		67.8	57.6	42.4				
4 500	73.2	62.2	44.8								
5 000											
PROPERTIES OF SINGLE ANGLES											
Area (mm^2)	1 720	1 450	1 170	2 100	1 600	1 350	1 090	1 940	1 480	1 250	1 010
r_x (mm)	31.9	32.1	32.3	31.8	32.2	32.4	32.7	27.3	27.7	27.9	28.1
r_y (mm)	26.8	27.1	27.3	21.9	22.3	22.5	22.7	22.4	22.8	23.0	23.2
r'_y (mm)	18.5	18.6	18.7	16.2	16.4	16.5	16.6	15.8	15.9	15.9	16.0
IMPERIAL SIZE AND WEIGHT											
Weight (lb/ft)	9.1	7.7	6.2	11.1	8.5	7.2	5.8	10.2	7.9	6.6	5.4
Thickness (in)	$^3/_8$	$^5/_{16}$	$^1/_4$	$^1/_2$	$^3/_8$	$^5/_{16}$	$^1/_4$	$^1/_2$	$^3/_8$	$^5/_{16}$	$^1/_4$
Size (in)	4 x 3 $^1/_2$			4 x 3				3 $^1/_2$ x 3			

This table is not intended for diagonal braces in braced frames.

‡ Class 4

SINGLE-ANGLE STRUTS

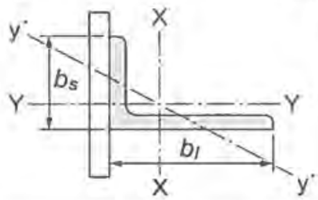
Unequal-Leg Angles

In Compliance with S16-14 Clause 13.3.3.1

Short Leg Connected

Factored Compressive Resistances, C_r (kN)

CSA G40.21-350W



Designation (mm x mm x mm)	L 89 x 64				L 76 x 64		L 76 x 51				
	13	9.5	7.9	‡ 6.4	9.5	7.9	13	9.5	7.9	6.4	‡ 4.8
Mass (kg/m)	13.9	10.7	9.0	7.3	9.8	8.3	11.5	8.8	7.4	6.1	4.6
INDIVIDUAL MEMBERS AND PLANAR TRUSSES											
Length of member (L) in mm: 0	330	253	213	168	237	200	266	205	173	140	95.5
500	274	211	178	140	191	161	214	165	140	114	77.5
1 000	228	176	148	117	154	130	172	134	113	92.0	62.9
1 500	190	147	124	97.9	125	106	130	99.9	85.6	69.4	48.0
2 000	142	109	92.9	73.9	95.5	80.6	78.8	60.8	52.2	42.3	29.3
2 500	96.0	73.7	62.9	50.1	64.4	54.4					
3 000											
3 500											
4 000											
4 500											
BOX AND SPACE TRUSSES											
Length of member (L) in mm: 0	376	289	243	191	273	230	301	232	196	159	108
500	311	239	202	159	219	185	240	186	157	128	87.1
1 000	255	197	166	131	174	147	190	148	125	102	69.6
1 500	209	162	137	108	138	117	151	118	100	81.6	55.9
2 000	173	134	114	89.8	108	92.3	102	78.9	67.7	54.8	37.9
2 500	125	95.6	81.6	65.0	83.8	70.7	68.6	52.9	45.4	36.8	25.5
3 000	90.0	69.1	59.0	47.0	60.4	51.0					
3 500											
4 000											
4 500											
5 000											
PROPERTIES OF SINGLE ANGLES											
Area (mm^2)	1 770	1 360	1 150	929	1 240	1 050	1 450	1 120	942	768	582
r_x (mm)	27.6	28.0	28.2	28.4	23.6	23.8	23.5	23.9	24.1	24.3	24.5
r_y (mm)	17.9	18.3	18.5	18.7	18.7	18.9	13.9	14.2	14.4	14.6	14.8
r'_y (mm)	13.6	13.6	13.7	13.8	13.3	13.3	10.9	10.9	11.0	11.0	11.1
IMPERIAL SIZE AND WEIGHT											
Weight (lb/ft)	9.4	7.2	6.1	4.9	6.6	5.6	7.7	5.9	5.0	4.1	3.1
Thickness (in)	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$
Size (in)	$3\frac{1}{2} \times 2\frac{1}{2}$				$3 \times 2\frac{1}{2}$		3×2				

This table is not intended for diagonal braces in braced frames.

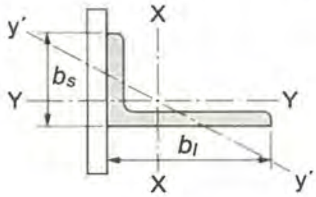
‡ Class 4

SINGLE-ANGLE STRUTS

In Compliance with S16-14 Clause 13.3.3.1

Factored Compressive Resistances, C_r (kN)

CSA G40.21-350W



Unequal-Leg Angles

Short Leg Connected

INDIVIDUAL MEMBERS AND PLANAR TRUSSES								
Designation (mm x mm x mm)		L 64 x 51				L 51 x 38		
		9.5	7.9	6.4	4.8	6.4	4.8	‡ 3.2
Mass (kg/m)		7.9	6.7	5.4	4.2	4.2	3.1	2.1
Length of member (L) in mm	0	190	160	130	99.0	98.4	75.2	46.2
	500	146	124	101	76.8	71.2	54.6	33.8
	1 000	112	95.6	78.0	59.6	51.9	39.9	24.8
	1 500	86.5	73.1	60.3	46.5	28.1	21.8	13.6
	2 000	52.4	44.3	36.6	28.3			
	2 500							
	3 000							
	3 500							
	4 000							
	4 500							

BOX AND SPACE TRUSSES								
Length of member (L) in mm	0	218	184	150	114	113	86.1	52.9
	500	166	141	115	87.7	80.4	61.7	38.2
	1 000	126	107	87.3	66.7	57.0	43.9	27.3
	1 500	95.4	81.5	66.8	51.3	36.5	28.2	17.7
	2 000	68.1	57.6	47.5	36.7			10.5
	2 500	45.6	38.6	31.8	24.6			
	3 000							
	3 500							
	4 000							
	4 500							
	5 000							

PROPERTIES OF SINGLE ANGLES							
Area (mm^2)	1 000	845	684	522	525	401	272
r_x (mm)	19.5	19.7	19.9	20.1	15.8	16.0	16.3
r_y (mm)	14.6	14.8	15.0	15.2	11.0	11.2	11.4
r'_y (mm)	10.7	10.7	10.8	10.9	8.12	8.18	8.27

IMPERIAL SIZE AND WEIGHT							
Weight (lb/ft)	5.3	4.5	3.6	2.8	2.8	2.1	1.4
Thickness (in)	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$
Size (in)	$2\frac{1}{2} \times 2$				$2 \times 1\frac{1}{2}$		

This table is not intended for diagonal braces in braced frames.

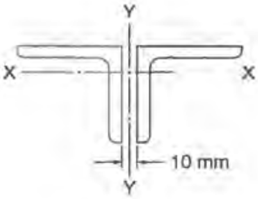
‡ Class 4

DOUBLE ANGLE STRUTS

Equal-Leg Angles

Factored Axial Compressive Resistances (kN)

Legs 10 mm Back-to-Back



CSA G40.21

350W

ϕ = 0.90

Designation (mm x mm x mm)		L 152 x 152				L 127 x 127				
		19	16	13	‡ 9.5	19	16	13	9.5	‡ 7.9
Mass (kg/m)		85.4	72.0	58.4	44.4	70.2	59.6	48.2	36.6	30.6
Effective length (KL) in millimetres with respect to indicated axis — X-X Axis	0	3 410	2 890	2 330	1 470	2 810	2 390	1 930	1 470	1 020
	1 000	3 320	2 810	2 270	1 440	2 700	2 290	1 850	1 410	983
	2 000	2 930	2 480	2 010	1 280	2 210	1 890	1 540	1 170	821
	3 000	2 320	1 980	1 610	1 020	1 610	1 380	1 130	865	607
	4 000	1 740	1 490	1 220	778	1 120	967	794	612	431
	5 000	1 300	1 110	910	583	799	689	567	438	309
	6 000	981	841	689	442	586	506	417	323	228
	7 000	757	650	533	342	445	384	317	245	173
	8 000	597	513	421	271					
	9 000	481	414	339	219					
	10 000									
	11 000									
	12 000									
Y-Y Axis	0	3 410	2 890	2 330	1 470	2 810	2 390	1 930	1 470	1 020
	1 000	2 980	2 350	1 670	811	2 570	2 070	1 530	952	553
	2 000	2 910	2 300	1 620	787	2 460	2 000	1 480	925	537
	3 000	2 730	2 190	1 570	767	2 170	1 790	1 360	879	517
	4 000	2 400	1 950	1 450	734	1 780	1 480	1 150	785	480
	5 000	2 010	1 660	1 260	679	1 420	1 180	926	656	420
	6 000	1 660	1 370	1 060	601	1 130	935	737	532	351
	7 000	1 370	1 130	883	515	899	746	589	429	287
	8 000	1 130	934	733	436	726	602	476	349	236
	9 000	937	778	612	368	594	493	390	287	195
	10 000	787	654	515	312	493	409	324	239	163
	11 000	667	555	438	267	415	344	273	202	138
	12 000	572	475	376	230					
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK										
Area (mm^2)		10 900	9 180	7 420	5 620	8 960	7 560	6 140	4 660	3 920
r_x (mm)		46.3	46.7	47.1	47.6	38.3	38.7	39.1	39.5	39.8
r_y (mm)		68.1	67.6	67.1	66.6	58.1	57.5	57.0	56.4	56.2
r_z (mm)		29.7	29.8	30.0	30.2	24.8	24.8	25.0	25.1	25.2
IMPERIAL SIZE AND WEIGHT										
Weight (lb/ft)		57.4	48.4	39.2	29.8	47.2	40.0	32.4	24.6	20.6
Thickness (in)		3/4	5/8	1/2	3/8	3/4	5/8	1/2	3/8	5/16
Size (in)		6 x 6				5 x 5				

Interconnectors are assumed to be closely spaced.

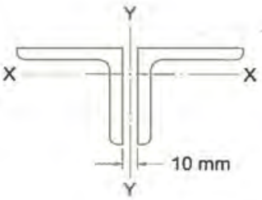
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Equal-Leg Angles

Factored Axial Compressive Resistances (kN)

Legs 10 mm Back-to-Back



CSA G40.21

350W

$\phi = 0.90$

Designation (mm x mm x mm)	L 102 x 102						L 89 x 89			
	19	13	11	9.5	7.9	‡ 6.4	13	9.5	7.9	‡ 6.4
Mass (kg/m)	55.0	38.0	33.6	29.2	24.4	19.6	33.0	25.2	21.4	17.2
Effective length (KL) in millimetres with respect to indicated axis — X-X Axis										
0	2 210	1 530	1 350	1 170	981	654	1 320	1 010	850	654
500	2 190	1 510	1 330	1 150	970	646	1 300	993	836	643
1 000	2 050	1 420	1 260	1 090	915	610	1 190	913	769	592
1 500	1 790	1 250	1 110	963	811	542	1 000	772	653	504
2 000	1 490	1 050	929	808	682	457	795	617	523	405
2 500	1 190	850	754	657	556	373	618	481	408	317
3 000	952	682	606	529	448	301	480	375	319	248
3 500	763	548	488	426	361	243	377	296	251	196
4 000	618	445	397	347	294	198	302	237	201	157
4 500	507	366	326	285	242	163	245	192	164	128
5 000	421	305	272	238	202	136	202	159	136	106
5 500	354	257	229	201	170	115			114	88.6
6 000	302	219	195	171	145	98.0				
Y-Y Axis										
0	2 210	1 530	1 350	1 170	981	654	1 320	1 010	850	654
1 000	2 080	1 320	1 100	878	646	346	1 180	813	616	395
2 000	1 890	1 230	1 040	837	621	334	1 050	748	578	377
3 000	1 530	1 010	872	724	559	313	802	589	473	329
4 000	1 170	774	670	565	452	271	583	430	351	256
5 000	884	582	505	429	348	218	425	314	258	191
6 000	673	443	384	327	268	171	316	234	193	144
7 000	522	343	298	254	209	135	242	179	148	111
8 000	414	272	236	202	166	108	190	141	116	87.5
9 000	334	220	191	163	135	87.5				
10 000										
11 000										
12 000										
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK										
Area (mm^2)	7 020	4 840	4 280	3 700	3 100	2 500	4 200	3 200	2 700	2 180
r_x (mm)	30.3	31.1	31.3	31.5	31.7	31.9	26.9	27.3	27.5	27.7
r_y (mm)	48.1	46.9	46.6	46.4	46.1	45.8	41.7	41.1	40.8	40.5
r_z (mm)	19.8	19.9	20.0	20.1	20.2	20.3	17.3	17.4	17.5	17.6
IMPERIAL SIZE AND WEIGHT										
Weight (lb/ft)	37.0	25.6	22.6	19.6	16.4	13.2	22.2	17.0	14.4	11.6
Thickness (in)	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{7}{16}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$
Size (in)	4 x 4						3 $\frac{1}{2}$ x 3 $\frac{1}{2}$			

Interconnectors are assumed to be closely spaced.

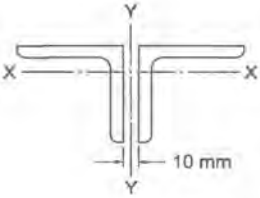
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Equal-Leg Angles

Factored Axial Compressive Resistances (kN)

Legs 10 mm Back-to-Back



CSA G40.21
350W
ϕ = 0.90

Designation (mm x mm x mm)		L 76 x 76					L 64 x 64				
		13	9.5	7.9	6.4	‡ 4.8	13	9.5	7.9	6.4	4.8
Mass (kg/m)		28.0	21.4	18.2	14.6	11.0	22.8	17.4	14.8	12.2	9.2
Effective length (KL) in millimetres with respect to indicated axis — X-X Axis	0	1 120	858	723	584	367	915	705	596	483	367
	500	1 090	836	705	570	359	874	676	571	463	352
	1 000	954	737	623	505	319	713	554	471	384	294
	1 500	752	585	496	404	256	512	402	343	281	217
	2 000	561	440	374	306	195	356	281	241	198	153
	2 500	416	328	279	229	146	252	199	171	141	110
	3 000	313	247	211	173	111	185	146	126	104	81.0
	3 500	241	191	163	134	86.1	140	111	95.5	78.9	61.6
	4 000	190	151	129	106	68.1					
	4 500	153	122	104	85.4	55.0					
	5 000										
	5 500										
	6 000										
Y-Y Axis	0	1 120	858	723	584	367	915	705	596	483	367
	500	1 040	743	580	408	195	870	639	511	375	230
	1 000	1 020	728	568	398	189	837	620	498	366	224
	1 500	952	694	547	387	185	745	559	458	344	216
	2 000	841	622	501	365	179	628	471	389	300	199
	2 500	716	532	434	327	169	512	383	317	247	173
	3 000	599	445	365	281	154	413	308	255	200	143
	3 500	498	370	304	236	135	334	249	206	161	117
	4 000	414	308	253	198	116	272	202	167	132	96.5
	4 500	347	257	212	167	99.3	224	167	138	108	79.9
	5 000	293	217	179	141	84.9	187	139	115	90.4	66.9
	5 500	249	185	153	120	73.0	158	117	96.9	76.3	56.6
	6 000	214	159	131	104	63.1	134	99.9	82.6	65.1	48.4
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK											
Area (mm^2)		3 540	2 720	2 300	1 860	1 410	2 900	2 240	1 880	1 540	1 160
r_x (mm)		22.8	23.2	23.4	23.6	23.9	18.8	19.1	19.3	19.5	19.8
r_y (mm)		36.6	36.0	35.7	35.4	35.2	31.6	31.0	30.7	30.3	30.1
r_z (mm)		14.8	14.9	15.0	15.0	15.1	12.4	12.4	12.4	12.5	12.6
IMPERIAL SIZE AND WEIGHT											
Weight (lb/ft)		18.8	14.4	12.2	9.80	7.42	15.4	11.8	10.0	8.20	6.14
Thickness (in)		1/2	3/8	5/16	1/4	3/16	1/2	3/8	5/16	1/4	3/16
Size (in)		3 x 3					2 1/2 x 2 1/2				

Interconnectors are assumed to be closely spaced.

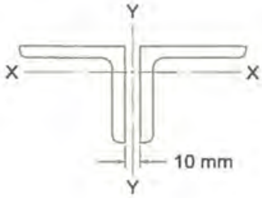
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Equal-Leg Angles

Factored Axial Compressive Resistances (kN)

Legs 10 mm Back-to-Back



CSA G40.21
350W
ϕ = 0.90

Designation (mm x mm x mm)		L 51 x 51					L 44 x 44			L 38 x 38		
		9.5	7.9	6.4	4.8	‡ 3.2	6.4	4.8	‡ 3.2	6.4	4.8	3.2
Mass (kg/m)		14.0	11.6	9.4	7.2	4.8	8.2	6.2	4.2	6.8	5.4	3.6
Effective length (KL) in millimetres with respect to indicated axis — X-X Axis	0	553	469	381	290	164	331	253	164	279	214	146
	500	511	434	354	270	153	297	228	149	238	184	126
	1 000	370	317	260	200	114	198	155	102	140	110	76.5
	1 500	237	204	169	131	75.2	119	94.3	62.5	78.4	61.8	43.4
	2 000	153	133	110	85.8	49.4	75.0	59.5	39.6	47.6	37.7	26.5
	2 500	104	90.7	75.5	58.8	34.0	50.3	40.0	26.7			
	3 000	74.9	65.1	54.2	42.3	24.4						
	3 500											
	4 000											
	4 500											
	5 000											
	5 500											
	6 000											
Y-Y Axis	0	553	469	381	290	164	331	253	164	279	214	146
	500	519	425	324	213	82.8	291	198	94.1	254	178	95.5
	1 000	483	401	309	206	80.6	270	188	91.0	221	162	90.6
	1 500	404	336	265	186	77.0	216	158	83.9	167	124	76.1
	2 000	318	264	209	151	69.9	163	120	69.8	120	88.9	56.7
	2 500	245	203	161	118	58.8	121	89.8	54.2	86.5	64.3	41.5
	3 000	189	156	124	91.3	47.4	91.8	68.0	41.7	64.1	47.6	30.9
	3 500	149	122	97.1	71.7	38.0	70.9	52.5	32.5	48.9	36.3	23.6
	4 000	118	97.5	77.4	57.2	30.7	56.0	41.5	25.8	38.3	28.5	
	4 500	96.2	79.1	62.8	46.5	25.1	45.1	33.5				
	5 000	79.3	65.3	51.8	38.4							
	5 500											
	6 000											
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK												
Area (mm^2)		1 750	1 480	1 210	922	624	1 050	802	544	888	680	464
r_x (mm)		15.1	15.3	15.5	15.7	15.9	13.4	13.7	13.9	11.4	11.6	11.8
r_y (mm)		26.0	25.6	25.3	25.0	24.7	22.8	22.5	22.2	20.3	20.0	19.6
r_z (mm)		9.89	9.90	9.93	10.0	10.1	8.68	8.73	8.82	7.42	7.45	7.52
IMPERIAL SIZE AND WEIGHT												
Weight (lb/ft)		9.40	7.84	6.38	4.88	3.30	5.54	4.24	2.88	4.68	3.60	2.46
Thickness (in)		$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$
Size (in)		2 x 2					$1\frac{3}{4}$ x $1\frac{3}{4}$			$1\frac{1}{2}$ x $1\frac{1}{2}$		

Interconnectors are assumed to be closely spaced.

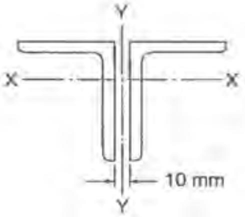
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Unequal-Leg Angles

Factored Axial Compressive Resistances (kN)

Long Legs 10 mm Back-to-Back



CSA G40.21
350W
ϕ = 0.90

Designation (mm x mm x mm)		L 178 x 102		L 152 x 102					L 152 x 89		
		‡13	‡9.5	19	16	13	‡9.5	‡7.9	13	‡9.5	‡7.9
Mass (kg/m)		53.0	40.4	70.0	59.2	48.0	36.4	30.6	45.4	34.6	29.0
Effective length (KL) in millimetres with respect to indicated axis: X-X Axis	0	2 070	1 320	2 810	2 390	1 930	1 320	1 000	1 830	1 240	936
	1 000	2 040	1 300	2 750	2 330	1 890	1 290	979	1 780	1 210	915
	2 000	1 890	1 210	2 440	2 070	1 680	1 150	878	1 590	1 090	821
	3 000	1 630	1 040	1 960	1 670	1 360	937	714	1 290	884	669
	4 000	1 320	850	1 490	1 270	1 040	719	549	988	679	514
	5 000	1 050	674	1 110	956	785	543	416	745	514	389
	6 000	825	532	846	727	598	414	317	567	392	297
	7 000	655	423	655	563	464	322	246	440	304	231
	8 000	527	340	518	446	367	255	195	349	241	183
	9 000	430	278	418	360	297	206	158	282	195	148
	10 000	356	231								
	11 000	299	194								
	12 000										
Y-Y Axis	0	2 070	1 320	2 810	2 390	1 930	1 320	1 000	1 830	1 240	936
	1 000	1 460	718	2 510	2 020	1 480	820	515	1 400	776	487
	2 000	1 300	652	2 220	1 800	1 340	755	479	1 190	681	436
	3 000	1 050	556	1 730	1 420	1 080	642	423	879	532	357
	4 000	787	442	1 280	1 050	810	504	347	623	389	272
	5 000	586	340	947	777	602	384	272	446	284	203
	6 000	443	262	711	584	455	294	212	330	211	153
	7 000	342	205	547	450	351	229	167	251	162	118
	8 000	270		431	355	277	182	133			
	9 000										
	10 000										
	11 000										
	12 000										
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK											
Area (mm^2)		6 780	5 140	8 960	7 560	6 120	4 660	3 900	5 800	4 420	3 700
r_x (mm)		57.3	57.8	47.6	48.0	48.5	48.9	49.2	48.6	49.1	49.3
r_y (mm)		40.2	39.7	43.3	42.7	42.1	41.6	41.3	36.0	35.4	35.2
r_z (mm)		22.2	22.4	21.9	22.0	22.2	22.4	22.5	19.3	19.5	19.6
IMPERIAL SIZE AND WEIGHT											
Weight (lb/ft)		35.8	27.2	47.2	40.0	32.4	24.6	20.6	30.6	23.4	19.6
Thickness (in)		½	⅜	¾	⅝	½	⅜	5/16	½	⅜	5/16
Size (in)		7 x 4		6 x 4					6 x 3 ½		

Interconnectors are assumed to be closely spaced.

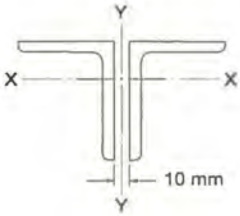
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Unequal-Leg Angles

Factored Axial Compressive Resistances (kN)

Long Legs 10 mm Back-to-Back



CSA G40.21
350W
ϕ = 0.90

Designation (mm x mm x mm)		L 127 x 89			L 127 x 76			
		9.5	‡ 7.9	‡ 6.4	13	9.5	‡ 7.9	‡ 6.4
Mass (kg/m)		30.8	25.8	20.8	38.0	29.0	24.2	19.6
Effective length (KL) in millimetres with respect to indicated axis: X-X Axis	0	1 240	936	654	1 520	1 160	872	619
	1 000	1 190	902	631	1 470	1 120	842	597
	2 000	1 010	761	533	1 230	946	711	506
	3 000	751	570	400	917	708	533	380
	4 000	536	408	287	653	507	383	273
	5 000	386	294	207	470	365	276	198
	6 000	286	218	153	347	271	205	146
	7 000	218	166	117	264	206	156	112
	8 000	170	130	91.6	207	161	122	87.5
	9 000							
	10 000							
	11 000							
	12 000							
Y-Y Axis	0	1 240	936	654	1 520	1 160	872	619
	500	911	597	330	1 310	866	566	320
	1 000	873	568	312	1 240	819	533	300
	1 500	836	546	301	1 130	759	499	284
	2 000	777	515	288	977	670	450	263
	2 500	696	472	271	813	568	390	236
	3 000	605	419	249	666	471	329	206
	3 500	518	364	223	544	388	275	176
	4 000	440	314	197	447	321	229	150
	4 500	374	269	172	371	267	192	127
	5 000	320	231	150	311	225	162	108
	5 500	275	200	131	264	191	138	93.1
	6 000	238	173	115	226	164	119	80.5
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK								
Area (mm^2)		3 940	3 300	2 660	4 840	3 700	3 100	2 500
r_x (mm)		40.6	40.8	41.0	40.3	40.8	41.0	41.2
r_y (mm)		37.4	37.1	36.8	32.0	31.4	31.1	30.8
r_z (mm)		19.3	19.4	19.6	16.5	16.6	16.7	16.8
IMPERIAL SIZE AND WEIGHT								
Weight (lb/ft)		20.8	17.4	14.0	25.6	19.6	16.4	13.2
Thickness (in)		3/8	5/16	1/4	1/2	3/8	5/16	1/4
Size (in)		5 x 3 1/2			5 x 3			

Interconnectors are assumed to be closely spaced.

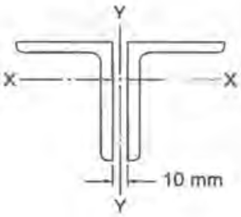
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Unequal-Leg Angles

Factored Axial Compressive Resistances (kN)

Long Legs 10 mm Back-to-Back



CSA G40.21

350W

ϕ = 0.90

Designation (mm x mm x mm)		L 102 x 89			L 102 x 76			
		9.5	7.9	‡ 6.4	13	9.5	7.9	‡ 6.4
Mass (kg/m)		27.0	22.8	18.4	32.8	25.2	21.4	17.2
Effective length (KL) in millimetres with respect to indicated axis — X-X Axis	0	1 090	915	654	1 320	1 010	852	619
	500	1 080	905	647	1 310	1 000	843	612
	1 000	1 020	856	612	1 240	947	797	580
	1 500	903	761	545	1 100	843	711	519
	2 000	761	643	461	923	713	602	441
	2 500	621	526	378	753	584	494	362
	3 000	501	425	306	607	472	400	294
	3 500	405	344	248	490	382	324	239
	4 000	330	280	202	399	312	265	195
	4 500	272	231	167	329	257	218	161
	5 000	227	193	139	274	214	182	135
	5 500	191	163	117	231	181	154	114
	6 000	163	139	100	197	155	131	97.1
Y-Y Axis	0	1 090	915	654	1 320	1 010	852	619
	500	867	651	383	1 190	824	625	379
	1 000	845	631	369	1 150	795	601	363
	1 500	818	613	360	1 060	744	569	347
	2 000	767	584	347	924	660	514	322
	2 500	689	536	328	775	559	444	288
	3 000	599	475	301	640	464	373	248
	3 500	513	412	268	526	383	310	210
	4 000	436	353	235	434	317	258	177
	4 500	371	302	203	361	264	215	149
	5 000	317	259	176	304	222	182	127
	5 500	273	223	153	258	189	155	108
	6 000	236	193	133	221	162	133	93.2
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK								
Area (mm^2)		3 440	2 900	2 340	4 200	3 200	2 700	2 180
r_x (mm)		31.9	32.1	32.3	31.8	32.2	32.4	32.7
r_y (mm)		39.7	39.4	39.1	34.0	33.4	33.1	32.8
r_z (mm)		18.5	18.6	18.7	16.2	16.4	16.5	16.6
IMPERIAL SIZE AND WEIGHT								
Weight (lb/ft)		18.2	15.4	12.4	22.2	17.0	14.4	11.6
Thickness (in)		3/8	5/16	1/4	1/2	3/8	5/16	1/4
Size (in)		4 x 3 1/2			4 x 3			

Interconnectors are assumed to be closely spaced.

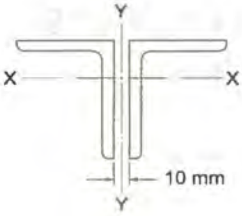
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Unequal-Leg Angles

Factored Axial Compressive Resistances (kN)

Long Legs 10 mm Back-to-Back



CSA G40.21
350W
$\phi = 0.90$

Designation (mm x mm x mm)		L 89 x 76				L 89 x 64			
		13	9.5	7.9	‡ 6.4	13	9.5	7.9	‡ 6.4
Mass (kg/m)		30.2	23.4	19.6	16.0	27.8	21.4	18.0	14.6
Effective length (KL) in millimetres with respect to indicated axis: X-X Axis	0	1 220	934	786	619	1 120	858	723	568
	500	1 200	919	774	609	1 100	844	712	560
	1 000	1 100	847	714	563	1 010	780	658	518
	1 500	932	720	609	481	860	666	563	444
	2 000	745	579	490	389	690	537	455	360
	2 500	581	453	385	306	540	422	358	284
	3 000	453	354	301	240	422	330	281	223
	3 500	357	280	238	190	333	261	223	177
	4 000	286	224	191	152	267	210	179	142
	4 500	232	183	156	124	217	171	146	116
	5 000	192	151	129	103	180	141	121	96.1
	5 500		127	108	86.2	151	119	101	80.7
	6 000								
Y-Y Axis	0	1 220	934	786	619	1 120	858	723	568
	500	1 120	788	607	408	1 030	735	573	390
	1 000	1 090	767	589	395	977	699	545	371
	1 500	1 010	723	562	381	854	619	491	342
	2 000	885	644	511	356	703	512	412	296
	2 500	747	548	441	318	562	409	332	244
	3 000	620	456	370	273	446	325	265	197
	3 500	512	377	308	231	356	260	212	159
	4 000	424	313	256	194	288	210	172	130
	4 500	354	261	214	163	236	172	141	107
	5 000	298	220	180	138	196	143	117	89.2
	5 500	253	187	154	118	165	120	98.8	75.3
	6 000	217	160	132	101				
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK									
Area (mm^2)		3 880	2 960	2 500	2 020	3 540	2 720	2 300	1 860
r_x (mm)		27.3	27.7	27.9	28.1	27.6	28.0	28.2	28.4
r_y (mm)		35.2	34.6	34.3	34.1	29.1	28.4	28.1	27.8
r_z (mm)		15.8	15.9	15.9	16.0	13.6	13.6	13.7	13.8
IMPERIAL SIZE AND WEIGHT									
Weight (lb/ft)		20.4	15.8	13.2	10.8	18.8	14.4	12.2	9.80
Thickness (in)		$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$
Size (in)		$3\frac{1}{2} \times 3$				$3\frac{1}{2} \times 2\frac{1}{2}$			

Interconnectors are assumed to be closely spaced.

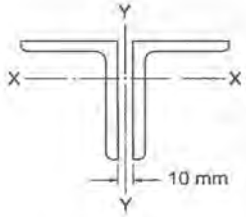
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Unequal-Leg Angles

Factored Axial Compressive Resistances (kN)

Long Legs 10 mm Back-to-Back



CSA G40.21
350W
ϕ = 0.90

Designation (mm x mm x mm)		L 76 x 64		L 76 x 51				
		9.5	7.9	13	9.5	7.9	6.4	‡ 4.8
Mass (kg/m)		19.6	16.6	23.0	17.6	14.8	12.2	9.2
Effective length (KL) in millimetres with respect to indicated axis: X-X Axis	0	782	659	915	705	596	483	329
	500	762	643	892	689	582	472	322
	1 000	676	571	789	612	519	421	288
	1 500	540	459	630	492	418	341	234
	2 000	409	348	476	374	319	261	179
	2 500	306	261	356	281	240	197	136
	3 000	232	198	269	213	183	150	103
	3 500	179	153	208	165	142	116	80.4
	4 000	142	121	165	131	112	92.2	63.7
	4 500	114	97.9	133	106	90.5	74.5	51.5
	5 000							
	5 500							
	6 000							
Y-Y Axis	0	782	659	915	705	596	483	329
	500	692	547	859	629	502	366	200
	1 000	664	527	775	571	459	338	187
	1 500	593	478	629	464	376	284	165
	2 000	494	403	483	354	288	221	135
	2 500	398	326	366	267	217	168	106
	3 000	318	261	279	204	166	129	82.3
	3 500	255	210	217	158	129	100	64.8
	4 000	207	170	172	125	102	79.9	51.8
	4 500	170	140	139	101	82.5	64.7	42.2
	5 000	141	116					
	5 500	119	97.9					
	6 000							

PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK

Area (mm^2)	2 480	2 100	2 900	2 240	1 880	1 540	1 160
r_x (mm)	23.6	23.8	23.5	23.9	24.1	24.3	24.5
r_y (mm)	29.6	29.3	24.2	23.5	23.1	22.8	22.5
r_z (mm)	13.3	13.3	10.9	10.9	11.0	11.0	11.1

IMPERIAL SIZE AND WEIGHT

Weight (lb/ft)	13.2	11.2	15.4	11.8	10.0	8.20	6.14
Thickness (in)	3/8	5/16	1/2	3/8	5/16	1/4	3/16
Size (in)	3 x 2 1/2		3 x 2				

Interconnectors are assumed to be closely spaced.

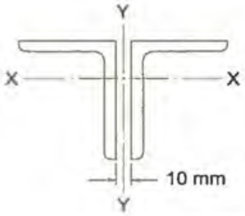
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Unequal-Leg Angles

Factored Axial Compressive Resistances (kN)

Long Legs 10 mm Back-to-Back



CSA G40.21
350W
ϕ = 0.90

Designation (mm x mm x mm)			L 64 x 51				L 51 x 38		
			9.5	7.9	6.4	4.8	6.4	4.8	‡ 3.2
Mass (kg/m)			15.8	13.4	10.8	8.4	8.4	6.2	4.2
Effective length (KL) in millimetres with respect to indicated axis	X-X Axis	0	629	532	432	328	330	252	155
		500	604	511	415	316	308	236	145
		1 000	500	425	347	265	229	177	110
		1 500	366	313	256	197	150	117	73.4
		2 000	258	221	182	140	98.6	76.9	48.7
		2 500	184	158	131	101	67.7	52.8	33.6
		3 000	135	117	96.3	74.5	48.7	38.1	24.2
		3 500	103	88.6	73.2	56.7			
		4 000				44.4			
		4 500							
		5 000							
		5 500							
		6 000							
	Y-Y Axis	0	629	532	432	328	330	252	155
		500	578	468	350	223	287	196	88.2
		1 000	531	434	328	211	245	172	81.5
		1 500	436	360	278	188	180	130	67.7
		2 000	338	278	217	152	127	92.1	51.0
		2 500	257	212	165	119	91.0	66.1	37.6
		3 000	197	162	127	91.9	67.0	48.8	28.2
		3 500	153	126	98.9	72.1	50.9	37.1	21.6
		4 000	122	100	78.6	57.6			
		4 500	98.6	81.3	63.7	46.8			
		5 000							
		5 500							
		6 000							
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK									
Area (mm^2)			2 000	1 690	1 370	1 040	1 050	802	544
r_x (mm)			19.5	19.7	19.9	20.1	15.8	16.0	16.3
r_y (mm)			24.6	24.3	23.9	23.6	19.0	18.6	18.3
r_z (mm)			10.7	10.7	10.8	10.9	8.12	8.18	8.27
IMPERIAL SIZE AND WEIGHT									
Weight (lb/ft)			10.6	9.00	7.24	5.50	5.54	4.24	2.88
Thickness (in)			$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$
Size (in)			$2\frac{1}{2} \times 2$				$2 \times 1\frac{1}{2}$		

Interconnectors are assumed to be closely spaced.

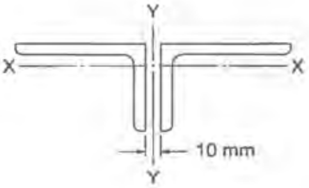
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Unequal-Leg Angles

Factored Axial Compressive Resistances (kN)

Short Legs 10 mm Back-to-Back



CSA G40.21
350W
$\phi = 0.90$

Designation (mm x mm x mm)			L 178 x 102		L 152 x 102					L 152 x 89		
			‡ 13	‡ 9.5	19	16	13	‡ 9.5	‡ 7.9	13	‡ 9.5	‡ 7.9
Mass (kg/m)			53.0	40.4	70.0	59.2	48.0	36.4	30.6	45.4	34.6	29.0
Effective length (KL) in millimetres with respect to indicated axis	X-X Axis	0	2 070	1 320	2 810	2 390	1 930	1 320	1 000	1 830	1 240	936
		500	2 040	1 300	2 770	2 350	1 900	1 300	988	1 790	1 220	917
		1 000	1 890	1 210	2 570	2 180	1 770	1 220	924	1 600	1 090	828
		1 500	1 620	1 040	2 210	1 880	1 540	1 060	807	1 310	898	681
		2 000	1 320	850	1 790	1 540	1 260	873	666	1 010	696	529
		2 500	1 040	674	1 420	1 220	1 000	699	534	762	529	404
		3 000	819	532	1 120	961	793	555	425	582	406	310
		3 500	649	423	886	764	632	443	340	453	316	241
		4 000	522	340	713	615	509	358	275	359	251	192
		4 500	426	278	582	503	417	293	225	290	203	155
		5 000	353	231	482	417	346	244	187		167	128
		5 500	296	194	405	350	291	205	157			
		6 000							134			
	Y-Y Axis	0	2 070	1 320	2 810	2 390	1 930	1 320	1 000	1 830	1 240	936
		1 000	1 440	698	2 530	2 030	1 470	805	502	1 400	764	473
		2 000	1 420	683	2 510	2 010	1 450	792	492	1 380	753	465
		3 000	1 410	678	2 410	1 960	1 430	784	488	1 370	748	462
		4 000	1 400	675	2 160	1 800	1 370	770	482	1 330	739	459
		5 000	1 370	669	1 850	1 550	1 210	735	471	1 190	716	452
		6 000	1 280	658	1 550	1 300	1 030	660	447	1 010	651	436
		7 000	1 130	633	1 290	1 080	859	564	402	846	557	397
		8 000	973	581	1 080	902	717	475	347	708	470	344
		9 000	836	512	904	756	601	400	296	595	396	292
		10 000	720	445	764	638	508	339	252	504	336	249
		11 000	622	387	651	544	433	289	216	430	287	213
		12 000	541	337	560	467	372	249	186	370	247	184
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK												
Area (mm^2)			6 780	5 140	8 960	7 560	6 120	4 660	3 900	5 800	4 420	3 700
r_x (mm)			28.5	28.9	28.6	28.9	29.3	29.8	30.0	24.7	25.1	25.3
r_y (mm)			87.7	87.1	74.7	74.1	73.5	72.9	72.6	75.5	74.9	74.6
r_z (mm)			22.2	22.4	21.9	22.0	22.2	22.4	22.5	19.3	19.5	19.6
IMPERIAL SIZE AND WEIGHT												
Weight (lb/ft)			35.8	27.2	47.2	40.0	32.4	24.6	20.6	30.6	23.4	19.6
Thickness (in)			$\frac{1}{2}$	$\frac{3}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$
Size (in)			7 x 4		6 x 4					6 x 3 $\frac{1}{2}$		

Interconnectors are assumed to be closely spaced.

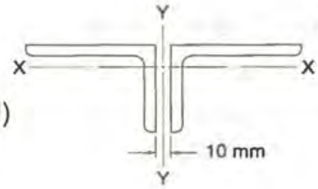
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Unequal-Leg Angles

Factored Axial Compressive Resistances (kN)

Short Legs 10 mm Back-to-Back



CSA G40.21
350W
ϕ = 0.90

Designation (mm x mm x mm)		L 127 x 89			L 127 x 76			
		9.5	‡ 7.9	‡ 6.4	13	9.5	‡ 7.9	‡ 6.4
Mass (kg/m)		30.8	25.8	20.8	38.0	29.0	24.2	19.6
Effective length (KL) in millimetres with respect to indicated axis — X-X Axis	0	1 240	936	654	1 520	1 160	872	619
	500	1 220	918	642	1 470	1 130	846	601
	1 000	1 100	836	585	1 260	969	730	520
	1 500	919	697	489	958	743	562	402
	2 000	721	549	386	695	543	412	296
	2 500	554	423	298	505	397	302	217
	3 000	428	327	231	376	296	225	162
	3 500	335	256	181	287	227	173	125
	4 000	267	204	144	225	178	136	98.0
	4 500	216	166	117				
	5 000	178	137	96.8				
	5 500							
	6 000							
Y-Y Axis	0	1 240	936	654	1 520	1 160	872	619
	1 000	866	559	303	1 270	821	527	291
	2 000	854	551	298	1 260	812	521	287
	3 000	835	542	295	1 210	801	515	285
	4 000	771	520	289	1 050	754	502	282
	5 000	649	464	275	861	638	456	273
	6 000	527	386	247	697	520	381	250
	7 000	426	315	209	564	421	311	212
	8 000	347	257	174	460	344	254	176
	9 000	286	212	145	379	284	210	146
	10 000	238	177	121	317	237	176	122
	11 000	201	150	103	267	200	148	104
	12 000	171	128	87.8	228	171	127	88.6
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK								
Area (mm^2)		3 940	3 300	2 660	4 840	3 700	3 100	2 500
r_x (mm)		26.0	26.2	26.4	21.1	21.5	21.7	21.9
r_y (mm)		61.3	61.0	60.7	63.8	63.2	62.9	62.6
r_z (mm)		19.3	19.4	19.6	16.5	16.6	16.7	16.8
IMPERIAL SIZE AND WEIGHT								
Weight (lb/ft)		20.8	17.4	14.0	25.6	19.6	16.4	13.2
Thickness (in)		$^3/_8$	$^5/_{16}$	$^1/_4$	$^1/_2$	$^3/_8$	$^5/_{16}$	$^1/_4$
Size (in)		5 x 3 $^1/_2$			5 x 3			

Interconnectors are assumed to be closely spaced.

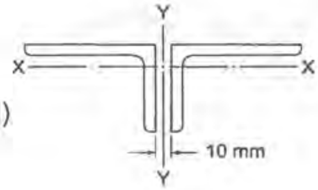
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Unequal-Leg Angles

Factored Axial Compressive Resistances (kN)

Short Legs 10 mm Back-to-Back



CSA G40.21
350W
$\phi = 0.90$

Designation (mm x mm x mm)		L 102 x 89			L 102 x 76			
		9.5	7.9	‡ 6.4	13	9.5	7.9	‡ 6.4
Mass (kg/m)		27.0	22.8	18.4	32.8	25.2	21.4	17.2
Effective length (KL) in millimetres with respect to indicated axis — X-X Axis	0	1 090	915	654	1 320	1 010	852	619
	500	1 070	900	643	1 280	984	828	602
	1 000	979	825	590	1 110	857	723	527
	1 500	823	697	500	861	669	567	415
	2 000	653	555	399	633	495	421	309
	2 500	507	432	311	464	365	311	229
	3 000	393	336	243	347	274	234	172
	3 500	309	265	191	267	210	180	133
	4 000	247	212	153	210	166	142	105
	4 500	201	172	125			114	84.2
	5 000	166	142	103				
	5 500							
	6 000							
Y-Y Axis	0	1 090	915	654	1 320	1 010	852	619
	1 000	845	628	365	1 170	800	600	359
	2 000	816	610	356	1 130	781	588	352
	3 000	712	556	337	943	693	547	339
	4 000	557	451	295	734	546	447	301
	5 000	424	347	236	560	418	344	241
	6 000	324	267	185	430	321	265	188
	7 000	252	208	145	336	250	207	147
	8 000	200	166	116	267	199	165	118
	9 000	162	134	94.2	216	161	134	95.5
	10 000				178			
	11 000							
	12 000							
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK								
Area (mm^2)		3 440	2 900	2 340	4 200	3 200	2 700	2 180
r_x (mm)		26.8	27.1	27.3	21.9	22.3	22.5	22.7
r_y (mm)		47.9	47.6	47.4	50.2	49.6	49.3	49.0
r_z (mm)		18.5	18.6	18.7	16.2	16.4	16.5	16.6
IMPERIAL SIZE AND WEIGHT								
Weight (lb/ft)		18.2	15.4	12.4	22.2	17.0	14.4	11.6
Thickness (in)		3/8	5/16	1/4	1/2	3/8	5/16	1/4
Size (in)		4 x 3 1/2			4 x 3			

Interconnectors are assumed to be closely spaced.

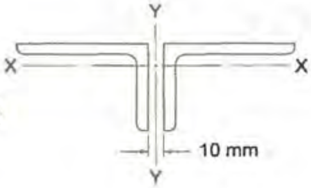
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Unequal-Leg Angles

Factored Axial Compressive Resistances (kN)

Short Legs 10 mm Back-to-Back



CSA G40.21
350W
ϕ = 0.90

Designation (mm x mm x mm)			L 89 x 76				L 89 x 64			
			13	9.5	7.9	‡ 6.4	13	9.5	7.9	‡ 6.4
Mass (kg/m)			30.2	23.4	19.6	16.0	27.8	21.4	18.0	14.6
Effective length (KL) in millimetres with respect to indicated axis	X-X Axis	0	1 220	934	786	619	1 120	858	723	568
		500	1 190	909	766	603	1 060	818	690	543
		1 000	1 030	797	673	532	846	658	558	441
		1 500	808	628	533	422	592	466	397	316
		2 000	599	469	399	317	405	321	275	219
		2 500	442	348	296	236	284	226	194	155
		3 000	332	262	224	179	207	165	142	114
		3 500	255	202	172	138	156	125	107	86.0
		4 000	201	159	136	109				
		4 500		128	110	87.7				
		5 000								
		5 500								
		6 000								
	Y-Y Axis	0	1 220	934	786	619	1 120	858	723	568
		1 000	1 100	772	591	393	1 020	720	555	372
		2 000	995	723	564	380	936	688	539	364
		3 000	772	573	467	338	738	553	455	333
		4 000	567	422	347	263	549	412	341	260
		5 000	416	309	256	196	406	304	253	194
		6 000	311	231	191	148	306	229	190	147
		7 000	238	178	147	114	235	176	146	113
		8 000	188	140	116	89.7	185	139	115	89.1
		9 000					149			
		10 000								
		11 000								
		12 000								
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK										
Area (mm^2)			3 880	2 960	2 500	2 020	3 540	2 720	2 300	1 860
r_x (mm)			22.4	22.8	23.0	23.2	17.9	18.3	18.5	18.7
r_y (mm)			43.2	42.6	42.3	42.1	45.0	44.4	44.1	43.8
r_z (mm)			15.8	15.9	15.9	16.0	13.6	13.6	13.7	13.8
IMPERIAL SIZE AND WEIGHT										
Weight (lb/ft)			20.4	15.8	13.2	10.8	18.8	14.4	12.2	9.80
Thickness (in)			1/2	3/8	5/16	1/4	1/2	3/8	5/16	1/4
Size (in)			3 1/2 x 3				3 1/2 x 2 1/2			

Interconnectors are assumed to be closely spaced.

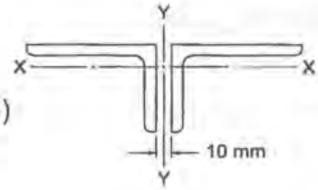
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Unequal-Leg Angles

Factored Axial Compressive Resistances (kN)

Short Legs 10 mm Back-to-Back



CSA G40.21
350W
$\phi = 0.90$

Designation (mm x mm x mm)		L 76 x 64		L 76 x 51				
		9.5	7.9	13	9.5	7.9	6.4	‡ 4.8
Mass (kg/m)		19.6	16.6	23.0	17.6	14.8	12.2	9.2
Effective length (KL) in millimetres with respect to indicated axis — X-X Axis	0	782	659	915	705	596	483	329
	500	747	631	830	643	545	443	303
	1 000	607	515	568	447	382	314	216
	1 500	435	371	349	277	239	197	137
	2 000	302	258	221	177	153	127	88.2
	2 500	213	183	149	119	103	85.9	60.0
	3 000	156	134					
	3 500	118	102					
	4 000							
	4 500							
	5 000							
	5 500							
	6 000							
Y-Y Axis	0	782	659	915	705	596	483	329
	500	689	543	862	627	497	358	192
	1 000	680	535	856	622	492	353	189
	1 500	656	522	815	609	486	350	187
	2 000	592	483	734	555	458	341	185
	2 500	510	421	639	483	402	313	180
	3 000	430	356	545	411	343	271	168
	3 500	360	298	461	347	289	229	148
	4 000	301	249	388	292	243	193	127
	4 500	253	209	328	246	205	163	108
	5 000	214	177	279	209	174	139	92.1
	5 500	182	151	239	179	149	119	79.0
	6 000	157	130	206	154	128	102	68.2
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK								
Area (mm^2)		2 480	2 100	2 900	2 240	1 880	1 540	1 160
r_x (mm)		18.7	18.9	13.9	14.2	14.4	14.6	14.8
r_y (mm)		37.6	37.3	40.1	39.4	39.1	38.8	38.5
r_z (mm)		13.3	13.3	10.9	10.9	11.0	11.0	11.1
IMPERIAL SIZE AND WEIGHT								
Weight (lb/ft)		13.2	11.2	15.4	11.8	10.0	8.20	6.14
Thickness (in)		3/8	5/16	1/2	3/8	5/16	1/4	3/16
Size (in)		3 x 2 1/2		3 x 2				

Interconnectors are assumed to be closely spaced.

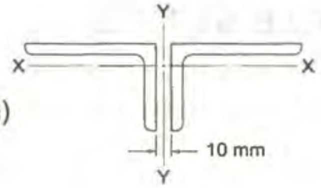
‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Unequal-Leg Angles

Factored Axial Compressive Resistances (kN)

Short Legs 10 mm Back-to-Back



CSA G40.21

350W

$\phi = 0.90$

Designation (mm x mm x mm)			L 64 x 51				L 51 x 38		
			9.5	7.9	6.4	4.8	6.4	4.8	‡ 3.2
Mass (kg/m)			15.8	13.4	10.8	8.4	8.4	6.2	4.2
Effective length (KL) in millimetres with respect to indicated axis	X-X Axis	0	629	532	432	328	330	252	155
		500	577	490	399	304	278	214	132
		1 000	409	350	287	221	159	124	77.9
		1 500	257	222	183	142	87.3	68.7	43.5
		2 000	165	143	118	92.1	52.7	41.6	26.4
		2 500	112	97.0	80.7	62.8			
		3 000			57.8	45.1			
		3 500							
		4 000							
		4 500							
		5 000							
		5 500							
		6 000							
	Y-Y Axis	0	629	532	432	328	330	252	155
		500	578	467	346	218	288	193	85.0
		1 000	568	460	341	215	281	190	83.8
		1 500	518	429	328	210	246	177	81.8
		2 000	442	368	289	197	197	146	76.5
		2 500	364	303	240	172	154	114	64.9
		3 000	296	246	196	142	120	88.8	51.9
		3 500	240	200	159	116	94.4	70.0	41.4
		4 000	196	163	130	95.6	75.5	56.0	33.3
		4 500	162	135	107	79.1	61.5	45.6	27.2
		5 000	136	113	89.7	66.2	50.8	37.7	22.5
		5 500	115	95.2	75.8	56.0			
		6 000	97.9	81.3	64.8	47.9			
PROPERTIES OF 2 ANGLES - 10 mm BACK-TO-BACK									
Area (mm^2)			2 000	1 690	1 370	1 040	1 050	802	544
r_x (mm)			14.6	14.8	15.0	15.2	11.0	11.2	11.4
r_y (mm)			32.6	32.3	32.0	31.6	27.0	26.6	26.3
r_z (mm)			10.7	10.7	10.8	10.9	8.12	8.18	8.27
IMPERIAL SIZE AND WEIGHT									
Weight (lb/ft)			10.6	9.00	7.24	5.50	5.54	4.24	2.88
Thickness (in)			3/8	5/16	1/4	3/16	1/4	3/16	1/8
Size (in)			2 1/2 x 2				2 x 1 1/2		

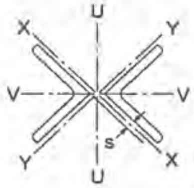
Interconnectors are assumed to be closely spaced.

‡ Factored axial compressive resistances calculated according to S16-14 Clause 13.3.5.

DOUBLE ANGLE STRUTS

Star-Shaped

Factored Axial Compressive Resistances (kN)



CSA G40.21 350W

$\phi = 0.90$

Designation (mm x mm x mm)		L152x152			L127x127			
		19	16	13	19	16	13	9.5
Mass (kg/m)		85.4	72.0	58.4	70.2	59.6	48.2	36.6
Spacing, s		12 mm			12 mm			
Effective length (KL) in millimetres with respect to indicated axis: U-U Axis	0	3 410	2 890	2 330	2 810	2 390	1 930	1 470
	1 000	2 910	2 270	1 590	2 520	2 020	1 470	897
	2 000	2 890	2 250	1 570	2 510	2 010	1 460	887
	3 000	2 890	2 250	1 560	2 390	2 010	1 460	885
	4 000	2 710	2 250	1 560	2 060	1 720	1 370	884
	5 000	2 340	1 950	1 550	1 710	1 420	1 120	834
	6 000	1 980	1 650	1 300	1 400	1 160	910	673
	7 000	1 670	1 380	1 090	1 140	941	737	543
	8 000	1 400	1 150	905	940	771	602	442
	9 000	1 180	969	759	779	638	497	364
	10 000	998	820	641	653	533	415	304
	11 000	852	699	546	553	451	351	256
	12 000	734	602	470	473	386	299	219
V-V Axis	0	3 410	2 890	2 330	2 810	2 390	1 930	1 470
	1 000	2 910	2 270	1 590	2 520	2 020	1 470	897
	2 000	2 890	2 250	1 570	2 450	2 010	1 460	887
	3 000	2 710	2 250	1 560	1 970	1 690	1 380	885
	4 000	2 210	1 890	1 540	1 500	1 290	1 060	817
	5 000	1 760	1 510	1 230	1 130	976	803	621
	6 000	1 390	1 190	978	860	743	614	475
	7 000	1 110	952	781	666	577	477	370
	8 000	892	768	631	527	457	378	294
	9 000	730	628	517	426	369	306	238
	10 000	605	522	429				
PROPERTIES OF 2 STARRED ANGLES								
Area (mm^2)		10 900	9 180	7 420	8 960	7 560	6 140	4 660
r_u (mm)		78.0	76.6	75.1	67.9	66.5	65.0	63.6
r_v (mm)		58.3	58.9	59.5	48.1	48.7	49.3	49.9
r_z (mm)		29.7	29.8	30.0	24.8	24.8	25.0	25.1
IMPERIAL SIZE AND WEIGHT								
Weight (lb/ft)		57.4	48.4	39.2	47.2	40.0	32.4	24.6
Thickness (in)		¾	⅝	½	¾	⅝	½	⅜
Size (in)		6 x 6			5 x 5			

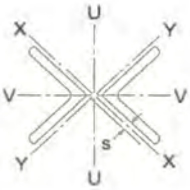
Interconnectors are assumed to be closely spaced.

See CSA S16-14 Clauses 19.2.4 and 19.2.5 for interconnecting requirements.

DOUBLE ANGLE STRUTS

Star-Shaped

Factored Axial Compressive Resistances (kN)



CSA G40.21
350W
$\phi = 0.90$

Designation (mm x mm x mm)		L102x102					L89x89		
		19	13	11	9.5	7.9	13	9.5	7.9
Mass (kg/m)		55.0	38.0	33.6	29.2	24.4	33.0	25.2	21.4
Spacing, s		10 mm					8 mm		
Effective length (KL) in millimetres with respect to indicated axis — U-U Axis	0	2 210	1 530	1 350	1 170	981	1 320	1 010	850
	1 000	2 070	1 290	1 070	846	615	1 170	796	597
	2 000	2 020	1 290	1 060	841	611	1 140	794	595
	3 000	1 730	1 160	1 010	840	610	910	678	564
	4 000	1 400	918	798	681	563	688	506	418
	5 000	1 100	712	616	524	432	514	375	309
	6 000	863	553	477	404	332	389	282	232
	7 000	684	435	374	316	260	301	217	178
	8 000	549	347	299	252	207	238	171	140
	9 000	448	282	243	205	167	192	138	
	10 000	371	233	200	169	138			
	11 000	311							
V-V Axis	0	2 210	1 530	1 350	1 170	981	1 320	1 010	850
	1 000	2 070	1 290	1 070	846	615	1 170	796	597
	2 000	1 740	1 220	1 060	841	611	963	745	595
	3 000	1 250	893	793	692	587	654	511	434
	4 000	875	629	561	490	418	438	345	294
	5 000	621	450	401	351	300	304	240	205
	6 000	455	331	295	259	221	220	174	149
	7 000	345	251	225	197	169			
	8 000					132			
PROPERTIES OF 2 STARRED ANGLES									
Area (mm^2)		7 020	4 840	4 280	3 700	3 100	4 200	3 200	2 700
r_u (mm)		56.5	53.6	52.9	52.1	51.4	47.0	45.5	44.9
r_v (mm)		38.0	39.1	39.4	39.7	40.1	33.8	34.4	34.7
r_z (mm)		19.8	19.9	20.0	20.1	20.2	17.3	17.4	17.5
IMPERIAL SIZE AND WEIGHT									
Weight (lb/ft)		37.0	25.6	22.6	19.6	16.4	22.2	17.0	14.4
Thickness (in)		$\frac{3}{4}$	$\frac{1}{2}$	$\frac{7}{16}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$
Size (in)		4 x 4					$3\frac{1}{2} \times 3\frac{1}{2}$		

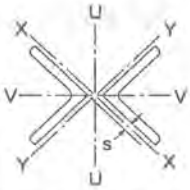
Interconnectors are assumed to be closely spaced.

See CSA S16-14 Clauses 19.2.4 and 19.2.5 for interconnecting requirements.

DOUBLE ANGLE STRUTS

Star-Shaped

Factored Axial Compressive Resistances (kN)



CSA G40.21
350W
$\phi = 0.90$

Designation (mm x mm x mm)		L76x76				L64x64				
		13	9.5	7.9	6.4	13	9.5	7.9	6.4	4.8
Mass (kg/m)		28.0	21.4	18.2	14.6	22.8	17.4	14.8	12.2	9.2
Spacing, s		8 mm				8 mm				
Effective length (KL) in millimetres with respect to indicated axis										
U-U Axis	0	1 120	858	723	584	915	705	596	483	367
U-U Axis	1 000	1 020	721	556	386	860	625	496	358	216
U-U Axis	2 000	920	694	555	385	704	529	441	352	215
U-U Axis	3 000	697	516	428	339	500	368	303	239	176
U-U Axis	4 000	504	368	304	239	345	251	204	160	117
U-U Axis	5 000	366	265	218	170	243	175	142	111	81.2
U-U Axis	6 000	272	196	160	125	178	127	103	80.4	58.6
U-U Axis	7 000	208	149	122	95.1	134	96.1			
U-U Axis	8 000	163	117							
V-V Axis	0	1 120	858	723	584	915	705	596	483	367
V-V Axis	500	1 030	726	561	391	861	627	499	361	218
V-V Axis	1 000	1 020	721	556	386	789	614	496	358	216
V-V Axis	1 500	878	681	555	385	630	496	422	345	215
V-V Axis	2 000	713	558	474	385	476	378	324	266	205
V-V Axis	2 500	564	443	378	309	356	284	244	201	156
V-V Axis	3 000	444	351	300	246	269	216	186	154	119
V-V Axis	3 500	352	279	239	196	208	168	145	120	92.7
V-V Axis	4 000	283	225	193	159	165	133	115	94.9	73.6
V-V Axis	4 500	231	184	158	130	133	107	92.6	76.7	59.6
V-V Axis	5 000	192	153	131	108					49.1
PROPERTIES OF 2 STARRED ANGLES										
Area (mm^2)		3 540	2 720	2 300	1 860	2 900	2 240	1 880	1 540	1 160
r_u (mm)		41.9	40.3	39.7	38.9	36.8	35.3	34.5	33.8	33.0
r_v (mm)		28.6	29.2	29.5	29.8	23.5	24.1	24.4	24.7	25.0
r_z (mm)		14.8	14.9	15.0	15.0	12.4	12.4	12.4	12.5	12.6
IMPERIAL SIZE AND WEIGHT										
Weight (lb/ft)		18.8	14.4	12.2	9.80	15.4	11.8	10.0	8.20	6.14
Thickness (in)		$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$
Size (in)		3 x 3				$2\frac{1}{2}$ x $2\frac{1}{2}$				

Interconnectors are assumed to be closely spaced.

See CSA S16-14 Clauses 19.2.4 and 19.2.5 for interconnecting requirements.

DOUBLE ANGLE STRUTS

Star-Shaped

Factored Axial Compressive Resistances (kN)

CSA G40.21 350W

$\phi = 0.90$

Designation (mm x mm x mm)			L51x51				L44x44		L38x38		
			9.5	7.9	6.4	4.8	6.4	4.8	6.4	4.8	3.2
Mass (kg/m)			14.0	11.6	9.4	7.2	8.2	6.2	6.8	5.4	3.6
Spacing, s			6 mm				6 mm		6 mm		
Effective length (KL) in millimetres with respect to indicated axis	U-U Axis	0	553	469	381	290	331	253	279	214	146
		500	516	421	319	208	289	195	253	177	93.1
		1 000	506	420	318	207	288	194	236	176	92.8
		1 500	437	364	292	207	237	178	184	137	90.6
		2 000	356	294	233	174	183	136	136	100	65.4
		2 500	282	231	182	134	138	102	100	73.3	47.3
		3 000	223	181	142	104	106	77.6	75.0	54.7	35.1
		3 500	177	144	112	81.9	82.3	60.1	57.6	41.9	26.8
		4 000	143	115	89.4	65.4	65.3	47.6	45.4	32.9	21.0
		4 500	116	94.0	72.7	53.1	52.8	38.5			
		5 000	96.5	77.8	60.1	43.9					
		5 500	81.1	65.3							
	V-V Axis	0	553	469	381	290	331	253	279	214	146
		500	516	421	319	208	289	195	253	177	93.1
		1 000	432	369	303	207	241	186	178	139	92.8
		1 500	311	268	222	172	164	128	111	87.6	61.5
		2 000	217	188	156	122	110	86.1	70.8	56.2	39.7
		2 500	154	134	111	87.1	76.2	60.0	47.9	38.1	27.0
		3 000	113	98.1	82.0	64.2	55.1	43.5			
		3 500	85.3	74.4	62.3	48.8					
PROPERTIES OF 2 STARRED ANGLES											
Area (mm^2)			1 750	1 480	1 210	922	1 050	802	888	680	464
r_u (mm)			28.9	28.1	27.3	26.7	24.8	24.1	22.2	21.6	20.8
r_v (mm)			18.9	19.2	19.5	19.8	16.9	17.2	14.3	14.6	14.9
r_z (mm)			9.89	9.90	9.93	10.0	8.68	8.73	7.42	7.45	7.52
IMPERIAL SIZE AND WEIGHT											
Weight (lb/ft)			9.40	7.84	6.38	4.88	5.54	4.24	4.68	3.60	2.46
Thickness (in)			$^3/_8$	$^5/_{16}$	$^1/_4$	$^3/_{16}$	$^1/_4$	$^3/_{16}$	$^1/_4$	$^3/_{16}$	$^1/_8$
Size (in)			2 x 2				$1\,^3/_4$ x $1\,^3/_4$		$1\,^1/_2$ x $1\,^1/_2$		

Interconnectors are assumed to be closely spaced.

See CSA S16-14 Clauses 19.2.4 and 19.2.5 for interconnecting requirements.

Single-Angle Strut – Design Example

General

Two design methods are available for eccentrically loaded single-angle members in compression:

(1) The factored compressive resistance may be calculated by neglecting the effects of eccentricity in accordance with CSA S16-14 Clause 13.3.3 for single angles satisfying the following conditions:

- The ratio of long leg width to short leg width is less than 1.7
- The angles are web members of planar, box or space trusses with adjacent web members attached to the same side of a gusset plate or chord.
- The members are loaded at the ends in compression through the same one leg.
- The members are attached by welding or by minimum two-bolt connections.
- There are no intermediate transverse loads.

This design method is not intended for single angles used as diagonal braces in a braced frame

(2) Members which do not satisfy the above conditions are designed for combined compression and bending by taking into account the effects of eccentricity in accordance with Clauses 13.3.2 and 13.3.3.4.

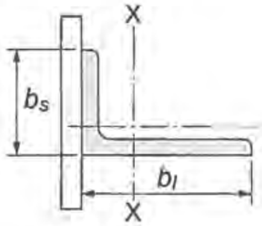
For single angles subject to elastic local buckling, Clause 13.3.5 is used.

Design Example

The following example illustrates the design of a single-angle web member in a planar truss according to S16-14 Clause 13.3.3.2. Also see the tables of Factored Compressive Resistances for single-angle struts in Part 4.

Given

Find the factored compressive resistance of a L102x76x6.4 unequal-leg angle connected through the shorter leg. The steel grade is CSA G40.21-350W (F_y = 350 MPa), and L = 2000 mm.



Long leg: $b_l = d = 102$ mm

Short leg: $b_s = b = 76.2$ mm

$b_l / b_s = 102 / 76.2 = 1.34 < 1.7$

r_x = radius of gyration about geometric axis parallel to the connected (i.e. shorter) leg

= 32.7 mm

Note: When the longer leg is connected, the axis parallel to the connected leg is defined as the X-axis, as opposed to the Y-axis as defined in the tables of Properties and Dimensions in Part 6.

r'_y = radius of gyration about minor principal axis = 16.6 mm

Solution

A. Width-to-Thickness Ratios

$$\frac{d}{t} = \frac{102}{6.35} = 16.1 > \frac{250}{\sqrt{F_y}} = 13.4\,, \quad \frac{b}{t} = \frac{76.2}{6.35} = 12.0 < \frac{250}{\sqrt{F_y}} = 13.4$$

The angle section exceeds the width-to-thickness ratio in S16-14 Table 1 and is therefore a Class 4 section. The resistance will be calculated according to Clause 13.3.5(a) using the effective area (A_e).

B. Effective Area

Calculate the effective area according to Clause 13.3.5(a).

$$A_e = A - \left(\frac{d}{t} - \frac{250}{\sqrt{F_y}}\right)t^2 = 1090 - (16.1 - 13.4)\,6.35^2 = 981\text{ mm}^2$$

C. Equivalent slenderness

$$\frac{L}{r_x} = \frac{2000}{32.7} = 61.2 < 80$$

For individual members and planar trusses, and for the shorter leg connected, the equivalent slenderness is:

$$\frac{KL}{r} = 72 + 0.75\frac{L}{r_x} + 4\left[\left(\frac{b_l}{b_s}\right)^2 - 1\right] = 72 + 0.75 \times 61.2 + 4\left[1.34^2 - 1\right]$$

$$= 121 > 0.95\frac{L}{r_y'} = \frac{0.95 \times 2000}{16.6} = 115$$

$$\frac{KL}{r} = 121 < 200 \quad \text{according to Clause 10.4.2.1.}$$

D. Factored Compressive Resistance

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 \times 200\,000}{121^2} = 135\text{ MPa}$$

$$\lambda = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{350}{135}} = 1.61$$

$$C_r = \frac{\phi A_e F_y}{\left(1 + \lambda^{2n}\right)^{\frac{1}{n}}} = \frac{0.9 \times 981 \times 350}{\left(1 + 1.61^{2\times 1.34}\right)^{\frac{1}{1.34}}} = 99.2\text{ kN}$$

Double-Angle Strut – Design Example

General

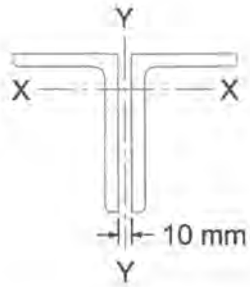
The following example illustrates the design of a double-angle strut in accordance with S16-14 Clauses 13.3 and 19.2. Also see *General Information* and the tables of *Factored Axial Compressive Resistances* for double-angle struts in Part 4.

Given:

Find the factored axial compressive resistance of a 2L102x89x6.4 double-angle strut, with long legs 10 mm back-to-back. The steel grade is G40.21 350W ($F_y = 350$ MPa), $L = 2\ 000$ mm, and there are two welded intermediate connectors at the one-third points.

Solution:

A. Class of Section



Width-to-thickness ratios, S16-14 Clause 11.2

$$\frac{d}{t} = \frac{102}{6.35} = 16.1 > \frac{250}{\sqrt{F_y}} = 13.4$$

$$\frac{b}{t} = \frac{88.9}{6.35} = 14.0 > \frac{250}{\sqrt{F_y}} = 13.4$$

The angle is therefore a Class 4 section in axial compression.

B. Effective Area

Effective area according to Clause 13.3.5(a)

$$A_e = A - 2\left(\frac{d}{t} + \frac{b}{t} - 2 \times \frac{250}{\sqrt{F_y}}\right) t^2$$

$$= 2340 - 2\,(16.1 + 14.0 - 2 \times 13.4)\,6.35^2 \approx 2\ 070 \text{ mm}^2$$

C. Compressive Resistance About Axis X-X, Flexural Mode

Slenderness parameter, Clause 13.3.1

$$\lambda = \left(\frac{KL}{r}\right)_x \sqrt{\frac{F_y}{\pi^2 E}} = \frac{2\ 000}{32.3}\sqrt{\frac{350}{\pi^2 \times 200 \times 10^3}} = 0.825$$

$$C_{rx} = \phi A_e F_y \left(1 + \lambda^{2n}\right)^{-1/n}$$

$$= 0.90 \times 2\ 070 \times 350 \left(1 + 0.825^{2 \times 1.34}\right)^{-1/1.34} = 460 \text{ kN}$$

By comparison, the tables of *Factored Axial Compressive Resistances* of double-angle struts, long legs back-to-back in Part 4 indicate a compressive resistance of 461 kN.

D. Compressive Resistance About Axis Y-Y, Torsional-Flexural Mode (Detailed Calculation)

Shear centre location, Clause 13.3.2

$$x_o = 0 \qquad y_o = y - \frac{t}{2} = 29.6 - \frac{6.35}{2} = 26.4 \text{ mm}$$

Torsional-flexural section properties

$$\bar{r}_o^2 = x_o^2 + y_o^2 + r_x^2 + r_y^2 = 0^2 + 26.4^2 + 32.3^2 + 39.1^2 = 3\,270 \text{ mm}^2$$

$$\Omega = 1 - \left(\frac{x_o^2 + y_o^2}{\bar{r}_o^2}\right) = 1 - \left(\frac{0^2 + 26.4^2}{3\,270}\right) = 0.787$$

Slenderness ratio of the built-up member, Clause 19.2.4(b)

$$\rho_o = \left(\frac{K\,L}{r}\right)_y = \frac{2\,000}{39.1} = 51.2$$

Slenderness ratio of a component angle, with two welded intermediate connectors spaced at $L\,/\,3 = 2\,000\,/\,3 = 667$ mm and $K = 0.65$

$$\rho_i = \left(\frac{K\,L}{r}\right)_z = \frac{0.65 \times 667}{18.7} = 23.2$$

Equivalent slenderness ratio

$$\rho_e = \sqrt{\rho_o^2 + \rho_i^2} = \sqrt{51.2^2 + 23.2^2} = 56.2$$

$$F_{ey} = \frac{\pi^2\,E}{\rho_e^2} = \frac{\pi^2 \times 200 \times 10^3}{56.2^2} = 625 \text{ MPa}$$

$$F_{ez} = \left(\frac{\pi^2\,E\,C_w}{(K_z\,L_z)^2} + G\,J\right)\frac{1}{A\,\bar{r}_o^2}$$

$$= \left(\frac{\pi^2 \times 200 \times 10^3 \times 22.7 \times 10^6}{2\,000^2} + 77 \times 10^3 \times 31.5 \times 10^3\right)\frac{1}{2\,340 \times 3\,270} = 318 \text{ MPa}$$

$$F_e = F_{eyz} = \frac{F_{ey} + F_{ez}}{2\Omega}\left(1 - \sqrt{1 - \frac{4\,F_{ey}F_{ez}\Omega}{(F_{ey} + F_{ez})^2}}\right)$$

$$= \frac{625 + 318}{2 \times 0.787}\left(1 - \sqrt{1 - \frac{4 \times 625 \times 318 \times 0.787}{(625 + 318)^2}}\right) = 273 \text{ MPa}$$

Slenderness parameter

$$\lambda = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{350}{273}} = 1.13$$

Compressive resistance

$$C_{ry} = \phi A_e F_y \left(1 + \lambda^{2n}\right)^{-1/n}$$
$$= 0.90 \times 2\,070 \times 350 \left(1 + 1.13^{2\times 1.34}\right)^{-1/1.34} = 341 \text{ kN}$$

E. *Resistance About Axis Y-Y Using the Tables of Factored Axial Compressive Resistances of Double-Angle Struts, Long Legs Back-to-Back in Part 4*

The actual length L = 2 000 mm is replaced by an equivalent length L_e that accounts for the slenderness of the component angles between the connectors.

$$L_e = r_y \sqrt{\left(\frac{KL}{r}\right)_y^2 + \left(\frac{KL}{r}\right)_z^2} = 39.1\sqrt{\left(\frac{2\,000}{39.1}\right)^2 + \left(\frac{0.65 \times 667}{18.7}\right)^2} = 2\,200 \text{ mm}$$

The table indicates C_r = 347 kN for L = 2 000 mm and C_r = 328 kN for L = 2 500 mm, for closely spaced interconnectors. The compressive resistance for interconnectors spaced at the one-third points is obtained by linear interpolation:

C_r = 340 kN

By comparison, the compressive resistance obtained previously by detailed calculation was 341 kN.

F. *Approximate Compressive Resistance of Struts with Back-to-Back Spacings Other Than 10 mm*

The actual length is replaced by an equivalent length based on the radius of gyration of the built-up section r'_y and the slenderness ratio of the component angles. Consider a double-angle strut with long legs spaced 16 mm back-to-back (r'_y = 41.3 mm).

$$L_e = r_y \sqrt{\left(\frac{KL}{r'}\right)_y^2 + \left(\frac{KL}{r}\right)_z^2} = 39.1\sqrt{\left(\frac{2\,000}{41.3}\right)^2 + \left(\frac{0.65 \times 667}{18.7}\right)^2} = 2\,100 \text{ mm}$$

By interpolation:

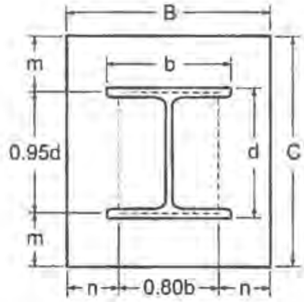
C_r = 343 kN

COLUMN BASE PLATES

When steel columns bear on concrete footings, steel base plates are required to distribute the column load to the footing without exceeding the bearing resistance of the concrete. In general, the ends of columns are saw-cut or milled to a plane surface so as to bear evenly on the base plate. Connection of the column to the base plate and then to the footing depends on the loading conditions. For columns carrying vertical gravity loads only, this connection is required only to hold the parts in line. However, for erection safety, four non-collinear anchor rods are required (CSA S16-14 Clause 25.2).

For base plates subjected to vertical gravity loads only, the following assumptions and design method are recommended:

1. The factored gravity load is assumed uniformly distributed over the base plate within a rectangle of $0.95d \times 0.80b$ (see diagram).
2. The base plate exerts a uniform pressure over the footing.
3. The base plate projecting beyond the area of $0.95d \times 0.80b$ acts as a cantilever subject to the uniform bearing pressure.



C_f = total factored column load (N)

A = $B \times C$ = area of plate (mm^2)

t_p = plate thickness (mm)

F_y = specified minimum yield strength of base plate steel (MPa)

f'_c = specified 28-day strength of concrete (MPa)

ϕ = 0.90 for steel

1. Determine the required area $A = C_f / B_r$ where B_r is the factored bearing resistance per unit of bearing area. For concrete, B_r is assumed to be $0.85\phi_c f'_c$ where $\phi_c = 0.65$ in bearing. (Clause 10.8 of CSA A23.3-14 states when a smaller area is acceptable.)
2. Determine B and C so that the dimensions m and n (the projections of the plate beyond the area, $0.95d \times 0.80b$) are approximately equal.
3. Determine m and n and solve for t_p, where

$$t_p = \sqrt{\frac{2C_f m^2}{BC\phi F_y}} \text{ or } \sqrt{\frac{2C_f n^2}{BC\phi F_y}}, \text{ whichever is greater.}$$

These formulas were derived by equating the factored moment acting on the portion of the plate taken as a cantilever to the factored moment resistance of the plate ($M_r = \phi Z F_y$) and solving for the plate thickness t_p. To minimize deflection of the base plate, the thickness should be generally not less than about 1/5 of the overhang, m or n.

The examples below illustrate the proportioning of base plates to limit bearing pressure on the concrete, and to resist plate bending and limit plate deflection. In addition, these base plates must be dimensioned to accommodate the anchor rods with ample clearances, taking into account the size of holes required for erection tolerances (see *Anchor Rods* in next section) and the presence of welds.

Examples

1. Given:

A W310x118 column subjected to a factored axial load of 2 500 kN is supported by a concrete foundation whose 28-day specified strength is 20 MPa. Design the base plate assuming 300 MPa steel.

Solution:

For W310x118, $b = 307$ mm, $d = 314$ mm.

Area of plate required $= \dfrac{2\,500 \times 10^3}{0.85 \times 0.65 \times 20} = 226\,000$ mm²

Try $B = C = 480$ mm; $A = 230\,000$ mm²

Determine m and n

$0.95\,d = 0.95 \times 314 = 298$ mm Therefore, $m = (480 - 298)/2 = 91$ mm

$0.80\,b = 0.80 \times 307 = 246$ mm Therefore, $n = (480 - 246)/2 = 117$ mm

Use n for design

Plate thickness required $= \sqrt{\dfrac{2 \times 2\,500 \times 10^3 \times 117^2}{480 \times 480 \times 0.9 \times 300}} = 33.2$ mm

$\dfrac{n}{5} = \dfrac{117}{5} = 23.4$ mm < 33.2 mm OK Use 35 mm.

Since the plate thickness of 35 mm is less than 65 mm, $F_y = 300$ MPa for G40.21 Grade 300W steel. For plates greater than 65 mm in thickness, $F_y = 280$ MPa for 300W steel (see Table 6-3). Therefore, use PL 35x480x480 for the base plate.

2. Given:

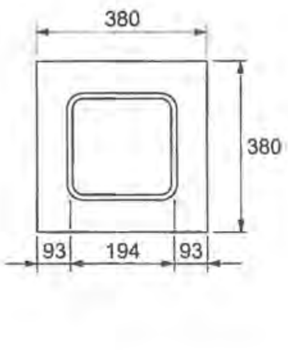
An HSS 203x203x9.5 column supports a factored axial load of 1 550 kN.

Select a base plate assuming $f'_c = 20$ MPa and $F_y = 300$ MPa.

Solution:

Area required is $\dfrac{1\,550 \times 10^3}{0.85 \times 0.65 \times 20} = 140\,000$ mm²

$B = C = \sqrt{A} = \sqrt{140 \times 10^3} = 374$ mm. Use 380 mm



$n = \dfrac{380 - (203 - 9.5)}{2} = 93.3$

Therefore, $t_p = \sqrt{\dfrac{2 \times 1\,550 \times 10^3 \times 93.3^2}{380 \times 380 \times 0.9 \times 300}} = 26.3$ mm

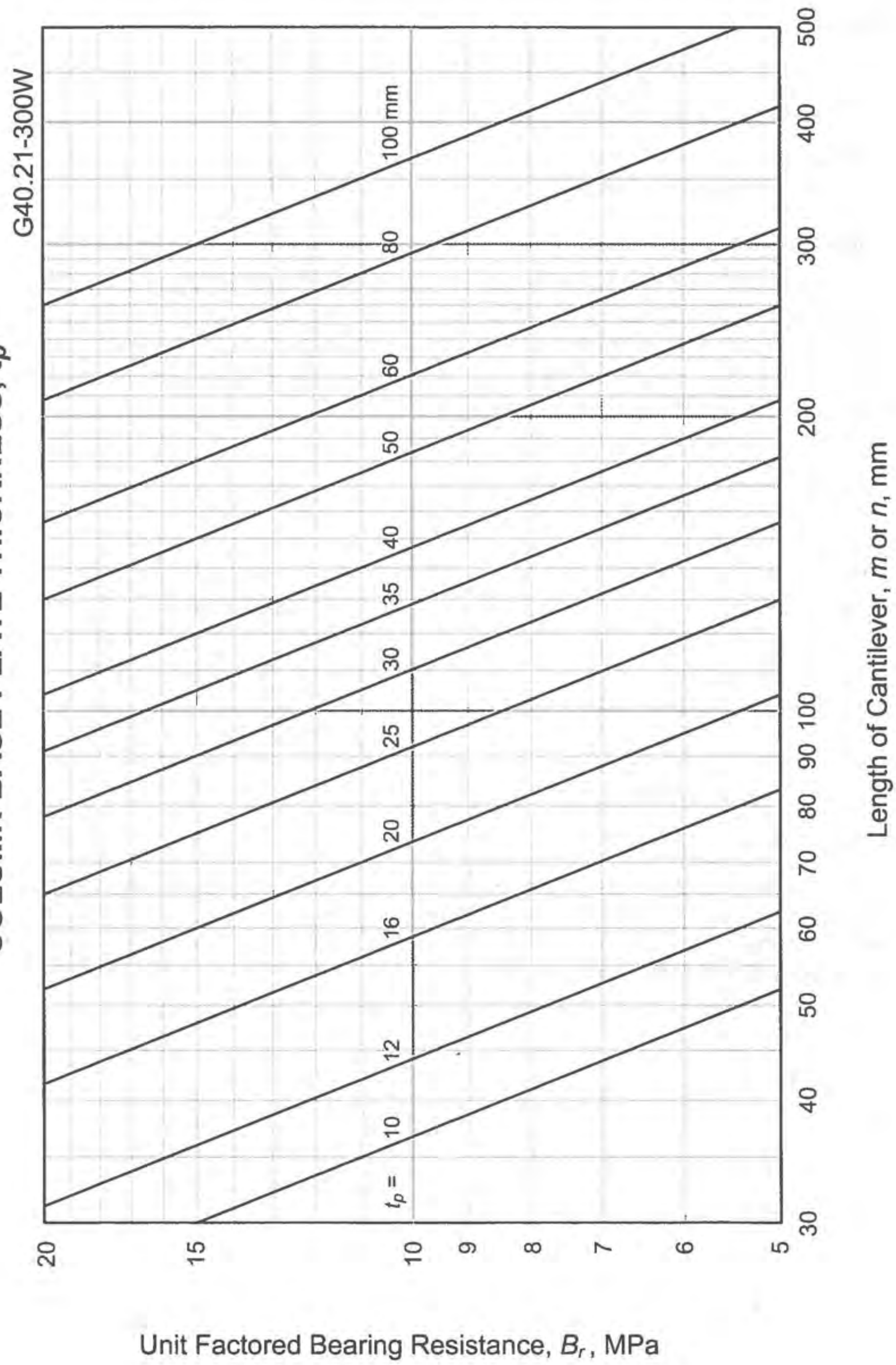
Use 30 mm.

Therefore, use PL 30x380x380 for the base plate.

Design Chart

As an alternative to computing the plate thickness, Figure 4-1 provides a means of selecting t_p knowing the length of cantilever m or n and the unit factored bearing resistance.

FIGURE 4-1
COLUMN BASE PLATE THICKNESS, t_p



Example

Given:

Same as example 1.

Solution:

Unit factored bearing resistance is $0.85 \times 0.65 \times 20 = 11.1$ MPa

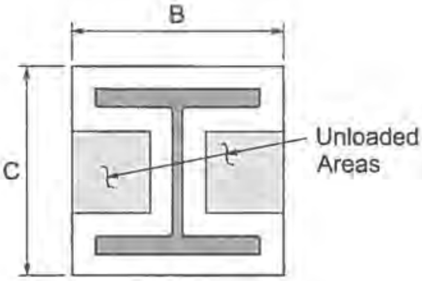
From Figure 4-1 for 11.1 MPa and $n = 117$, select $t_p = 35$ mm.

Base plate assemblies (including anchor rods) that are subjected to applied bending moments, uplift tension, and shear forces must be designed to resist all such forces.

Lightly Loaded Base Plates

For lightly loaded base plates where the required bearing area is less than or about equal to the area bounded by the column dimensions b and d, the above method does not give realistic results for the base plate thickness, and other methods have been proposed in the literature. Fling (1970) uses a yield line theory to derive an equation for the plate thickness. When modified for limit states design, the equation becomes:

$$t_p = 0.43 b\beta \sqrt{\frac{B_r}{\phi F_y\left(1-\beta^2\right)}}$$



where

$B_r = 0.85\, \phi_c f'_c$

$\beta = \sqrt{0.75+\frac{1}{4\lambda^2}} - \frac{1}{2\lambda}$

$\lambda = 2d/b$

b = column width (mm)

d = column depth (mm)

Stockwell (1975) assumes an effective bearing area where only an H-shaped pattern under a W-shape column is loaded, and the remainder of the base plate is unloaded. The assumed width of flange strips can be derived from the required bearing area, and the plate thickness can be determined by the expression:

$$t_p = \sqrt{\frac{2C_f m^2}{A\phi F_y}}$$

where

A = effective bearing area

m = half the width of the bearing strips

References

FLING, R.S. 1970. Design of steel bearing plates. Engineering Journal, American Institute of Steel Construction, **7**(2), April.

STOCKWELL, F.J.JR. 1975. Preliminary base plate selection. Engineering Journal, American Institute of Steel Construction, **12**(3), Third Quarter.

ANCHOR RODS

The vast majority of anchor rods are used at the bases of gravity columns. Theoretically, neither end moments, uplift forces, nor horizontal forces are present at the base of a concentrically loaded column carrying gravity loads only. These anchor rods serve to position, level and secure the base plate, and to resist nominal end moments and horizontal forces which may occur. As a measure for erection safety, CSA S16-14 Clause 25.2 requires that each column base be fitted with at least four non-collinear anchor rods to ensure an adequate resistance against overturning (in any direction) during erection, unless otherwise accounted for. *Note: the expression "anchor rod" has replaced "anchor bolt" in order to avoid confusion with bolts produced to ASTM A325 and A490.*

Fabricators traditionally supply anchor rods manufactured from round bar stock. The bars are threaded at one end to receive a washer and nut and may be bent at the other end to form a hook, or both ends may be threaded. The material used for most common applications is usually produced to CSA G40.21 Grade 300W (F_y = 300 MPa) or to ASTM A36 (F_y = 248 MPa). However, ASTM A36 round bar stock is generally more readily available. Since the introduction of *ASTM F1554 Standard Specification for Anchor Bolts, Steel, 36, 55 and 105-ksi Yield Strength*, Grade 36 anchor rods fill this role.

The diameter of anchor rod holes in base plates should provide for possible horizontal adjustments for alignment purposes. The following table, adopted from the reference below, can be used as a guide for maximum hole sizes (diameters), although actual sizes used by fabricators may vary depending on shop and field practices. When holes smaller than the tabulated maximum sizes are desired, they may be sized to accommodate anchor rod placement tolerances as a minimum. The CISC Code of Standard Practice indicates a tolerance of 6 mm for an entire anchor rod group, and a further 3 mm (1.5 mm on either side) for an individual anchor rod within a group. Therefore, the combined tolerance is 6 + 1.5, or 7.5 mm and thus the minimum hole size allowance may be 8 mm. e.g. 27-mm holes for ¾-inch rods. For the larger rod sizes, a 10-mm or 12-mm minimum hole size allowance is suggested.

SUGGESTED MAXIMUM ANCHOR ROD HOLE SIZES AND MINIMUM WASHER SIZES FOR GRAVITY COLUMNS

Anchor Rod Diameter	Maximum Hole Diameter	Minimum Washer Size [1]	Minimum Washer Thickness
in.	mm	mm	mm
¾	33	51	6.4
⅞	40	64	7.9
1	46	76	9.5
1¼	52	76	12.7
1½	59	89	12.7
1¾	70	102	15.9
2	83	127	19.1
2½	95	140	22.2

Notes:

1. Circular (diameter) or square washers (sides) meeting the washer size are acceptable.
2. Anchor rod holes should be located with ample clearance to accommodate the rod position in the hole with respect to the column, welds and other interferences.
3. For ¾ inch anchor rods, 27 mm diameter holes may be used with ASTM F844 washers in place of fabricated plate washers.

CSA S16-14 Clause 25 covers design requirements for column bases in situations where anchor rods transfer end moments, uplift and horizontal forces due to lateral loads, etc. If such requirements are necessary, they should be clearly identified in the contract documents.

Important mechanical properties for F1554 products are summarized in the table below. When specified in the purchased order as a "supplementary requirement", Grades 55 and 105 rods are supplied to meet specific Charpy notch-toughness with test values. Grade 36 is inherently weldable, while weldable Grade 55 rods are also available when specified as a "supplementary requirement". In addition, F1554 includes provisions for stress area, minimum body diameter, recommended nuts for each rod grade and size range, minimum cross-sectional area at the bend for hooked rods, zinc coating requirements when specified, etc.

MECHANICAL PROPERTIES FOR ASTM F1554 ANCHOR RODS

Grade	36	55	105
Tensile Strength, MPa	400-552	517-655	862-1034
Yield Strength, min, MPa (0.2 % offset)	248	380	724
Elongation in 200 mm, min, %[1]	20	18	12
Elongation in 50 mm, min, %[1]	23	21	15
Reduction of Area, min, %			
6.35 to 50.8 mm (¼ to 2 in.), incl.	40	30	45
over 50.8 to 63.5 mm (2 to 2½ in.), incl.	40	22	45
over 63.5 to 76.2 mm (2½ to 3 in.), incl.	40	20	45
over 76.2 to 102 mm (3 to 4 in.), incl.	40	18	---
Supplementary Requirements			
Min. Average Charpy V-Notch Energy, J			
S4: at +5°C Test Temperature	N.A.	20	20
S5: at -29°C Test Temperature	N.A.	N.A.	20

Notes:

1. Elongation in 200 mm applies to bars. Elongation in 50 mm applies to tests on machined specimens.
2. The round bars from which anchor rods are made shall conform to the tensile properties listed above, except when heat-treated after bending or threading.

For specialized applications, fastener suppliers or fabricators should be consulted.

References

AISC. 2011. Steel construction manual. 14th Edition. American Institute of Steel Construction, Chicago, IL.

BRACING ASSEMBLIES

General

The following example illustrates the design of a bracing assembly in accordance with CSA S16-14 Clause 9.2.6.2.

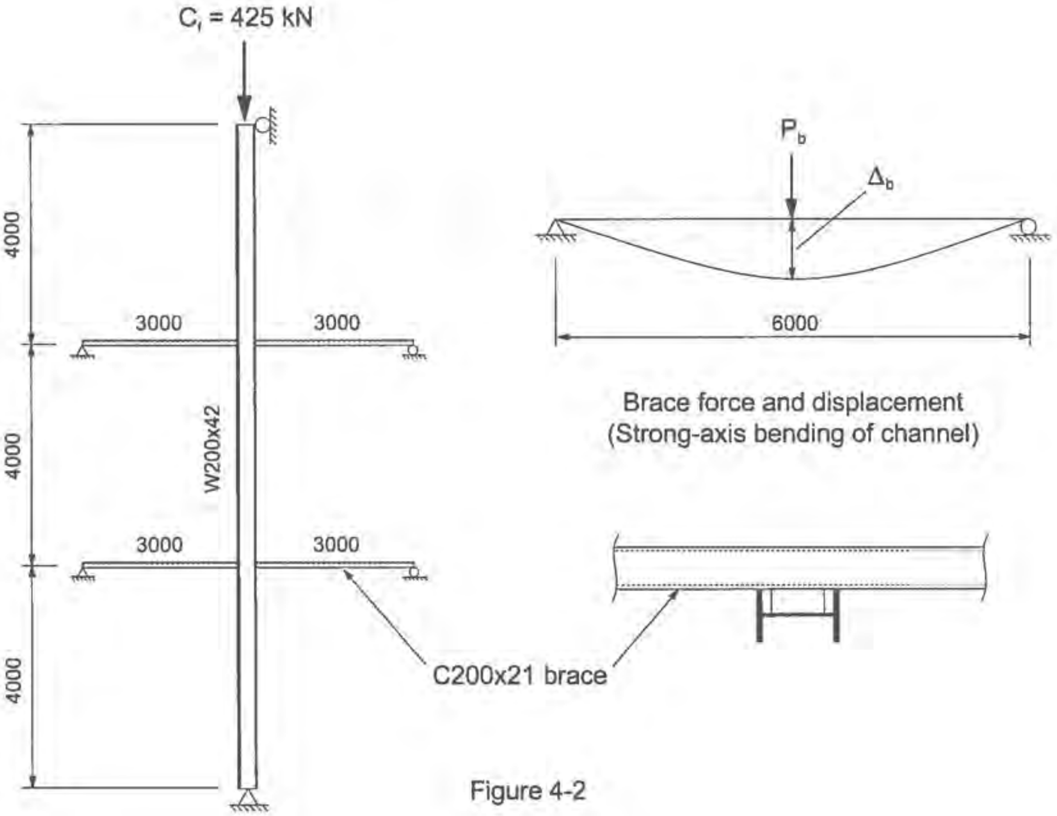


Figure 4-2

Example

A W200x42 column is braced about its weak axis by channels located at the one-third points, as shown on Fig. 4-2. Given a factored axial load of 425 kN acting on the 12-m column, select channel sections with flexural strength and stiffness to provide adequate weak-axis bracing at the one-third points of the column.

Solution

The design objective consists in sizing the channel braces with sufficient strength and stiffness to force the column into a buckling mode between bracing points.

A. Initial Imperfections

For the assumed imperfect shape shown on Fig. 4-3, the initial imperfection is taken to be:

$$\Delta_o = 0.001\,L_b = 0.001 \times 4000 = 4.0 \text{ mm}$$

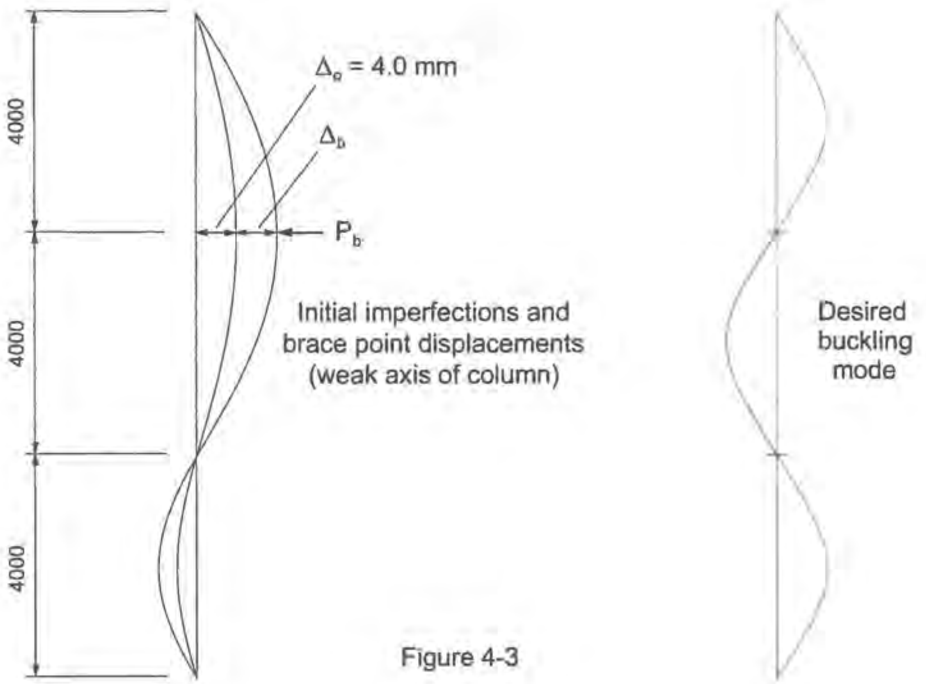


Figure 4-3

B. Strength Requirement

The required flexural strength of the channel braces is calculated using the "Direct Method" in Clause 9.2.6.2. For two equally spaced braces, β = 3. The brace point displacement is assumed to be equal to the initial imperfection:

$\Delta_b = \Delta_o = 4.0$ mm

The factored bracing force is given by:

$$P_b = \frac{\beta(\Delta_o + \Delta_b)\, C_f}{L_b} = \frac{3(4.0 + 4.0)\, 425}{4000} = 2.55\ \text{kN}$$

Factored moment acting on a channel: $M_f = P_b\, L / 4 = 2.55 \times 6.0 / 4 = 3.83\ \text{kN} \cdot \text{m}$

Try C200x21 channels. The factored moment resistance of a C200x21 channel with unbraced length $L/2$ = 3000 mm and F_y = 350 MPa may be determined using the Beam Selection Table in Part 5.

M_r' = 28.0 kN·m > 3.83 kN·m

C. Stiffness Requirement

For channels of length L = 6000 mm and strong-axis moment of inertia $I_x = 14.9 \times 10^6$ mm^4, the brace point displacement is given by:

$$\Delta_b = \frac{P_b L^3}{48 E I_x} = \frac{2.55 \times 10^3 \times 6000^3}{48 \times 200 \times 10^3 \times 14.9 \times 10^6} = 3.85\ \text{mm} < \Delta_o = 4.0\ \text{mm}$$

The selected channel section is adequate.

Note: a wind column is usually oriented with the channel girts running parallel to its strong axis instead.

NOTES

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FLEXURAL MEMBERS

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GENERAL INFORMATION

Part 5 covers flexural members, and its contents are informally grouped in this order: steel members, composite members and general aids for the analysis of beams in common applications. The contents for steel members include design aids for unstiffened-web members and stiffened plate girders.

While most C-shapes (channels) are produced in Canada, imported sections are identified as such in the *Beam Selection Tables*. Since all W-shapes and S-shapes are imported, they need not be identified. W-shapes that are readily available are highlighted in yellow.

Class of Sections in Bending

See page 5-5.

Factored Resistance of Beams

The *Beam Selection Tables*, which list the factored moment resistance of beams under various conditions of lateral support, are provided on pages 5-14 to 5-29 to facilitate the design of flexural members. See page 5-9 for the explanatory text.

The *Beam Load Tables*, which list total uniformly distributed factored loads for laterally supported beams of various spans, are provided on pages 5-30 to 5-50. For the explanatory text, see page 5-10.

Beams with Web Holes

See page 5-51 for design information, design tables and an illustrative example.

Factored Shear Resistance of Girder Webs

The tables on pages 5-66 to 5-67 list the factored shear resistance ϕF_s in a girder web, computed in accordance with the requirements of Clause 13.4.1.1 of CSA S16-14, and the required gross area of pairs of intermediate stiffeners, computed in accordance with the requirements of Clause 14.5.3. Values are provided for minimum specified yield strength levels F_y of 300 and 350 MPa, for aspect ratios (a / h) from 0.50 to 3.00, and for web slenderness ratios (h / w) varying between 50 and 260 for $F_y = 300$, and between 50 and 220 for $F_y = 350$. The required gross area of stiffeners is provided as a percentage of the web area $(h\,w)$ and is shown in *italics*.

Design Example for Stiffened Girder Webs

For design information on the web shear resistance and an illustrative example, see page 5-68.

Beam Bearing Plates

See page 5-71 for design information, a design chart and an illustrative example.

Composite Beams

Tables for the Factored Shear Resistance of Shear Studs in solid slabs and in deck-slabs are given on pages 5-77 to 5-79. Formulas for calculating the area of the concrete pull-out pyramid and an illustrative example are given on pages 5-80 to 5-82. The calculation of the factored resistance of a shear stud is illustrated on page 5-83. *Trial Selection Tables* for composite beams with various combinations of cover slab and steel deck (hollow composite construction) are given on pages 5-86 to 5-125. See page 5-74 for the explanatory text. A composite beam design example is given on page 5-84.

Deflection of Flexural Members

See page 5-126 for a design chart, table and illustrative examples.

Beam Diagrams and Formulas

Pages 5-130 to 5-148 contain diagrams and formulas to facilitate the design of flexural members in accordance with elastic theory.

CLASS OF SECTIONS IN BENDING

Table 5-1 lists the class of section in bending of W-shapes for grades of steel including ASTM A992 and A572 grade 50 ($F_y = 345$ MPa). Listed are the W-shape sizes provided in Part 6 of this Handbook. For these steel grades, all S-shapes are Class 1, and all C and MC-shapes are Class 3.

Table 5-1 also lists for each section size the ratios b_{el}/t and h/w, where b_{el} = one-half the flange width, t = flange thickness, h = clear distance between flanges and w = web thickness. See also *Limits on Width-to-Thickness Ratios* in Part 4.

CLASS OF SECTIONS IN BENDING

Table 5-1

ASTM A992, A572 grade 50

Designation	Class	b_{el}/t	h / w
W1100x499	1	4.50	39.5
x433	1	5.03	46.7
x390	1	5.56	51.4
x343	1	6.45	57.1
W1000x976	1	2.38	18.6
x883	1	2.59	20.4
x748	1	2.98	23.8
x642	1	3.43	27.3
x591	1	3.66	29.9
x554	1	3.92	31.5
x539	1	3.98	32.7
x483	1	4.39	36.5
x443	1	4.80	39.3
x412	1	5.03	44.0
x371	1	5.54	48.8
x321	1	6.45	56.2
x296	1	7.38	56.2
W1000x584	1	2.45	25.8
x494	1	2.86	29.9
x486	1	2.85	30.9
x438	1	3.11	34.5
x415	1	3.30	35.7
x393	1	3.45	38.0
x350	1	3.78	44.0
x314	1	4.18	48.6
x272	1	4.84	56.2
x249	1	5.77	56.2
x222	1	7.11	58.0
W920x1377	1	2.05	11.2
x1269	1	2.00	13.5
x1194	1	2.10	14.3
x1077	1	2.28	15.7
x970	1	2.48	17.3
x787	1	2.96	21.1
x725	1	3.19	22.6
x656	1	3.48	25.0
x588	1	3.82	27.8
x537	1	4.16	30.4
x491	1	4.49	33.3
x449	1	4.95	35.9
x420	1	5.29	38.4
x390	1	5.74	40.5
x368	1	6.11	42.5
x344	1	6.53	44.7
W920x381	1	3.53	35.4
x345	1	3.86	39.1
x313	1	4.48	40.9
x289	1	4.81	44.5
x271	1	5.12	46.9
x253	1	5.48	49.9
x238	1	5.89	52.3
x223	1	6.36	54.3
x201	1	7.56	56.8

Designation	Class	b_{el}/t	h / w
W840x576	1	3.55	24.9
x527	1	3.85	27.0
x473	1	4.23	30.2
x433	1	4.60	32.7
x392	1	5.03	36.1
x359	1	5.66	37.8
x329	1	6.19	40.5
x299	1	6.85	43.8
W840x251	1	4.71	46.9
x226	1	5.49	49.5
x210	1	6.00	51.8
x193	1	6.73	54.2
x176	1	7.77	57.0
W760x582	1	3.19	20.8
x531	1	3.45	22.8
x484	1	3.74	24.8
x434	1	4.12	27.8
x389	1	4.59	30.5
x350	1	5.01	34.1
x314	1	5.75	36.5
x284	1	6.35	39.9
x257	1	7.03	43.3
W760x220	1	4.43	43.6
x196	1	5.28	46.1
x185	1	5.66	48.2
x173	1	6.18	49.9
x161	1	6.89	52.1
x147	1	7.79	54.5
x134	2	8.52	60.4
W690x802	1	2.15	12.9
x548	1	2.95	18.4
x500	1	3.19	20.2
x457	1	3.46	21.9
x419	1	3.71	24.0
x384	1	4.02	25.9
x350	1	4.40	28.0
x323	1	4.71	30.6
x289	1	5.24	34.0
x265	1	5.93	35.1
x240	1	6.50	38.5
x217	1	7.16	41.9
W690x192	1	4.55	41.7
x170	1	5.42	44.5
x152	1	6.02	49.3
x140	1	6.72	52.1
x125	1	7.76	55.2

Class	b_{el}/t limit	h / w limit
1	7.81	59.2
2	9.15	91.5
3	10.77	102.3

This table applies to major-axis bending. For seismic applications, see CSA S16-14 Clause 27.1.7. F_y = 345 MPa

CLASS OF SECTIONS IN BENDING

Table 5-1

ASTM A992, A572 grade 50

Designation	Class	b_{el}/t	h / w
W610x551	1	2.51	14.8
x498	1	2.72	16.3
x455	1	2.94	17.9
x415	1	3.18	19.4
x372	1	3.49	21.7
x341	1	3.79	23.5
x307	1	4.14	25.9
x285	1	4.43	27.8
x262	1	4.81	30.2
x241	1	5.31	32.0
x217	1	5.92	34.7
x195	1	6.70	37.2
x174	1	7.52	40.9
x155	2	8.53	45.1
W610x153	1	4.60	40.9
x140	1	5.18	43.7
x125	1	5.84	48.1
x113	1	6.59	51.2
x101	1	7.65	54.6
W610x92	1	5.97	52.6
x82	1	6.95	57.3
W530x409	1	2.94	16.2
x369	1	3.21	18.0
x332	1	3.54	19.8
x300	1	3.85	21.7
x272	1	4.22	23.8
x248	1	4.57	26.4
x219	1	5.45	27.4
x196	1	6.01	30.4
x182	1	6.45	33.0
x165	1	7.05	35.8
x150	1	7.68	39.6
W530x138	1	4.53	34.1
x123	1	5.00	38.3
x109	1	5.61	43.2
x101	1	6.03	46.1
x92	1	6.70	49.2
x82	2	7.86	52.8
x72	3	9.50	55.8
W530x85	1	5.03	48.7
x74	1	6.10	51.7
x66	1	7.24	56.4
W460x464	1	2.19	11.1
x421	1	2.38	12.0
x384	1	2.56	13.2
x349	1	2.76	14.5
x315	1	3.02	15.9
x286	1	3.28	17.5
x260	1	3.58	18.9
x235	1	3.92	20.8
x213	1	4.25	23.1
x193	1	4.64	25.2
x177	1	5.32	25.8
x158	1	5.94	28.5
x144	1	6.40	31.5
x128	1	7.19	35.1
x113	2	8.09	39.7
W460x106	1	4.71	34.0
x97	1	5.08	37.5
x89	1	5.42	40.7
x82	1	5.97	43.2
x74	1	6.55	47.6
W460x68	1	5.00	47.1
x60	1	5.75	53.6
x52	1	7.04	56.4
W410x149	1	5.30	25.6
x132	1	5.92	28.6
x114	1	6.76	32.9
x100	1	7.69	38.1
W410x85	1	4.97	34.9
x74	1	5.63	39.3
x67	1	6.22	43.3
x60	1	6.95	49.5
x54	2	8.12	50.8
W410x46	1	6.25	54.4
x39	2	7.95	59.6
W360x1299	1	1.70	3.20
x1202	1	1.81	3.37
x1086	1	1.82	4.09
x990	1	1.95	4.45
x900	1	2.08	4.84
x818	1	2.25	5.29
x744	1	2.43	5.76
x677	1	2.63	6.25
x634	1	2.75	6.72
x592	1	2.91	7.12
x551	1	3.09	7.61
x509	1	3.32	8.20
x463	1	3.59	8.94
x421	1	3.89	9.75
x382	1	4.23	10.7
x347	1	4.62	11.8
x314	1	5.06	12.8
x287	1	5.45	14.2
x262	1	5.98	15.2
x237	1	6.54	16.9
x216	1	7.11	18.5

Class	b_{el}/t limit	h / w limit
1	7.81	59.2
2	9.15	91.5
3	10.77	102.3

This table applies to major-axis bending. For seismic applications, see CSA S16-14 Clause 27.1.7. F_y = 345 MPa

CLASS OF SECTIONS IN BENDING

ASTM A992, A572 grade 50

Table 5-1

Designation	Class	b_{el}/t	h/w
W360x196	1	7.14	19.5
x179	1	7.80	21.3
x162	2	8.51	24.1
x147	3	9.34	26.0
x134	3	10.3	28.6
W360x122	1	5.92	24.6
x110	1	6.43	28.1
x101	1	6.97	30.5
x91	1	7.74	33.7
W360x79	1	6.10	34.1
x72	1	6.75	37.2
x64	1	7.52	41.6
W360x57	1	6.56	42.0
x51	1	7.37	46.1
x45	2	8.72	48.2
W360x39	1	5.98	51.0
x33	1	7.47	57.2
W310x500	1	2.26	6.14
x454	1	2.45	6.72
x415	1	2.66	7.14
x375	1	2.88	7.81
x342	1	3.12	8.49
x313	1	3.36	9.25
x283	1	3.65	10.3
x253	1	4.03	11.3
x226	1	4.45	12.5
x202	1	4.95	13.8
x179	1	5.57	15.4
x158	1	6.18	17.9
x143	1	6.75	19.8
x129	1	7.48	21.1
x118	2	8.21	23.2
x107	2	9.00	25.4
x97	3	9.90	28.0
W310x86	1	7.79	30.5
x79	2	8.70	31.5
W310x74	1	6.29	29.5
x67	1	6.99	32.6
x60	1	7.75	36.9
W310x52	1	6.33	38.2
x45	1	7.41	44.0
x39	2	8.51	50.1
W310x33	1	4.72	44.2
x28	1	5.73	48.5
x24	1	7.54	52.1
x21	2	8.86	57.2

Designation	Class	b_{el}/t	h/w
W250x167	1	4.17	11.7
x149	1	4.63	13.0
x131	1	5.20	14.6
x115	1	5.86	16.7
x101	1	6.56	18.9
x89	1	7.40	21.1
x80	2	8.17	23.9
x73	2	8.94	26.1
W250x67	1	6.50	25.3
x58	1	7.52	28.1
x49	3	9.18	30.4
W250x45	1	5.69	31.6
x39	1	6.56	36.3
x33	2	8.02	39.3
W250x28	1	5.10	37.5
x25	1	6.07	39.4
x22	1	7.39	41.4
x18	3	9.53	50.1
W200x100	1	4.43	12.5
x86	1	5.07	13.9
x71	1	5.92	17.8
x59	1	7.22	20.0
x52	2	8.10	22.9
x46	3	9.23	25.1
W200x42	1	7.03	25.2
x36	2	8.09	29.1
W200x31	1	6.57	29.6
x27	2	7.92	32.8
W200x22	1	6.38	30.6
x19	2	7.85	32.8
x15	3	9.62	44.1
W150x37	1	6.64	17.1
x30	2	8.23	21.0
x22	4	11.5	23.9
W150x24	1	4.95	21.1
x18	1	7.18	23.9
x14	2	9.09	32.3
x13	3	10.2	32.1
W130x28	1	5.87	15.8
x24	1	6.98	17.8
W100x19	1	5.85	12.5

Class	b_{el}/t limit	h/w limit
1	7.81	59.2
2	9.15	91.5
3	10.77	102.3

This table applies to major-axis bending. For seismic applications, see CSA S16-14 Clause 27.1.7. F_y = 345 MPa

FACTORED RESISTANCE OF BEAMS

General

The following pages contain the *Beam Selection Tables*, which can be used to select flexural members, and the *Beam Load Tables* for estimating maximum beam reactions. The *Beam Selection Tables* facilitate the proportioning of flexural members subject to forces and moments determined by elastic analysis. The *Beam Load Tables* list total uniformly distributed factored loads for laterally supported beams. W-shape sections that are commonly used and readily available at the time of preparation of this Handbook are highlighted in yellow.

When using these tables, factored moments or forces must be equal to or less than the appropriate factored resistances, V_r, M_r or M_r' given in the tables. V_r values for uncoped beams are tabulated. A general discussion on coped beams is provided under *Supported Beams with Copes* in the section on *Double-Angle Beam Connections* in Part 3.

Beam Selection Tables

Tables

The *Beam Selection Tables* list beam sizes in descending order of their factored moment resistance M_r (shown in bold) based on full lateral support (CSA S16-14 Clause 13.5). Listed beams include W-shapes and S-shapes normally used as beams, and all C-shapes (channels) produced in Canada. Tables for W-shapes and S-shapes are based on F_y = 345 MPa, corresponding to ASTM A992 and A572 grade 50, while those for C-shapes are based on G40.21-350W with F_y = 350 MPa. Data for welded wide-flange (WWF) beams is no longer provided in this Handbook.

Shapes are grouped, and those shown in bold type are the lightest in each group (i.e. with the largest ratio of moment resistance to mass). Other shapes listed below a bold-type section are heavier sections, but may be more suitable sections when depth limitations dictate a shallower beam, or when the shear resistance of a coped beam influences the beam selection.

For each beam size, the tables list the maximum unsupported length L_u for which the factored moment resistance M_r is applicable. In addition, the tables list the factored moment resistance M_r' for laterally unsupported beams (Clause 13.6) for selected values of the unbraced beam length greater than L_u. For other values of unbraced length greater than L_u, M_r' can be interpolated.

In place of a more accurate analysis, the destabilizing effect of beams supporting gravity loads acting above the shear centre can be taken into account by increasing the effective unbraced length by a factor of 1.2 or 1.4 (depending on the end conditions), as specified in Clause 13.6.

The following items are included in the table:

V_r = factored shear resistance (kN) = $\phi A_w F_s$ (Clause 13.4.1.1)

I_x = moment of inertia about X-X axis (10^6 mm^4)

b = flange width (mm)

L_u = maximum unsupported beam length for which M_r is applicable (mm)

M_r = factored moment resistance for laterally supported members (kN·m)

= $\phi Z_x F_y$ for Class 1 and Class 2 sections

= $\phi S_x F_y$ for Class 3 sections, Clause 13.5

= $\phi S_e F_y$ for Class 4 sections, Clause 13.5(c)(iii)

M_r' = factored moment resistance for tabulated unbraced beam length when greater than L_u, and computed according to Clause 13.6 using ω_2 = 1.0 in the expression for M_u (kN·m)

Least Mass Design – Elastically Analyzed Structures

Compute the maximum bending moment in the beam under factored loads M_f and the required moment of inertia I_{reqd} to meet the deflection limit using the specified loads. (For I_{reqd}, see *Deflection of Flexural Members* in Part 5).

For a laterally supported beam, proceed up the M_r column until a value of $M_r > M_f$ is obtained. Any beam above will satisfy the factored moment requirement. Check to ensure that $V_r > V_f$, the maximum factored beam shear, that $I_x > I_{reqd}$, and that L_u is greater than the maximum unsupported beam length.

For a laterally unsupported beam, proceed up the M_r column until a value of $M_r > M_f$ is reached. Then move to the right across the table to the column headed by the unsupported length (or the first listed unsupported length greater than that required) to obtain a value of M_r'. Proceed up this column comparing a few beams that have an $M_r' > M_f$ and choose the appropriate section. Check $V_r > V_f$ and $I_x > I_{reqd}$ if a deflection check is necessary.

Plastically Analyzed Structures

For beams analyzed plastically (S16-14 Clause 8.3.2), the *Beam Selection Tables* may be used to facilitate the selection of a beam size as follows:

Proceed up the M_r column until a value of $M_r > M_f$ is obtained.

Any beam above will satisfy the factored moment requirement, provided the beam is a Class 1 section. (See Table 5-1 for the Class of Sections in Bending for various steel grades.)

Check to ensure that 0.8 $V_r > V_f$, the maximum factored beam shear (Clause 13.4.2).

Provide suitable lateral bracing (Clause 13.7), etc.

Beam Load Tables — W-Shapes

Loads

The *Beam Load Tables* list total uniformly distributed factored loads for simply supported W-shape beams with the top flange fully supported (i.e. the unsupported length of beam is less than or equal to L_u). Tables are based on F_y = 345 MPa for steel grades ASTM A992 and A572 grade 50. Tabulated values may also be used for beams produced to CSA G40.21-350W, although grade 350W does not appear in the table headings. To obtain the net supported load (factored), the beam factored dead load should be deducted from the total tabulated load.

For laterally supported beams, the *Beam Load Tables* may also be used to estimate loads for other loading conditions by dividing the tabulated values by the coefficient of the "Equivalent Tabular Load" value for the particular loading condition. (See *Beam Diagrams and Formulas* in Part 5.) Thus, for a simple beam, laterally supported and carrying equal concentrated loads at the third points (loading condition 9), each factored concentrated load is

⅜ of the tabulated uniformly distributed factored load, and the total load is ¾ of the tabulated load for the same span.

For steel grades with a value of F_y less than that used in the table, reduce the tabulated values by the ratio of the yield-stress values.

The *Beam Load Tables* (sometimes referred to as "book loads") are frequently used to estimate the maximum reactions for the design of connections, when beam reactions are not provided on the structural design drawings. Since compositely designed beams possess a greater flexural capacity, these tables should not be used for composite beams.

Vertical Deflection

The column headed "Approximate Deflection" lists the approximate theoretical mid-span deflection, at an assumed bending stress level of 240 MPa for ASTM A992 and A572 grade 50 steels, for beams of various spans designed to support the tabulated factored loads.

The listed deflections are based on the nominal depth of the beam, and are calculated using the formula:

$$\Delta = \frac{5}{384}\frac{W L^3}{E I}$$

For E = 200 000 MPa and an assumed bending unit stress of 240 MPa this formula reduces to:

$$\Delta = \frac{250 \times 10^{-6} \times L^2}{d}$$ where:

Δ = deflection (mm)

W = total uniform load including the dead load of the beam (kN)

L = beam span (mm)

E = modulus of elasticity (MPa)

I = moment of inertia of beam (mm^4)

d = depth of beam (mm)

More accurate deflections can be determined by multiplying the approximate deflection values listed by the ratio of actual bending stress to the assumed unit bending stress of 240 MPa for ASTM A992 and A572 grade 50 steels. (See also *Deflection of Flexural Members* in Part 5)

Web Shear

For beams with very short spans, high end shear and coped flanges at the supports, the loads for beams may be limited by the shear capacity of the web rather than by the bending capacity of the section. The designer should consider the effect of copes on the load-carrying capacity of beams when selecting appropriate member sizes.

Both the depth and length of copes can vary considerably depending on the relative size and elevation of intersecting beams. The *Beam Load Tables* list the factored shear resistance V_r for uncoped W-shape beams (S16-14 Clause 13.4.1.1). The factored shear resistance of singly and doubly coped beams should be adjusted for the depth and length of the copes, to account for possible non-uniform shear distribution across the effective web depth (including Clauses 13.4.3 and 13.11) and to account for local web buckling.

Web Crippling and Yielding

Bearing stiffeners are required when the factored compressive resistance (S16-14 Clause 14.3.2) of the web is exceeded. For common light beam sizes, the unstiffened bearing resistance is usually governed by web crippling, except for very short bearing lengths when web yielding may govern.

The *Beam Load Tables* list values of R (kN), the maximum factored end reaction for 100 mm of bearing based on web yielding according to Clause 14.3.2(b)(i), and values of the increment in bearing resistance, G (kN), per 10 millimetres of bearing length. For steels with a minimum specified yield stress other than F_y = 345 MPa, values of R and G can be computed by multiplying the values listed by the ratio $F_y / 345$. Also listed is the value of B_r' (kN), the factored bearing resistance based on web crippling (Clause 14.3.2(b)(ii)).

Proper lateral support must be provided for the top flanges of beams at the reaction point to ensure that the web crippling strength is not decreased.

Properties and Dimensions

The properties and dimensions listed in the *Beam Load Tables* for rolled shapes include the beam depth d, the flange width b, the flange thickness t, and the web thickness w (all in millimetres). Dimensions t and w are required for calculating the compressive resistance of the web (yielding or crippling) according to Clause 14.3.2.

Examples

1. Given:

Design a simply-supported beam spanning 8 m to carry a uniformly distributed load of 15 kN/m specified live load and 7 kN/m specified dead load. The dead load includes an assumed beam dead load of 0.7 kN/m. Live load deflection is limited to L/300. The beam frames into supporting members, and the beam is laterally supported and uncoped. Use ASTM A992 steel.

Solution:

(a) Using the *Beam Selection Tables* — Elastic Analysis:

Factored load = $\alpha_D D + \alpha_L L = (1.25 \times 7) + (1.50 \times 15) = 31.3$ kN/m

M_f (factored load moment) = $wL^2 / 8 = 31.3 \times 8^2 / 8 = 250$ kN·m

V_f (factored end shear) = $wL/2 = 31.3 \times 8/2 = 125$ kN

Compute I_{reqd} to meet the deflection limit (see *Deflection of Flexural Members*)

For UDL, B_d = 1.0; from Figure 5-2, for $L/\Delta = 300$, $C_d = 1.25 \times 10^6$

$I_{reqd} = W C_d B_d = 15 \times 8 \times 1.25 \times 10^6 \times 1.0 = 150 \times 10^6$ mm^4

From the *Beam Selection Table*, select a W410x46 beam.

M_r = 274 kN·m > 250 kN·m V_r = 578 kN > 125 kN

$I_x = 156 \times 10^6$ mm$^4 > 150 \times 10^6$ mm^4. The W410x46 beam is adequate.

(b) Using the *Beam Load Tables*:

Total factored load, $W_f = 31.3 \times 8 = 250$ kN

End reaction is $V_f = 250 / 2 = 125$ kN

From the *Beam Load Tables*, select a W410x46 beam.

W_r (8 000) = 274 kN > 250 kN, V_r = 578 kN > 125 kN (uncoped)

(When the beam is bearing on supports rather than framing into supporting members, it is necessary to check that the factored bearing resistance is greater than or equal to the factored end reaction.)

Approximate deflection listed at an assumed stress of 240 MPa = 39 mm

$$\text{Stress at specified live load} = \frac{M_{Live}}{S_x} = \frac{15}{31.3} \times \frac{250 \times 10^6}{772 \times 10^3} = 155 \text{ MPa}$$

Live load deflection = 39 × 155 / 240 = 25.2 mm

$L / 300$ = 8 000 / 300 = 26.7 mm > 25.2 mm. The W410x46 beam is adequate.

2. Given:

Same as in Example (1), except that the beam is laterally supported at mid-span and at the ends of the beam only. In this case, the uniformly distributed gravity loading is applied on the top flange but the manner of load transfer does not provide lateral support (not a common situation).

Solution:

The point of application of the loading is located above the shear centre. In the absence of a more accurate analysis, the effective length is taken equal to 1.2 times the unbraced length (assuming pin-ended segments between the bracing points) in accordance with S16-14 Clause 13.6:

Effective unbraced length = 1.2 (8 000 / 2) = 4 800 mm.

As in Example (1), consider a W410x46 beam on the basis of M_r and check M_r' for an unbraced length of 4 800 mm ≈ 5 000 mm. Using the *Beam Selection Table*:

M_r' (5 000) = 99.9 kN·m < 250 kN·m

The W410x46 is not adequate. Check further up the table for the lightest section with M_r' > 250 kN·m for an unbraced length of 5 000 mm.

For a W360x64 beam, M_r' (5 000) = 273 kN·m

A more accurate value could be obtained by linear interpolation between 4 500 and 5 000 mm, if desired:

M_r' (4 800) = 281 kN·m > 250 kN·m

V_r = 548 kN > 125 kN (uncoped)

$I_x = 178 \times 10^6$ mm^4 > 150×10^6 The W360x64 beam is adequate.

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m)					
						Unbraced length (mm)					
	kN	10^6 mm^4	mm	mm	$\leq L_u$	4 500	5 000	5 500	6 000	6 500	7 000
# **W920x1377**	17 200	30 300	473	9 570	**21 000**	—	—	—	—	—	—
# **W920x1269**	14 300	29 000	461	9 170	**19 800**	—	—	—	—	—	—
# **W920x1194**	13 400	26 900	457	8 800	**18 600**	—	—	—	—	—	—
# **W920x1077**	12 000	23 800	451	8 260	**16 600**	—	—	—	—	—	—
# **W1000x976**	11 400	23 500	428	7 190	**15 600**	—	—	—	—	—	—
# **W920x970**	10 700	21 000	446	7 790	**14 800**	—	—	—	—	—	—
# **W1000x883**	10 200	21 000	424	6 850	**14 100**	—	—	—	—	—	14 000
W920x787	8 470	16 500	437	7 050	**11 800**	—	—	—	—	—	—
# **W1000x748**	8 540	17 300	417	6 390	**11 800**	—	—	—	—	11 700	11 500
W920x725	7 800	14 900	434	6 830	**10 800**	—	—	—	—	—	10 700
W1000x642	7 300	14 500	412	6 060	**9 970**	—	—	—	—	9 790	9 580
W920x656	6 980	13 400	431	6 590	9 720	—	—	—	—	—	9 570
# W690x802	8 460	10 600	387	7 980	9 590	—	—	—	—	—	—
W1000x591	6 610	13 300	409	5 920	**9 160**	—	—	—	9 130	8 930	8 730
# **W1000x584**	7 790	12 500	314	4 530	**8 690**	—	8 480	8 240	8 000	7 760	7 520
W920x588	6 190	11 800	427	6 370	8 630	—	—	—	—	8 590	8 420
W1000x554	6 240	12 300	408	5 810	**8 540**	—	—	—	8 470	8 280	8 080
W1000x539	5 990	12 000	407	5 790	**8 320**	—	—	—	8 240	8 050	7 860
W1100x499	5 960	12 900	405	5 490	**8 260**	—	—	8 250	8 050	7 830	7 600
W840x576	5 990	10 100	411	6 320	7 950	—	—	—	—	7 900	7 750
W920x537	5 620	10 700	425	6 210	7 860	—	—	—	—	7 760	7 600
W1000x483	5 310	10 700	404	5 650	**7 420**	—	—	—	7 300	7 120	6 930
W760x582	5 960	8 620	396	6 460	7 390	—	—	—	—	7 380	7 260
# W1000x494	6 580	10 300	309	4 280	7 270	7 170	6 950	6 720	6 480	6 250	6 010
W840x527	5 460	9 150	409	6 150	7 230	—	—	—	—	7 140	6 990
W1100x433	5 000	11 300	402	5 400	**7 200**	—	—	7 170	6 970	6 770	6 560
W1000x486	6 370	10 200	308	4 270	7 200	7 100	6 880	6 660	6 420	6 190	5 950
W920x491	5 080	9 660	422	6 070	7 140	—	—	—	—	7 010	6 850
W1000x443	4 890	9 670	402	5 530	6 770	—	—	—	6 610	6 440	6 250
W760x531	5 380	7 770	393	6 230	6 710	—	—	—	—	6 640	6 520
W920x449	4 660	8 750	423	6 000	6 490	—	—	—	—	6 350	6 200

May be produced to ASTM A913 Grade 50, for which this table also applies.

Nominal mass	Factored moment resistance M_r' (kN·m)									Imperial designation
	Unbraced length (mm)									
kg/m	8 000	9 000	10 000	11 000	12 000	14 000	16 000	18 000	20 000	
1 377	—	—	20 800	20 400	20 100	19 300	18 500	17 800	17 000	**W36x925**
1 269	—	—	19 500	19 100	18 700	18 000	17 200	16 500	15 700	**W36x853**
1 194	—	18 500	18 100	17 700	17 300	16 600	15 800	15 100	14 300	**W36x802**
1 077	—	16 300	15 900	15 500	15 100	14 400	13 600	12 900	12 200	**W36x723**
976	15 300	14 800	14 300	13 900	13 500	12 600	11 700	10 800	9 830	**W40x655**
970	14 700	14 300	14 000	13 600	13 200	12 500	11 700	11 000	10 200	**W36x652**
883	13 600	13 100	12 700	12 200	11 800	10 900	10 000	9 070	8 080	**W40x593**
787	11 400	11 100	10 700	10 300	9 950	9 210	8 480	7 710	6 860	**W36x529**
748	11 100	10 600	10 200	9 760	9 320	8 450	7 490	6 550	5 830	**W40x503**
725	10 300	9 980	9 610	9 240	8 880	8 150	7 420	6 570	5 840	**W36x487**
642	9 160	8 730	8 300	7 860	7 420	6 520	5 550	4 830	4 280	**W40x431**
656	9 220	8 850	8 490	8 120	7 750	7 020	6 230	5 430	4 820	W36x441
802	—	9 380	9 160	8 950	8 740	8 330	7 920	7 510	7 100	W27x539
591	8 320	7 900	7 470	7 040	6 610	5 650	4 790	4 160	3 680	**W40x397**
584	7 030	6 540	6 050	5 500	4 960	4 150	3 580	3 150	2 810	**W40x392**
588	8 070	7 720	7 360	7 000	6 640	5 920	5 070	4 410	3 900	W36x395
554	7 670	7 250	6 830	6 400	5 970	4 980	4 220	3 660	3 230	**W40x372**
539	7 460	7 040	6 620	6 200	5 770	4 790	4 050	3 510	3 100	**W40x362**
499	7 120	6 620	6 100	5 570	4 920	3 960	3 310	2 840	2 490	**W44x335**
576	7 440	7 130	6 820	6 510	6 210	5 590	4 890	4 270	3 790	W33x387
537	7 260	6 910	6 560	6 200	5 840	5 070	4 290	3 710	3 280	W36x361
483	6 540	6 140	5 730	5 310	4 860	3 940	3 320	2 860	2 520	**W40x324**
582	7 000	6 750	6 500	6 250	6 000	5 500	5 010	4 420	3 940	W30x391
494	5 520	5 040	4 470	3 970	3 570	2 970	2 550	2 240	1 990	W40x331
527	6 690	6 390	6 080	5 780	5 470	4 860	4 140	3 610	3 200	W33x354
433	6 110	5 640	5 140	4 580	4 010	3 190	2 650	2 260	1 970	**W44x290**
486	5 460	4 980	4 410	3 910	3 510	2 930	2 510	2 200	1 960	W40x327
491	6 520	6 180	5 840	5 480	5 130	4 320	3 640	3 140	2 770	W36x330
443	5 880	5 480	5 070	4 660	4 160	3 360	2 810	2 420	2 130	W40x297
531	6 270	6 020	5 770	5 520	5 270	4 780	4 230	3 700	3 300	W30x357
449	5 880	5 550	5 220	4 870	4 520	3 720	3 120	2 680	2 360	W36x302

Note: For unbraced beam segments loaded above the shear centre, see CSA S16-14 Clause 13.6.

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m)					
					Unbraced length (mm)						
	kN	10^6 mm^4	mm	mm	≤ L_u	4 000	4 500	5 000	5 500	6 000	7 000
W1100x390	4 510	10 100	400	5 310	**6 460**	—	—	—	6 390	6 210	5 820
W840x473	4 830	8 130	406	5 980	6 460	—	—	—	—	6 450	6 170
W1000x438	5 660	9 090	305	4 160	6 430	—	6 290	6 080	5 860	5 630	5 170
W1000x412	4 360	9 100	402	5 530	6 370	—	—	—	—	6 220	5 880
W690x548	5 550	6 730	372	6 400	6 330	—	—	—	—	—	6 210
W1000x415	5 430	8 530	304	4 080	6 090	—	5 920	5 710	5 490	5 260	4 800
W920x420	4 350	8 130	422	5 920	6 050	—	—	—	—	6 030	5 750
W760x484	4 890	6 990	390	6 040	6 050	—	—	—	—	—	5 830
W840x433	4 430	7 360	404	5 850	5 870	—	—	—	—	5 830	5 560
W610x551	5 620	5 570	347	6 620	5 780	—	—	—	—	—	5 710
W1000x393	5 080	8 080	303	4 050	5 740	—	5 570	5 370	5 160	4 940	4 480
W690x500	5 000	6 060	369	6 140	5 740	—	—	—	—	—	5 570
W1000x371	3 890	8 140	400	5 440	**5 710**	—	—	—	5 700	5 550	5 220
W1100x343	3 850	8 670	400	5 230	**5 620**	—	—	—	5 530	5 370	5 010
W920x390	4 090	7 420	420	5 810	5 560	—	—	—	—	5 510	5 230
W760x434	4 320	6 190	387	5 830	5 400	—	—	—	—	5 360	5 130
W920x381	4 760	6 960	310	4 250	5 280	—	5 190	5 020	4 840	4 660	4 280
W840x392	3 970	6 600	401	5 720	5 280	—	—	—	—	5 210	4 950
W920x368	3 870	6 920	419	5 750	5 220	—	—	—	—	5 150	4 880
W690x457	4 550	5 470	367	5 920	5 220	—	—	—	—	5 200	5 000
W610x498	5 030	4 950	343	6 230	5 190	—	—	—	—	—	5 060
W1000x350	4 360	7 230	302	4 010	5 150	—	4 980	4 780	4 580	4 370	3 940
W1000x321	3 250	6 960	400	5 360	**4 910**	—	—	—	4 870	4 730	4 440
W920x344	3 670	6 450	418	5 680	4 870	—	—	—	—	4 800	4 540
W760x389	3 880	5 450	385	5 640	4 810	—	—	—	—	4 730	4 510
W840x359	3 750	5 920	403	5 630	4 780	—	—	—	—	4 700	4 450
W920x345	4 270	6 250	308	4 170	4 750	—	4 650	4 480	4 310	4 130	3 760
W690x419	4 100	4 950	364	5 730	4 750	—	—	—	—	4 700	4 510
W610x455	4 520	4 440	340	5 940	4 690	—	—	—	—	4 680	4 520
W1000x314	3 910	6 440	300	3 910	**4 630**	4 600	4 430	4 240	4 050	3 850	3 420
W1000x296	3 230	6 200	400	5 230	**4 440**	—	—	—	4 370	4 240	3 960
W840x329	3 480	5 360	401	5 530	4 350	—	—	—	—	4 240	4 010
W690x384	3 760	4 490	362	5 550	4 350	—	—	—	—	4 260	4 070
W760x350	3 440	4 870	382	5 510	4 320	—	—	—	—	4 220	4 000
W610x415	4 100	4 000	338	5 700	4 250	—	—	—	—	4 210	4 050
W920x313	4 030	5 480	309	4 060	4 220	—	4 090	3 940	3 770	3 600	3 240
W1000x272	3 250	5 540	300	3 870	**3 970**	3 940	3 780	3 620	3 440	3 260	2 860
W840x299	3 190	4 800	400	5 430	3 940	—	—	—	3 930	3 820	3 600
W690x350	3 450	4 030	360	5 410	3 910	—	—	—	3 900	3 810	3 620
W920x289	3 690	5 040	308	4 030	3 880	—	3 750	3 600	3 440	3 280	2 930
W460x464	4 490	2 900	305	7 060	3 850	—	—	—	—	—	—
W760x314	3 170	4 290	384	5 420	3 820	—	—	—	3 800	3 710	3 500
W610x372	3 620	3 530	335	5 450	3 790	—	—	—	3 780	3 700	3 550
W530x409	3 890	3 170	327	6 030	3 760	—	—	—	—	—	3 640
W920x271	3 480	4 710	307	3 970	**3 660**	—	3 520	3 380	3 220	3 060	2 710
W690x323	3 120	3 710	359	5 300	3 630	—	—	—	3 600	3 510	3 330

Nominal mass	Factored moment resistance M_r' (kN·m)									Imperial designation
	Unbraced length (mm)									
kg/m	8 000	9 000	10 000	11 000	12 000	14 000	16 000	18 000	20 000	
390	5 400	4 950	4 470	3 890	3 390	2 690	2 220	1 880	1 640	**W44x262**
473	5 890	5 590	5 290	4 990	4 680	4 010	3 400	2 950	2 610	W33x318
438	4 700	4 190	3 640	3 220	2 890	2 400	2 050	1 790	1 600	W40x294
412	5 510	5 130	4 730	4 330	3 830	3 080	2 570	2 200	1 930	W40x277
548	6 010	5 800	5 600	5 400	5 200	4 810	4 410	3 960	3 540	W27x368
415	4 320	3 780	3 280	2 890	2 590	2 140	1 830	1 600	1 420	W40x278
420	5 440	5 120	4 790	4 440	4 100	3 310	2 760	2 370	2 080	W36x282
484	5 580	5 340	5 090	4 850	4 600	4 110	3 540	3 100	2 760	W30x326
433	5 280	4 990	4 690	4 400	4 100	3 410	2 880	2 490	2 200	W33x291
551	5 550	5 390	5 230	5 070	4 910	4 590	4 280	3 970	3 600	W24x370
393	4 020	3 470	3 000	2 650	2 370	1 950	1 670	1 450	1 290	W40x264
500	5 370	5 170	4 970	4 770	4 570	4 180	3 770	3 310	2 960	W27x336
371	4 880	4 510	4 120	3 700	3 240	2 590	2 150	1 840	1 600	**W40x249**
343	4 610	4 190	3 730	3 180	2 760	2 160	1 770	1 490	1 290	**W44x230**
390	4 940	4 620	4 290	3 960	3 590	2 870	2 380	2 040	1 780	W36x262
434	4 890	4 650	4 410	4 160	3 920	3 380	2 880	2 520	2 230	W30x292
381	3 880	3 470	3 000	2 640	2 360	1 950	1 660	1 450	1 290	W36x256
392	4 680	4 400	4 110	3 820	3 520	2 850	2 400	2 070	1 820	W33x263
368	4 590	4 290	3 970	3 630	3 240	2 580	2 140	1 820	1 590	W36x247
457	4 800	4 610	4 410	4 210	4 010	3 620	3 170	2 790	2 490	W27x307
498	4 900	4 740	4 580	4 420	4 260	3 950	3 640	3 280	2 940	W24x335
350	3 490	2 950	2 540	2 230	1 980	1 630	1 380	1 200	1 070	W40x235
321	4 120	3 780	3 420	2 980	2 600	2 050	1 690	1 430	1 240	**W40x215**
344	4 260	3 960	3 640	3 320	2 920	2 310	1 910	1 620	1 410	W36x231
389	4 270	4 040	3 790	3 550	3 300	2 740	2 330	2 030	1 790	W30x261
359	4 190	3 910	3 630	3 340	3 000	2 420	2 020	1 740	1 520	W33x241
345	3 380	2 930	2 530	2 220	1 980	1 630	1 380	1 200	1 070	W36x232
419	4 310	4 110	3 910	3 720	3 520	3 120	2 680	2 350	2 090	W27x281
455	4 360	4 200	4 040	3 880	3 730	3 420	3 100	2 740	2 450	W24x306
314	2 940	2 460	2 110	1 840	1 640	1 340	1 130	979	865	**W40x211**
296	3 650	3 320	2 960	2 530	2 190	1 720	1 410	1 190	1 030	**W40x199**
329	3 760	3 490	3 210	2 930	2 580	2 060	1 720	1 470	1 290	W33x221
384	3 880	3 680	3 480	3 290	3 090	2 650	2 270	1 990	1 770	W27x258
350	3 770	3 540	3 300	3 060	2 800	2 290	1 930	1 680	1 480	W30x235
415	3 890	3 730	3 570	3 420	3 260	2 960	2 600	2 290	2 050	W24x279
313	2 860	2 400	2 060	1 800	1 600	1 310	1 100	958	847	W36x210
272	2 390	1 990	1 690	1 470	1 300	1 050	883	761	670	**W40x183**
299	3 350	3 090	2 820	2 520	2 200	1 750	1 450	1 240	1 080	W33x201
350	3 430	3 240	3 040	2 850	2 660	2 210	1 890	1 650	1 470	W27x235
289	2 550	2 130	1 820	1 580	1 400	1 140	960	831	732	W36x194
464	3 760	3 670	3 580	3 490	3 400	3 230	3 050	2 870	2 690	W18x311
314	3 290	3 060	2 830	2 600	2 310	1 870	1 570	1 360	1 190	W30x211
372	3 390	3 230	3 080	2 920	2 770	2 440	2 110	1 860	1 660	W24x250
409	3 520	3 400	3 280	3 160	3 050	2 810	2 580	2 310	2 070	W21x275
271	2 310	1 920	1 640	1 420	1 260	1 020	857	739	651	**W36x182**
323	3 140	2 940	2 750	2 560	2 340	1 930	1 650	1 430	1 270	W27x217

Note: For unbraced beam segments loaded above the shear centre, see CSA S16-14 Clause 13.6.

BEAM SELECTION TABLE
W Shapes

ASTM A992, A572 Grade 50
F_y = 345 MPa

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m)					
					Unbraced length (mm)						
	kN	10^6 mm^4	mm	mm	≤ L_u	3 500	4 000	4 500	5 000	5 500	6 000
W1000x249	3 220	4 810	300	3 740	**3 510**	—	3 440	3 290	3 130	2 960	2 780
W760x284	2 870	3 830	382	5 300	3 450	—	—	—	—	3 410	3 320
W610x341	3 310	3 180	333	5 250	3 450	—	—	—	—	3 410	3 330
W460x421	4 050	2 570	302	6 600	3 450	—	—	—	—	—	—
W920x253	3 260	4 370	306	3 950	3 380	—	3 370	3 240	3 110	2 960	2 800
W530x369	3 450	2 810	324	5 720	3 350	—	—	—	—	—	3 320
W840x251	2 990	3 860	292	3 890	3 200	—	3 170	3 050	2 920	2 790	2 650
W690x289	2 780	3 260	356	5 160	3 200	—	—	—	—	3 140	3 060
W920x238	3 090	4 060	305	3 890	**3 170**	—	3 140	3 020	2 880	2 740	2 590
W460x384	3 630	2 290	299	6 190	3 110	—	—	—	—	—	—
W760x257	2 630	3 430	381	5 230	3 100	—	—	—	—	3 050	2 960
W610x307	2 960	2 840	330	5 080	3 080	—	—	—	—	3 020	2 950
W1000x222	3 000	4 080	300	3 590	**3 040**	—	2 940	2 800	2 650	2 480	2 310
W530x332	3 090	2 480	322	5 430	3 000	—	—	—	—	2 990	2 930
W920x223	2 970	3 760	304	3 830	2 960	—	2 920	2 800	2 670	2 530	2 380
W690x265	2 660	2 920	358	5 060	2 900	—	—	—	—	2 830	2 750
W610x285	2 730	2 610	329	4 980	2 850	—	—	—	2 840	2 770	2 700
W840x226	2 810	3 400	294	3 830	2 840	—	2 810	2 700	2 570	2 450	2 310
W460x349	3 230	2 040	296	5 840	2 800	—	—	—	—	—	2 780
W530x300	2 770	2 210	319	5 210	2 690	—	—	—	—	2 660	2 600
W840x210	2 670	3 110	293	3 770	**2 620**	—	2 570	2 460	2 350	2 220	2 090
W690x240	2 410	2 630	356	4 960	2 620	—	—	—	2 610	2 540	2 460
W610x262	2 500	2 360	327	4 850	2 590	—	—	—	2 570	2 500	2 430
W920x201	2 710	3 250	304	3 720	**2 590**	—	2 530	2 420	2 300	2 170	2 030
W760x220	2 630	2 780	266	3 570	2 540	—	2 460	2 350	2 230	2 120	1 990
W460x315	2 890	1 800	293	5 480	2 490	—	—	—	—	—	2 440
W530x272	2 490	1 970	317	5 020	2 420	—	—	—	—	2 370	2 310
W610x241	2 330	2 150	329	4 790	2 380	—	—	—	2 350	2 290	2 220
W840x193	2 530	2 780	292	3 690	**2 370**	—	2 310	2 200	2 090	1 970	1 850
W690x217	2 190	2 360	355	4 890	2 360	—	—	—	2 350	2 280	2 210
W460x286	2 590	1 610	291	5 220	2 250	—	—	—	—	2 220	2 180
W760x196	2 460	2 400	268	3 500	2 230	—	2 130	2 030	1 920	1 810	1 690
W530x248	2 220	1 770	315	4 880	2 190	—	—	—	2 180	2 120	2 070
W610x217	2 120	1 910	328	4 680	2 130	—	—	—	2 090	2 030	1 960
W840x176	2 300	2 460	292	3 610	**2 110**	—	2 050	1 950	1 840	1 730	1 610
W760x185	2 340	2 230	267	3 450	2 080	2 070	1 980	1 880	1 780	1 670	1 550
W460x260	2 360	1 440	289	4 980	2 030	—	—	—	—	1 980	1 940
W690x192	2 230	1 980	254	3 440	2 010	2 000	1 910	1 830	1 730	1 640	1 540
W760x173	2 250	2 060	267	3 410	**1 930**	1 910	1 830	1 730	1 630	1 520	1 410
W530x219	2 100	1 510	318	4 720	1 900	—	—	—	1 870	1 820	1 760
W610x195	1 960	1 680	327	4 570	1 880	—	—	—	1 840	1 780	1 710
W460x235	2 120	1 270	287	4 770	1 810	—	—	—	1 790	1 750	1 710

Sections highlighted in yellow are commonly used sizes and are generally readily available.

ASTM A992, A572 Grade 50 — BEAM SELECTION TABLE

F_y = 345 MPa — W Shapes

Nominal mass	Factored moment resistance M_r' (kN·m)									Imperial designation
	Unbraced length (mm)									
kg/m	7 000	8 000	9 000	10 000	11 000	12 000	14 000	16 000	18 000	
249	2 400	1 940	1 600	1 360	1 170	1 030	831	694	596	**W40x167**
284	3 120	2 910	2 690	2 460	2 210	1 940	1 560	1 310	1 120	W30x191
341	3 180	3 020	2 870	2 710	2 560	2 400	2 050	1 760	1 550	W24x229
421	3 410	3 320	3 230	3 140	3 050	2 970	2 790	2 620	2 450	W18x283
253	2 470	2 080	1 720	1 460	1 270	1 120	902	756	650	W36x170
369	3 200	3 080	2 960	2 850	2 730	2 620	2 390	2 130	1 880	W21x248
251	2 350	2 010	1 680	1 440	1 260	1 120	911	770	668	W33x169
289	2 880	2 700	2 510	2 320	2 130	1 900	1 560	1 320	1 150	W27x194
238	2 270	1 870	1 550	1 310	1 130	996	800	668	573	**W36x160**
384	3 030	2 940	2 860	2 770	2 680	2 600	2 420	2 250	2 080	W18x258
257	2 780	2 580	2 360	2 140	1 880	1 650	1 320	1 100	938	W30x173
307	2 800	2 640	2 490	2 340	2 180	2 020	1 690	1 450	1 270	W24x207
222	1 910	1 520	1 250	1 050	906	794	634	527	451	**W40x149**
332	2 810	2 700	2 580	2 460	2 340	2 230	2 000	1 730	1 530	W21x223
223	2 060	1 680	1 380	1 170	1 000	881	705	587	502	W36x150
265	2 580	2 400	2 220	2 030	1 810	1 600	1 310	1 100	957	W27x178
285	2 550	2 400	2 250	2 100	1 940	1 760	1 470	1 260	1 100	W24x192
226	2 020	1 680	1 400	1 190	1 030	914	741	623	537	W33x152
349	2 700	2 610	2 520	2 440	2 350	2 270	2 100	1 930	1 730	W18x234
300	2 480	2 370	2 250	2 130	2 020	1 900	1 650	1 420	1 250	W21x201
210	1 810	1 470	1 220	1 040	899	792	639	535	461	**W33x141**
240	2 300	2 120	1 940	1 760	1 530	1 360	1 100	924	797	W27x161
262	2 290	2 140	1 990	1 830	1 670	1 490	1 240	1 060	924	W24x176
201	1 720	1 360	1 120	940	807	705	560	463	394	**W36x135**
220	1 740	1 430	1 210	1 050	922	823	679	579	504	W30x148
315	2 360	2 270	2 190	2 100	2 020	1 930	1 770	1 580	1 400	W18x211
272	2 190	2 080	1 960	1 850	1 730	1 620	1 360	1 170	1 030	W21x182
241	2 080	1 930	1 780	1 630	1 450	1 300	1 070	911	795	W24x162
193	1 580	1 260	1 040	877	758	666	534	445	382	**W33x130**
217	2 050	1 880	1 710	1 510	1 310	1 150	930	778	669	W27x146
286	2 090	2 010	1 920	1 840	1 760	1 670	1 510	1 310	1 160	W18x192
196	1 430	1 160	973	835	731	650	532	451	391	W30x132
248	1 960	1 840	1 730	1 610	1 500	1 360	1 140	979	859	W21x166
217	1 820	1 680	1 530	1 370	1 200	1 070	878	745	647	W24x146
176	1 330	1 060	868	731	629	551	439	364	311	**W33x118**
185	1 280	1 040	867	743	649	576	470	397	344	W30x124
260	1 850	1 770	1 690	1 600	1 520	1 440	1 250	1 080	958	W18x175
192	1 330	1 090	924	802	708	634	525	449	392	W27x129
173	1 150	924	770	657	572	506	411	346	299	**W30x116**
219	1 650	1 540	1 430	1 310	1 180	1 060	880	754	659	W21x147
195	1 580	1 440	1 300	1 120	981	870	709	598	518	W24x131
235	1 630	1 540	1 460	1 380	1 290	1 210	1 020	887	783	W18x158

Note: For unbraced beam segments loaded above the shear centre, see CSA S16-14 Clause 13.6.

BEAM SELECTION TABLE
W Shapes

ASTM A992, A572 Grade 50
F_y = 345 MPa

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m)					
						Unbraced length (mm)					
	kN	10^6 mm^4	mm	mm	≤ L_u	2 500	3 000	3 500	4 000	4 500	5 000
W760x161	2 140	1 860	266	3 330	**1 760**	—	—	1 730	1 650	1 560	1 460
W690x170	2 060	1 700	256	3 380	1 750	—	—	1 730	1 650	1 570	1 480
W530x196	1 870	1 340	316	4 600	1 700	—	—	—	—	—	1 660
W610x174	1 770	1 470	325	4 480	1 660	—	—	—	—	—	1 610
W460x213	1 880	1 140	285	4 590	1 640	—	—	—	—	—	1 600
W760x147	2 040	1 660	265	3 260	**1 580**	—	—	1 550	1 470	1 380	1 290
W530x182	1 720	1 240	315	4 530	1 560	—	—	—	—	—	1 520
W690x152	1 850	1 510	254	3 320	1 550	—	—	1 530	1 460	1 380	1 290
W460x193	1 700	1 020	283	4 440	1 470	—	—	—	—	—	1 430
W610x155	1 590	1 290	324	4 400	1 470	—	—	—	—	1 460	1 410
W760x134	1 650	1 500	264	3 230	**1 440**	—	—	1 400	1 330	1 250	1 160
W610x153	1 790	1 250	229	3 110	1 430	—	—	1 380	1 310	1 240	1 160
W690x140	1 740	1 360	254	3 270	1 410	—	—	1 380	1 320	1 240	1 160
W530x165	1 570	1 110	313	4 440	1 410	—	—	—	—	—	1 360
W460x177	1 640	910	286	4 330	1 330	—	—	—	—	1 320	1 280
W610x140	1 660	1 120	230	3 070	1 290	—	—	1 240	1 170	1 100	1 030
W530x150	1 410	1 010	312	4 380	1 290	—	—	—	—	1 280	1 240
W690x125	1 610	1 180	253	3 190	**1 250**	—	—	1 210	1 140	1 070	999
W460x158	1 460	796	284	4 200	1 170	—	—	—	—	1 150	1 110
W610x125	1 490	985	229	3 020	1 140	—	—	1 090	1 020	959	889
W530x138	1 650	861	214	2 930	1 120	—	1 110	1 060	1 000	945	884
W460x144	1 320	726	283	4 130	1 070	—	—	—	—	1 050	1 010
W610x113	1 400	875	228	2 950	**1 020**	—	—	964	906	843	775
W410x149	1 320	618	265	4 080	1 010	—	—	—	—	983	952
W530x123	1 460	761	212	2 860	997	—	984	933	879	822	762
W360x162	992	515	371	5 980	975	—	—	—	—	—	—
W460x128	1 170	637	282	4 040	947	—	—	—	—	917	884
W610x101	1 300	764	228	2 890	**900**	—	891	842	787	728	664
W410x132	1 160	538	263	3 940	885	—	—	—	882	853	823
W530x109	1 280	667	211	2 810	879	—	862	815	764	709	652
W460x113	1 020	556	280	3 950	829	—	—	—	826	796	765
W530x101	1 200	617	210	2 770	814	—	794	749	699	647	591
†W360x147	907	463	370	6 190	798	—	—	—	—	—	—
W610x92	1 350	646	179	2 180	**779**	744	683	614	540	448	376
W410x114	998	461	261	3 810	764	—	—	—	754	726	698
W460x106	1 210	488	194	2 690	742	—	719	679	637	594	549
W530x92	1 110	552	209	2 720	733	—	711	668	621	570	516
†W360x134	817	415	369	6 030	723	—	—	—	—	—	—
W360x122	967	365	257	4 040	705	—	—	—	—	686	664

Sections highlighted in yellow are commonly used sizes and are generally readily available.

† Class 3

ASTM A992, A572 Grade 50 — BEAM SELECTION TABLE
F_y = 345 MPa — W Shapes

Nominal mass	Factored moment resistance M_r' (kN·m)									Imperial designation
	Unbraced length (mm)									
kg/m	6 000	7 000	8 000	9 000	10 000	12 000	14 000	16 000	18 000	
161	1 250	988	793	658	560	429	347	291	251	**W30x108**
170	1 290	1 070	875	736	634	497	408	347	302	W27x114
196	1 550	1 450	1 340	1 230	1 110	873	721	615	537	W21x132
174	1 490	1 370	1 230	1 090	924	709	574	482	415	W24x117
213	1 520	1 440	1 350	1 270	1 190	1 010	848	733	646	W18x143
147	1 090	840	671	555	470	358	288	241	207	**W30x99**
182	1 420	1 320	1 210	1 100	969	759	625	531	463	W21x122
152	1 110	898	728	610	523	406	332	281	244	W27x102
193	1 350	1 270	1 180	1 100	1 020	836	702	606	533	W18x130
155	1 300	1 180	1 050	901	762	579	465	388	333	W24x104
134	967	738	587	483	408	308	246	205	175	**W30x90**
153	1 000	815	675	576	502	400	333	286	251	W24x103
140	987	778	628	523	447	345	280	236	204	W27x94
165	1 270	1 170	1 060	954	823	641	525	445	386	W21x111
177	1 200	1 120	1 030	953	865	694	581	500	439	W18x119
140	874	695	573	486	422	334	277	237	207	W24x94
150	1 150	1 050	943	829	709	548	446	377	326	W21x101
125	834	640	513	425	362	277	223	187	161	**W27x84**
158	1 040	955	875	794	696	555	462	397	348	W18x106
125	733	575	470	396	342	269	222	189	165	W24x84
138	759	616	515	444	390	314	263	227	200	W21x93
144	936	858	779	693	602	478	396	339	297	W18x97
113	617	481	391	328	282	220	180	153	133	**W24x76**
149	889	825	760	696	621	501	421	363	320	W16x100
123	631	505	421	361	316	253	211	182	160	W21x83
162	974	935	895	855	814	733	653	558	488	W14x109
128	812	736	658	566	489	385	318	271	236	W18x86
101	512	396	320	267	228	176	144	121	105	**W24x68**
132	761	698	635	565	495	397	332	286	251	W16x89
109	520	413	342	291	254	202	168	144	126	W21x73
113	696	623	545	458	394	307	252	213	185	W18x76
101	462	365	301	255	222	176	146	125	109	W21x68
147	—	773	740	708	675	609	544	467	407	W14x99
92	281	222	183	156	135	107	88.9	76.1	66.5	**W24x62**
114	638	576	513	438	382	304	253	217	190	W16x77
106	450	366	308	266	235	190	160	138	122	W18x71
92	393	309	253	214	185	146	120	103	89.6	W21x62
134	—	694	663	630	598	532	461	392	341	W14x90
122	621	577	534	491	441	357	301	260	229	W14x82

Note: For unbraced beam segments loaded above the shear centre, see CSA S16-14 Clause 13.6.

BEAM SELECTION TABLE
W Shapes

ASTM A992, A572 Grade 50
F_y = 345 MPa

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m)					
						Unbraced length (mm)					
	kN	10^6 mm^4	mm	mm	$\leq L_u$	2 000	2 500	3 000	3 500	4 000	5 000
W610x82	1 170	560	178	2 110	**683**	—	644	587	522	448	304
W460x97	1 090	445	193	2 650	677	—	—	652	614	574	488
W310x129	854	308	308	5 080	671	—	—	—	—	—	—
W410x100	850	398	260	3 730	661	—	—	—	—	648	596
W530x85	1 130	485	166	2 110	652	—	616	564	507	446	316
W530x82	1 030	477	209	2 660	640	—	—	616	576	531	433
W360x110	841	331	256	3 940	640	—	—	—	—	637	596
W460x89	996	409	192	2 620	624	—	—	598	562	523	439
W310x118	766	275	307	4 920	605	—	—	—	—	—	603
W360x101	768	301	255	3 860	584	—	—	—	—	578	538
W460x82	933	370	191	2 560	568	—	—	540	505	466	384
W530x74	1 050	411	166	2 040	**562**	—	523	474	420	357	247
W310x107	695	248	306	4 800	546	—	—	—	—	—	541
W410x85	931	315	181	2 530	534	—	—	507	475	443	375
W360x91	687	267	254	3 760	522	—	—	—	—	513	475
W460x74	843	332	190	2 530	512	—	—	484	450	414	332
W530x66	927	351	165	1 980	**484**	483	444	398	347	284	195
†W530x72	947	401	207	2 750	475	—	—	462	434	402	331
W410x74	821	275	180	2 470	469	—	467	440	410	379	312
W460x68	856	297	154	2 010	463	—	429	390	348	301	213
†W310x97	625	222	305	4 970	447	—	—	—	—	—	446
W360x79	682	226	205	3 010	444	—	—	—	425	404	361
W310x86	578	198	254	3 900	441	—	—	—	—	438	409
W250x101	644	164	257	4 470	435	—	—	—	—	—	424
W410x67	739	245	179	2 420	422	—	418	392	364	333	264
W460x60	746	255	153	1 970	**397**	396	364	329	289	242	169
W360x72	617	201	204	2 940	397	—	—	395	377	357	315
W310x79	552	177	254	3 810	397	—	—	—	—	392	364
W250x89	570	143	256	4 260	382	—	—	—	—	—	367
W410x60	642	216	178	2 390	369	—	365	341	314	286	218
W310x74	597	164	205	3 100	366	—	—	—	354	339	307
W200x100	680	113	210	4 460	357	—	—	—	—	—	349
W360x64	548	178	203	2 870	354	—	—	350	332	313	273
W460x52	680	212	152	1 890	**338**	333	303	269	231	185	128
W250x80	493	126	255	4 130	338	—	—	—	—	—	321
W410x54	619	186	177	2 310	326	—	318	295	269	241	176
W310x67	533	144	204	3 020	326	—	—	—	312	297	266
W360x57	580	160	172	2 360	314	—	309	289	267	244	192
W250x73	446	113	254	4 010	306	—	—	—	—	—	287
W200x86	591	94.7	209	4 110	305	—	—	—	—	—	292
W310x60	466	128	203	2 960	290	—	—	289	275	261	231
W250x67	469	104	204	3 260	280	—	—	—	275	265	244
W360x51	524	141	171	2 320	**277**	—	271	252	232	210	159
W410x46	578	156	140	1 790	**274**	265	239	210	177	142	99.9
W310x52	494	118	167	2 380	260	—	256	240	223	206	167
W200x71	452	76.6	206	3 730	249	—	—	—	—	246	232

Sections highlighted in yellow are commonly used sizes and are generally readily available.

† Class 3

Nominal mass	Factored moment resistance M_r' (kN·m)									Imperial designation
	Unbraced length (mm)									
kg/m	6 000	7 000	8 000	9 000	10 000	11 000	12 000	14 000	16 000	
82	225	177	145	123	106	93.4	83.4	68.9	58.7	**W24x55**
97	389	314	264	227	200	178	161	135	117	W18x65
129	643	612	581	551	520	490	460	390	336	W12x87
100	539	479	411	348	302	266	238	197	168	W16x67
85	240	193	162	139	122	108	97.8	81.9	70.6	W21x57
82	320	249	203	170	147	129	115	94.0	79.8	W21x55
110	553	510	467	423	372	332	300	252	217	W14x74
89	343	276	231	198	174	155	140	117	101	W18x60
118	574	543	513	482	452	422	388	324	279	W12x79
101	497	454	411	363	318	283	255	214	184	W14x68
82	292	234	195	166	146	129	116	97.3	83.6	W18x55
74	186	148	123	105	91.7	81.4	73.2	61.0	52.4	**W21x50**
107	512	483	453	423	392	362	325	271	233	W12x72
85	297	242	205	177	157	140	127	107	92.8	W16x57
91	434	392	350	299	261	232	209	174	150	W14x61
74	249	198	164	140	122	108	96.8	80.6	69.1	W18x50
66	145	115	94.9	80.6	70.0	61.9	55.5	46.0	39.4	**W21x44**
72	246	191	154	129	110	96.3	85.4	69.6	58.8	W21x48
74	239	194	163	140	123	110	99.7	83.8	72.3	W16x50
68	164	133	112	96.7	85.2	76.1	68.9	57.9	50.1	W18x46
97	423	400	375	351	326	302	272	226	193	W12x65
79	317	267	225	194	171	153	139	117	101	W14x53
86	379	347	316	282	248	221	199	167	144	W12x58
101	403	382	362	342	322	302	279	236	205	W10x68
67	201	161	135	116	102	90.4	81.6	68.3	58.9	W16x45
60	129	104	86.6	74.3	65.2	58.1	52.4	43.9	37.8	**W18x40**
72	272	222	186	160	141	126	114	95.6	82.5	W14x48
79	334	304	273	237	207	184	166	139	119	W12x53
89	346	326	306	285	265	242	220	186	161	W10x60
60	165	131	109	93.1	81.3	72.1	64.9	54.1	46.4	W16x40
74	274	240	204	177	156	140	127	107	93.0	W12x50
100	335	321	307	294	280	266	253	222	194	W8x67
64	228	183	153	131	115	102	92.2	77.2	66.5	W14x43
52	96.2	76.7	63.6	54.3	47.4	42.0	37.8	31.5	27.0	**W18x35**
80	302	282	262	242	221	197	179	151	130	W10x54
54	132	104	86.0	73.1	63.5	56.2	50.4	41.8	35.8	W16x36
67	234	198	167	144	127	114	103	86.8	75.1	W12x45
57	147	119	99.7	85.9	75.6	67.5	61.0	51.2	44.2	W14x38
73	268	248	228	209	185	165	149	126	108	W10x49
86	279	265	252	238	225	212	197	168	147	W8x58
60	199	163	137	118	104	92.5	83.6	70.2	60.5	W12x40
67	223	202	180	157	139	125	114	96.5	83.8	W10x45
51	121	96.9	80.9	69.4	60.8	54.1	48.8	40.8	35.2	**W14x34**
46	76.4	61.7	51.8	44.6	39.2	35.0	31.6	26.5	22.9	**W16x31**
52	130	106	89.4	77.4	68.4	61.3	55.5	46.8	40.5	W12x35
71	219	205	192	179	166	150	137	116	101	W8x48

Note: For unbraced beam segments loaded above the shear centre, see CSA S16-14 Clause 13.6.

BEAM SELECTION TABLE — ASTM A992, A572 Grade 50
W Shapes — F_y = 345 MPa

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m)					
						Unbraced length (mm)					
	kN	10^6 mm^4	mm	mm	$\leq L_u$	1 500	2 000	2 500	3 000	3 500	4 000
W360x45	498	122	171	2 260	**242**	—	—	234	217	197	176
W250x58	413	87.3	203	3 130	239	—	—	—	—	232	222
W410x39	480	126	140	1 730	**227**	—	216	193	166	132	105
W310x45	423	99.2	166	2 310	220	—	—	215	200	184	167
W360x39	470	102	128	1 660	206	—	193	172	148	120	97.0
W200x59	392	61.1	205	3 430	203	—	—	—	—	202	195
W310x39	368	85.1	165	2 260	189	—	—	184	170	155	139
W250x45	414	71.1	148	2 170	187	—	—	179	167	155	142
†W250x49	375	70.6	202	3 160	178	—	—	—	—	173	165
W200x52	334	52.7	204	3 300	177	—	—	—	—	174	168
W360x33	396	82.6	127	1 600	**168**	—	155	135	113	87.5	70.2
W250x39	354	60.1	147	2 110	159	—	—	151	140	128	115
W310x33	423	65.0	102	1 330	149	143	125	104	80.5	64.2	53.3
†W200x46	300	45.4	203	3 370	139	—	—	—	—	138	133
W200x42	302	40.9	166	2 610	138	—	—	—	133	126	120
W250x33	323	48.9	146	2 020	132	—	—	122	112	100	88.4
W310x28	380	54.3	102	1 290	**126**	120	103	83.1	61.9	48.8	40.2
W200x36	255	34.4	165	2 510	118	—	—	—	112	105	99.0
W250x28	341	40.0	102	1 370	110	107	94.5	81.0	65.4	52.6	44.0
W200x31	275	31.4	134	1 980	104	—	—	96.7	89.3	81.7	74.0
W310x24	350	42.7	101	1 210	**102**	94.2	78.3	58.1	42.8	33.5	27.3
W150x37	269	22.2	154	2 630	96.3	—	—	—	93.3	89.4	85.4
W250x25	321	34.2	102	1 330	95.3	91.7	80.0	66.9	51.6	41.2	34.2
W310x21	303	37.0	101	1 190	**89.1**	81.5	66.7	47.9	35.0	27.2	22.0
W200x27	246	25.8	133	1 890	86.6	—	85.3	78.7	71.5	64.1	56.0
W250x22	302	28.9	102	1 280	81.7	77.6	66.6	53.9	40.3	31.9	26.3
W150x30	212	17.1	153	2 440	75.8	—	—	75.3	71.6	67.8	63.9
W200x22	262	20.0	102	1 390	68.9	67.4	60.1	52.1	43.2	35.0	29.4
W150x24	216	13.4	102	1 630	59.3	—	56.2	51.9	47.7	43.4	39.1
W200x19	241	16.6	102	1 340	**58.1**	56.0	49.2	41.5	32.7	26.2	21.9
†W250x18	247	22.4	101	1 330	**55.6**	53.4	46.1	37.5	27.8	21.7	17.7
‡W150x22	181	12.0	152	2 480	46.2	—	—	46.1	43.7	41.1	38.5
W150x18	182	9.15	102	1 480	42.2	42.1	38.2	34.2	30.1	25.4	21.5
†W200x15	176	12.7	100	1 380	**39.4**	38.5	33.8	28.5	22.1	17.4	14.4
W150x14	132	6.85	100	1 400	**31.7**	31.0	27.6	23.8	19.6	15.7	13.1
†W150x13	130	6.13	100	1 460	**25.7**	25.5	22.9	20.0	16.9	13.6	11.3

Sections highlighted in yellow are commonly used sizes and are generally readily available.

† Class 3 ‡ Class 4

Nominal mass	Factored moment resistance M_r' (kN·m)									Imperial designation
	Unbraced length (mm)									
kg/m	4 500	5 000	6 000	7 000	8 000	9 000	10 000	12 000	14 000	
45	152	128	96.0	76.4	63.3	54.0	47.1	37.5	31.3	**W14x30**
58	212	202	181	161	137	119	105	85.7	72.4	W10x39
39	86.5	73.0	55.1	44.0	36.5	31.2	27.3	21.8	18.2	**W16x26**
45	150	128	98.2	79.3	66.5	57.3	50.4	40.6	34.1	W12x30
39	81.2	69.7	54.1	44.2	37.4	32.5	28.7	23.3	19.7	W14x26
59	189	182	169	155	142	128	114	93.2	79.1	W8x40
39	121	103	77.7	62.2	51.8	44.3	38.8	31.1	25.9	W12x26
45	129	114	90.6	75.2	64.4	56.3	50.1	41.1	34.9	W10x30
49	157	149	133	115	97.2	84.0	74.1	60.0	50.5	W10x33
52	161	155	142	129	116	101	89.5	73.3	62.1	W8x35
33	58.3	49.6	38.0	30.8	25.8	22.3	19.6	15.8	13.3	**W14x22**
39	102	88.0	69.2	57.0	48.6	42.3	37.6	30.7	26.0	W10x26
33	45.5	39.7	31.7	26.4	22.7	19.9	17.7	14.6	12.4	W12x22
46	128	122	112	101	90.2	78.4	69.5	56.6	47.9	W8x31
42	113	106	92.9	77.5	66.5	58.3	51.9	42.7	36.3	W8x28
33	74.1	63.6	49.4	40.3	34.1	29.6	26.1	21.2	17.9	W10x22
28	34.1	29.5	23.4	19.4	16.5	14.5	12.8	10.5	8.93	**W12x19**
36	92.5	85.9	71.3	59.0	50.4	44.0	39.1	32.1	27.2	W8x24
28	37.8	33.1	26.6	22.3	19.2	16.9	15.1	12.4	10.6	W10x19
31	65.2	57.0	45.6	38.0	32.7	28.7	25.6	21.0	17.9	W8x21
24	23.0	19.8	15.5	12.8	10.9	9.45	8.37	6.83	5.78	**W12x16**
37	81.5	77.6	69.8	61.5	53.2	46.9	42.0	34.7	29.6	W6x25
25	29.2	25.5	20.4	17.0	14.6	12.8	11.4	9.40	7.99	W10x17
21	18.4	15.8	12.3	10.0	8.49	7.36	6.50	5.29	4.46	**W12x14**
27	47.5	41.2	32.6	27.1	23.1	20.2	18.0	14.7	12.5	W8x18
22	22.4	19.5	15.4	12.8	11.0	9.60	8.54	7.01	5.95	W10x15
30	60.1	56.3	48.1	40.2	34.6	30.4	27.2	22.4	19.1	W6x20
22	25.3	22.3	18.0	15.1	13.0	11.5	10.3	8.47	7.22	W8x15
24	34.2	30.4	25.0	21.2	18.4	16.3	14.6	12.1	10.4	W6x16
19	18.7	16.4	13.2	11.0	9.48	8.33	7.44	6.13	5.22	**W8x13**
18	14.9	12.8	10.0	8.24	7.00	6.09	5.40	4.40	3.73	**W10x12**
22	35.9	33.2	27.2	22.5	19.2	16.8	14.9	12.2	10.4	W6x15
18	18.6	16.5	13.4	11.3	9.80	8.64	7.74	6.41	5.47	W6x12
15	12.2	10.6	8.35	6.92	5.91	5.17	4.59	3.76	3.19	**W8x10**
14	11.3	9.91	7.97	6.67	5.75	5.06	4.51	3.72	3.17	**W6x9**
13	9.70	8.49	6.81	5.70	4.90	4.31	3.84	3.17	2.70	**W6x8.5**

Note: For unbraced beam segments loaded above the shear centre, see CSA S16-14 Clause 13.6.

BEAM SELECTION TABLE
S Shapes

ASTM A992, A572 Grade 50
F_y = 345 MPa

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m)					
						Unbraced length (mm)					
	kN	10^6 mm^4	mm	mm	$\leq L_u$	1 500	2 000	2 500	3 000	3 500	4 000
S610x180	2 590	1 310	204	2 540	**1 560**	—	—	—	1 480	1 400	1 310
S610x158	2 000	1 220	200	2 530	**1 420**	—	—	—	1 350	1 270	1 190
S610x149	2 360	996	184	2 130	**1 220**	—	—	1 160	1 080	987	895
S610x134	1 990	939	181	2 120	**1 130**	—	—	1 080	995	908	818
S610x119	1 590	879	178	2 130	**1 040**	—	—	992	915	833	747
S510x143	2 150	700	183	2 290	1 010	—	—	986	929	870	810
S510x128	1 780	658	179	2 250	935	—	—	908	852	794	735
S510x112	1 680	532	162	1 940	**776**	—	770	715	656	595	533
S510x98.2	1 330	497	159	1 940	**711**	—	705	652	596	537	477
S460x104	1 700	387	159	1 880	637	—	626	581	534	485	437
S460x81.4	1 100	335	152	1 840	**531**	—	518	476	431	384	331
S380x74	1 090	203	143	1 740	**394**	—	379	348	316	284	248
S380x64	812	187	140	1 730	**354**	—	339	310	279	248	211
S310x74	1 090	127	139	1 930	311	—	308	291	273	256	239
S310x60.7	731	113	133	1 820	**270**	—	263	245	227	209	191
S310x52	681	95.8	129	1 640	**229**	—	216	197	178	158	136
S310x47	556	91.0	127	1 630	**214**	—	201	183	164	145	123
S250x52	786	61.5	126	1 690	181	—	174	162	150	138	127
S250x38	411	51.4	118	1 560	**144**	—	134	122	109	97.0	82.5
S200x34	466	27.0	106	1 460	**98.1**	97.5	90.0	82.5	75.1	67.8	59.3
S200x27	287	24.0	102	1 390	**84.1**	82.6	75.1	67.5	59.9	51.3	44.0
S150x26	368	10.9	91	1 400	**53.7**	52.9	49.2	45.6	42.0	38.4	34.5
S150x19	184	9.16	85	1 230	**42.8**	40.7	36.7	32.7	28.8	24.3	21.0
S130x15	141	5.11	76	1 150	**28.8**	26.9	24.2	21.6	18.9	16.0	13.9
S100x14.1	173	2.85	71	1 220	**20.6**	19.8	18.3	16.8	15.3	13.9	12.1
S100x11	102	2.56	68	1 100	**18.0**	16.7	15.1	13.5	11.9	10.1	8.8
S75x11	139	1.22	64	1 380	12.0	11.9	11.1	10.5	9.8	9.1	8.4
S75x8	67.0	1.04	59	1 090	**9.9**	9.2	8.4	7.6	6.8	5.9	5.2

Nominal mass	Factored moment resistance M_r' (kN·m)									Imperial designation
	Unbraced length (mm)									
kg/m	5 000	6 000	7 000	8 000	9 000	10 000	12 000	14 000	16 000	
180	1 130	930	766	652	569	504	412	349	303	**S24x121**
158	1 010	818	671	569	495	438	357	301	261	**S24x106**
149	689	542	447	381	332	295	241	204	177	**S24x100**
134	617	482	396	336	293	259	211	179	155	**S24x90**
119	555	431	352	298	258	228	185	156	135	**S24x80**
143	691	557	465	399	350	312	256	218	190	S20x96
128	612	486	404	346	303	269	221	188	163	S20x86
112	401	319	266	228	200	178	146	124	108	**S20x75**
98.2	351	278	230	197	172	153	125	106	92.5	**S20x66**
104	334	268	225	194	171	152	126	107	93.2	S18x70
81.4	245	194	161	138	121	108	88.3	74.9	65.2	**S18x54.7**
74	188	152	128	110	96.9	86.6	71.5	61.0	53.2	**S15x50**
64	158	127	106	91.3	80.2	71.6	59.0	50.2	43.7	**S15x42.9**
74	204	167	142	123	109	97.6	81.0	69.2	60.5	S12x50
60.7	152	124	104	90.6	80.0	71.7	59.4	50.7	44.3	**S12x40.8**
52	104	84.5	71.2	61.6	54.4	48.7	40.3	34.4	30.0	**S12x35**
47	93.4	75.6	63.6	55.0	48.4	43.3	35.8	30.5	26.6	**S12x31.8**
52	101	83.1	70.6	61.5	54.4	48.8	40.6	34.7	30.3	S10x35
38	63.4	51.7	43.7	37.9	33.5	30.0	24.8	21.2	18.5	**S10x25.4**
34	46.6	38.4	32.7	28.5	25.2	22.7	18.8	16.1	14.1	**S8x23**
27	34.3	28.1	23.9	20.8	18.4	16.5	13.7	11.7	10.2	**S8x18.4**
26	27.4	22.7	19.4	16.9	15.0	13.5	11.3	9.64	8.43	**S6x17.25**
19	16.5	13.7	11.6	10.2	9.00	8.09	6.73	5.76	5.03	**S6x12.5**
15	11.0	9.09	7.77	6.78	6.02	5.41	4.50	3.86	3.37	**S5x10**
14.1	9.7	8.03	6.87	6.01	5.34	4.80	4.00	3.43	3.00	**S4x9.5**
11	7.0	5.82	4.97	4.35	3.86	3.47	2.89	2.48	2.17	**S4x7.7**
11	6.8	5.68	4.86	4.25	3.78	3.40	2.83	2.43	2.12	S3x7.5
8	4.1	3.43	2.93	2.57	2.28	2.05	1.71	1.46	1.28	**S3x5.7**

BEAM SELECTION TABLE
C Shapes

G40.21-350W
F_y = 350 MPa

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m)					
						Unbraced length (mm)					
	kN	10^6 mm^4	mm	mm	≤ L_u	1 500	2 000	2 500	3 000	3 500	4 000
†C380x74*	1 440	168	94	1 830	**278**	—	272	255	238	221	204
†C380x60*	1 050	145	89	1 690	**239**	—	228	210	191	172	152
†C380x50*	808	131	86	1 620	**216**	—	203	184	165	145	122
†C310x45	824	67.3	80	1 530	**139**	—	129	118	107	95.9	83.2
†C310x37	621	59.9	77	1 440	**124**	123	111	99.5	87.6	74.0	62.8
†C310x31	457	53.5	74	1 380	**111**	108	96.6	84.5	71.6	58.4	49.3
†C250x45	903	42.8	76	1 630	106	—	102	95.5	89.5	83.5	77.6
†C250x37	708	37.9	73	1 460	94.2	93.7	86.7	79.8	73.0	66.2	58.5
†C250x30	507	32.7	69	1 330	**81.0**	78.4	70.7	63.0	55.2	46.4	39.8
†C230x30*	543	25.5	67	1 340	69.9	68.2	62.4	56.7	51.1	45.2	39.0
†C250x23	322	27.8	65	1 220	**69.0**	64.8	56.7	48.3	38.8	32.0	27.3
†C230x22	343	21.3	63	1 210	**58.6**	55.0	48.6	42.1	34.9	29.0	24.9
†C200x28	523	18.2	64	1 370	56.7	55.6	51.5	47.5	43.5	39.5	35.0
†C230x20	281	19.8	61	1 160	**54.5**	50.3	43.8	37.1	29.7	24.7	21.1
†C200x21	325	14.9	59	1 160	46.3	43.1	38.2	33.4	28.0	23.5	20.3
†C200x17	236	13.5	57	1 100	**41.9**	38.0	32.9	27.7	22.2	18.5	15.9
†C180x22*	392	11.3	58	1 260	40.0	38.4	35.3	32.2	29.1	25.9	22.5
†C180x18	296	10.0	55	1 120	35.6	32.9	29.4	25.9	22.1	18.7	16.2
†C180x15	196	8.86	53	1 040	**31.4**	28.0	24.2	20.2	16.4	13.7	11.9
†C150x19	351	7.11	54	1 290	29.5	28.6	26.6	24.5	22.5	20.5	18.3
†C150x16	253	6.21	51	1 100	25.8	23.8	21.5	19.1	16.7	14.1	12.3
†C150x12	161	5.36	48	976	**22.2**	19.4	16.8	14.0	11.4	9.62	8.34
†C130x13	219	3.66	47	1 110	18.1	17.0	15.5	14.0	12.5	10.9	9.46
†C130x10	127	3.09	44	936	**15.3**	13.3	11.6	9.77	8.03	6.82	5.93
†C100x11	174	1.91	43	1 160	11.8	11.2	10.3	9.49	8.65	7.80	6.81
†C100x9	134	1.68	42	1 000	**10.4**	9.43	8.48	7.55	6.52	5.56	4.85
†C100x8	99.7	1.61	40	924	**9.95**	8.75	7.73	6.73	5.59	4.76	4.15
†C100x7	67.9	1.53	40	877	**9.45**	8.08	7.01	5.85	4.83	4.11	3.58
†C75x9	142	0.847	40	1 440	7.02	6.98	6.60	6.22	5.84	5.47	5.09
†C75x7	104	0.749	37	1 130	6.21	5.88	5.44	5.00	4.57	4.13	3.61
†C75x6	67.9	0.670	35	939	**5.54**	4.97	4.47	3.98	3.43	2.93	2.56
†C75x5	53.7	0.651	35	897	**5.39**	4.75	4.23	3.72	3.13	2.67	2.33

* Imported section

† Class 3

BEAM SELECTION TABLE
C Shapes

Nominal mass	Factored moment resistance M_r' (kN·m)									Imperial designation
	Unbraced length (mm)									
kg/m	4 500	5 000	6 000	7 000	8 000	9 000	10 000	11 000	12 000	
74	187	167	137	116	101	89.3	80.1	72.7	66.5	**C15x50**
60	132	116	94.5	79.8	69.1	61.0	54.6	49.5	45.2	**C15x40**
50	106	93.0	75.3	63.3	54.7	48.2	43.1	39.0	35.6	**C15x33.9**
45	72.7	64.7	53.1	45.0	39.2	34.7	31.1	28.2	25.8	**C12x30**
37	54.7	48.4	39.5	33.4	29.0	25.6	22.9	20.8	19.0	**C12x25**
31	42.8	37.8	30.7	25.9	22.4	19.7	17.7	16.0	14.6	**C12x20.7**
45	71.7	64.6	53.5	45.7	39.9	35.4	31.8	28.9	26.5	C10x30
37	51.5	46.1	38.0	32.4	28.2	25.0	22.5	20.4	18.7	C10x25
30	34.8	31.0	25.5	21.7	18.8	16.7	15.0	13.6	12.4	**C10x20**
30	34.4	30.7	25.4	21.6	18.9	16.7	15.0	13.6	12.5	C9x20
23	23.8	21.1	17.3	14.6	12.7	11.2	10.1	9.12	8.34	**C10x15.3**
22	21.8	19.4	16.0	13.6	11.8	10.4	9.38	8.51	7.79	**C9x15**
28	30.9	27.7	23.0	19.6	17.1	15.2	13.7	12.4	11.4	C8x18.75
20	18.5	16.5	13.5	11.5	9.98	8.83	7.93	7.19	6.58	**C9x13.4**
21	17.8	15.9	13.1	11.2	9.76	8.65	7.77	7.06	6.46	C8x13.75
17	14.0	12.5	10.3	8.75	7.62	6.75	6.06	5.50	5.03	**C8x11.5**
22	19.9	17.8	14.8	12.6	11.0	9.79	8.81	8.00	7.33	C7x14.75
18	14.3	12.8	10.6	9.04	7.89	7.00	6.29	5.71	5.23	C7x12.25
15	10.5	9.35	7.72	6.58	5.74	5.09	4.57	4.15	3.80	**C7x9.8**
19	16.2	14.5	12.1	10.3	9.03	8.02	7.21	6.55	6.00	C6x13
16	10.9	9.75	8.09	6.91	6.04	5.36	4.82	4.38	4.01	C6x10.5
12	7.37	6.60	5.47	4.67	4.08	3.62	3.25	2.95	2.71	**C6x8.2**
13	8.39	7.54	6.26	5.36	4.69	4.16	3.74	3.40	3.12	C5x9
10	5.25	4.71	3.91	3.34	2.92	2.59	2.33	2.12	1.94	**C5x6.7**
11	6.04	5.43	4.52	3.87	3.38	3.01	2.71	2.46	2.25	C4x7.25
9	4.30	3.87	3.21	2.75	2.41	2.14	1.92	1.75	1.60	**C4x6.25**
8	3.68	3.31	2.75	2.36	2.06	1.83	1.65	1.50	1.37	**C4x5.4**
7	3.18	2.85	2.37	2.03	1.77	1.57	1.42	1.29	1.18	**C4x4.5**
9	4.72	4.25	3.54	3.03	2.65	2.36	2.12	1.93	1.77	C3x6
7	3.21	2.88	2.40	2.06	1.80	1.60	1.44	1.31	1.20	C3x5
6	2.27	2.04	1.70	1.46	1.27	1.13	1.02	0.926	0.849	**C3x4.1**
5	2.07	1.86	1.55	1.33	1.16	1.03	0.929	0.844	0.774	**C3x3.5**

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W1100				Approx. Deflect. (mm)	W1000						Approx. Deflect. (mm)
Mass (kg/m)	499	433	390	343		642	591	554	539	483	443	
Span in Millimetres												
5 000					6	14 600		12 500				6
5 500	11 900	10 000	9 020	7 700	7	14 500	13 200	12 400	12 000	10 600	9 780	8
6 000	11 000	9 600	8 610	7 490	8	13 300	12 200	11 400	11 100	9 890	9 030	9
6 500	10 200	8 870	7 950	6 920	10	12 300	11 300	10 500	10 200	9 130	8 330	11
7 000	9 440	8 230	7 380	6 420	11	11 400	10 500	9 760	9 510	8 480	7 740	12
7 500	8 810	7 680	6 890	5 990	13	10 600	9 770	9 110	8 880	7 920	7 220	14
8 000	8 260	7 200	6 460	5 620	15	9 970	9 160	8 540	8 320	7 420	6 770	16
8 500	7 770	6 780	6 080	5 290	16	9 380	8 620	8 040	7 830	6 980	6 370	18
9 000	7 340	6 400	5 740	5 000	18	8 860	8 140	7 590	7 400	6 600	6 020	20
9 500	6 960	6 070	5 440	4 730	21	8 390	7 710	7 190	7 010	6 250	5 700	23
10 000	6 610	5 760	5 170	4 500	23	7 970	7 330	6 830	6 660	5 940	5 420	25
10 500	6 290	5 490	4 920	4 280	25	7 590	6 980	6 510	6 340	5 650	5 160	28
11 000	6 010	5 240	4 700	4 090	28	7 250	6 660	6 210	6 050	5 400	4 920	30
11 500	5 750	5 010	4 490	3 910	30	6 930	6 370	5 940	5 790	5 160	4 710	33
12 000	5 510	4 800	4 310	3 750	33	6 640	6 110	5 690	5 550	4 950	4 510	36
12 500	5 290	4 610	4 130	3 600	36	6 380	5 860	5 460	5 330	4 750	4 330	39
13 000	5 080	4 430	3 970	3 460	38	6 130	5 640	5 250	5 120	4 570	4 170	42
13 500	4 890	4 270	3 830	3 330	41	5 910	5 430	5 060	4 930	4 400	4 010	46
14 000	4 720	4 120	3 690	3 210	45	5 700	5 230	4 880	4 760	4 240	3 870	49
14 500	4 560	3 970	3 560	3 100	48	5 500	5 050	4 710	4 590	4 090	3 730	53
15 000	4 400	3 840	3 440	3 000	51	5 320	4 890	4 550	4 440	3 960	3 610	56
15 500	4 260	3 720	3 330	2 900	55	5 140	4 730	4 410	4 290	3 830	3 490	60
16 000	4 130	3 600	3 230	2 810	58	4 980	4 580	4 270	4 160	3 710	3 380	64
16 500	4 000	3 490	3 130	2 720	62	4 830	4 440	4 140	4 030	3 600	3 280	68
17 000	3 890	3 390	3 040	2 640	66	4 690	4 310	4 020	3 920	3 490	3 190	72
17 500	3 780	3 290	2 950	2 570	70	4 560	4 190	3 900	3 800	3 390	3 090	77
18 000	3 670	3 200	2 870	2 500	74	4 430	4 070	3 800	3 700	3 300	3 010	81
18 500	3 570	3 120	2 790	2 430	78	4 310	3 960	3 690	3 600	3 210	2 930	86
19 000	3 480	3 030	2 720	2 370	82	4 200	3 860	3 600	3 500	3 120	2 850	90
19 500	3 390	2 960	2 650	2 310	86	4 090	3 760	3 500	3 410	3 040	2 780	95
20 000	3 300	2 880	2 580	2 250	91	3 990	3 660	3 420	3 330	2 970	2 710	100
20 500	3 220	2 810	2 520	2 190	96	3 890	3 570	3 330	3 250	2 900	2 640	105
21 000	3 150	2 740	2 460	2 140	100	3 800	3 490	3 250	3 170	2 830	2 580	110
21 500	3 070	2 680	2 400	2 090	105	3 710	3 410	3 180	3 100	2 760	2 520	116
22 000	3 000	2 620	2 350	2 040	110	3 620	3 330	3 110	3 030	2 700	2 460	121
PROPERTIES AND DESIGN DATA												
V_r (kN)	5 960	5 000	4 510	3 850		7 300	6 610	6 240	5 990	5 310	4 890	
R (kN)	1 880	1 480	1 260	1 040		2 990	2 600	2 350	2 240	1 870	1 630	
G (kN)	67.3	56.9	51.8	46.6		88.0	80.2	76.3	73.5	65.7	61.1	
B_r' (kN)	2 530	1 810	1 500	1 210		4 320	3 590	3 250	3 010	2 410	2 080	
L_u (mm)	5 490	5 400	5 310	5 230		6 060	5 920	5 810	5 790	5 650	5 530	
d (mm)	1 118	1 108	1 100	1 090		1 048	1 040	1 032	1 030	1 020	1 012	
b (mm)	405	402	400	400		412	409	408	407	404	402	
t (mm)	45.0	40.0	36.0	31.0		60.0	55.9	52.0	51.1	46.0	41.9	
w (mm)	26.0	22.0	20.0	18.0		34.0	31.0	29.5	28.4	25.4	23.6	
IMPERIAL SIZE AND WEIGHT												
Weight (lb/ft)	335	290	262	230		431	397	372	362	324	297	
Nominal Depth (in.)	44					40						

BEAM LOAD TABLES
W Shapes

ASTM A992, A572 grade 50
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W1000											Approx. Deflect. (mm)
Mass (kg/m)	412	371	321	296	486	438	415	393	350	314	272	
Span in Millimetres												
4 000							10 900					4
4 500					12 700	11 300	10 800	10 200	8 720	7 820	6 500	5
5 000					11 500	10 300	9 740	9 190	8 250	7 400	6 360	6
5 500	8 720	7 780		6 460	10 500	9 350	8 850	8 360	7 500	6 730	5 780	8
6 000	8 490	7 620	6 500	5 920	9 600	8 570	8 110	7 660	6 870	6 170	5 300	9
6 500	7 830	7 030	6 040	5 460	8 870	7 910	7 490	7 070	6 340	5 690	4 890	11
7 000	7 270	6 530	5 610	5 070	8 230	7 350	6 960	6 560	5 890	5 290	4 540	12
7 500	6 790	6 090	5 230	4 740	7 680	6 860	6 490	6 130	5 500	4 930	4 240	14
8 000	6 370	5 710	4 910	4 440	7 200	6 430	6 090	5 740	5 150	4 630	3 970	16
8 500	5 990	5 380	4 620	4 180	6 780	6 050	5 730	5 410	4 850	4 350	3 740	18
9 000	5 660	5 080	4 360	3 950	6 400	5 710	5 410	5 110	4 580	4 110	3 530	20
9 500	5 360	4 810	4 130	3 740	6 070	5 410	5 120	4 840	4 340	3 900	3 350	23
10 000	5 090	4 570	3 920	3 550	5 760	5 140	4 870	4 600	4 120	3 700	3 180	25
10 500	4 850	4 350	3 740	3 380	5 490	4 900	4 640	4 380	3 930	3 520	3 030	28
11 000	4 630	4 160	3 570	3 230	5 240	4 670	4 430	4 180	3 750	3 360	2 890	30
11 500	4 430	3 970	3 410	3 090	5 010	4 470	4 230	4 000	3 590	3 220	2 760	33
12 000	4 240	3 810	3 270	2 960	4 800	4 280	4 060	3 830	3 440	3 080	2 650	36
12 500	4 070	3 660	3 140	2 840	4 610	4 110	3 890	3 680	3 300	2 960	2 540	39
13 000	3 920	3 520	3 020	2 730	4 430	3 960	3 750	3 530	3 170	2 850	2 450	42
13 500	3 770	3 390	2 910	2 630	4 270	3 810	3 610	3 400	3 050	2 740	2 360	46
14 000	3 640	3 260	2 800	2 540	4 120	3 670	3 480	3 280	2 950	2 640	2 270	49
14 500	3 510	3 150	2 710	2 450	3 970	3 550	3 360	3 170	2 840	2 550	2 190	53
15 000	3 390	3 050	2 620	2 370	3 840	3 430	3 250	3 060	2 750	2 470	2 120	56
15 500	3 290	2 950	2 530	2 290	3 720	3 320	3 140	2 960	2 660	2 390	2 050	60
16 000	3 180	2 860	2 450	2 220	3 600	3 210	3 040	2 870	2 580	2 310	1 990	64
16 500	3 090	2 770	2 380	2 150	3 490	3 120	2 950	2 790	2 500	2 240	1 930	68
17 000	3 000	2 690	2 310	2 090	3 390	3 020	2 860	2 700	2 430	2 180	1 870	72
17 500	2 910	2 610	2 240	2 030	3 290	2 940	2 780	2 630	2 360	2 110	1 820	77
18 000	2 830	2 540	2 180	1 970	3 200	2 860	2 700	2 550	2 290	2 060	1 770	81
18 500	2 750	2 470	2 120	1 920	3 120	2 780	2 630	2 480	2 230	2 000	1 720	86
19 000	2 680	2 410	2 070	1 870	3 030	2 710	2 560	2 420	2 170	1 950	1 670	90
19 500	2 610	2 340	2 010	1 820	2 960	2 640	2 500	2 360	2 110	1 900	1 630	95
20 000	2 550	2 290	1 960	1 780	2 880	2 570	2 430	2 300	2 060	1 850	1 590	100
20 500	2 480	2 230	1 910	1 730	2 810	2 510	2 370	2 240	2 010	1 810	1 550	105
21 000	2 420	2 180	1 870	1 690	2 740	2 450	2 320	2 190	1 960	1 760	1 510	110
PROPERTIES AND DESIGN DATA												
V_r (kN)	4 360	3 890	3 250	3 230	6 370	5 660	5 430	5 080	4 360	3 910	3 250	
R (kN)	1 420	1 200	956	890	2 460	2 060	1 910	1 740	1 420	1 200	956	
G (kN)	54.6	49.2	42.7	42.7	77.6	69.6	67.3	63.1	54.6	49.4	42.7	
B_r' (kN)	1 660	1 350	1 020	1 020	3 360	2 700	2 530	2 230	1 660	1 360	1 020	
L_u (mm)	5 530	5 440	5 360	5 230	4 270	4 160	4 080	4 050	4 010	3 910	3 870	
d (mm)	1 008	1 000	990	982	1 036	1 026	1 020	1 016	1 008	1 000	990	
b (mm)	402	400	400	400	308	305	304	303	302	300	300	
t (mm)	40.0	36.1	31.0	27.1	54.1	49.0	46.0	43.9	40.0	35.9	31.0	
w (mm)	21.1	19.0	16.5	16.5	30.0	26.9	26.0	24.4	21.1	19.1	16.5	
IMPERIAL SIZE AND WEIGHT												
Weight (lb/ft)	277	249	215	199	327	294	278	264	235	211	183	
Nominal Depth (in.)	40											

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation		W1000		Approx. Deflect. (mm)	W920							Approx. Deflect. (mm)
Mass (kg/m)		249	222		656	588	537	491	449	420	390	
Span in Millimetres	4 000	6 440	6 000	4								4
	4 500	6 240	5 410	5								6
	5 000	5 610	4 870	6							8 180	7
	5 500	5 100	4 430	8	14 000	12 400	11 200	10 200	9 320	8 700	8 080	8
	6 000	4 680	4 060	9	13 000	11 500	10 500	9 520	8 650	8 070	7 410	10
	6 500	4 320	3 750	11	12 000	10 600	9 670	8 790	7 990	7 450	6 840	11
	7 000	4 010	3 480	12	11 100	9 870	8 980	8 160	7 420	6 920	6 350	13
	7 500	3 740	3 250	14	10 400	9 210	8 380	7 620	6 920	6 460	5 930	15
	8 000	3 510	3 040	16	9 720	8 630	7 860	7 140	6 490	6 050	5 560	17
	8 500	3 300	2 860	18	9 150	8 120	7 390	6 720	6 110	5 700	5 230	20
	9 000	3 120	2 700	20	8 640	7 670	6 980	6 350	5 770	5 380	4 940	22
	9 500	2 950	2 560	23	8 180	7 270	6 620	6 010	5 460	5 100	4 680	25
	10 000	2 810	2 430	25	7 770	6 910	6 280	5 710	5 190	4 840	4 450	27
	10 500	2 670	2 320	28	7 400	6 580	5 990	5 440	4 940	4 610	4 230	30
	11 000	2 550	2 210	30	7 070	6 280	5 710	5 190	4 720	4 400	4 040	33
	11 500	2 440	2 120	33	6 760	6 000	5 460	4 970	4 510	4 210	3 870	36
	12 000	2 340	2 030	36	6 480	5 750	5 240	4 760	4 330	4 040	3 710	39
	12 500	2 250	1 950	39	6 220	5 520	5 030	4 570	4 150	3 880	3 560	42
	13 000	2 160	1 870	42	5 980	5 310	4 830	4 390	3 990	3 730	3 420	46
	13 500	2 080	1 800	46	5 760	5 120	4 660	4 230	3 850	3 590	3 290	50
	14 000	2 000	1 740	49	5 550	4 930	4 490	4 080	3 710	3 460	3 180	53
	14 500	1 940	1 680	53	5 360	4 760	4 330	3 940	3 580	3 340	3 070	57
	15 000	1 870	1 620	56	5 180	4 600	4 190	3 810	3 460	3 230	2 960	61
	15 500	1 810	1 570	60	5 020	4 460	4 050	3 690	3 350	3 130	2 870	65
	16 000	1 750	1 520	64	4 860	4 320	3 930	3 570	3 240	3 030	2 780	70
	16 500	1 700	1 480	68	4 710	4 190	3 810	3 460	3 150	2 940	2 690	74
	17 000	1 650	1 430	72	4 570	4 060	3 700	3 360	3 050	2 850	2 620	79
	17 500	1 600	1 390	77	4 440	3 950	3 590	3 260	2 970	2 770	2 540	83
	18 000	1 560	1 350	81	4 320	3 840	3 490	3 170	2 880	2 690	2 470	88
	18 500	1 520	1 320	86	4 200	3 730	3 400	3 090	2 810	2 620	2 400	93
	19 000	1 480	1 280	90	4 090	3 630	3 310	3 010	2 730	2 550	2 340	98
	19 500	1 440	1 250	95	3 990	3 540	3 220	2 930	2 660	2 480	2 280	103
	20 000	1 400	1 220	100	3 890	3 450	3 140	2 860	2 600	2 420	2 220	109
	20 500	1 370	1 190	105	3 790	3 370	3 070	2 790	2 530	2 360	2 170	114
	21 000	1 340	1 160	110	3 700	3 290	2 990	2 720	2 470	2 310	2 120	120
PROPERTIES AND DESIGN DATA												
V_r (kN)		3 220	3 000		6 980	6 190	5 620	5 080	4 660	4 350	4 090	
R (kN)		871	763		3 110	2 600	2 240	1 930	1 680	1 510	1 360	
G (kN)		42.7	41.4		89.3	80.2	73.5	67.0	62.1	58.2	55.1	
B_r' (kN)		1 020	957		4 450	3 590	3 010	2 510	2 150	1 890	1 700	
L_u (mm)		3 740	3 590		6 590	6 370	6 210	6 070	6 000	5 920	5 810	
d (mm)		980	970		987	975	965	957	948	943	936	
b (mm)		300	300		431	427	425	422	423	422	420	
t (mm)		26.0	21.1		62.0	55.9	51.1	47.0	42.7	39.9	36.6	
w (mm)		16.5	16.0		34.5	31.0	28.4	25.9	24.0	22.5	21.3	
IMPERIAL SIZE AND WEIGHT												
Weight (lb/ft)		167	149		441	395	361	330	302	282	262	
Nominal Depth (in.)		40			36							

BEAM LOAD TABLES
W Shapes

ASTM A992, A572 grade 50
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation		W920											Approx. Deflect. (mm)
Mass (kg/m)		368	344	381	345	313	289	271	253	238	223	201	
Span in Millimetres	3 000												2
	3 500										5 940	5 420	3
	4 000			9 520	8 540	8 060	7 380	6 960	6 520	6 180	5 910	5 180	4
	4 500			9 380	8 450	7 510	6 900	6 510	6 020	5 630	5 260	4 600	6
	5 000	7 740	7 340	8 450	7 600	6 760	6 210	5 860	5 420	5 070	4 730	4 140	7
	5 500	7 590	7 090	7 680	6 910	6 140	5 650	5 330	4 920	4 610	4 300	3 770	8
	6 000	6 960	6 500	7 040	6 330	5 630	5 180	4 890	4 510	4 220	3 940	3 450	10
	6 500	6 420	6 000	6 500	5 850	5 200	4 780	4 510	4 170	3 900	3 640	3 190	11
	7 000	5 960	5 570	6 030	5 430	4 830	4 440	4 190	3 870	3 620	3 380	2 960	13
	7 500	5 560	5 200	5 630	5 070	4 500	4 140	3 910	3 610	3 380	3 150	2 760	15
	8 000	5 220	4 870	5 280	4 750	4 220	3 880	3 660	3 380	3 170	2 960	2 590	17
	8 500	4 910	4 590	4 970	4 470	3 970	3 650	3 450	3 190	2 980	2 780	2 440	20
	9 000	4 640	4 330	4 690	4 220	3 750	3 450	3 260	3 010	2 820	2 630	2 300	22
	9 500	4 390	4 110	4 450	4 000	3 560	3 270	3 090	2 850	2 670	2 490	2 180	25
	10 000	4 170	3 900	4 220	3 800	3 380	3 110	2 930	2 710	2 530	2 360	2 070	27
	10 500	3 970	3 710	4 020	3 620	3 220	2 960	2 790	2 580	2 410	2 250	1 970	30
	11 000	3 790	3 550	3 840	3 460	3 070	2 820	2 660	2 460	2 300	2 150	1 880	33
	11 500	3 630	3 390	3 670	3 300	2 940	2 700	2 550	2 350	2 200	2 060	1 800	36
	12 000	3 480	3 250	3 520	3 170	2 820	2 590	2 440	2 260	2 110	1 970	1 730	39
	12 500	3 340	3 120	3 380	3 040	2 700	2 480	2 340	2 170	2 030	1 890	1 660	42
	13 000	3 210	3 000	3 250	2 920	2 600	2 390	2 250	2 080	1 950	1 820	1 590	46
	13 500	3 090	2 890	3 130	2 820	2 500	2 300	2 170	2 010	1 880	1 750	1 530	50
	14 000	2 980	2 790	3 020	2 710	2 410	2 220	2 090	1 930	1 810	1 690	1 480	53
	14 500	2 880	2 690	2 910	2 620	2 330	2 140	2 020	1 870	1 750	1 630	1 430	57
	15 000	2 780	2 600	2 820	2 530	2 250	2 070	1 950	1 810	1 690	1 580	1 380	61
	15 500	2 690	2 520	2 720	2 450	2 180	2 000	1 890	1 750	1 630	1 530	1 340	65
	16 000	2 610	2 440	2 640	2 380	2 110	1 940	1 830	1 690	1 580	1 480	1 290	70
	16 500	2 530	2 360	2 560	2 300	2 050	1 880	1 780	1 640	1 540	1 430	1 260	74
	17 000	2 450	2 290	2 480	2 240	1 990	1 830	1 720	1 590	1 490	1 390	1 220	79
	17 500	2 380	2 230	2 410	2 170	1 930	1 770	1 670	1 550	1 450	1 350	1 180	83
	18 000	2 320	2 170	2 350	2 110	1 880	1 730	1 630	1 500	1 410	1 310	1 150	88
	18 500	2 260	2 110	2 280	2 050	1 830	1 680	1 580	1 460	1 370	1 280	1 120	93
	19 000	2 200	2 050	2 220	2 000	1 780	1 630	1 540	1 430	1 330	1 240	1 090	98
	19 500	2 140	2 000	2 170	1 950	1 730	1 590	1 500	1 390	1 300	1 210	1 060	103
	20 000	2 090	1 950	2 110	1 900	1 690	1 550	1 470	1 350	1 270	1 180	1 040	109
PROPERTIES AND DESIGN DATA													
V_r (kN)		3 870	3 670	4 760	4 270	4 030	3 690	3 480	3 260	3 090	2 970	2 710	
R (kN)		1 250	1 140	1 740	1 480	1 300	1 140	1 050	947	869	805	710	
G (kN)		52.5	49.9	63.1	57.2	54.6	50.2	47.6	44.8	42.7	41.1	39.3	
B_r' (kN)		1 540	1 390	2 230	1 830	1 660	1 410	1 270	1 120	1 020	945	864	
L_u (mm)		5 750	5 680	4 250	4 170	4 060	4 030	3 970	3 950	3 890	3 830	3 720	
d (mm)		931	927	951	943	932	927	923	919	915	911	903	
b (mm)		419	418	310	308	309	308	307	306	305	304	304	
t (mm)		34.3	32.0	43.9	39.9	34.5	32.0	30.0	27.9	25.9	23.9	20.1	
w (mm)		20.3	19.3	24.4	22.1	21.1	19.4	18.4	17.3	16.5	15.9	15.2	
IMPERIAL SIZE AND WEIGHT													
Weight (lb/ft)		247	231	256	232	210	194	182	170	160	150	135	
Nominal Depth (in.)		36											

Sections highlighted in yellow are commonly used sizes and are generally readily available.

BEAM LOAD TABLES
W Shapes

ASTM A992, A572 grade 50
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation		W840								Approx. Deflect. (mm)
Mass (kg/m)		576	527	473	433	392	359	329	299	
Span in Millimetres	4 000									5
	4 500								6 380	6
	5 000	12 000	10 900	9 660	8 860	7 940	7 500	6 960	6 310	7
	5 500	11 600	10 500	9 390	8 540	7 680	6 960	6 320	5 740	9
	6 000	10 600	9 650	8 610	7 820	7 040	6 380	5 800	5 260	11
	6 500	9 780	8 900	7 950	7 220	6 500	5 890	5 350	4 850	13
	7 000	9 080	8 270	7 380	6 710	6 030	5 460	4 970	4 510	15
	7 500	8 480	7 720	6 890	6 260	5 630	5 100	4 640	4 210	17
	8 000	7 950	7 230	6 460	5 870	5 280	4 780	4 350	3 940	19
	8 500	7 480	6 810	6 080	5 520	4 970	4 500	4 090	3 710	22
	9 000	7 070	6 430	5 740	5 220	4 690	4 250	3 860	3 510	24
	9 500	6 690	6 090	5 440	4 940	4 450	4 030	3 660	3 320	27
	10 000	6 360	5 790	5 170	4 690	4 220	3 830	3 480	3 150	30
	10 500	6 060	5 510	4 920	4 470	4 020	3 640	3 310	3 000	33
	11 000	5 780	5 260	4 700	4 270	3 840	3 480	3 160	2 870	36
	11 500	5 530	5 030	4 490	4 080	3 670	3 330	3 020	2 740	39
	12 000	5 300	4 820	4 310	3 910	3 520	3 190	2 900	2 630	43
	12 500	5 090	4 630	4 130	3 760	3 380	3 060	2 780	2 520	47
	13 000	4 890	4 450	3 970	3 610	3 250	2 940	2 680	2 430	50
	13 500	4 710	4 290	3 830	3 480	3 130	2 830	2 580	2 340	54
	14 000	4 540	4 130	3 690	3 350	3 020	2 730	2 480	2 250	58
	14 500	4 390	3 990	3 560	3 240	2 910	2 640	2 400	2 180	63
	15 000	4 240	3 860	3 440	3 130	2 820	2 550	2 320	2 100	67
	15 500	4 100	3 730	3 330	3 030	2 720	2 470	2 240	2 040	72
	16 000	3 970	3 620	3 230	2 930	2 640	2 390	2 170	1 970	76
	16 500	3 850	3 510	3 130	2 850	2 560	2 320	2 110	1 910	81
	17 000	3 740	3 400	3 040	2 760	2 480	2 250	2 050	1 860	86
	17 500	3 630	3 310	2 950	2 680	2 410	2 190	1 990	1 800	91
	18 000	3 530	3 220	2 870	2 610	2 350	2 130	1 930	1 750	96
	18 500	3 440	3 130	2 790	2 540	2 280	2 070	1 880	1 710	102
	19 000	3 350	3 050	2 720	2 470	2 220	2 010	1 830	1 660	107
	19 500	3 260	2 970	2 650	2 410	2 170	1 960	1 780	1 620	113
	20 000	3 180	2 890	2 580	2 350	2 110	1 910	1 740	1 580	119
	20 500	3 100	2 820	2 520	2 290	2 060	1 870	1 700	1 540	125
	21 000	3 030	2 760	2 460	2 240	2 010	1 820	1 660	1 500	131
PROPERTIES AND DESIGN DATA										
V_r (kN)		5 990	5 460	4 830	4 430	3 970	3 750	3 480	3 190	
R (kN)		2 750	2 380	1 990	1 740	1 480	1 320	1 170	1 020	
G (kN)		82.8	76.3	68.3	63.1	57.2	54.6	51.0	47.1	
B_r' (kN)		3 830	3 250	2 610	2 230	1 830	1 660	1 450	1 240	
L_u (mm)		6 320	6 150	5 980	5 850	5 720	5 630	5 530	5 430	
d (mm)		913	903	893	885	877	868	862	855	
b (mm)		411	409	406	404	401	403	401	400	
t (mm)		57.9	53.1	48.0	43.9	39.9	35.6	32.4	29.2	
w (mm)		32.0	29.5	26.4	24.4	22.1	21.1	19.7	18.2	
IMPERIAL SIZE AND WEIGHT										
Weight (lb/ft)		387	354	318	291	263	241	221	201	
Nominal Depth (in.)		33								

BEAM LOAD TABLES
W Shapes

ASTM A992, A572 grade 50
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W840					Approx. Deflect. (mm)	W760					Approx. Deflect. (mm)
Mass (kg/m)	251	226	210	193	176		582	531	484	434	389	
Span in Millimetres												
3 000						3						3
3 500			5 340	5 060	4 600	4						4
4 000	5 980	5 620	5 240	4 730	4 230	5						5
4 500	5 690	5 060	4 650	4 210	3 760	6	11 900	10 800	9 780		7 760	7
5 000	5 120	4 550	4 190	3 790	3 380	7	11 800	10 700	9 690	8 640	7 700	8
5 500	4 650	4 140	3 810	3 440	3 080	9	10 700	9 760	8 810	7 860	7 000	10
6 000	4 260	3 790	3 490	3 150	2 820	11	9 850	8 940	8 070	7 200	6 420	12
6 500	3 940	3 500	3 220	2 910	2 600	13	9 100	8 250	7 450	6 650	5 920	14
7 000	3 660	3 250	2 990	2 700	2 420	15	8 450	7 660	6 920	6 170	5 500	16
7 500	3 410	3 030	2 790	2 520	2 260	17	7 880	7 150	6 460	5 760	5 130	19
8 000	3 200	2 840	2 620	2 370	2 110	19	7 390	6 710	6 050	5 400	4 810	21
8 500	3 010	2 680	2 460	2 230	1 990	22	6 960	6 310	5 700	5 080	4 530	24
9 000	2 840	2 530	2 330	2 100	1 880	24	6 570	5 960	5 380	4 800	4 280	27
9 500	2 690	2 400	2 200	1 990	1 780	27	6 220	5 650	5 100	4 550	4 050	30
10 000	2 560	2 280	2 090	1 890	1 690	30	5 910	5 370	4 840	4 320	3 850	33
10 500	2 440	2 170	1 990	1 800	1 610	33	5 630	5 110	4 610	4 120	3 670	36
11 000	2 330	2 070	1 900	1 720	1 540	36	5 370	4 880	4 400	3 930	3 500	40
11 500	2 220	1 980	1 820	1 650	1 470	39	5 140	4 670	4 210	3 760	3 350	44
12 000	2 130	1 900	1 750	1 580	1 410	43	4 930	4 470	4 040	3 600	3 210	47
12 500	2 050	1 820	1 680	1 510	1 350	47	4 730	4 290	3 880	3 460	3 080	51
13 000	1 970	1 750	1 610	1 460	1 300	50	4 550	4 130	3 730	3 320	2 960	56
13 500	1 900	1 690	1 550	1 400	1 250	54	4 380	3 970	3 590	3 200	2 850	60
14 000	1 830	1 630	1 500	1 350	1 210	58	4 220	3 830	3 460	3 090	2 750	64
14 500	1 760	1 570	1 440	1 310	1 170	63	4 080	3 700	3 340	2 980	2 660	69
15 000	1 710	1 520	1 400	1 260	1 130	67	3 940	3 580	3 230	2 880	2 570	74
15 500	1 650	1 470	1 350	1 220	1 090	72	3 810	3 460	3 130	2 790	2 480	79
16 000	1 600	1 420	1 310	1 180	1 060	76	3 690	3 350	3 030	2 700	2 410	84
16 500	1 550	1 380	1 270	1 150	1 030	81	3 580	3 250	2 940	2 620	2 330	90
17 000	1 510	1 340	1 230	1 110	995	86	3 480	3 160	2 850	2 540	2 260	95
17 500	1 460	1 300	1 200	1 080	967	91	3 380	3 070	2 770	2 470	2 200	101
18 000	1 420	1 260	1 160	1 050	940	96	3 280	2 980	2 690	2 400	2 140	107
18 500	1 380	1 230	1 130	1 020	914	102	3 200	2 900	2 620	2 340	2 080	113
19 000	1 350	1 200	1 100	996	890	107	3 110	2 820	2 550	2 270	2 030	119
19 500	1 310	1 170	1 070	971	867	113	3 030	2 750	2 480	2 220	1 970	125
20 000	1 280	1 140	1 050	946	846	119	2 960	2 680	2 420	2 160	1 930	132
PROPERTIES AND DESIGN DATA												
V_r (kN)	2 990	2 810	2 670	2 530	2 300		5 960	5 380	4 890	4 320	3 880	
R (kN)	985	863	787	711	635		3 110	2 670	2 310	1 930	1 630	
G (kN)	44.0	41.7	39.8	38.0	36.2		89.3	81.5	75.0	67.0	61.1	
B_r' (kN)	1 080	969	886	808	733		4 450	3 710	3 140	2 510	2 080	
L_u (mm)	3 890	3 830	3 770	3 690	3 610		6 460	6 230	6 040	5 830	5 640	
d (mm)	859	851	846	840	835		843	833	823	813	803	
b (mm)	292	294	293	292	292		396	393	390	387	385	
t (mm)	31.0	26.8	24.4	21.7	18.8		62.0	56.9	52.1	47.0	41.9	
w (mm)	17.0	16.1	15.4	14.7	14.0		34.5	31.5	29.0	25.9	23.6	
IMPERIAL SIZE AND WEIGHT												
Weight (lb/ft)	169	152	141	130	118		391	357	326	292	261	
Nominal Depth (in.)	33						30					

Sections highlighted in yellow are commonly used sizes and are generally readily available.

BEAM LOAD TABLES
W Shapes

ASTM A992, A572 grade 50
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W760											Approx. Deflect. (mm)
Mass (kg/m)	350	314	284	257	220	196	185	173	161	147	134	
Span in Millimetres 3 000								4 500	4 280	4 080	3 300	3
3 500					5 260	4 920	4 680	4 410	4 020	3 620	3 290	4
4 000					5 090	4 450	4 150	3 860	3 510	3 170	2 880	5
4 500		6 340	5 740	5 260	4 520	3 960	3 690	3 430	3 120	2 820	2 560	7
5 000	6 880	6 110	5 510	4 950	4 070	3 560	3 320	3 090	2 810	2 530	2 300	8
5 500	6 280	5 560	5 010	4 500	3 700	3 240	3 020	2 800	2 560	2 300	2 090	10
6 000	5 750	5 090	4 600	4 130	3 390	2 970	2 770	2 570	2 340	2 110	1 920	12
6 500	5 310	4 700	4 240	3 810	3 130	2 740	2 560	2 370	2 160	1 950	1 770	14
7 000	4 930	4 360	3 940	3 540	2 910	2 540	2 370	2 200	2 010	1 810	1 640	16
7 500	4 600	4 070	3 680	3 300	2 710	2 370	2 220	2 060	1 870	1 690	1 530	19
8 000	4 320	3 820	3 450	3 100	2 540	2 230	2 080	1 930	1 760	1 580	1 440	21
8 500	4 060	3 590	3 240	2 910	2 390	2 100	1 960	1 810	1 650	1 490	1 350	24
9 000	3 840	3 390	3 060	2 750	2 260	1 980	1 850	1 710	1 560	1 410	1 280	27
9 500	3 630	3 220	2 900	2 610	2 140	1 870	1 750	1 620	1 480	1 330	1 210	30
10 000	3 450	3 060	2 760	2 480	2 030	1 780	1 660	1 540	1 410	1 270	1 150	33
10 500	3 290	2 910	2 630	2 360	1 940	1 700	1 580	1 470	1 340	1 210	1 100	36
11 000	3 140	2 780	2 510	2 250	1 850	1 620	1 510	1 400	1 280	1 150	1 050	40
11 500	3 000	2 660	2 400	2 150	1 770	1 550	1 450	1 340	1 220	1 100	1 000	44
12 000	2 880	2 550	2 300	2 060	1 700	1 480	1 380	1 290	1 170	1 060	958	47
12 500	2 760	2 440	2 210	1 980	1 630	1 420	1 330	1 230	1 120	1 010	920	51
13 000	2 660	2 350	2 120	1 910	1 560	1 370	1 280	1 190	1 080	974	885	56
13 500	2 560	2 260	2 040	1 830	1 510	1 320	1 230	1 140	1 040	938	852	60
14 000	2 470	2 180	1 970	1 770	1 450	1 270	1 190	1 100	1 000	905	821	64
14 500	2 380	2 110	1 900	1 710	1 400	1 230	1 150	1 060	970	874	793	69
15 000	2 300	2 040	1 840	1 650	1 360	1 190	1 110	1 030	937	845	767	74
15 500	2 230	1 970	1 780	1 600	1 310	1 150	1 070	995	907	817	742	79
16 000	2 160	1 910	1 720	1 550	1 270	1 110	1 040	964	879	792	719	84
16 500	2 090	1 850	1 670	1 500	1 230	1 080	1 010	935	852	768	697	90
17 000	2 030	1 800	1 620	1 460	1 200	1 050	978	907	827	745	677	95
17 500	1 970	1 750	1 580	1 420	1 160	1 020	950	881	803	724	657	101
18 000	1 920	1 700	1 530	1 380	1 130	989	923	857	781	704	639	107
18 500	1 870	1 650	1 490	1 340	1 100	963	898	834	760	685	622	113
19 000	1 820	1 610	1 450	1 300	1 070	937	875	812	740	667	605	119
19 500	1 770	1 570	1 410	1 270	1 040	913	852	791	721	650	590	125
20 000	1 730	1 530	1 380	1 240	1 020	891	831	771	703	633	575	132
PROPERTIES AND DESIGN DATA												
V_r (kN)	3 440	3 170	2 870	2 630	2 630	2 460	2 340	2 250	2 140	2 040	1 650	
R (kN)	1 380	1 190	1 030	895	939	814	749	695	633	574	499	
G (kN)	54.6	51.0	46.6	43.0	42.7	40.4	38.6	37.3	35.7	34.2	30.8	
B_r' (kN)	1 660	1 450	1 210	1 030	1 020	910	830	775	712	651	529	
L_u (mm)	5 510	5 420	5 300	5 230	3 570	3 500	3 450	3 410	3 330	3 260	3 230	
d (mm)	795	786	779	773	779	770	766	762	758	753	750	
b (mm)	382	384	382	381	266	268	267	267	266	265	264	
t (mm)	38.1	33.4	30.1	27.1	30.0	25.4	23.6	21.6	19.3	17.0	15.5	
w (mm)	21.1	19.7	18.0	16.6	16.5	15.6	14.9	14.4	13.8	13.2	11.9	
IMPERIAL SIZE AND WEIGHT												
Weight (lb/ft)	235	211	191	173	148	132	124	116	108	99	90	
Nominal Depth (in.)	30											

Sections highlighted in yellow are commonly used sizes and are generally readily available.

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation		W690											Approx. Deflect. (mm)
Mass (kg/m)		548	500	457	419	384	350	323	289	265	240	217	
Span in Millimetres	4 000									5 320	4 820	4 380	6
	4 500	11 100	10 000	9 100	8 200	7 520	6 900	6 240	5 560	5 150	4 650	4 200	7
	5 000	10 100	9 190	8 350	7 600	6 960	6 260	5 810	5 120	4 640	4 190	3 780	9
	5 500	9 210	8 360	7 590	6 910	6 320	5 690	5 280	4 650	4 210	3 810	3 440	11
	6 000	8 450	7 660	6 960	6 330	5 800	5 220	4 840	4 260	3 860	3 490	3 150	13
	6 500	7 800	7 070	6 420	5 850	5 350	4 820	4 470	3 940	3 570	3 220	2 910	15
	7 000	7 240	6 560	5 960	5 430	4 970	4 470	4 150	3 660	3 310	2 990	2 700	18
	7 500	6 760	6 130	5 560	5 070	4 640	4 170	3 880	3 410	3 090	2 790	2 520	20
	8 000	6 330	5 740	5 220	4 750	4 350	3 910	3 630	3 200	2 900	2 620	2 360	23
	8 500	5 960	5 410	4 910	4 470	4 090	3 680	3 420	3 010	2 730	2 460	2 220	26
	9 000	5 630	5 110	4 640	4 220	3 860	3 480	3 230	2 840	2 580	2 330	2 100	29
	9 500	5 330	4 840	4 390	4 000	3 660	3 290	3 060	2 690	2 440	2 200	1 990	33
	10 000	5 070	4 600	4 170	3 800	3 480	3 130	2 910	2 560	2 320	2 090	1 890	36
	10 500	4 830	4 380	3 970	3 620	3 310	2 980	2 770	2 440	2 210	1 990	1 800	40
	11 000	4 610	4 180	3 790	3 460	3 160	2 850	2 640	2 330	2 110	1 900	1 720	44
	11 500	4 410	4 000	3 630	3 300	3 020	2 720	2 530	2 220	2 020	1 820	1 640	48
	12 000	4 220	3 830	3 480	3 170	2 900	2 610	2 420	2 130	1 930	1 750	1 580	52
	12 500	4 050	3 680	3 340	3 040	2 780	2 500	2 330	2 050	1 850	1 680	1 510	57
	13 000	3 900	3 530	3 210	2 920	2 680	2 410	2 240	1 970	1 780	1 610	1 450	61
	13 500	3 750	3 400	3 090	2 820	2 580	2 320	2 150	1 900	1 720	1 550	1 400	66
	14 000	3 620	3 280	2 980	2 710	2 480	2 240	2 080	1 830	1 660	1 500	1 350	71
	14 500	3 490	3 170	2 880	2 620	2 400	2 160	2 000	1 760	1 600	1 440	1 300	76
	15 000	3 380	3 060	2 780	2 530	2 320	2 090	1 940	1 710	1 550	1 400	1 260	82
	15 500	3 270	2 960	2 690	2 450	2 240	2 020	1 880	1 650	1 500	1 350	1 220	87
	16 000	3 170	2 870	2 610	2 380	2 170	1 960	1 820	1 600	1 450	1 310	1 180	93
	16 500	3 070	2 790	2 530	2 300	2 110	1 900	1 760	1 550	1 400	1 270	1 150	99
	17 000	2 980	2 700	2 450	2 240	2 050	1 840	1 710	1 510	1 360	1 230	1 110	105
	17 500	2 900	2 630	2 380	2 170	1 990	1 790	1 660	1 460	1 320	1 200	1 080	111
	18 000	2 820	2 550	2 320	2 110	1 930	1 740	1 610	1 420	1 290	1 160	1 050	117
	18 500	2 740	2 480	2 260	2 050	1 880	1 690	1 570	1 380	1 250	1 130	1 020	124
	19 000	2 670	2 420	2 200	2 000	1 830	1 650	1 530	1 350	1 220	1 100	995	131
	19 500	2 600	2 360	2 140	1 950	1 780	1 610	1 490	1 310	1 190	1 070	969	138
	20 000	2 530	2 300	2 090	1 900	1 740	1 560	1 450	1 280	1 160	1 050	945	145
	20 500	2 470	2 240	2 040	1 850	1 700	1 530	1 420	1 250	1 130	1 020	922	152
	21 000	2 410	2 190	1 990	1 810	1 660	1 490	1 380	1 220	1 100	997	900	160
PROPERTIES AND DESIGN DATA													
V_r (kN)		5 550	5 000	4 550	4 100	3 760	3 450	3 120	2 780	2 660	2 410	2 190	
R (kN)		3 200	2 750	2 380	2 060	1 800	1 580	1 380	1 160	1 050	911	794	
G (kN)		90.8	82.8	76.3	69.6	64.4	59.8	54.6	49.2	47.6	43.5	39.8	
B_r' (kN)		4 610	3 830	3 250	2 700	2 320	1 990	1 660	1 350	1 270	1 060	886	
L_u (mm)		6 400	6 140	5 920	5 730	5 550	5 410	5 300	5 160	5 060	4 960	4 890	
d (mm)		772	762	752	744	736	728	722	714	706	701	695	
b (mm)		372	369	367	364	362	360	359	356	358	356	355	
t (mm)		63.0	57.9	53.1	49.0	45.0	40.9	38.1	34.0	30.2	27.4	24.8	
w (mm)		35.1	32.0	29.5	26.9	24.9	23.1	21.1	19.0	18.4	16.8	15.4	
IMPERIAL SIZE AND WEIGHT													
Weight (lb/ft)		368	336	307	281	258	235	217	194	178	161	146	
Nominal Depth (in.)		27											

BEAM LOAD TABLES
W Shapes

ASTM A992, A572 grade 50
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W690					Approx. Deflect. (mm)	W610					Approx. Deflect. (mm)
Mass (kg/m)	192	170	152	140	125		551	498	455	415	372	
Span in Millimetres												
3 000		4 120	3 700	3 480	3 220	3						4
3 500	4 460	3 990	3 550	3 230	2 850	4						5
4 000	4 010	3 490	3 110	2 830	2 490	6	11 200	10 100	9 040	8 200	7 240	7
4 500	3 570	3 100	2 760	2 510	2 210	7	10 300	9 220	8 340	7 560	6 730	8
5 000	3 210	2 790	2 480	2 260	1 990	9	9 240	8 300	7 500	6 810	6 060	10
5 500	2 920	2 540	2 260	2 050	1 810	11	8 400	7 540	6 820	6 190	5 510	12
6 000	2 670	2 330	2 070	1 880	1 660	13	7 700	6 910	6 250	5 670	5 050	15
6 500	2 470	2 150	1 910	1 740	1 530	15	7 110	6 380	5 770	5 240	4 660	17
7 000	2 290	1 990	1 770	1 610	1 420	18	6 600	5 930	5 360	4 860	4 330	20
7 500	2 140	1 860	1 660	1 510	1 330	20	6 160	5 530	5 000	4 540	4 040	23
8 000	2 010	1 750	1 550	1 410	1 250	23	5 780	5 190	4 690	4 250	3 790	26
8 500	1 890	1 640	1 460	1 330	1 170	26	5 440	4 880	4 410	4 000	3 570	30
9 000	1 780	1 550	1 380	1 260	1 110	29	5 130	4 610	4 170	3 780	3 370	33
9 500	1 690	1 470	1 310	1 190	1 050	33	4 860	4 370	3 950	3 580	3 190	37
10 000	1 600	1 400	1 240	1 130	996	36	4 620	4 150	3 750	3 400	3 030	41
10 500	1 530	1 330	1 180	1 080	949	40	4 400	3 950	3 570	3 240	2 890	45
11 000	1 460	1 270	1 130	1 030	906	44	4 200	3 770	3 410	3 090	2 750	50
11 500	1 400	1 210	1 080	983	866	48	4 020	3 610	3 260	2 960	2 640	54
12 000	1 340	1 160	1 040	942	830	52	3 850	3 460	3 130	2 840	2 530	59
12 500	1 280	1 120	994	904	797	57	3 700	3 320	3 000	2 720	2 420	64
13 000	1 230	1 070	955	869	766	61	3 550	3 190	2 890	2 620	2 330	69
13 500	1 190	1 030	920	837	738	66	3 420	3 070	2 780	2 520	2 240	75
14 000	1 150	997	887	807	711	71	3 300	2 960	2 680	2 430	2 160	80
14 500	1 110	963	857	779	687	76	3 190	2 860	2 590	2 350	2 090	86
15 000	1 070	931	828	753	664	82	3 080	2 770	2 500	2 270	2 020	92
15 500	1 040	901	801	729	643	87	2 980	2 680	2 420	2 200	1 960	98
16 000	1 000	873	776	706	623	93	2 890	2 590	2 340	2 130	1 890	105
16 500	973	846	753	685	604	99	2 800	2 510	2 270	2 060	1 840	112
17 000	944	821	731	665	586	105	2 720	2 440	2 210	2 000	1 780	118
17 500	917	798	710	646	569	111	2 640	2 370	2 140	1 940	1 730	126
18 000	891	776	690	628	553	117	2 570	2 300	2 080	1 890	1 680	133
18 500	867	755	671	611	538	124	2 500	2 240	2 030	1 840	1 640	140
19 000	845	735	654	595	524	131	2 430	2 180	1 970	1 790	1 590	148
19 500	823	716	637	580	511	138	2 370	2 130	1 920	1 750	1 550	156
20 000	802	698	621	565	498	145	2 310	2 070	1 880	1 700	1 520	164
PROPERTIES AND DESIGN DATA												
V_r (kN)	2 230	2 060	1 850	1 740	1 610		5 620	5 030	4 520	4 100	3 620	
R (kN)	849	729	625	563	500		3 760	3 200	2 750	2 380	1 990	
G (kN)	40.1	37.5	33.9	32.1	30.3		99.9	90.8	82.8	76.3	68.3	
B_r' (kN)	898	786	641	575	512		5 570	4 610	3 830	3 250	2 610	
L_u (mm)	3 440	3 380	3 320	3 270	3 190		6 620	6 230	5 940	5 700	5 450	
d (mm)	702	693	688	684	678		711	699	689	679	669	
b (mm)	254	256	254	254	253		347	343	340	338	335	
t (mm)	27.9	23.6	21.1	18.9	16.3		69.1	63.0	57.9	53.1	48.0	
w (mm)	15.5	14.5	13.1	12.4	11.7		38.6	35.1	32.0	29.5	26.4	
IMPERIAL SIZE AND WEIGHT												
Weight (lb/ft)	129	114	102	94	84		370	335	306	279	250	
Nominal Depth (in.)	27						24					

Sections highlighted in yellow are commonly used sizes and are generally readily available.

BEAM LOAD TABLES
W Shapes

ASTM A992, A572 grade 50
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W610									Approx. Deflect. (mm)
Mass (kg/m)	341	307	285	262	241	217	195	174	155	
Span in Millimetres										
3 000										4
3 500							3 920	3 540	3 180	5
4 000	6 620	5 920	5 460	5 000	4 660	4 240	3 770	3 330	2 940	7
4 500	6 130	5 480	5 060	4 610	4 230	3 780	3 350	2 960	2 610	8
5 000	5 510	4 930	4 560	4 150	3 810	3 400	3 020	2 660	2 350	10
5 500	5 010	4 480	4 140	3 770	3 460	3 090	2 740	2 420	2 140	12
6 000	4 600	4 110	3 800	3 460	3 180	2 840	2 510	2 220	1 960	15
6 500	4 240	3 790	3 500	3 190	2 930	2 620	2 320	2 050	1 810	17
7 000	3 940	3 520	3 250	2 960	2 720	2 430	2 150	1 900	1 680	20
7 500	3 680	3 290	3 040	2 770	2 540	2 270	2 010	1 780	1 570	23
8 000	3 450	3 080	2 850	2 590	2 380	2 130	1 880	1 660	1 470	26
8 500	3 240	2 900	2 680	2 440	2 240	2 000	1 770	1 570	1 380	30
9 000	3 060	2 740	2 530	2 300	2 120	1 890	1 680	1 480	1 310	33
9 500	2 900	2 600	2 400	2 180	2 010	1 790	1 590	1 400	1 240	37
10 000	2 760	2 470	2 280	2 070	1 910	1 700	1 510	1 330	1 170	41
10 500	2 630	2 350	2 170	1 980	1 810	1 620	1 440	1 270	1 120	45
11 000	2 510	2 240	2 070	1 890	1 730	1 550	1 370	1 210	1 070	50
11 500	2 400	2 140	1 980	1 800	1 660	1 480	1 310	1 160	1 020	54
12 000	2 300	2 060	1 900	1 730	1 590	1 420	1 260	1 110	979	59
12 500	2 210	1 970	1 820	1 660	1 520	1 360	1 210	1 070	940	64
13 000	2 120	1 900	1 750	1 600	1 470	1 310	1 160	1 020	904	69
13 500	2 040	1 830	1 690	1 540	1 410	1 260	1 120	986	870	75
14 000	1 970	1 760	1 630	1 480	1 360	1 220	1 080	951	839	80
14 500	1 900	1 700	1 570	1 430	1 310	1 170	1 040	918	810	86
15 000	1 840	1 640	1 520	1 380	1 270	1 130	1 010	888	783	92
15 500	1 780	1 590	1 470	1 340	1 230	1 100	973	859	758	98
16 000	1 720	1 540	1 420	1 300	1 190	1 060	942	832	734	105
16 500	1 670	1 490	1 380	1 260	1 150	1 030	914	807	712	112
17 000	1 620	1 450	1 340	1 220	1 120	1 000	887	783	691	118
17 500	1 580	1 410	1 300	1 190	1 090	972	862	761	671	126
18 000	1 530	1 370	1 270	1 150	1 060	945	838	740	653	133
18 500	1 490	1 330	1 230	1 120	1 030	920	815	720	635	140
19 000	1 450	1 300	1 200	1 090	1 000	896	794	701	618	148
PROPERTIES AND DESIGN DATA										
V_r (kN)	3 310	2 960	2 730	2 500	2 330	2 120	1 960	1 770	1 590	
R (kN)	1 740	1 480	1 320	1 160	1 040	900	787	675	578	
G (kN)	63.1	57.2	53.3	49.2	46.3	42.7	39.8	36.2	32.9	
B_r' (kN)	2 230	1 830	1 590	1 350	1 200	1 020	886	733	603	
L_u (mm)	5 250	5 080	4 980	4 850	4 790	4 680	4 570	4 480	4 400	
d (mm)	661	653	647	641	635	628	622	616	611	
b (mm)	333	330	329	327	329	328	327	325	324	
t (mm)	43.9	39.9	37.1	34.0	31.0	27.7	24.4	21.6	19.0	
w (mm)	24.4	22.1	20.6	19.0	17.9	16.5	15.4	14.0	12.7	
IMPERIAL SIZE AND WEIGHT										
Weight (lb/ft)	229	207	192	176	162	146	131	117	104	
Nominal Depth (in.)	24									

Sections highlighted in yellow are commonly used sizes and are generally readily available.

BEAM LOAD TABLES
W Shapes

ASTM A992, A572 grade 50
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W610							Approx. Deflect. (mm)	W530			Approx. Deflect. (mm)
Mass (kg/m)	153	140	125	113	101	92	82		409	369	332	
Span in Millimetres 2 000						2 700	2 340	2				2
2 500				2 800	2 600	2 490	2 190	3				3
3 000	3 580	3 320	2 980	2 720	2 400	2 080	1 820	4				4
3 500	3 260	2 950	2 600	2 330	2 060	1 780	1 560	5	7 780	6 900	6 180	6
4 000	2 860	2 580	2 280	2 040	1 800	1 560	1 370	7	7 510	6 710	6 000	8
4 500	2 540	2 290	2 030	1 820	1 600	1 390	1 210	8	6 680	5 960	5 330	10
5 000	2 290	2 060	1 820	1 630	1 440	1 250	1 090	10	6 010	5 370	4 800	12
5 500	2 080	1 870	1 660	1 490	1 310	1 130	994	12	5 460	4 880	4 360	14
6 000	1 900	1 720	1 520	1 360	1 200	1 040	911	15	5 010	4 470	4 000	17
6 500	1 760	1 590	1 400	1 260	1 110	959	841	17	4 620	4 130	3 690	20
7 000	1 630	1 470	1 300	1 170	1 030	891	781	20	4 290	3 830	3 430	23
7 500	1 520	1 370	1 220	1 090	960	831	729	23	4 010	3 580	3 200	27
8 000	1 430	1 290	1 140	1 020	900	779	683	26	3 760	3 350	3 000	30
8 500	1 340	1 210	1 070	961	847	734	643	30	3 540	3 160	2 820	34
9 000	1 270	1 150	1 010	908	800	693	607	33	3 340	2 980	2 670	38
9 500	1 200	1 090	960	860	758	656	575	37	3 160	2 820	2 530	43
10 000	1 140	1 030	912	817	720	623	546	41	3 010	2 680	2 400	47
10 500	1 090	982	868	778	686	594	520	45	2 860	2 550	2 290	52
11 000	1 040	937	829	743	655	567	497	50	2 730	2 440	2 180	57
11 500	994	896	793	711	626	542	475	54	2 610	2 330	2 090	62
12 000	952	859	760	681	600	520	455	59	2 500	2 240	2 000	68
12 500	914	825	729	654	576	499	437	64	2 400	2 150	1 920	74
13 000	879	793	701	629	554	480	420	69	2 310	2 060	1 850	80
13 500	846	764	675	605	534	462	405	75	2 230	1 990	1 780	86
14 000	816	736	651	584	515	445	390	80	2 150	1 920	1 710	92
14 500	788	711	629	564	497	430	377	86	2 070	1 850	1 650	99
15 000	762	687	608	545	480	416	364	92	2 000	1 790	1 600	106
15 500	737	665	588	527	465	402	353	98	1 940	1 730	1 550	113
16 000	714	644	570	511	450	390	342	105	1 880	1 680	1 500	121
16 500	693	625	553	495	437	378	331	112	1 820	1 630	1 450	128
17 000	672	606	536	481	424	367	321	118	1 770	1 580	1 410	136
17 500	653	589	521	467	412	356	312	126	1 720	1 530	1 370	144
18 000	635	573	506	454	400	346	304	133	1 670	1 490	1 330	153
18 500	618	557	493	442	389	337	295	140	1 620	1 450	1 300	161
19 000	601	543	480	430	379	328	288	148	1 580	1 410	1 260	170
PROPERTIES AND DESIGN DATA												
V_r (kN)	1 790	1 660	1 490	1 400	1 300	1 350	1 170		3 890	3 450	3 090	
R (kN)	723	640	549	490	434	451	391		2 590	2 180	1 850	
G (kN)	36.2	33.9	30.8	29.0	27.2	28.2	25.9		80.2	72.2	65.7	
B_r' (kN)	733	641	529	469	412	444	374		3 590	2 910	2 410	
L_u (mm)	3 110	3 070	3 020	2 950	2 890	2 180	2 110		6 030	5 720	5 430	
d (mm)	623	617	612	608	603	603	599		613	603	593	
b (mm)	229	230	229	228	228	179	178		327	324	322	
t (mm)	24.9	22.2	19.6	17.3	14.9	15.0	12.8		55.6	50.5	45.5	
w (mm)	14.0	13.1	11.9	11.2	10.5	10.9	10.0		31.0	27.9	25.4	
IMPERIAL SIZE AND WEIGHT												
Weight (lb/ft)	103	94	84	76	68	62	55		275	248	223	
Nominal Depth (in.)	24								21			

Sections highlighted in yellow are commonly used sizes and are generally readily available.

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W530								Approx. Deflect. (mm)
Mass (kg/m)	300	272	248	219	196	182	165	150	
Span in Millimetres									
3 000									4
3 500	5 540	4 980	4 440	4 200	3 740	3 440	3 140	2 820	6
4 000	5 380	4 840	4 380	3 790	3 390	3 130	2 830	2 580	8
4 500	4 790	4 300	3 900	3 370	3 010	2 780	2 510	2 290	10
5 000	4 310	3 870	3 510	3 040	2 710	2 500	2 260	2 060	12
5 500	3 920	3 520	3 190	2 760	2 470	2 280	2 050	1 870	14
6 000	3 590	3 230	2 920	2 530	2 260	2 090	1 880	1 720	17
6 500	3 310	2 980	2 700	2 330	2 090	1 930	1 740	1 590	20
7 000	3 080	2 760	2 510	2 170	1 940	1 790	1 610	1 470	23
7 500	2 870	2 580	2 340	2 020	1 810	1 670	1 510	1 370	27
8 000	2 690	2 420	2 190	1 900	1 700	1 560	1 410	1 290	30
8 500	2 530	2 280	2 060	1 790	1 600	1 470	1 330	1 210	34
9 000	2 390	2 150	1 950	1 690	1 510	1 390	1 260	1 150	38
9 500	2 270	2 040	1 850	1 600	1 430	1 320	1 190	1 090	43
10 000	2 150	1 940	1 750	1 520	1 360	1 250	1 130	1 030	47
10 500	2 050	1 840	1 670	1 450	1 290	1 190	1 080	982	52
11 000	1 960	1 760	1 590	1 380	1 230	1 140	1 030	937	57
11 500	1 870	1 680	1 520	1 320	1 180	1 090	983	896	62
12 000	1 790	1 610	1 460	1 260	1 130	1 040	942	859	68
12 500	1 720	1 550	1 400	1 210	1 090	1 000	904	825	74
13 000	1 660	1 490	1 350	1 170	1 040	963	869	793	80
13 500	1 600	1 430	1 300	1 120	1 000	927	837	764	86
14 000	1 540	1 380	1 250	1 080	969	894	807	736	92
14 500	1 490	1 330	1 210	1 050	935	863	779	711	99
15 000	1 440	1 290	1 170	1 010	904	835	753	687	106
15 500	1 390	1 250	1 130	979	875	808	729	665	113
16 000	1 350	1 210	1 100	949	848	782	706	644	121
PROPERTIES AND DESIGN DATA									
V_r (kN)	2 770	2 490	2 220	2 100	1 870	1 720	1 570	1 410	
R (kN)	1 590	1 370	1 170	1 030	876	777	684	595	
G (kN)	59.8	54.6	49.2	47.4	42.7	39.3	36.2	32.9	
B_r' (kN)	1 990	1 660	1 350	1 250	1 020	864	733	603	
L_u (mm)	5 210	5 020	4 880	4 720	4 600	4 530	4 440	4 380	
d (mm)	585	577	571	560	554	551	546	543	
b (mm)	319	317	315	318	316	315	313	312	
t (mm)	41.4	37.6	34.5	29.2	26.3	24.4	22.2	20.3	
w (mm)	23.1	21.1	19.0	18.3	16.5	15.2	14.0	12.7	
IMPERIAL SIZE AND WEIGHT									
Weight (lb/ft)	201	182	166	147	132	122	111	101	
Nominal Depth (in.)	21								

BEAM LOAD TABLES
W Shapes

ASTM A992, A572 grade 50
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W530										Approx. Deflect. (mm)
Mass (kg/m)	138	123	109	101	92	82	† 72	85	74	66	
Span in Millimetres											
2 000						2 060	1 890	2 260	2 100	1 850	2
2 500	3 300	2 920	2 560	2 400	2 220	2 050	1 520	2 090	1 800	1 550	3
3 000	2 990	2 660	2 340	2 170	1 950	1 710	1 270	1 740	1 500	1 290	4
3 500	2 560	2 280	2 010	1 860	1 670	1 460	1 090	1 490	1 280	1 110	6
4 000	2 240	1 990	1 760	1 630	1 470	1 280	950	1 300	1 120	969	8
4 500	1 990	1 770	1 560	1 450	1 300	1 140	845	1 160	999	861	10
5 000	1 790	1 590	1 410	1 300	1 170	1 020	760	1 040	899	775	12
5 500	1 630	1 450	1 280	1 180	1 070	930	691	948	817	705	14
6 000	1 490	1 330	1 170	1 080	977	853	633	869	749	646	17
6 500	1 380	1 230	1 080	1 000	902	787	585	803	692	596	20
7 000	1 280	1 140	1 000	930	837	731	543	745	642	554	23
7 500	1 200	1 060	937	868	782	682	507	696	599	517	27
8 000	1 120	997	879	814	733	640	475	652	562	484	30
8 500	1 050	938	827	766	690	602	447	614	529	456	34
9 000	996	886	781	723	651	569	422	580	500	431	38
9 500	944	839	740	685	617	539	400	549	473	408	43
10 000	897	797	703	651	586	512	380	522	450	388	47
10 500	854	759	669	620	558	487	362	497	428	369	52
11 000	815	725	639	592	533	465	346	474	409	352	57
11 500	780	693	611	566	510	445	330	454	391	337	62
12 000	747	664	586	542	489	426	317	435	375	323	68
12 500	717	638	562	521	469	409	304	417	360	310	74
13 000	690	613	541	501	451	394	292	401	346	298	80
13 500	664	591	521	482	434	379	282	386	333	287	86
14 000	641	570	502	465	419	366	271	373	321	277	92
14 500	618	550	485	449	404	353	262	360	310	267	99
15 000	598	532	469	434	391	341	253	348	300	258	106
15 500	579	514	454	420	378	330	245	337	290	250	113
16 000	560	498	439	407	366	320	238	326	281	242	121
PROPERTIES AND DESIGN DATA											
V_r (kN)	1 650	1 460	1 280	1 200	1 110	1 030	947	1 130	1 050	927	
R (kN)	739	626	526	478	429	377	334	442	388	335	
G (kN)	38.0	33.9	30.0	28.2	26.4	24.6	23.3	26.7	25.1	23.0	
B_r' (kN)	808	641	503	444	389	337	303	397	352	296	
L_u (mm)	2 930	2 860	2 810	2 770	2 720	2 660	2 750	2 110	2 040	1 980	
d (mm)	549	544	539	537	533	528	524	535	529	525	
b (mm)	214	212	211	210	209	209	207	166	166	165	
t (mm)	23.6	21.2	18.8	17.4	15.6	13.3	10.9	16.5	13.6	11.4	
w (mm)	14.7	13.1	11.6	10.9	10.2	9.5	9.0	10.3	9.7	8.9	
IMPERIAL SIZE AND WEIGHT											
Weight (lb/ft)	93	83	73	68	62	55	48	57	50	44	
Nominal Depth (in.)	21										

Sections highlighted in yellow are commonly used sizes and are generally readily available.

† Class 3

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W460									Approx. Deflect. (mm)
Mass (kg/m)	260	235	213	193	177	158	144	128	113	
Span in Millimetres 3 000	4 720	4 240	3 760	3 400	3 280	2 920	2 640	2 340	2 040	5
3 500	4 630	4 140	3 740	3 370	3 040	2 680	2 450	2 160	1 890	7
4 000	4 060	3 630	3 270	2 950	2 660	2 340	2 140	1 890	1 660	9
4 500	3 600	3 220	2 910	2 620	2 360	2 080	1 900	1 680	1 470	11
5 000	3 240	2 900	2 620	2 360	2 130	1 870	1 710	1 520	1 330	14
5 500	2 950	2 640	2 380	2 150	1 930	1 700	1 560	1 380	1 210	16
6 000	2 700	2 420	2 180	1 970	1 770	1 560	1 430	1 260	1 110	20
6 500	2 500	2 230	2 010	1 820	1 640	1 440	1 320	1 170	1 020	23
7 000	2 320	2 070	1 870	1 690	1 520	1 340	1 220	1 080	947	27
7 500	2 160	1 930	1 750	1 570	1 420	1 250	1 140	1 010	884	31
8 000	2 030	1 810	1 640	1 470	1 330	1 170	1 070	947	829	35
8 500	1 910	1 710	1 540	1 390	1 250	1 100	1 010	891	780	39
9 000	1 800	1 610	1 450	1 310	1 180	1 040	952	842	737	44
9 500	1 710	1 530	1 380	1 240	1 120	986	902	797	698	49
10 000	1 620	1 450	1 310	1 180	1 060	936	857	758	663	54
10 500	1 540	1 380	1 250	1 120	1 010	892	816	722	632	60
11 000	1 470	1 320	1 190	1 070	967	851	779	689	603	66
11 500	1 410	1 260	1 140	1 030	924	814	745	659	577	72
12 000	1 350	1 210	1 090	983	886	780	714	631	553	78
12 500	1 300	1 160	1 050	944	851	749	686	606	531	85
13 000	1 250	1 120	1 010	908	818	720	659	583	510	92
13 500	1 200	1 070	970	874	788	694	635	561	491	99
14 000	1 160	1 040	935	843	759	669	612	541	474	107
PROPERTIES AND DESIGN DATA										
V_r (kN)	2 360	2 120	1 880	1 700	1 640	1 460	1 320	1 170	1 020	
R (kN)	1 530	1 310	1 120	977	892	759	663	563	473	
G (kN)	58.5	53.3	47.9	44.0	43.0	38.8	35.2	31.6	27.9	
B_r' (kN)	1 910	1 590	1 280	1 080	1 030	841	691	556	436	
L_u (mm)	4 980	4 770	4 590	4 440	4 330	4 200	4 130	4 040	3 950	
d (mm)	509	501	495	489	482	476	472	467	463	
b (mm)	289	287	285	283	286	284	283	282	280	
t (mm)	40.4	36.6	33.5	30.5	26.9	23.9	22.1	19.6	17.3	
w (mm)	22.6	20.6	18.5	17.0	16.6	15.0	13.6	12.2	10.8	
IMPERIAL SIZE AND WEIGHT										
Weight (lb/ft)	175	158	143	130	119	106	97	86	76	
Nominal Depth (in.)	18									

Sections highlighted in yellow are commonly used sizes and are generally readily available.

BEAM LOAD TABLES
W Shapes

ASTM A992, A572 grade 50
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W460								Approx. Deflect. (mm)
Mass (kg/m)	106	97	89	82	74	68	60	52	
Span in Millimetres									
2 000	2 420	2 180		1 870	1 690	1 710	1 490	1 350	2
2 500	2 370	2 170	1 990	1 820	1 640	1 480	1 270	1 080	3
3 000	1 980	1 810	1 660	1 520	1 370	1 230	1 060	903	5
3 500	1 700	1 550	1 430	1 300	1 170	1 060	908	774	7
4 000	1 480	1 350	1 250	1 140	1 020	925	795	677	9
4 500	1 320	1 200	1 110	1 010	911	822	707	602	11
5 000	1 190	1 080	999	909	820	740	636	542	14
5 500	1 080	985	908	826	745	673	578	492	16
6 000	989	903	832	758	683	617	530	451	20
6 500	913	833	768	699	631	569	489	417	23
7 000	848	774	713	649	586	529	454	387	27
7 500	792	722	666	606	546	493	424	361	31
8 000	742	677	624	568	512	463	397	338	35
8 500	698	637	587	535	482	435	374	319	39
9 000	660	602	555	505	455	411	353	301	44
9 500	625	570	526	478	431	390	335	285	49
10 000	594	542	499	455	410	370	318	271	54
10 500	565	516	476	433	390	352	303	258	60
11 000	540	492	454	413	373	336	289	246	66
11 500	516	471	434	395	356	322	276	235	72
12 000	495	451	416	379	342	308	265	226	78
12 500	475	433	399	364	328	296	254	217	85
13 000	457	417	384	350	315	285	245	208	92
13 500	440	401	370	337	304	274	236	201	99
14 000	424	387	357	325	293	264	227	193	107

PROPERTIES AND DESIGN DATA

	106	97	89	82	74	68	60	52
V_r (kN)	1 210	1 090	996	933	843	856	746	680
R (kN)	595	519	464	420	368	381	317	282
G (kN)	32.6	29.5	27.2	25.6	23.3	23.5	20.7	19.7
B_r' (kN)	593	486	412	366	303	310	239	216
L_u (mm)	2 690	2 650	2 620	2 560	2 530	2 010	1 970	1 890
d (mm)	469	466	463	460	457	459	455	450
b (mm)	194	193	192	191	190	154	153	152
t (mm)	20.6	19.0	17.7	16.0	14.5	15.4	13.3	10.8
w (mm)	12.6	11.4	10.5	9.9	9.0	9.1	8.0	7.6

IMPERIAL SIZE AND WEIGHT

Weight (lb/ft)	71	65	60	55	50	46	40	35
Nominal Depth (in.)	18							

Sections highlighted in yellow are commonly used sizes and are generally readily available.

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W410									Approx. Deflect. (mm)
Mass (kg/m)	149	132	114	100	85	74	67	60	54	
Span in Millimetres										
2 000					1 860	1 640	1 480	1 280	1 240	2
2 500					1 710	1 500	1 350	1 180	1 040	4
3 000	2 640	2 320	2 000	1 700	1 420	1 250	1 130	985	869	5
3 500	2 310	2 020	1 750	1 510	1 220	1 070	965	845	745	7
4 000	2 020	1 770	1 530	1 320	1 070	938	845	739	652	10
4 500	1 790	1 570	1 360	1 180	949	834	751	657	580	12
5 000	1 610	1 420	1 220	1 060	854	750	676	591	522	15
5 500	1 470	1 290	1 110	962	777	682	614	537	474	18
6 000	1 350	1 180	1 020	882	712	625	563	493	435	22
6 500	1 240	1 090	940	814	657	577	520	455	401	26
7 000	1 150	1 010	873	756	610	536	483	422	373	30
7 500	1 080	944	815	705	570	500	450	394	348	34
8 000	1 010	885	764	661	534	469	422	369	326	39
8 500	950	833	719	622	503	441	397	348	307	44
9 000	897	787	679	588	475	417	375	328	290	49
9 500	850	745	643	557	450	395	356	311	275	55
10 000	807	708	611	529	427	375	338	296	261	61
10 500	769	674	582	504	407	357	322	282	248	67
11 000	734	644	556	481	388	341	307	269	237	74
11 500	702	616	531	460	372	326	294	257	227	81
12 000	673	590	509	441	356	313	282	246	217	88
12 500	646	566	489	423	342	300	270	236	209	95
13 000	621	545	470	407	329	289	260	227	201	103
PROPERTIES AND DESIGN DATA										
V_r (kN)	1 320	1 160	998	850	931	821	739	642	619	
R (kN)	771	650	532	434	487	412	359	301	279	
G (kN)	38.6	34.4	30.0	25.9	28.2	25.1	22.8	19.9	19.4	
B_r' (kN)	830	661	503	374	444	352	289	222	210	
L_u (mm)	4 080	3 940	3 810	3 730	2 530	2 470	2 420	2 390	2 310	
d (mm)	431	425	420	415	417	413	410	407	403	
b (mm)	265	263	261	260	181	180	179	178	177	
t (mm)	25.0	22.2	19.3	16.9	18.2	16.0	14.4	12.8	10.9	
w (mm)	14.9	13.3	11.6	10.0	10.9	9.7	8.8	7.7	7.5	
IMPERIAL SIZE AND WEIGHT										
Weight (lb/ft)	100	89	77	67	57	50	45	40	36	
Nominal Depth (in.)	16									

Sections highlighted in yellow are commonly used sizes and are generally readily available.

BEAM LOAD TABLES
ASTM A992, A572 grade 50
W Shapes
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W410		Approx. Deflect. (mm)	W360							Approx. Deflect. (mm)
Mass (kg/m)	46	39		122	110	101	91	79	72	64	
Span in Millimetres											
1 000			1								1
1 500	1 160	960	1								2
2 000	1 100	907	2								3
2 500	878	725	4	1 930				1 360	1 230	1 100	4
3 000	732	604	5	1 880	1 680	1 540	1 370	1 180	1 060	944	6
3 500	627	518	7	1 610	1 460	1 330	1 190	1 010	908	809	9
4 000	549	453	10	1 410	1 280	1 170	1 040	888	795	708	11
4 500	488	403	12	1 250	1 140	1 040	927	789	707	629	14
5 000	439	363	15	1 130	1 020	934	835	710	636	566	17
5 500	399	330	18	1 030	930	849	759	646	578	515	21
6 000	366	302	22	940	853	778	696	592	530	472	25
6 500	338	279	26	867	787	718	642	546	489	436	29
7 000	314	259	30	806	731	667	596	507	454	405	34
7 500	293	242	34	752	682	623	556	474	424	378	39
8 000	274	227	39	705	640	584	522	444	397	354	44
8 500	258	213	44	663	602	549	491	418	374	333	50
9 000	244	201	49	627	569	519	464	395	353	315	56
9 500	231	191	55	594	539	492	439	374	335	298	63
10 000	220	181	61	564	512	467	417	355	318	283	69
10 500	209	173	67	537	487	445	397	338	303	270	77
11 000	200	165	74	513	465	425	379	323	289	257	84
11 500	191	158	81	490	445	406	363	309	276	246	92
12 000	183	151	88	470	426	389	348	296	265	236	100
12 500	176	145	95	451	409	374	334	284	254	227	109
13 000	169	139	103	434	394	359	321	273	245	218	117
PROPERTIES AND DESIGN DATA											
V_r (kN)	578	480		967	841	768	687	682	617	548	
R (kN)	262	224		628	530	471	407	407	357	307	
G (kN)	18.1	16.6		33.6	29.5	27.2	24.6	24.3	22.3	19.9	
B_r' (kN)	183	153		632	486	412	337	330	276	222	
L_u (mm)	1 790	1 730		4 040	3 940	3 860	3 760	3 010	2 940	2 870	
d (mm)	403	399		363	360	357	353	354	350	347	
b (mm)	140	140		257	256	255	254	205	204	203	
t (mm)	11.2	8.8		21.7	19.9	18.3	16.4	16.8	15.1	13.5	
w (mm)	7.0	6.4		13.0	11.4	10.5	9.5	9.4	8.6	7.7	
IMPERIAL SIZE AND WEIGHT											
Weight (lb/ft)	31	26		82	74	68	61	53	48	43	
Nominal Depth (in.)	16			14							

Sections highlighted in yellow are commonly used sizes and are generally readily available.

BEAM LOAD TABLES
W Shapes

ASTM A992, A572 grade 50
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W360					Approx. Deflect. (mm)	W310					Approx. Deflect. (mm)
Mass (kg/m)	57	51	45	39	33		86	79	74	67	60	
Span in Millimetres												
1 000						1						1
1 500			996	940	792	2						2
2 000	1 160	1 050	966	822	672	3			1 190	1 070	932	3
2 500	1 000	887	773	658	538	4		1 100	1 170	1 040	927	5
3 000	836	739	644	548	448	6	1 160	1 060	977	869	773	7
3 500	717	634	552	470	384	9	1 010	908	837	745	662	10
4 000	627	555	483	411	336	11	882	795	733	652	579	13
4 500	558	493	429	365	299	14	784	707	651	580	515	16
5 000	502	444	387	329	269	17	705	636	586	522	464	20
5 500	456	403	351	299	244	21	641	578	533	474	421	24
6 000	418	370	322	274	224	25	588	530	489	435	386	29
6 500	386	341	297	253	207	29	543	489	451	401	357	34
7 000	358	317	276	235	192	34	504	454	419	373	331	40
7 500	335	296	258	219	179	39	470	424	391	348	309	45
8 000	314	277	242	206	168	44	441	397	366	326	290	52
8 500	295	261	227	193	158	50	415	374	345	307	273	58
9 000	279	246	215	183	149	56	392	353	326	290	258	65
9 500	264	233	203	173	141	63	371	335	309	275	244	73
10 000	251	222	193	164	134	69	353	318	293	261	232	81
10 500	239	211	184	157	128	77	336	303	279	248	221	89
11 000	228	202	176	149	122	84	321	289	266	237	211	98
PROPERTIES AND DESIGN DATA												
V_r (kN)	580	524	498	470	396		578	552	597	533	466	
R (kN)	312	273	249	240	201		389	361	402	348	296	
G (kN)	20.4	18.6	17.9	16.8	15.0		23.5	22.8	24.3	22.0	19.4	
B_r' (kN)	233	194	178	158	126		310	289	330	270	210	
L_u (mm)	2 360	2 320	2 260	1 660	1 600		3 900	3 810	3 100	3 020	2 960	
d (mm)	358	355	352	353	349		310	306	310	306	303	
b (mm)	172	171	171	128	127		254	254	205	204	203	
t (mm)	13.1	11.6	9.8	10.7	8.5		16.3	14.6	16.3	14.6	13.1	
w (mm)	7.9	7.2	6.9	6.5	5.8		9.1	8.8	9.4	8.5	7.5	
IMPERIAL SIZE AND WEIGHT												
Weight (lb/ft)	38	34	30	26	22		58	53	50	45	40	
Nominal Depth (in.)	14						12					

Sections highlighted in yellow are commonly used sizes and are generally readily available.

BEAM LOAD TABLES
W Shapes

ASTM A992, A572 grade 50
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W310							Approx. Deflect. (mm)	W250			Approx. Deflect. (mm)
Mass (kg/m)	52	45	39	33	28	24	21		67	58	† 49	
Span in Millimetres												
1 000				846	760	700	606	1				1
1 500				795	674	543	475	2			750	2
2 000	988	846	736	596	505	407	356	3	938	826	710	4
2 500	832	703	606	477	404	326	285	5	895	765	568	6
3 000	693	586	505	397	337	272	238	7	746	638	474	9
3 500	594	502	433	341	289	233	204	10	639	546	406	12
4 000	520	440	379	298	253	204	178	13	560	478	355	16
4 500	462	391	337	265	225	181	158	16	497	425	316	20
5 000	416	352	303	238	202	163	143	20	448	383	284	25
5 500	378	320	275	217	184	148	130	24	407	348	258	30
6 000	347	293	253	199	168	136	119	29	373	319	237	36
6 500	320	271	233	183	156	125	110	34	344	294	219	42
7 000	297	251	216	170	144	116	102	40	320	273	203	49
7 500	277	234	202	159	135	109	95	45	298	255	189	56
8 000	260	220	189	149	126	102	89	52	280	239	178	64
8 500	245	207	178	140	119	96	84	58	263	225	167	72
9 000	231	195	168	132	112	91	79	65	249	213	158	81
9 500	219	185	159	126	106	86	75	73	236	201	150	90
10 000	208	176	152	119	101	82	71	81	224	191	142	100
PROPERTIES AND DESIGN DATA												
V_r (kN)	494	423	368	423	380	350	303		469	413	375	
R (kN)	300	247	208	245	211	184	162		375	319	276	
G (kN)	19.7	17.1	15.0	17.1	15.5	14.5	13.2		23.0	20.7	19.1	
B_r' (kN)	216	163	126	163	135	117	97.2		296	239	205	
L_u (mm)	2 380	2 310	2 260	1 330	1 290	1 210	1 190		3 260	3 130	3 160	
d (mm)	317	313	310	313	309	305	303		257	252	247	
b (mm)	167	166	165	102	102	101	101		204	203	202	
t (mm)	13.2	11.2	9.7	10.8	8.9	6.7	5.7		15.7	13.5	11.0	
w (mm)	7.6	6.6	5.8	6.6	6.0	5.6	5.1		8.9	8.0	7.4	
IMPERIAL SIZE AND WEIGHT												
Weight (lb/ft)	35	30	26	22	19	16	14		45	39	33	
Nominal Depth (in.)	12								10			

Sections highlighted in yellow are commonly used sizes and are generally readily available.

† Class 3

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation		W250							Approx. Deflect. (mm)
Mass (kg/m)		45	39	33	28	25	22	† 18	
Span in Millimetres	1 000				682	642	604	445	1
	1 500	828	708	646	585	508	436	296	2
	2 000	748	637	527	438	381	327	222	4
	2 500	598	510	421	351	305	261	178	6
	3 000	498	425	351	292	254	218	148	9
	3 500	427	364	301	251	218	187	127	12
	4 000	374	319	263	219	191	163	111	16
	4 500	332	283	234	195	169	145	99	20
	5 000	299	255	211	175	153	131	89	25
	5 500	272	232	191	159	139	119	81	30
	6 000	249	212	176	146	127	109	74	36
	6 500	230	196	162	135	117	101	68	42
	7 000	214	182	150	125	109	93	64	49
	7 500	199	170	140	117	102	87	59	56
	8 000	187	159	132	110	95	82	56	64
PROPERTIES AND DESIGN DATA									
V_r (kN)		414	354	323	341	321	302	247	
R (kN)		299	247	215	232	211	191	151	
G (kN)		19.7	17.1	15.8	16.6	15.8	15.0	12.4	
B_r' (kN)		216	163	139	153	139	126	86.1	
L_u (mm)		2 170	2 110	2 020	1 370	1 330	1 280	1 330	
d (mm)		266	262	258	260	257	254	251	
b (mm)		148	147	146	102	102	102	101	
t (mm)		13.0	11.2	9.1	10.0	8.4	6.9	5.3	
w (mm)		7.6	6.6	6.1	6.4	6.1	5.8	4.8	
IMPERIAL SIZE AND WEIGHT									
Weight (lb/ft)		30	26	22	19	17	15	12	
Nominal Depth (in.)		10							

Sections highlighted in yellow are commonly used sizes and are generally readily available.

† Class 3

BEAM LOAD TABLES
W Shapes

ASTM A992, A572 grade 50
F_y = 345 MPa

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W200							Approx. Deflect. (mm)
Mass (kg/m)	42	36	31	27	22	19	† 15	
Span in Millimetres								
1 000				492	524	465	315	1
1 500	604	510	550	462	368	310	210	3
2 000	553	471	416	347	276	232	158	5
2 500	442	377	333	277	221	186	126	8
3 000	368	314	277	231	184	155	105	11
3 500	316	269	238	198	158	133	90	15
4 000	276	235	208	173	138	116	79	20
4 500	246	209	185	154	123	103	70	25
5 000	221	188	166	139	110	93	63	31
5 500	201	171	151	126	100	85	57	38
6 000	184	157	139	116	92	77	53	45
6 500	170	145	128	107	85	72	49	53
7 000	158	134	119	99	79	66	45	61
PROPERTIES AND DESIGN DATA								
V_r (kN)	302	255	275	246	262	241	176	
R (kN)	274	226	233	201	212	189	134	
G (kN)	18.6	16.0	16.6	15.0	16.0	15.0	11.1	
B_r' (kN)	194	144	153	126	144	126	69.1	
L_u (mm)	2 610	2 510	1 980	1 890	1 390	1 340	1 380	
d (mm)	205	201	210	207	206	203	200	
b (mm)	166	165	134	133	102	102	100	
t (mm)	11.8	10.2	10.2	8.4	8.0	6.5	5.2	
w (mm)	7.2	6.2	6.4	5.8	6.2	5.8	4.3	
IMPERIAL SIZE AND WEIGHT								
Weight (lb/ft)	28	24	21	18	15	13	10	
Nominal Depth (in.)	8							

Sections highlighted in yellow are commonly used sizes and are generally readily available.

† Class 3

BEAMS WITH WEB HOLES

General

Structures may support a variety of pipes, ducts, conduits and other services, and efforts to reduce floor heights have led to these items being placed in the same plane as the structural floor members. Structural systems using stub girders, trusses and open-web steel joists provide openings for structural/mechanical integration; however, when beams with solid webs are used it may be necessary to cut openings through the webs. This section, based on research summarized by Redwood and Shrivastava (1980), describes a method to account for web holes during design of the member.

Special precautions may be required if it becomes necessary to cut holes in beam webs after construction is complete.

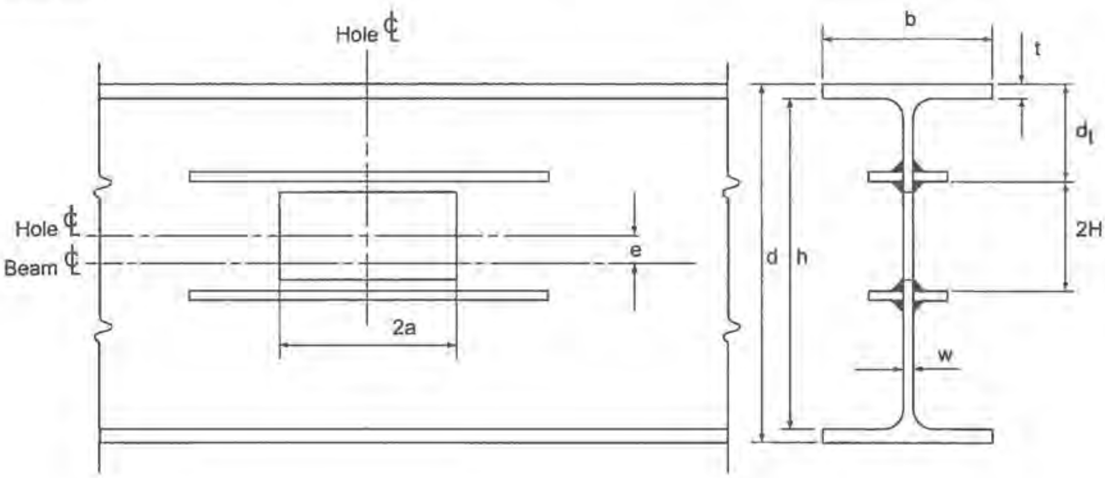
Design

The formulas are applicable for beams of Class 1 and Class 2 sections with openings between 0.3 and 0.7 times the depth of the beam, and hole lengths up to three times the hole height. The steel should meet the requirements of Clause 8.3.2(a) of CSA S16-14 and exhibit the characteristics necessary to achieve moment redistribution, such as ASTM A992.

The hole corner should have a radius at least equal to the larger of 16 mm or twice the web thickness. Fatigue loading considerations have not been accounted for in the formulas of this section and if holes are necessary in a member subjected to fatigue, some guidance is available from Frost and Leffler (1971).

Special design considerations are required if concentrated loads are to be located within the hole length or within one beam depth from either end of a hole.

The width-to-thickness ratio of outstanding reinforcing plates should meet Class 1 requirements.



Nomenclature

A_f = area of one flange ($b\,t$)

A_r = area of reinforcement along top or bottom edge of the hole

A_w = area of web ($d\,w$)

e = eccentricity of centreline of the hole above or below beam centreline (*always positive*)

M_f = bending moment due to factored loads at centreline of hole

M_r = factored moment resistance of an unperforated beam

M_o, M_l = values of moment resistance defined in web hole formulas

R = radius of a circular hole

s = length of web between adjacent holes

V_f = shear force at centreline of hole due to factored loads

V_r' = factored shear resistance based on plastic analysis of an unperforated beam = $0.8\,\phi\,A_w F_s$ (S16-14 Clause 13.4.2)

V_o, V_l = values of shear resistance defined in web hole formulas

Web Stability

This section of the Handbook is valid for the following range of values:

For Class 1 Sections	For Class 2 Sections
$V_f \leq 0.67\,V_r'$	$V_f \leq 0.45\,V_r'$
and in addition, for rectangular holes	and in addition, for rectangular holes
$a/H \leq 3.0$	$a/H \leq 2.2$
$(a/H) + 6\,(2H/d) \leq 5.6$	$(a/H) + 6\,(2H/d) \leq 5.6$

If these values are exceeded, refer to Redwood and Shrivastava (1980).

Deflections

One or two small circular holes normally result in negligible additional deflections; however, deflections of beams with large holes will increase because of local deformations caused by:

(a) effect of rotation produced by change in length of the tee sections above and below the hole

(b) local bending over the length of the hole

(c) shear deformations.

Multiple Holes

To avoid effects of interaction between two adjacent holes which may occur with high shear, the length of the web between the holes should satisfy the following, where s = clear length of solid web between the holes:

Rectangular holes

$$s \geq 2H, \qquad s \geq 2a\left[\frac{V_f/V_r'}{1-(V_f/V_r')}\right]$$

Circular holes

$$s \geq 3R, \qquad s \geq 2R\left[\frac{V_f/V_r'}{1-(V_f/V_r')}\right]$$

where in each case the length, height or radius refers to that of the larger of the two holes.

Lateral Stability

The presence of a web hole has only a minor effect on the lateral stability of a beam, when the strength of the beam is governed by the resistance of a section remote from the hole. For members that may be susceptible to lateral buckling, refer to the paper by Redwood and Shrivastava (1980).

Unreinforced Holes

According to Clause 14.3.3.2 of S16-14 (see Part Two), unreinforced circular openings may be used under stipulated conditions. Round holes that are not covered by Clause 14.3.3.2 may be checked using the unreinforced hole formulas below by equating a and H to hole radius R as follows:

$2a = 0.9R$ and $2H = 1.8R$

Beam Resistance — Unreinforced Holes

Web stability must always be confirmed (see previous page), and compression zone stability of the tee section must be checked when $2a > 4\,d_t$.

The factored shear force V_f and factored moment M_f applied at the web hole centreline must satisfy:

$$V_f \leq V_l \qquad [1]$$

$$M_f \leq M_o - (M_o - M_l)\, V_f/V_l \qquad [2]$$

in which

$$\frac{M_o}{M_r} = 1 - \frac{\dfrac{A_w}{4A_f}\left[\left(\dfrac{2H}{d}\right)^2 + \left(\dfrac{4e}{d}\right)\left(\dfrac{2H}{d}\right)\right]}{1+\dfrac{A_w}{4A_f}} \qquad [3]$$

$$\frac{M_l}{M_r} = \frac{1-\dfrac{2}{\sqrt{3}}\left(\dfrac{A_w}{A_f}\right)\left(\dfrac{a}{d}\right)\sqrt{\dfrac{\alpha_2}{1+\alpha_2}}}{1+\dfrac{A_w}{4A_f}} \qquad [4]$$

$$\frac{V_l}{V_r'} = \frac{2}{\sqrt{3}}\left(\frac{a}{d}\right)\left(\frac{\alpha_1}{\sqrt{1+\alpha_1}} + \frac{\alpha_2}{\sqrt{1+\alpha_2}}\right) \qquad [5]$$

where

$$\alpha_1 = \frac{3}{16}\left(\frac{d}{a}\right)^2\left(1 - \frac{2H}{d} - \frac{2e}{d}\right)^2 \qquad [6]$$

$$\alpha_2 = \frac{3}{16}\left(\frac{d}{a}\right)^2\left(1 - \frac{2H}{d} + \frac{2e}{d}\right)^2 \qquad [7]$$

Tables 5-2 and 5-3 provide means of evaluating equations [1] and [2]. For further explanation of these tables, see Design Tables.

Reinforced Holes

Horizontal Bars Only

Equal areas of reinforcement should be placed above and below the opening, with the reinforcement as close as possible to the edges of the hole. Welds attaching the reinforcement to the beam web should be continuous and may be placed on only one side of the reinforcing bar (with a short weld at each end on the opposite side of the bar to maintain alignment). Within the length of the hole, the welds should develop twice the factored tensile resistance of the reinforcement except that the weld capacity need not exceed $1.15\,a\,w\,F_y$. The reinforcement should extend past the hole far enough for the weld to develop the factored tensile resistance of the reinforcement but not less than a distance of $a/2$.

Reinforcement may be placed on only one side of the web of Class 1 sections (for economy) providing the following conditions are satisfied:

$A_r \le 0.333\,A_f$, $\quad M_f \le 20\,V_f\,d$ (at the hole centreline)

$a/H \le 2.5$, $\quad d_t / w \le 370/\sqrt{F_y}$

Round holes may be checked using the reinforced hole formulas by relating a and H to R as follows:

$2a = 0.9R$ and $2H = 2R$

Once it is established that hole reinforcement is required, Table 5-4 provides a means of checking the resistance of a beam with a reinforced hole for an assumed area of reinforcement.

Vertical Bars

The compression zone stability of the reinforced tee should be checked by treating it as an axially loaded column with effective length equal to $2a$.

If it is determined that web instability could be a problem, vertical reinforcing at the ends of the hole will be required. Attachment of both vertical and horizontal bars is generally more economical when the horizontal bars are placed on one side of the web with the vertical bars on the other side.

Beam Resistance — Holes with Horizontal Reinforcing Bars

Web stability and compression zone stability must be checked in addition to the following strength criteria. The factored shear force V_f and factored moment M_f at the web hole centreline must satisfy the following, where A_r is less than A_f:

$$V_f \le V_l \qquad [8a]$$

$$V_f / V_r' \le 1 - \frac{2H}{d} \qquad [8b]$$

$$M_f \le M_o - (M_o - M_l) V_f / V_l \qquad [9a]$$

$$M_f \le M_r \qquad [9b]$$

in which

$$\left(\frac{M_o}{M_r}\right)_a = 1 + \frac{\frac{A_r}{A_f}\left(\frac{2H}{d}\right) - \frac{A_w}{4A_f}\left[\left(\frac{2H}{d}\right)^2 + 4\left(\frac{2H}{d}\right)\left(\frac{e}{d}\right) - 4\left(\frac{e}{d}\right)^2\right]}{1 + \frac{A_w}{4A_f}} \quad \text{for } \frac{e}{d} \le \frac{A_r}{A_w} \qquad [10a]$$

$$\text{or} \quad \left(\frac{M_o}{M_r}\right)_b = \left(\frac{M_o}{M_r}\right)_a - \frac{\frac{A_w}{A_f}\left(\frac{e}{d} - \frac{A_r}{A_w}\right)^2}{1 + \frac{A_w}{4A_f}} \quad \text{for } \frac{e}{d} > \frac{A_r}{A_w} \qquad [10b]$$

$$\left(\frac{M_l}{M_r}\right) = \frac{1 - \frac{A_r}{A_f}}{1 + \frac{A_w}{4A_f}} \qquad [11]$$

$$\frac{V_l}{V_r'} = \sqrt{3}\left(\frac{d}{a}\right)\frac{A_r}{A_w}\left(1 - \frac{2H}{d}\right) \qquad [12]$$

Flow Chart

The flowchart on the next page is provided as a guide in developing computer programs. The logic provided determines the minimum reinforcement, A_r, which will satisfy equation [9a].

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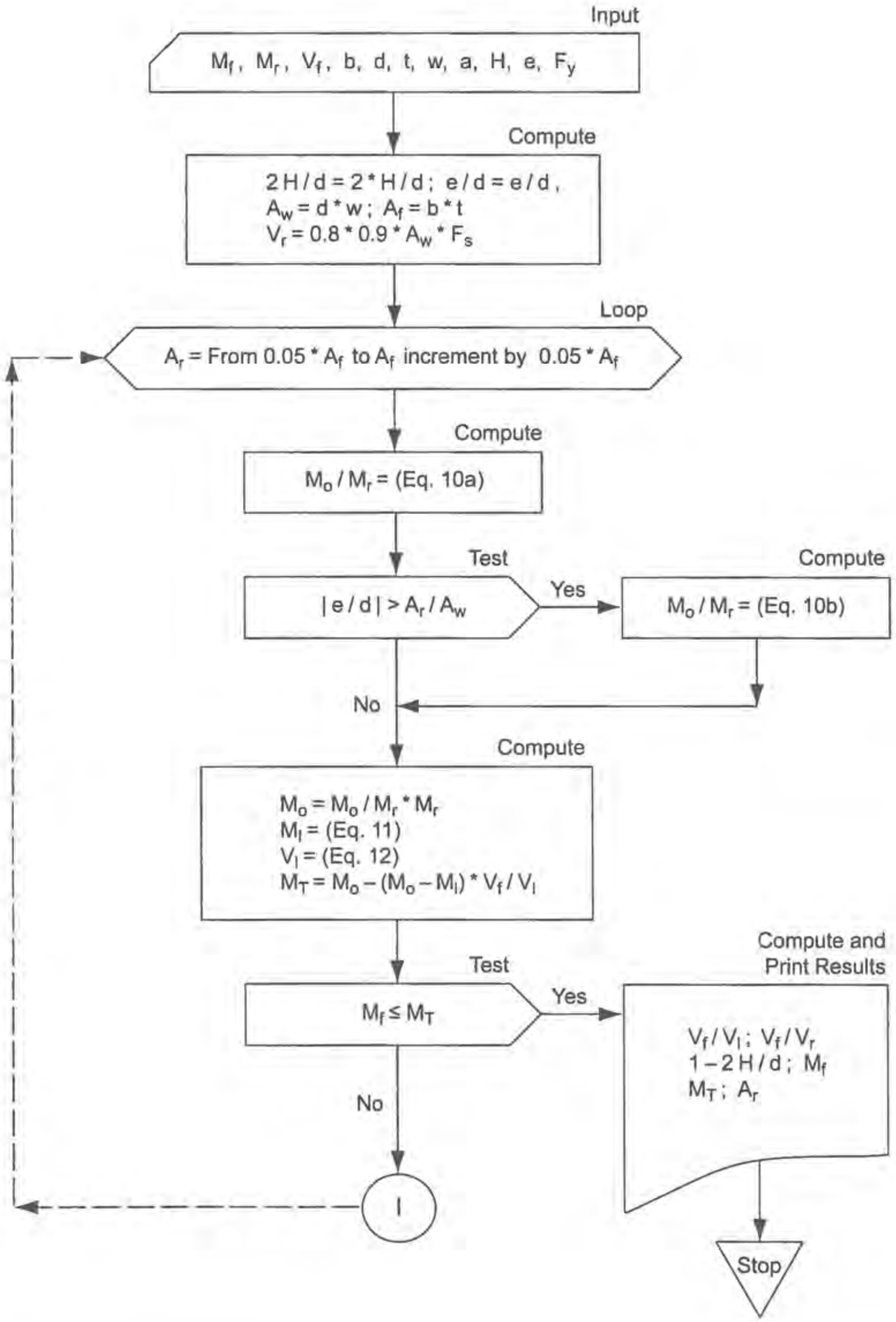
REDWOOD, R.G., and WONG, P. 1982. Web holes in composite beams with steel deck. Proceedings, Canadian Structural Engineering Conference, Canadian Steel Construction Council, Willowdale, Ontario.

REINFORCED HOLE PROGRAM FLOWCHART



Design Tables

(*a*) *Unreinforced Holes*

Table 5-2 gives the values of constants C_1 and C_2 for unreinforced holes where

$$C_1 = \frac{M_o}{M_r} \text{ and } C_2 = \frac{M_o / M_r - M_l / M_r}{V_l / V_r'}$$

where M_o, M_l and V_l are defined in equations [3] to [5].

Table 5-3 gives the value of constant C_3 taken as V_l/V_r'.

A_w/A_f varies from 0.5 to 2.25, $2H/d$ from 0.3 to 0.6, a/H from 0.50 to 2.2, and $e/d = 0$.

Written in terms of the constants C_1 and C_2, [2] becomes

$$\frac{M_f}{M_r} \le C_1 - C_2\left(\frac{V_f}{V_r'}\right) \qquad [13]$$

and [1] becomes

$$\frac{V_f}{V_r'} \le C_3 \qquad [14]$$

Use

For concentric ($e/d = 0$) unreinforced holes, compute A_w/A_f, $2H/d$ and a/H. Determine C_1, C_2 and C_3 with the aid of Tables 5-2 and 5-3 for use in equations [13] and [14].

(*b*) *Reinforced Holes*

Table 5-4 gives the values of the constants C_4 and C_5 for reinforced holes where

$$C_4 = \frac{M_o}{M_r} \text{ and } C_5 = \frac{M_o / M_r - M_l / M_r}{V_l / V_r'}$$

where M_o, M_l and V_l are defined in equations [10] to [12] for concentric holes ($e/d = 0$).

A_w/A_f varies from 0.5 to 2.25, $2H/d$ from 0.3 to 0.6, and a/H from 0.45 to 2.2, for the three values of A_r/A_f, 0.333, 0.667 and 1.0.

Written in terms of the constants C_4 and C_5, [9a] becomes

$$\frac{M_f}{M_r} \le C_4 - C_5\left(\frac{V_f}{V_r'}\right) \qquad [15]$$

Use

For concentric ($e/d = 0$) reinforced holes, compute A_w/A_f, $2H/d$ and a/H. Determine C_4 and C_5, for one of the assumed values of A_r/A_f, for use in [15].

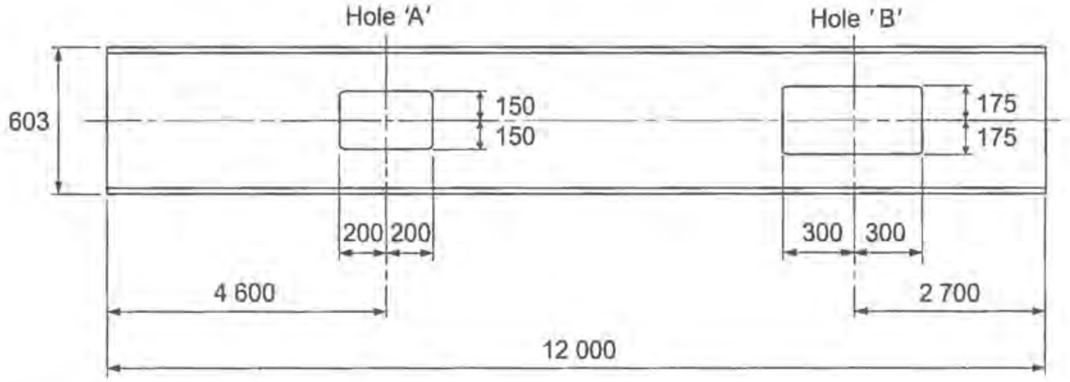
Calculate V_l from [12] for use in [8a].

Then check equations [8b] and [9b].

Example

Given:

A simple-span W610x101 beam of ASTM A992 grade steel (F_y = 345 MPa) spanning 12 m supports a factored total uniformly distributed load of 480 kN (40 kN/m). Check the adequacy of the section for two rectangular holes located as shown. Lateral support to the compression flange is provided.



Solution for Hole 'A':

Class of beam: from Table 5-1, W610x101 is a Class 1

From the Beam Selection Table, $M_r = 900$ kN·m

$V_r' = 0.8\phi A_w F_s = 0.8 \times 0.9 \times (603 \times 10.5) \times 0.66 \times 345 = 1040$ kN

At centreline of hole

$M_f = 40$ kN/m $\times$ 4.6 m $\times (12 - 4.6)/2 = 681$ kN·m

$V_f = 40$ kN/m $\times ((12/2) - 4.6) = 56.0$ kN

$$\frac{M_f}{M_r} = \frac{681}{900} = 0.757 \text{ and } \frac{V_f}{V_r'} = \frac{56.0}{1\,040} = 0.0538$$

Check stability of web (see Web Stability)

$$\frac{V_f}{V_r'} = 0.0538 < 0.67$$

$a/H = 1.33 < 3.0$ (limit for Class 1 beam)

$a/H + 6(2H/d) = 200/150 + 6\,(300/603) = 1.33 + 6\,(0.498) = 4.32 < 5.6$

Check compression zone stability

OK, if $2a \leq 4\,d_t = 4\,((603/2) - 150) = 606$ mm

$2a = 400$ mm $<$ 606 mm

Check for unreinforced hole

$$\frac{A_w}{A_f} = \frac{10.5 \times 603}{228 \times 14.9} = 1.86 \text{ and } \frac{2H}{d} = 0.50$$

$C_1 = 0.92$, from Table 5-2

For $a / H = 1.33$ Use 1.4

$C_2 = 1.9$, from Table 5-2

$C_3 = 0.263$, from Table 5-3

$$\frac{M_f}{M_r} \leq C_1 - C_2\left(\frac{V_f}{V_r'}\right) \quad [13]$$

$$\leq 0.92 - 1.9\,(0.0538) = 0.818$$

$$M_f / M_r = 0.757 < 0.818$$

$$\frac{V_f}{V_r'} \leq C_3 \quad [14]$$

$0.0538 < 0.263$ OK

Therefore, reinforcement is not required.

Solution for Hole 'B'

At centreline of hole

$M_f = 40$ kN/m × 2.7 m × (12 – 2.7) / 2 = 502 kN·m

$V_f = 40$ kN/m × ((12 / 2) – 2.7) = 132 kN

$$\frac{M_f}{M_r} = \frac{502}{900} = 0.558 \text{ and } \frac{V_f}{V_r'} = \frac{132}{1040} = 0.127$$

Check spacing between holes

Use $2H$ of larger hole.

OK, if $s \geq 2H = 350$ and $s \geq 2a\left[\frac{V_f/V_r'}{1-(V_f/V_r')}\right] = 600\left[\frac{0.127}{1-0.127}\right] = 87.3$ mm

$s = 12\,000 - (2\,700 + 4\,600) = 4\,700$ mm > 350 mm

Check stability of web (see Web Stability)

$$\frac{V_f}{V_r'} = 0.127 < 0.67$$

$a / H = 1.71 < 3.0$ (limit for Class 1 beam)

$a / H + 6\,(2H / d) = 300 / 175 + 6\,(350 / 603) = 1.71 + 6\,(0.580) = 5.19 < 5.6$

Check compression zone stability

OK, if $2a \le 4\,d_t$ (unreinforced tee)

$\le 4\,((603\,/\,2) - 175) = 506$ mm

$2a = 600$ mm > 506 mm (not adequate)

Check for unreinforced hole

From Table 5-2, for $A_w / A_f = 1.86$ (use 2.0)

and $2H / d = 0.58$ (use 0.60), $C_1 = 0.88$

For $a / H = 1.71$ (use 1.8), $C_2 = 3.83$

$$\frac{M_f}{M_r} \le C_1 - C_2\left(\frac{V_f}{V_r'}\right) \quad [13]$$

$\le 0.88 - 3.83\,(0.127) = 0.394$

$M_f / M_r = 0.558 > 0.394$ (reinforcement required)

Reinforcement

Assume $A_r / A_f = 0.333$ (maximum permitted for one-sided reinforcement)

Reinforcing plate either CSA G40.21 350W or ASTM A572 grade 50 ksi steel.

From Table 5-4,

for $\dfrac{A_r}{A_f} = 0.333$, $\dfrac{A_w}{A_f} = 2.0$, $\dfrac{2H}{d} \approx 0.60$

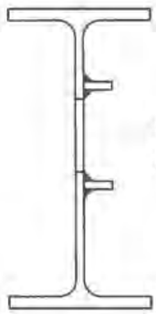
$C_4 = 1.013$

For $a / H = 1.71$, $C_5 = 2.53$ (by interpolation)

$$\frac{M_f}{M_r} \le C_4 - C_5\left(\frac{V_f}{V_r'}\right) \quad [15]$$

$\le 1.013 - 2.53\,(0.127) = 0.692$

$M_f / M_r = 0.558 < 0.692$



Section through beam at hole 'B'

Further refinement of A_r / A_f can be accomplished by using the expressions previously given.

Check one-sided reinforcement

$M_f \le 20\,V_f\,d$ at hole centreline (see Reinforced Holes)

$\le 20 \times 132 \times 603 = 1\,590$ kN·m

$M_f = 502$ kN·m $< 1\,590$ kN·m

$a / H = 1.71 < 2.5$

VALUES OF C_1 AND C_2

Table 5-2

For Unreinforced Concentric Holes in Beam Webs

$\frac{A_w}{A_f}$	$\frac{2H}{d}$	C_1	C_2 For following a/H values							
			0.50	1.0	1.2	1.4	1.6	1.8	2.0	2.2
	0.30	0.990	0.204	0.271	0.300	0.330	0.360	0.391	0.423	0.455
	0.35	0.986	0.226	0.315	0.353	0.392	0.433	0.474	0.516	0.558
	0.40	0.982	0.252	0.367	0.417	0.468	0.520	0.574	0.628	0.682
0.50	0.45	0.978	0.283	0.432	0.495	0.561	0.628	0.696	0.764	0.833
	0.50	0.972	0.321	0.511	0.593	0.676	0.761	0.846	0.933	1.020
	0.55	0.966	0.368	0.612	0.715	0.820	0.927	1.035	1.143	1.252
	0.60	0.960	0.428	0.740	0.872	1.005	1.140	1.275	1.411	*
	0.30	0.986	0.290	0.385	0.426	0.468	0.512	0.556	0.601	0.647
	0.35	0.981	0.321	0.447	0.502	0.557	0.615	0.673	0.733	0.793
	0.40	0.975	0.358	0.522	0.593	0.665	0.740	0.815	0.892	0.970
0.75	0.45	0.968	0.402	0.613	0.704	0.797	0.892	0.989	1.086	1.184
	0.50	0.961	0.456	0.726	0.842	0.961	1.081	1.203	1.325	1.449
	0.55	0.952	0.522	0.869	1.016	1.166	1.317	1.470	1.624	1.779
	0.60	0.943	0.608	1.052	1.239	1.428	1.619	1.812	2.005	*
	0.30	0.982	0.367	0.488	0.540	0.593	0.648	0.705	0.762	0.820
	0.35	0.976	0.407	0.567	0.635	0.706	0.779	0.853	0.928	1.005
	0.40	0.968	0.454	0.661	0.751	0.843	0.937	1.033	1.130	1.228
1.00	0.45	0.960	0.510	0.777	0.892	1.010	1.130	1.252	1.376	1.500
	0.50	0.950	0.577	0.920	1.067	1.217	1.369	1.523	1.679	1.835
	0.55	0.940	0.662	1.101	1.287	1.477	1.669	1.862	2.057	2.253
	0.60	0.928	0.770	1.333	1.569	1.809	2.051	2.295	2.539	*
	0.30	0.979	0.437	0.581	0.643	0.706	0.772	0.839	0.907	0.976
	0.35	0.971	0.485	0.675	0.756	0.841	0.927	1.015	1.105	1.196
	0.40	0.962	0.540	0.787	0.894	1.003	1.115	1.229	1.345	1.462
1.25	0.45	0.952	0.607	0.925	1.062	1.202	1.346	1.491	1.638	1.786
	0.50	0.940	0.687	1.095	1.270	1.449	1.630	1.813	1.999	2.185
	0.55	0.928	0.788	1.310	1.532	1.758	1.987	2.217	2.449	2.682
	0.60	0.914	0.916	1.587	1.868	2.154	2.442	2.732	3.023	*
	0.30	0.975	0.500	0.666	0.736	0.809	0.884	0.961	1.039	1.118
	0.35	0.967	0.555	0.773	0.866	0.963	1.062	1.163	1.266	1.370
	0.40	0.956	0.619	0.902	1.024	1.149	1.278	1.408	1.541	1.675
1.50	0.45	0.945	0.695	1.059	1.216	1.377	1.541	1.708	1.876	2.046
	0.50	0.932	0.787	1.255	1.455	1.659	1.867	2.077	2.289	2.502
	0.55	0.918	0.902	1.501	1.755	2.014	2.276	2.540	2.805	3.072
	0.60	0.902	1.050	1.817	2.140	2.467	2.797	3.129	3.463	*
	0.30	0.973	0.558	0.743	0.822	0.903	0.987	1.072	1.159	1.248
	0.35	0.963	0.619	0.862	0.967	1.075	1.185	1.298	1.413	1.529
	0.40	0.951	0.691	1.006	1.142	1.282	1.426	1.572	1.719	1.869
1.75	0.45	0.938	0.776	1.182	1.357	1.537	1.720	1.906	2.094	2.283
	0.50	0.924	0.879	1.400	1.624	1.852	2.084	2.318	2.555	2.793
	0.55	0.908	1.007	1.675	1.959	2.247	2.539	2.834	3.131	3.429
	0.60	0.890	1.171	2.028	2.388	2.753	3.122	3.492	3.864	*
	0.30	0.970	0.611	0.813	0.900	0.989	1.081	1.174	1.270	1.366
	0.35	0.959	0.678	0.944	1.059	1.177	1.298	1.421	1.547	1.674
	0.40	0.947	0.757	1.102	1.251	1.405	1.561	1.721	1.883	2.047
2.00	0.45	0.933	0.849	1.295	1.486	1.683	1.884	2.087	2.293	2.500
	0.50	0.917	0.962	1.534	1.778	2.028	2.282	2.539	2.798	3.059
	0.55	0.899	1.103	1.835	2.145	2.461	2.781	3.104	3.429	3.755
	0.60	0.880	1.283	2.221	2.616	3.016	3.419	3.825	4.232	*
	0.30	0.968	0.660	0.878	0.972	1.068	1.167	1.268	1.371	1.476
	0.35	0.956	0.733	1.020	1.144	1.271	1.402	1.535	1.671	1.808
	0.40	0.942	0.817	1.190	1.351	1.517	1.686	1.859	2.034	2.211
2.25	0.45	0.927	0.917	1.398	1.605	1.818	2.034	2.254	2.476	2.700
	0.50	0.910	1.039	1.656	1.920	2.190	2.465	2.742	3.022	3.303
	0.55	0.891	1.191	1.981	2.317	2.658	3.004	3.352	3.703	4.056
	0.60	0.870	1.386	2.399	2.825	3.257	3.692	4.131	4.571	*

* a/H plus 6(2H/d) exceeds 5.6.

$$\frac{d_t}{w} = \frac{(603/2) - 175}{10.5} = 12.0 \leq \frac{370}{\sqrt{F_y}} = 19.9$$

Therefore, one-sided reinforcement is adequate.

$A_r = 0.33 \times A_f = 0.33\,(228 \times 14.9) = 1\,120 \text{ mm}^2$

Check shear

$$V_t = \sqrt{3}\left(\frac{d}{a}\right)\left(\frac{A_r}{A_w}\right)\left(1 - \frac{2H}{d}\right)V_r' \quad [12]$$

$$= \sqrt{3}\left(\frac{603}{300}\right)\left(\frac{1120}{10.5 \times 603}\right)(1 - 0.58)\,1\,040 = 269$$

$V_f \leq V_t$ [8a]

$132 < 269$

$V_f / V_r' \leq 1 - 2H/d$ [8b]

$\leq 1 - 0.58 = 0.42$

$V_f / V_r' = 0.127 < 0.42$

Try 16 × 70 reinforcement

$$\frac{b}{t} \leq \frac{145}{\sqrt{F_y}} \quad (\textit{for Class } 1)$$

≤ 7.81

$b / t = 70 / 16 = 4.38 < 7.81$

Therefore, use 16 × 70 one-sided reinforcement.

VALUES OF C_3 — Table 5-3

For Unreinforced Concentric Holes in Beam Webs

2H / d	a / H							
	0.5	1.0	1.2	1.4	1.6	1.8	2.0	2.2
0.30	0.680	0.627	0.602	0.575	0.549	0.523	0.498	0.474
0.35	0.621	0.552	0.521	0.490	0.461	0.433	0.407	0.384
0.40	0.560	0.475	0.441	0.408	0.378	0.351	0.327	0.305
0.45	0.497	0.400	0.364	0.332	0.303	0.279	0.257	0.238
0.50	0.433	0.327	0.293	0.263	0.238	0.217	0.199	0.183
0.55	0.368	0.260	0.229	0.203	0.182	0.165	0.150	0.138
0.60	0.302	0.200	0.173	0.152	0.136	0.122	0.111	0.102

VALUES OF C_4 AND C_5 — Table 5-4

For Reinforced Concentric Holes in Beam Webs — $A_r/A_f = 0.333$

$\frac{A_w}{A_r}$	$\frac{2H}{d}$	C_4	C_5 For following a/H values							
			0.45	1.0	1.2	1.4	1.6	1.8	2.0	2.2
	0.30	1.079	0.041	0.090	0.108	0.126	0.144	0.162	0.181	0.199
	0.35	1.090	0.052	0.116	0.139	0.162	0.186	0.209	0.232	0.255
	0.40	1.101	0.066	0.147	0.176	0.205	0.235	0.264	0.293	0.323
0.50	0.45	1.111	0.083	0.184	0.220	0.257	0.294	0.331	0.367	0.404
	0.50	1.120	0.103	0.229	0.274	0.320	0.366	0.411	0.457	0.503
	0.55	1.129	0.128	0.284	0.341	0.398	0.455	0.511	0.568	0.625
	0.60	1.138	0.159	0.354	0.425	0.496	0.567	0.637	0.708	*
	0.30	1.070	0.064	0.142	0.170	0.198	0.227	0.255	0.283	0.312
	0.35	1.079	0.081	0.181	0.217	0.253	0.290	0.326	0.362	0.398
	0.40	1.087	0.102	0.228	0.273	0.319	0.364	0.410	0.455	0.501
0.75	0.45	1.094	0.127	0.283	0.340	0.397	0.453	0.510	0.567	0.623
	0.50	1.101	0.158	0.350	0.421	0.491	0.561	0.631	0.701	0.771
	0.55	1.106	0.195	0.433	0.519	0.606	0.693	0.779	0.866	0.952
	0.60	1.111	0.241	0.536	0.643	0.751	0.858	0.965	1.072	*
	0.30	1.062	0.088	0.196	0.236	0.275	0.314	0.353	0.393	0.432
	0.35	1.069	0.112	0.250	0.300	0.350	0.400	0.450	0.500	0.550
	0.40	1.075	0.141	0.313	0.375	0.438	0.500	0.563	0.625	0.688
1.00	0.45	1.079	0.174	0.387	0.465	0.542	0.619	0.697	0.774	0.852
	0.50	1.083	0.214	0.476	0.572	0.667	0.762	0.858	0.953	1.048
	0.55	1.086	0.263	0.585	0.702	0.819	0.936	1.054	1.171	1.288
	0.60	1.088	0.324	0.721	0.865	1.009	1.153	1.297	1.441	*
	0.30	1.055	0.114	0.254	0.305	0.355	0.406	0.457	0.508	0.558
	0.35	1.060	0.145	0.322	0.386	0.450	0.515	0.579	0.644	0.708
	0.40	1.063	0.180	0.401	0.481	0.562	0.642	0.722	0.802	0.882
1.25	0.45	1.066	0.223	0.495	0.593	0.692	0.791	0.890	0.989	1.088
	0.50	1.067	0.273	0.606	0.727	0.848	0.969	1.091	1.212	1.333
	0.55	1.068	0.333	0.741	0.889	1.037	1.185	1.333	1.482	1.630
	0.60	1.067	0.408	0.908	1.089	1.271	1.452	1.634	1.815	*
	0.30	1.048	0.141	0.314	0.377	0.439	0.502	0.565	0.628	0.690
	0.35	1.051	0.178	0.396	0.476	0.555	0.634	0.714	0.793	0.872
	0.40	1.053	0.222	0.493	0.591	0.690	0.788	0.887	0.985	1.084
1.50	0.45	1.054	0.272	0.605	0.726	0.847	0.968	1.089	1.210	1.331
	0.50	1.053	0.332	0.738	0.886	1.034	1.181	1.329	1.477	1.624
	0.55	1.051	0.405	0.899	1.079	1.258	1.438	1.618	1.798	1.978
	0.60	1.047	0.493	1.096	1.316	1.535	1.754	1.973	2.193	*
	0.30	1.042	0.169	0.376	0.451	0.526	0.601	0.677	0.752	0.827
	0.35	1.044	0.213	0.474	0.568	0.663	0.758	0.853	0.947	1.042
	0.40	1.044	0.264	0.587	0.704	0.821	0.938	1.056	1.173	1.290
1.75	0.45	1.043	0.323	0.718	0.862	1.005	1.149	1.293	1.436	1.580
	0.50	1.040	0.393	0.873	1.048	1.223	1.397	1.572	1.747	1.922
	0.55	1.035	0.477	1.059	1.271	1.483	1.695	1.907	2.119	2.331
	0.60	1.029	0.579	1.287	1.544	1.801	2.059	2.316	2.573	*
	0.30	1.037	0.198	0.440	0.528	0.616	0.704	0.792	0.880	0.968
	0.35	1.037	0.249	0.553	0.663	0.774	0.885	0.995	1.106	1.216
	0.40	1.035	0.307	0.683	0.819	0.956	1.093	1.229	1.366	1.502
2.00	0.45	1.032	0.375	0.834	1.000	1.167	1.334	1.501	1.667	1.834
	0.50	1.028	0.455	1.011	1.213	1.415	1.617	1.819	2.022	2.224
	0.55	1.021	0.550	1.222	1.466	1.711	1.955	2.199	2.444	2.688
	0.60	1.013	0.665	1.479	1.774	2.070	2.366	2.661	2.957	*
	0.30	1.032	0.227	0.505	0.607	0.708	0.809	0.910	1.011	1.112
	0.35	1.030	0.285	0.634	0.761	0.888	1.014	1.141	1.268	1.395
	0.40	1.028	0.352	0.781	0.937	1.094	1.250	1.406	1.562	1.719
2.25	0.45	1.023	0.428	0.951	1.142	1.332	1.522	1.712	1.903	2.093
	0.50	1.017	0.518	1.150	1.380	1.610	1.840	2.070	2.300	2.530
	0.55	1.008	0.624	1.386	1.663	1.941	2.218	2.495	2.772	3.049
	0.60	0.998	0.752	1.672	2.006	2.340	2.675	3.009	3.344	*

* a/H plus 6(2H/d) exceeds 5.6.

VALUES OF C_4 AND C_5

Table 5-4

For Reinforced Concentric Holes in Beam Webs

$A_r/A_f = 0.667$

$\frac{A_w}{A_f}$	$\frac{2H}{d}$	C_4	C_5 For following a/H values							
			0.45	1.0	1.2	1.4	1.6	1.8	2.0	2.2
0.50	0.30	1.168	0.036	0.081	0.097	0.113	0.129	0.146	0.162	0.178
	0.35	1.194	0.047	0.105	0.126	0.146	0.167	0.188	0.209	0.230
	0.40	1.219	0.060	0.133	0.160	0.186	0.213	0.240	0.266	0.293
	0.45	1.244	0.076	0.168	0.201	0.235	0.269	0.302	0.336	0.369
	0.50	1.269	0.095	0.210	0.253	0.295	0.337	0.379	0.421	0.463
	0.55	1.292	0.119	0.264	0.316	0.369	0.422	0.474	0.527	0.580
	0.60	1.316	0.149	0.331	0.397	0.463	0.530	0.596	0.662	*
0.75	0.30	1.154	0.055	0.122	0.146	0.170	0.195	0.219	0.243	0.267
	0.35	1.177	0.071	0.157	0.188	0.219	0.251	0.282	0.314	0.345
	0.40	1.199	0.089	0.199	0.239	0.278	0.318	0.358	0.398	0.438
	0.45	1.221	0.112	0.250	0.300	0.350	0.400	0.450	0.499	0.549
	0.50	1.241	0.140	0.312	0.374	0.437	0.499	0.561	0.624	0.686
	0.55	1.261	0.175	0.389	0.467	0.545	0.623	0.700	0.778	0.856
	0.60	1.280	0.219	0.487	0.584	0.681	0.779	0.876	0.974	*
1.00	0.30	1.142	0.073	0.162	0.195	0.227	0.260	0.292	0.325	0.357
	0.35	1.162	0.094	0.209	0.251	0.292	0.334	0.376	0.418	0.459
	0.40	1.181	0.119	0.264	0.317	0.370	0.422	0.475	0.528	0.581
	0.45	1.200	0.149	0.330	0.397	0.463	0.529	0.595	0.661	0.727
	0.50	1.217	0.185	0.411	0.494	0.576	0.658	0.740	0.823	0.905
	0.55	1.233	0.230	0.511	0.614	0.716	0.818	0.920	1.023	1.125
	0.60	1.248	0.287	0.637	0.765	0.892	1.020	1.147	1.275	*
1.25	0.30	1.131	0.092	0.203	0.244	0.285	0.325	0.366	0.407	0.448
	0.35	1.149	0.117	0.261	0.313	0.365	0.417	0.469	0.521	0.574
	0.40	1.165	0.148	0.329	0.394	0.460	0.526	0.592	0.657	0.723
	0.45	1.180	0.185	0.410	0.492	0.574	0.656	0.738	0.820	0.902
	0.50	1.195	0.229	0.509	0.611	0.713	0.814	0.916	1.018	1.120
	0.55	1.207	0.284	0.631	0.757	0.883	1.009	1.135	1.261	1.387
	0.60	1.219	0.353	0.783	0.940	1.097	1.254	1.410	1.567	*
1.50	0.30	1.121	0.110	0.245	0.293	0.342	0.391	0.440	0.489	0.538
	0.35	1.136	0.141	0.313	0.375	0.438	0.500	0.563	0.625	0.688
	0.40	1.150	0.177	0.393	0.472	0.550	0.629	0.708	0.786	0.865
	0.45	1.163	0.220	0.489	0.587	0.685	0.783	0.880	0.978	1.076
	0.50	1.174	0.272	0.605	0.726	0.847	0.968	1.089	1.210	1.331
	0.55	1.184	0.336	0.748	0.897	1.047	1.196	1.346	1.495	1.645
	0.60	1.193	0.417	0.926	1.111	1.296	1.481	1.666	1.852	*
1.75	0.30	1.112	0.129	0.286	0.343	0.400	0.457	0.514	0.571	0.629
	0.35	1.125	0.164	0.364	0.437	0.510	0.583	0.656	0.729	0.802
	0.40	1.137	0.206	0.457	0.549	0.640	0.731	0.823	0.914	1.006
	0.45	1.147	0.255	0.567	0.681	0.794	0.908	1.021	1.135	1.248
	0.50	1.156	0.315	0.700	0.840	0.980	1.120	1.260	1.400	1.540
	0.55	1.163	0.388	0.862	1.035	1.207	1.380	1.552	1.725	1.897
	0.60	1.169	0.479	1.065	1.278	1.491	1.704	1.917	2.129	*
2.00	0.30	1.103	0.147	0.327	0.392	0.458	0.523	0.589	0.654	0.719
	0.35	1.115	0.187	0.416	0.499	0.583	0.666	0.749	0.832	0.915
	0.40	1.125	0.234	0.521	0.625	0.729	0.833	0.937	1.042	1.146
	0.45	1.133	0.290	0.645	0.774	0.903	1.032	1.161	1.290	1.419
	0.50	1.139	0.357	0.794	0.952	1.111	1.270	1.429	1.587	1.746
	0.55	1.144	0.439	0.975	1.170	1.365	1.560	1.755	1.950	2.145
	0.60	1.147	0.540	1.201	1.441	1.681	1.921	2.161	2.402	*
2.25	0.30	1.096	0.166	0.368	0.442	0.516	0.589	0.663	0.737	0.810
	0.35	1.105	0.211	0.468	0.561	0.655	0.749	0.842	0.936	1.029
	0.40	1.113	0.263	0.584	0.701	0.818	0.935	1.052	1.169	1.285
	0.45	1.119	0.325	0.722	0.866	1.011	1.155	1.299	1.444	1.588
	0.50	1.123	0.399	0.886	1.064	1.241	1.418	1.596	1.773	1.950
	0.55	1.126	0.489	1.086	1.304	1.521	1.738	1.955	2.173	2.390
	0.60	1.127	0.600	1.334	1.601	1.868	2.135	2.402	2.668	*

* a/H plus 6(2H/d) exceeds 5.6.

VALUES OF C_4 AND C_5

For Reinforced Concentric Holes in Beam Webs

Table 5-4

$A_r/A_f = 1.00$

$\frac{A_w}{A_f}$	$\frac{2H}{d}$	C_4	C_5							
			For following a/H values							
			0.45	1.0	1.2	1.4	1.6	1.8	2.0	2.2
	0.30	1.257	0.035	0.078	0.093	0.109	0.124	0.140	0.155	0.171
	0.35	1.298	0.045	0.101	0.121	0.141	0.161	0.182	0.202	0.222
	0.40	1.338	0.058	0.129	0.154	0.180	0.206	0.232	0.257	0.283
0.50	0.45	1.378	0.073	0.163	0.195	0.228	0.260	0.293	0.325	0.358
	0.50	1.417	0.092	0.204	0.245	0.286	0.327	0.368	0.409	0.450
	0.55	1.455	0.116	0.257	0.308	0.359	0.411	0.462	0.513	0.565
	0.60	1.493	0.145	0.323	0.388	0.453	0.517	0.582	0.647	*
	0.30	1.238	0.052	0.115	0.138	0.161	0.184	0.207	0.230	0.253
	0.35	1.275	0.067	0.149	0.178	0.208	0.238	0.268	0.297	0.327
	0.40	1.312	0.085	0.189	0.227	0.265	0.303	0.341	0.379	0.416
0.75	0.45	1.347	0.107	0.239	0.286	0.334	0.382	0.429	0.477	0.525
	0.50	1.382	0.135	0.299	0.359	0.419	0.479	0.538	0.598	0.658
	0.55	1.415	0.169	0.375	0.449	0.524	0.599	0.674	0.749	0.824
	0.60	1.448	0.212	0.470	0.564	0.659	0.753	0.847	0.941	*
	0.30	1.222	0.068	0.151	0.181	0.212	0.242	0.272	0.302	0.333
	0.35	1.256	0.088	0.195	0.234	0.273	0.312	0.351	0.390	0.429
	0.40	1.288	0.112	0.248	0.297	0.347	0.397	0.446	0.496	0.545
1.00	0.45	1.320	0.140	0.312	0.374	0.436	0.499	0.561	0.623	0.686
	0.50	1.350	0.175	0.390	0.468	0.546	0.624	0.701	0.779	0.857
	0.55	1.380	0.219	0.487	0.584	0.681	0.779	0.876	0.973	1.071
	0.60	1.408	0.274	0.610	0.732	0.854	0.975	1.097	1.219	*
	0.30	1.207	0.084	0.187	0.224	0.261	0.299	0.336	0.373	0.411
	0.35	1.238	0.108	0.240	0.289	0.337	0.385	0.433	0.481	0.529
	0.40	1.267	0.137	0.305	0.366	0.427	0.488	0.548	0.609	0.670
1.25	0.45	1.295	0.172	0.382	0.459	0.535	0.612	0.688	0.764	0.841
	0.50	1.321	0.215	0.477	0.572	0.668	0.763	0.858	0.954	1.049
	0.55	1.347	0.267	0.594	0.713	0.832	0.951	1.069	1.188	1.307
	0.60	1.371	0.334	0.742	0.891	1.039	1.188	1.336	1.485	*
	0.30	1.194	0.100	0.222	0.266	0.310	0.354	0.399	0.443	0.487
	0.35	1.221	0.128	0.285	0.342	0.399	0.456	0.512	0.569	0.626
	0.40	1.247	0.162	0.360	0.432	0.504	0.576	0.648	0.720	0.792
1.50	0.45	1.272	0.203	0.451	0.541	0.631	0.721	0.811	0.901	0.991
	0.50	1.295	0.252	0.561	0.673	0.785	0.898	1.010	1.122	1.234
	0.55	1.318	0.314	0.697	0.837	0.976	1.116	1.255	1.395	1.534
	0.60	1.338	0.391	0.869	1.043	1.217	1.391	1.565	1.738	*
	0.30	1.181	0.115	0.256	0.307	0.358	0.409	0.460	0.512	0.563
	0.35	1.206	0.148	0.328	0.394	0.459	0.525	0.591	0.656	0.722
	0.40	1.230	0.186	0.414	0.497	0.580	0.663	0.745	0.828	0.911
1.75	0.45	1.251	0.233	0.517	0.621	0.724	0.828	0.931	1.034	1.138
	0.50	1.272	0.289	0.642	0.771	0.899	1.028	1.156	1.285	1.413
	0.55	1.291	0.359	0.797	0.956	1.116	1.275	1.434	1.594	1.753
	0.60	1.308	0.446	0.991	1.189	1.387	1.586	1.784	1.982	*
	0.30	1.170	0.130	0.289	0.347	0.405	0.463	0.521	0.579	0.637
	0.35	1.193	0.167	0.371	0.445	0.519	0.593	0.667	0.741	0.816
	0.40	1.213	0.210	0.467	0.560	0.654	0.747	0.841	0.934	1.027
2.00	0.45	1.233	0.262	0.582	0.699	0.815	0.932	1.048	1.164	1.281
	0.50	1.250	0.325	0.722	0.866	1.010	1.155	1.299	1.443	1.588
	0.55	1.266	0.402	0.893	1.072	1.251	1.429	1.608	1.786	1.965
	0.60	1.280	0.499	1.109	1.330	1.552	1.774	1.995	2.217	*
	0.30	1.160	0.145	0.323	0.387	0.452	0.516	0.581	0.646	0.710
	0.35	1.180	0.186	0.413	0.495	0.578	0.660	0.743	0.825	0.908
	0.40	1.198	0.234	0.519	0.623	0.726	0.830	0.934	1.038	1.142
2.25	0.45	1.215	0.291	0.646	0.775	0.904	1.033	1.162	1.291	1.421
	0.50	1.230	0.360	0.799	0.959	1.118	1.278	1.438	1.598	1.758
	0.55	1.243	0.444	0.987	1.184	1.382	1.579	1.776	1.974	2.171
	0.60	1.254	0.550	1.222	1.467	1.711	1.955	2.200	2.444	*

* a/H plus 6(2H/d) exceeds 5.6.

Factored Shear Resistance of Girder Webs

F_y = 300 MPa

ϕ = 0.90

Top number = Factored shear stress, ϕF_s (MPa)

Bottom number = Required Gross Area of Pairs of Intermediate Stiffeners, Percent of Web Area, h x w

Web h / w Ratio	Panel Aspect Ratio: **a / h** = Stiffener Spacing / Web Depth										No Inter-mediate Stiffeners
	0.50	**0.67**	**0.75**	**1.00**	**1.25**	**1.50**	**1.75**	**2.00**	**2.50**	**3.00**	
50											178
60										178 *0.77*	174
70					178 *1.37*	172 *1.26*	166 *1.15*	163 *1.06*	158 *0.89*	155 *0.77*	149
80			178 *1.50*	173 *1.46*	159 *1.37*	153 *1.26*	150 *1.15*	148 *1.06*	144 *0.89*	141 *0.77*	131
90			178 *1.50*	155 *1.46*	149 *1.37*	145 *1.26*	141 *1.75*	138 *2.02*	131 *2.12*	127 *2.02*	107
100		178 *1.49*	166 *1.50*	149 *1.46*	143 *2.52*	135 *3.33*	128 *3.61*	123 *3.64*	116 *3.42*	111 *3.10*	86.5
110	178 *1.38*	164 *1.49*	154 *1.50*	144 *2.96*	133 *4.46*	124 *4.94*	118 *4.99*	112 *4.84*	104 *4.37*	98.9 *3.90*	71.5
120	178 *1.38*	154 *1.49*	151 *1.50*	136 *4.83*	125 *5.93*	116 *6.16*	110 *6.03*	104 *5.75*	95.8 *5.10*	89.9 *4.50*	60.1
130	175 *1.38*	151 *1.49*	147 *2.62*	130 *6.28*	119 *7.08*	110 *7.11*	103 *6.84*	97.6 *6.46*	89.0 *5.67*	83.0 *4.98*	51.2
140	163 *1.38*	148 *2.41*	142 *4.33*	125 *7.43*	114 *7.99*	105 *7.87*	98.3 *7.49*	92.5 *7.03*	83.7 *6.12*	77.4 *5.35*	44.1
150	155 *1.38*	144 *4.01*	138 *5.70*	122 *8.36*	110 *8.73*	102 *8.48*	94.3 *8.01*	88.3 *7.48*	79.3 *6.49*	72.9 *5.65*	38.4
160	153 *1.38*	140 *5.32*	134 *6.83*	118 *9.12*	107 *9.33*	98.3 *8.98*	91.0 *8.44*	85.0 *7.86*	75.8 *6.78*		33.8
180	149 *2.64*	134 *7.32*	129 *8.54*	114 *10.3*	102 *10.2*	93.4 *9.74*	85.9 *9.09*	79.8 *8.42*			26.7
200	144 *4.77*	130 *8.75*	125 *9.77*	110 *11.1*	99.0 *10.9*	89.9 *10.3*					21.6
220	140 *6.34*	127 *9.81*	122 *10.7*	108 *11.7*	96.5 *11.4*						17.9
240	137 *7.53*	125 *10.6*	120 *11.4*	106 *12.2*			**Not Permitted**				15.0
260	134 *8.46*	123 *11.2*	118 *11.9*								12.8

Notes:

- For shear resistance and stiffener area, see S16-14 Clauses 13.4.1.1 and 14.5.3, respectively.
- For maximum web slenderness and stiffener spacing, see S16-14 Clauses 14.3.1 and 14.5.2, respectively.
- For single stiffeners on one side of web only, multiply percentages shown by 1.8 for angle stiffeners and by 2.4 for plate stiffeners.
- When the stiffener F_y is not the same as the web F_y, multiply gross area by the ratio ($F_{y\,web}$ / $F_{y\,stiffener}$).

Factored Shear Resistance of Girder Webs

$F_y = 350$ MPa

$\phi = 0.90$

Top number = Factored shear stress, ϕF_s (MPa)

Bottom number = Required Gross Area of Pairs of Intermediate Stiffeners, Percent of Web Area, h x w

Web h / w Ratio	Panel Aspect Ratio: **a / h** = Stiffener Spacing / Web Depth										No Inter-mediate Stiffeners
	0.50	**0.67**	**0.75**	**1.00**	**1.25**	**1.50**	**1.75**	**2.00**	**2.50**	**3.00**	
50										208 *0.77*	208
60							208 *1.15*	205 *1.06*	199 *0.89*	196 *0.77*	188
70				208 *1.46*	196 *1.37*	186 *1.26*	181 *1.15*	178 *1.06*	174 *0.89*	172 *0.77*	161
80			208 *1.50*	187 *1.46*	177 *1.37*	172 *1.26*	168 *1.15*	165 *1.29*	160 *1.54*	156 *1.54*	135
90		208 *1.49*	199 *1.50*	176 *1.46*	168 *1.86*	161 *2.79*	154 *3.15*	148 *3.24*	140 *3.09*	134 *2.83*	107
100	208 *1.38*	195 *1.49*	181 *1.50*	169 *2.53*	157 *4.11*	147 *4.66*	140 *4.74*	133 *4.63*	124 *4.21*	118 *3.75*	86.5
110	208 *1.38*	180 *1.49*	176 *1.50*	160 *4.63*	147 *5.78*	137 *6.03*	129 *5.92*	122 *5.66*	113 *5.03*	106 *4.44*	71.5
120	205 *1.38*	176 *1.49*	172 *2.55*	152 *6.23*	139 *7.04*	129 *7.08*	121 *6.82*	114 *6.44*	104 *5.65*	97.1 *4.96*	60.1
130	189 *1.38*	173 *2.48*	166 *4.39*	146 *7.48*	133 *8.03*	123 *7.90*	114 *7.51*	108 *7.05*	97.4 *6.14*	90.1 *5.36*	51.2
140	180 *1.38*	168 *4.18*	160 *5.85*	141 *8.46*	128 *8.81*	118 *8.54*	109 *8.07*	103 *7.53*	92.0 *6.52*	84.5 *5.69*	44.1
150	178 *1.38*	163 *5.56*	156 *7.03*	137 *9.26*	124 *9.44*	114 *9.07*	105 *8.51*	98.4 *7.92*	87.7 *6.84*	80.0 *5.94*	38.4
160	176 *1.69*	159 *6.68*	152 *8.00*	134 *9.91*	121 *9.95*	111 *9.49*	102 *8.88*	95.0 *8.24*	84.2 *7.09*		33.8
170	173 *3.08*	156 *7.62*	149 *8.80*	132 *10.5*	119 *10.4*	108 *9.85*	99.4 *9.18*	92.2 *8.51*			29.9
180	169 *4.24*	153 *8.40*	147 *9.47*	129 *10.9*	117 *10.7*	106 *10.1*	97.1 *9.43*	89.9 *8.73*			26.7
190	167 *5.22*	151 *9.06*	145 *10.0*	128 *11.3*	115 *11.0*	104 *10.4*	95.2 *9.65*				24.0
200	164 *6.06*	149 *9.63*	143 *10.5*	126 *11.6*	113 *11.3*	102 *10.6*		**Not Permitted**			21.6
220	160 *7.41*	146 *10.5*	140 *11.3*	123 *12.1*	111 *11.7*						17.9

Notes:

- For shear resistance and stiffener area, see S16-14 Clauses 13.4.1.1 and 14.5.3, respectively.
- For maximum web slenderness and stiffener spacing, see S16-14 Clauses 14.3.1 and 14.5.2, respectively.
- For single stiffeners on one side of web only, multiply percentages shown by 1.8 for angle stiffeners and by 2.4 for plate stiffeners.
- When the stiffener F_y is not the same as the web F_y, multiply gross area by the ratio ($F_{y\,web}$ / $F_{y\,stiffener}$).

DESIGN EXAMPLE FOR STIFFENED GIRDER WEBS

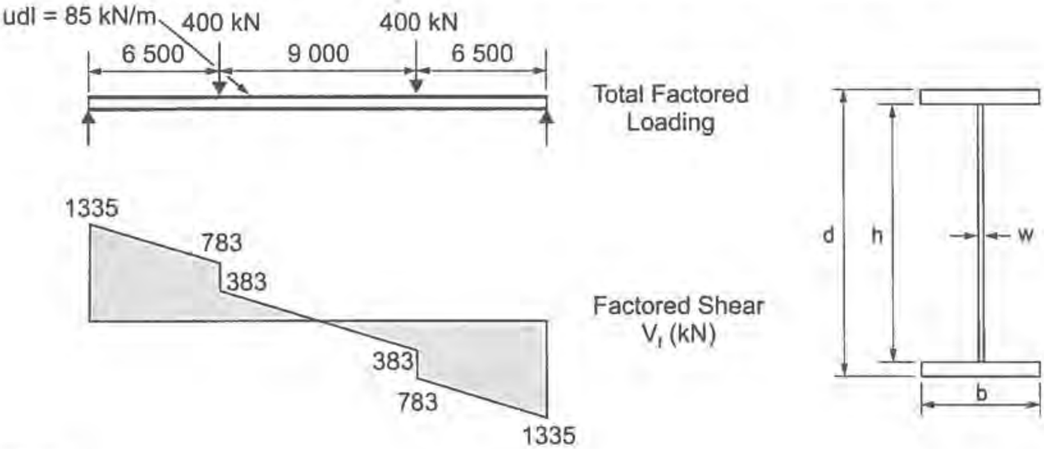
Design Example – Web Shear Resistance

Given:

Find the shear resistance of a simply supported welded plate girder spanning 22 m and loaded as shown. The grade of steel is G40.21-350W.

Girder cross-sectional dimensions:

$d = 1800$ mm, $b = 500$ mm, $t = 30$ mm, $w = 10$ mm, $h = d - 2t = 1740$ mm



Solution:

a) Shear resistance of end panels

Factored ultimate shear force in girder web: $V_f = 1335$ kN

Maximum h/w permitted $= 83\,000 / F_y = 83\,000 / 350 = 237$ (S16-14 Clause 14.3.1)

Web slenderness ratio: $h/w = 1740 / 10 = 174 < 237$

Maximum a/h permitted $= 67\,500 / (h/w)^2$

$$= 67\,500 / 174^2 = 2.23 \quad \text{for } h/w > 150 \quad \text{(Clause 14.5.2)}$$

Size the end panel without the tension-field action, in accordance with Clause 14.5.1.

Find the maximum stiffener spacing:

Assuming that Clause 13.4.1.1(b)(iv) applies, for $h/w = 174$ and $F_s = F_{cre}$

$$V_r = \phi A_w \frac{180\,000 k_v}{(h/w)^2} = 0.9 \times 17\,400 \times \frac{180\,000 k_v}{174^2} = 93\,100 k_v$$

Equating V_r to $V_f = 1335$ kN gives $k_v = 14.34$

$$k_v = 4 + \frac{5.34}{(a/h)^2} = 14.34, \quad \frac{a}{h} = \sqrt{\frac{5.34}{14.34 - 4}} = 0.719$$

Therefore, the maximum end panel length is $0.719 \times 1740 = 1250$ mm.

Try $a = 1000$ mm. The panel aspect ratio is $a/h = 1000 / 1740 = 0.575$

$$k_v = 4 + \frac{5.34}{(a/h)^2} = 4 + \frac{5.34}{0.575^2} = 20.2$$

$$\frac{h}{w} = 174 > 621\sqrt{\frac{k_v}{F_y}} = 621\sqrt{\frac{20.2}{350}} = 149$$

This confirms that Clause 13.4.1.1(b)(iv) applies, as assumed above.

b) Shear resistance between end panel and concentrated load

Using an end panel length of a = 1000 mm and two equal panels between the end stiffener and the stiffener at the interior concentrated load gives an intermediate stiffener spacing of: (6500 – 1000) / 2 = 2750 mm

Factored shear force at the first intermediate stiffener, by linear interpolation:

V_f = 1335 – (1335 – 783) (1000 / 6500) = 1250 kN

From the table for *Factored Shear Resistance of Girder Webs* (F_y = 350 MPa) for h/w = 174 and a/h = 2750 / 1740 = 1.58

ϕF_s = 104 MPa (by interpolation)

Factored shear resistance: $V_r = A_w(\phi F_s)$ = 17 400 × 104 = 1810 kN > V_f = 1250 kN

c) Intermediate Stiffener Size

For h/w = 174 and a/h = 1.58, by interpolation from the tabulated values:

The required total area of a pair of intermediate stiffeners is 9.74% of the web area.

A_s = 0.0974 × 17 400 = 1690 mm²

Required $I_s = (h/50)^4 = (1740/50)^4 = 1.47 \times 10^6$ mm⁴ (Clause 14.5.3)

Maximum $\dfrac{b}{t} = \dfrac{200}{\sqrt{F_y}} = 10.7$ Use two 10 × 100 stiffeners

$A_s = 2 \times 10 \times 100 = 2000 > 1690$ mm² $I_s = 10 \times 210^3 / 12 = 7.72 \times 10^6 > 1.47 \times 10^6$ mm⁴

$b/t = 100/10 = 10.0 < 10.7$

d) Shear resistance between concentrated loads

Factored ultimate shear force in girder web: V_f = 383 kN Try an unstiffened web.

For h/w = 174: ϕF_s = 28.6 MPa (by interpolation from the tabulated values)

Factored shear resistance: $V_r = A_w(\phi F_s)$ = 17 400 × 28.6 = 498 kN > 383 kN

Therefore, stiffeners are not required between the two concentrated loads (but are required at the concentrated load locations). Design checks for bearing at concentrated loads, moment, and combined shear and moment are not shown. See "Limit States Design in Structural Steel", Kulak and Grondin, CISC, for examples.

NOTES

BEAM BEARING PLATES

General

When a flexural member is supported by a masonry wall or pier, the beam reaction must be distributed over sufficient area to avoid exceeding the bearing capacity of the masonry or concrete. Steel bearing plates may be used for this purpose.

Bearing plates are usually set in place and grouted level at the required elevation before positioning the beam. Thus, even though the beam flange may be able to distribute the reaction to supporting masonry or concrete, a bearing plate can be useful to facilitate erection. Some form of anchorage is required to ensure that the beam is connected to the pier or wall either longitudinally or for uplift forces.

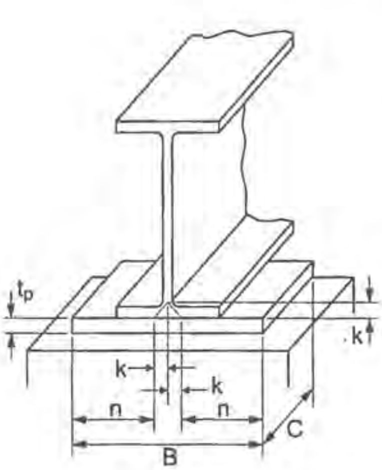
Design Chart

Figure 5-1 provides a graph to determine the thickness of bearing plates using CSA G40.21-300W steel, for beams without bearing stiffeners, based on the following assumptions:

- The beam reaction P_f is uniformly distributed to the bearing plate over an effective area of width $2k$ and length C.
- The bearing pressure between the effective area of the bearing plate and the concrete or masonry support is uniform over the area of the plate.
- The bearing pressure under the portion of plate projecting beyond the k-distance from the centre line of the beam is ignored, since in practice the flange may be slightly "curled".

Equating the factored moment acting on the portion of the bearing plate, taken as a cantilever, to the factored moment resistance of the plate, ($M_r = \phi\, Z\, F_y$), the bearing plate thickness is calculated as:

$$t_p = \sqrt{\frac{2\,P_f\,n^2}{A\,\phi\,F_y}}$$



where:

P_f = factored end reaction

F_y = specified minimum yield strength of the bearing plate steel (MPa)

A = $B \times C$ = area of plate (mm^2)

t_p = required thickness of bearing plate (mm)

k = beam k-distance = distance from web toe of fillet to outer face of flange (mm)

n = $B/2 - k$, (mm)

b = width of beam flange (mm)

To minimize deflection of the bearing plate, the thickness generally should not be less than about one fifth of the overhang, i.e., $t_p \geq (B - b)/10$.

Use of chart

1. Required area, A = beam reaction due to factored loads divided by the unit factored concrete bearing resistance, $(0.85\,\phi_c f'_c)$, where $\phi_c = 0.65$
2. Determine C and solve for B. (C, the length of bearing, is usually governed by the available wall thickness or other structural considerations.)
3. Determine n and enter Figure 5-1 to determine t_p.

Example

Given:

A W610x140 of ASTM A992 steel beam has a factored end reaction of 600 kN and is supported on a concrete pier with 28-day compressive strength of 20 MPa. Design the bearing plate assuming G40.21-300W steel and a concrete bearing length of 200 mm.

Solution:

Unit factored bearing resistance of concrete is:

$0.85 \times 0.65 \times 20 = 11.1$ MPa

Area required is $(600 \times 10^3)/11.1 = 54\,100$ mm^2

Therefore, required B is: $54\,100/200 = 271$ mm

For W610x140, $b = 230$ mm, $t = 22.2$ mm, $w = 13.1$ mm, $k = 54$ mm

Select $B = 280$ mm (greater than flange width, $b = 230$ mm)

$n = (B/2) - k = (280/2) - 54 = 86$ mm

From Figure 5-1, for unit factored bearing resistance of 11.1 MPa and n of 86 mm,

minimum $t_p \approx 24$ mm Select $t_p = 25$ mm

Use plate $25 \times 200 \times 280$

Check for web crippling and web yielding (Clause 14.3.2(b), S16-14)

Web crippling:

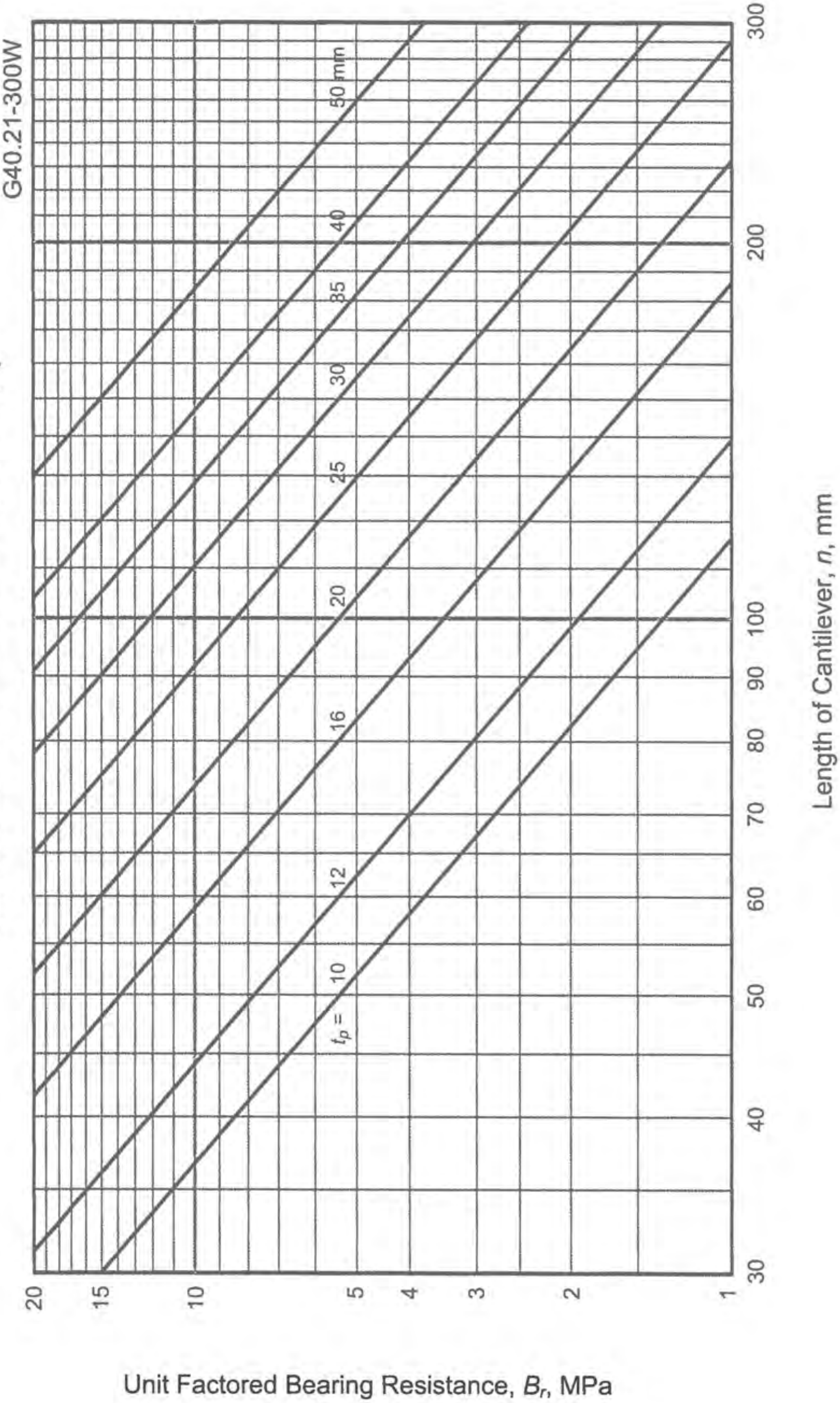
$B_r = 0.60\,\phi_{be}\,w^2\,(F_y\,E)^{0.5} = 0.60 \times 0.75 \times 13.1^2\,(345 \times 2 \times 10^5)^{0.5} = 641$ kN

Web yielding:

$B_r = \phi_{be}\,w\,(N + 4\,t)\,F_y = 0.75 \times 13.1\,(200 + 4 \times 22.2)\,345 = 979$ kN

Therefore web crippling, Clause 14.3.2(b)(ii), governs and $B_r = 641$ kN > 600 kN.

FIGURE 5-1
BEAM BEARING PLATE THICKNESS, t_p



COMPOSITE BEAMS

General

A composite beam, in general, consists of a steel beam and a concrete slab so interconnected that the steel beam and the slab jointly resist bending through composite action. Several combinations which effectively act as composite beams occur in practice. These include a steel beam or girder with a concrete slab interconnected with mechanical shear connectors, a steel beam or girder with a ribbed concrete slab formed by steel deck interconnected by mechanical shear connectors, and a steel beam or girder fully encased by the concrete in such a way that the encased beam and the concrete slab behave monolithically. Clause 17 of CSA S16-14 contains requirements for composite beams.

Some advantages of composite construction are:

- Reduced weight of steel members
- Reduced depth of steel members
- Reduced deflections under superimposed load
- Simplified changes to electrical services when steel deck is used

Composite construction is most advantageous when heavy loads and long spans are involved. For this reason composite construction is widely used for bridges. For building construction, composite beams consisting of steel beams with steel deck and concrete cover slab utilizing steel stud shear connectors welded to the beam top flange are most frequently used. Other types of composite construction used in buildings include composite trusses and joists, and stub-girders.

Tables

The Composite Beam Trial Selection Tables on the following pages are based on ASTM A992 and A572 grade 50 (F_y = 345 MPa). The tabulated values may also be used for W-shapes produced to CSA G40.21-350W, although grade 350W does not appear in the table headings. The tables list composite members for the practical range of rolled W-shapes from 200 mm to 1000 mm nominal depth. Tables are provided for the following combinations of deck-slab concrete strength and concrete density:

- 75 mm steel deck with 65 mm cover slab with f'_c of 25 MPa, 2350 kg/m^3 concrete
- 75 mm steel deck with 75 mm cover slab with f'_c of 25 MPa, 2350 kg/m^3 concrete
- 75 mm steel deck with 90 mm cover slab with f'_c of 25 MPa, 2350 kg/m^3 concrete
- 75 mm steel deck with 85 mm cover slab with f'_c of 25 MPa, 1850 kg/m^3 concrete

The tables show steel shapes listed in descending order of nominal depth and mass, and include the following properties, design data and resistances:

b	=	flange width of steel shape (mm)
t	=	flange thickness of steel shape (mm)
d	=	overall depth of steel shape (mm)
b_1	=	effective width of slab used in computing values of M_{rc}, Q_r, I_t, S_t and I_{ts} (mm). (Refer to Clause 17.4 of S16-14 for appropriate design effective width.)

M_{rc} = factored moment resistance of composite beam for percentage of full shear connection equal to 100%, 70% and 40% (kN·m)

Q_r = required sum of factored shear resistances between adjacent points of maximum and zero moment for 100% shear connection, (kN). Q_r = lesser of $\phi A_s F_y$ or $\phi_c \alpha_1 b_1 t_c f'_c$, where t_c = effective slab thickness or effective cover slab thickness

I_t = moment of inertia of the composite section, transformed into steel properties, computed using mass density as shown on each table (10^6 mm^4)

S_t = section modulus of the composite section related to the extreme fibre of the bottom flange of the steel beam based on the value of I_t (10^3 mm^3)

I_{ts} = transformed moment of inertia for calculating shrinkage deflections, based on the modular ratio n_s. (See S16-14 Annex H for further information.)

M_r = factored moment resistance of laterally supported bare steel section (kN·m)

V_r = factored shear resistance of the bare steel beam; also taken to be the factored shear resistance of the composite section (kN)

L_u = maximum unsupported length of compression flange of the steel beam alone for which no reduction in M_r is required (mm)

I_x = moment of inertia about the x-x axis of the bare steel beam (10^6 mm^4)

S_x = section modulus of the bare steel beam (10^3 mm^3)

M_r' = factored moment resistance of the bare steel beam for an unsupported length L' (kN·m).

Since the concrete slab and/or the steel deck prevent movement of the top flange, lateral buckling is not a consideration at composite action. During construction, however, the unsupported length of the compression flange may be greater than L_u, and the moment resistance for the non-composite shape for the appropriate unsupported length of compression flange must be used.

The tabulated factored shear resistance V_r is computed according to Clause 13.4.1.1 of S16-14 for the appropriate h/w ratio.

Shear Connectors

Clauses 17.9.5 and 17.9.6 stipulate the amount of total factored horizontal shear force that must be resisted by shear connectors.

For full (i.e. 100%) shear connection, the total factored horizontal shear force V_h to be transferred between the point of maximum positive moment and adjacent points of zero moment is either:

- $\phi A_s F_y$ when the plastic neutral axis is in the slab, or
- $\phi_c \alpha_1 b_1 t_c f'_c$ when the plastic neutral axis is in the steel section.

For partial shear connection the total factored horizontal shear force V_h is the sum of the factored resistances of all the shear connectors between the point of maximum positive moment and each adjacent point of zero moment. S16-14 Clause 17.9.4 limits the minimum amount of partial shear connection to 40% of either $\phi A_s F_y$ or $\phi_c \alpha_1 b_1 t_c f'_c$, whichever is the lesser, when computing flexural strength.

Generally, shear connectors may be uniformly spaced in regions of positive or negative bending. However, when a concentrated load occurs within a region of positive bending, the number of shear connectors and the shear connector spacing is determined by Clause 17.9.8.

Tables 5-5 and 5-6 provide values of the factored shear resistance q_r for the most common sizes of end-welded shear studs according to the requirements of Clause 17.7 when the stud height is at least four stud diameters, and when the stud projection in a ribbed slab is at least two stud diameters above the top surface of the steel deck.

Table 5-5 gives values of q_r for stud diameters of ¾ inch (19 mm), ⅝ inch (15.9 mm), and ½ inch (12.7 mm) in solid slabs, or in deck-slabs with ribs parallel to the beam, based on three concrete strength levels f'_c of 20 MPa, 25 MPa, and 30 MPa for both normal density (2350 kg/m^3) and semi-low density (1850 kg/m^3) concrete. Values are calculated according to Clause 17.7.2.2 and Clause 17.7.2.3.

Tables 5-6a and 5-6b give values of q_{rr} for ¾ inch (19 mm) and ⅝ inch (15.9 mm) diameter studs, respectively, in ribbed slabs for 75 mm or 38 mm deck, with ribs perpendicular to the beam, calculated according to Clause 17.7.2.4. Values are given for three concrete strength levels f'_c of 20 MPa, 25 MPa, and 30 MPa for both normal density (2350 kg/m^3) and semi-low density (1850 kg/m^3) concrete.

Deflections

Composite beams are stiffer than similar non-composite beams, and deflections are reduced when composite construction is used. Due to creep of the concrete slab over time, maximum deflections may increase, especially if the full load is sustained. Annex H of CSA S16-14 provides guidance for estimating deflections caused by shrinkage of the concrete slab. Beam deflection during construction, due to loads supported prior to hardening of the concrete while the steel beam alone supports the loads, should be checked. Cambering or the use of temporary shores will reduce the total final deflection.

For steel beams unshored during construction, S16-14 Clause 17.11 limits the stress (caused by the total of the specified loads applied before the concrete strength reaches $0.75f'_c$ and, at the same location, the remaining specified loads acting on the composite section) in the tension flange to F_y.

Other Composite Members

Other composite members suitable for floor construction include composite trusses, composite open-web steel joists, and stub-girders. Optimum spans for performance and economy depend on overall building considerations such as storey height restrictions and integration of building services.

For composite trusses and joists, Clause 17.9.2 of S16-14 stipulates that the area of the top chord shall be neglected in determining the properties of the composite section, and that the factored moment resistance of the composite truss or joist shall be computed on the basis of full shear connection with the plastic neutral axis in the slab.

Composite stub-girders use wide-flange column shapes with short W-shape stubs shop-welded to the top of the girders and interconnected with the deck-slab by shear connectors to provide Vierendeel girder action. Deck-slabs usually consist of a 75 mm composite steel deck with 75 mm or 85 mm cover slabs.

Availability

Beam sizes that are commonly used and readily available are highlighted in yellow.

References

PART TWO of this Handbook. See CISC Commentary on Clause 17.

CHIEN, E.Y.L., RITCHIE, J.K., 1984. Design and Construction of Composite Floor Systems. Canadian Institute of Steel Construction, Willowdale, Ontario.

KULAK, G.L., GRONDIN, G.Y. 2014. Limit States Design in Structural Steel, 9th Edition. Canadian Institute of Steel Construction, Markham, Ontario.

BEAULIEU, D., PICARD, A., TREMBLAY, R., GRONDIN, G., MASSICOTTE, B. 2010. Calcul des charpentes d'acier - Tome II. Institut canadien de la construction en acier, Markham, Ontario.

Factored Shear Resistance of Shear Studs — Table 5-5

In Solid Slabs and in Deck-Slabs with Ribs Parallel to Beam ($3.0 > w_d / h_d \geq 1.5$)

Stud in a Solid Slab, q_{rs} (kN)

Stud Diameter	f'_c (γ_c = 2350 kg/m^3)			f'_c (γ_c = 1850 kg/m^3)		
	20 MPa	25 MPa	30 MPa	20 MPa	25 MPa	30 MPa
¾ in. (19 mm)	75.9	88.2	99.8	63.4	73.7	83.4
⅝ in. (15.9 mm)	53.1	61.7	69.9	44.4	51.6	58.4
½ in. (12.7 mm)	33.9	39.4	44.6	28.3	32.9	37.3

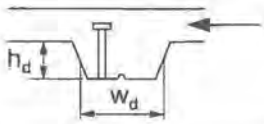
Stud in a Deck-Slab with Ribs Parallel to Beam, q_{rr} (kN)

Stud Diameter	w_d / h_d	f'_c (γ_c = 2350 kg/m^3)			f'_c (γ_c = 1850 kg/m^3)		
		20 MPa	25 MPa	30 MPa	20 MPa	25 MPa	30 MPa
¾ in. (19 mm)	2.5	69.6	80.8	91.5	58.1	67.6	76.5
	2.4	68.3	79.4	89.8	57.1	66.3	75.1
	2.0	63.2	73.5	83.2	52.8	61.4	69.5
⅝ in. (15.9 mm)	2.5	48.7	56.6	64.1	40.7	47.3	53.5
	2.4	47.8	55.6	62.9	40.0	46.5	52.6
	2.0	44.3	51.5	58.2	37.0	43.0	48.7
½ in. (12.7 mm)	2.5	31.1	36.1	40.9	26.0	30.2	34.2
	2.4	30.5	35.5	40.1	25.5	29.6	33.5
	2.0	28.2	32.8	37.2	23.6	27.4	31.0

Factored Resistance of Shear Studs

Table 5-6a

Ribs Perpendicular to Beam
¾ in. (19 mm) Diameter Studs, F_u = 450 MPa
75 mm or 38 mm-Deep Steel Deck



Deck		Stud connector(s)				Pull-out area	Factored shear resistance of stud(s), q_{rr} (kN)					
h_d	$\frac{w_d}{h_d}$	Dia.	Length	n	Edge distance	A_p	f'_c (γ_c = 2350 kg/m^3)			f'_c (γ_c = 1850 kg/m^3)		
mm		mm	mm		mm	10^3 mm^2	20 MPa	25 MPa	30 MPa	20 MPa	25 MPa	30 MPa
75	2.4	¾ in.	115	1	Int.	52.0	65.2	72.9	79.8	55.4	61.9	67.8
		(19)		1	65	40.7	51.0	57.0	62.5	43.4	48.5	53.1
				1	35	33.9	42.5	47.5	52.1	36.1	40.4	44.2
				2	Int.	69.2	86.7	96.9	106	73.7	82.4	90.3
			150	1	Int.	82.7	75.9	88.2	99.8	63.4	73.7	83.4
				1	65	60.7	75.9	84.9	93.0	63.4	72.2	79.1
				1	35	51.8	64.8	72.5	79.4	55.1	61.6	67.5
				2	Int.	105	132	147	161	112	125	137
75	2.0	¾ in.	115	1	Int.	49.1	61.4	68.7	75.3	52.2	58.4	64.0
		(19)		1	65	38.5	48.2	53.9	59.0	40.9	45.8	50.1
				1	35	32.1	40.2	44.9	49.2	34.2	38.2	41.8
				2	Int.	65.5	82.0	91.6	100	69.7	77.9	85.3
			150	1	Int.	71.6	75.9	88.2	99.8	63.4	73.7	83.4
				1	65	53.0	66.4	74.2	81.3	56.4	63.1	69.1
				1	35	45.1	56.4	63.1	69.1	48.0	53.6	58.8
				2	Int.	91.7	115	128	141	97.7	109	120
38	2.5	¾ in.	75	1	Int.	20.2	44.0	49.2	53.9	37.4	41.8	45.8
		(19)		1	65	18.8	41.0	45.9	50.3	34.9	39.0	42.7
				1	35	14.8	32.3	36.1	39.5	27.4	30.7	33.6
				2	Int.	30.4	66.3	74.1	81.2	56.3	63.0	69.0
			100	1	Int.	32.0	69.7	78.0	85.4	59.3	66.3	72.6
				1	65	27.3	59.5	66.5	72.8	50.6	56.5	61.9
				1	35	22.1	48.3	54.0	59.1	41.0	45.9	50.2
				2	Int.	45.3	98.8	110	121	84.0	93.9	103
38	1.4	¾ in.	75	1	Int.	13.5	29.4	32.8	36.0	25.0	27.9	30.6
		(19)		1	65	12.7	27.7	31.0	34.0	23.6	26.4	28.9
				1	35	10.4	22.7	25.4	27.8	19.3	21.6	23.6
				2	Int.	21.4	46.7	52.3	57.2	39.7	44.4	48.7
			100	1	Int.	27.5	59.9	67.0	73.4	50.9	57.0	62.4
				1	65	24.8	54.2	60.6	66.4	46.1	51.5	56.4
				1	35	19.9	43.4	48.5	53.1	36.9	41.2	45.1
				2	Int.	40.8	89.0	99.5	109	75.7	84.6	92.7

Factored shear resistances are calculated in accordance with CSA S16-14 Clause 17.7.2.4.

Notes:

1. n = number of studs per rib, γ_c = density of concrete
2. Stud length listed is the length after welding.
   Minimum length prior to welding = stud length listed + 10 mm fusion allowance.
3. Double studs transversely spaced at minimum 4 stud diameters.
4. Int. = interior condition
5. Studs placed off-centre in ribs of 75 mm deck and on-centre in ribs of 38 mm deck.

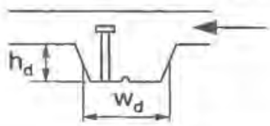
Factored Resistance of Shear Studs

Table 5-6b

Ribs Perpendicular to Beam

⅝ in. (15.9 mm) Diameter Studs, F_u = 450 MPa

75 mm or 38 mm-Deep Steel Deck



Deck		Stud connector(s)				Pull-out area	Factored shear resistance of stud(s), q_{rr} (kN)					
h_d	$\frac{w_d}{h_d}$	Dia.	Length	n	Edge distance	A_p	f'_c (γ_c = 2350 kg/m^3)			f'_c (γ_c = 1850 kg/m^3)		
mm		mm	mm		mm	10^3 mm^2	20 MPa	25 MPa	30 MPa	20 MPa	25 MPa	30 MPa
75	2.4	⅝ in.	115	1	Int.	52.0	53.1	61.7	69.9	44.4	51.6	58.4
		(15.9)		1	65	40.7	51.0	57.0	62.5	43.4	48.5	53.1
				1	35	33.9	42.5	47.5	52.1	36.1	40.4	44.2
				2	Int.	66.4	83.2	93.0	102	70.7	79.1	86.6
			150	1	Int.	82.7	53.1	61.7	69.9	44.4	51.6	58.4
				1	65	60.7	53.1	61.7	69.9	44.4	51.6	58.4
				1	35	51.8	53.1	61.7	69.9	44.4	51.6	58.4
				2	Int.	102	106	123	140	88.8	103	117
75	2.0	⅝ in.	115	1	Int.	49.1	53.1	61.7	69.9	44.4	51.6	58.4
		(15.9)		1	65	38.5	48.2	53.9	59.0	40.9	45.8	50.1
				1	35	32.1	40.2	44.9	49.2	34.2	38.2	41.8
				2	Int.	62.8	78.6	87.9	96.3	66.8	74.7	81.9
			150	1	Int.	71.6	53.1	61.7	69.9	44.4	51.6	58.4
				1	65	53.0	53.1	61.7	69.9	44.4	51.6	58.4
				1	35	45.1	53.1	61.7	69.1	44.4	51.6	58.4
				2	Int.	88.5	106	123	136	88.8	103	115
38	2.5	⅝ in.	75	1	Int.	20.2	44.0	49.2	53.9	37.4	41.8	45.8
		(15.9)		1	65	18.8	41.0	45.9	50.3	34.9	39.0	42.7
				1	35	14.8	32.3	36.1	39.5	27.4	30.7	33.6
				2	Int.	28.7	62.6	70.0	76.7	53.2	59.5	65.2
			100	1	Int.	32.0	53.1	61.7	69.9	44.4	51.6	58.4
				1	65	27.3	53.1	61.7	69.9	44.4	51.6	58.4
				1	35	22.1	48.3	54.0	59.1	41.0	45.9	50.2
				2	Int.	43.1	94.1	105	115	80.0	89.4	97.9
38	1.4	⅝ in.	75	1	Int.	13.5	29.4	32.8	36.0	25.0	27.9	30.6
		(15.9)		1	65	12.7	27.7	31.0	34.0	23.6	26.4	28.9
				1	35	10.4	22.7	25.4	27.8	19.3	21.6	23.6
				2	Int.	20.1	43.9	49.1	53.8	37.3	41.7	45.7
			100	1	Int.	27.5	53.1	61.7	69.9	44.4	51.6	58.4
				1	65	24.8	53.1	60.6	66.4	44.4	51.5	56.4
				1	35	19.9	43.4	48.5	53.1	36.9	41.2	45.1
				2	Int.	38.6	84.3	94.2	103	71.6	80.1	87.7

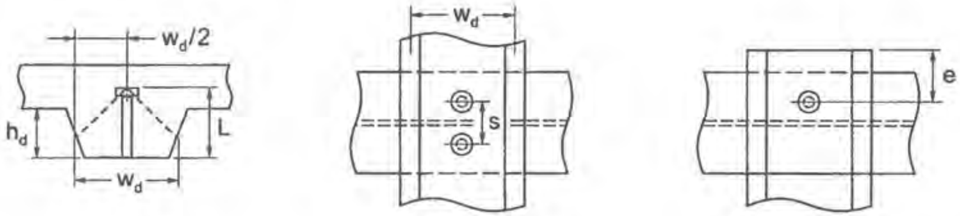
Factored shear resistances are calculated in accordance with CSA S16-14 Clause 17.7.2.4.

Notes:

1. n = number of studs per rib, γ_c = density of concrete
2. Stud length listed is the length after welding.
   Minimum length prior to welding ≈ stud length listed + 10 mm fusion allowance.
3. Double studs transversely spaced at minimum 4 stud diameters.
4. Int. = interior condition
5. Studs placed off-centre in ribs of 75 mm deck and on-centre in ribs of 38 mm deck.

TABLE 5-7a Pull-out Area, A_p

Studs placed on-centre in ribs



n = 2, s ≥ 4 dia.

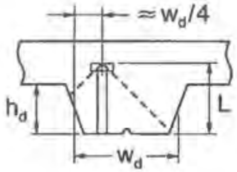
Spandrel beam

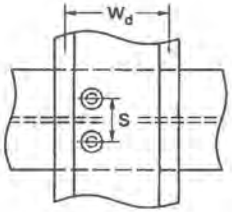
Stud length, L	A_p	If double studs (n=2), add:	If spandrel beam and e < L, subtract:	
a) $L \leq w_d/2$	$4\sqrt{2}\,L^2$	$2\sqrt{2}\,s\,L$	$2\sqrt{2}\,L\,(L-e)$	
b) $w_d/2 < L \leq w_d/2 + h_d$	$2\sqrt{2}\,L\,w_d$	$\sqrt{2}\,s\,w_d$	$\sqrt{2}\,w_d\,(L-e)$	
c) $L > w_d/2 + h_d$	$2\sqrt{2}\,[2\,(L-h_d)^2 + h_d\,w_d]$	$2\sqrt{2}\,s\,(L-h_d)$	**i)** $e \geq L - h_d$	$\sqrt{2}\,w_d\,(L-e)$
			ii) $e < L - h_d$	$\sqrt{2}\,[2\,(L-h_d)^2 + w_d\,h_d - 2\,e\,(L-h_d)]$

See CSA S16-14 Clause 17.7.2.4.

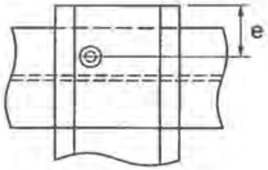
TABLE 5-7b Pull-out Area, A_p

Studs placed off-centre in ribs





$n = 2$, $s \geq 4$ dia.

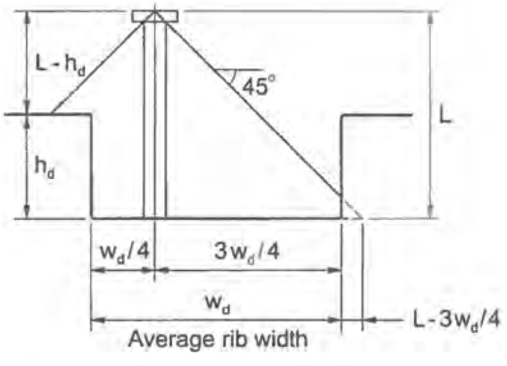


Spandrel beam

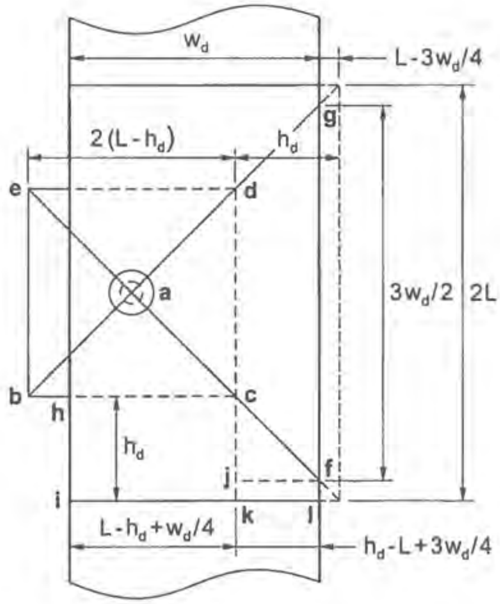
Stud length, L	A_p	If double studs (n=2), add:	If spandrel beam and e < L, subtract:	
a) $L \leq w_d/4$	$4\sqrt{2}\,L^2$	$2\sqrt{2}\,s\,L$	$2\sqrt{2}\,L\,(L-e)$	
b) $w_d/4 < L \leq 3\,w_d/4$ and $L \leq w_d/4 + h_d$	$\sqrt{2}\,L\,(2L + w_d/2)$	$\sqrt{2}\,s\,(L + w_d/4)$	$\sqrt{2}\,(w_d/4 + L)\,(L-e)$	
c) $3\,w_d/4 < L \leq w_d/4 + h_d$	$2\sqrt{2}\,L\,w_d$	$\sqrt{2}\,s\,w_d$	$\sqrt{2}\,w_d\,(L-e)$	
d) $w_d/4 + h_d < L \leq 3\,w_d/4$	$\sqrt{2}\,(4L^2 - 4L\,h_d + 2\,h_d^2 + h_d\,w_d/2)$	$\sqrt{2}\,s\,(2L - h_d)$	i) $e \geq L - h_d$	$\sqrt{2}\,(w_d/4 + L)\,(L-e)$
			ii) $e < L - h_d$	$\sqrt{2}\,[(w_d/4 + L)(L-e) + (L - h_d - e)(L - h_d - w_d/4)]$
e) $L > w_d/4 + h_d$ and $3\,w_d/4 < L \leq 3\,w_d/4 + h_d$	$\sqrt{2}\,[2\,(L-h_d)^2 + h_d\,w_d/2 + 3\,L\,w_d/2]$	$\sqrt{2}\,s\,(L - h_d + 3\,w_d/4)$	i) $e \geq L - h_d$	$\sqrt{2}\,w_d\,(L-e)$
			ii) $e < L - h_d$	$\sqrt{2}\,[w_d\,(L-e) + (L - h_d - e)(L - h_d - w_d/4)]$
f) $L > 3\,w_d/4 + h_d$	$\sqrt{2}\,[4\,(L-h_d)^2 + 2\,h_d\,w_d]$	$2\sqrt{2}\,s\,(L - h_d)$	i) $e \geq L - h_d$	$\sqrt{2}\,w_d\,(L-e)$
			ii) $e < L - h_d$	$\sqrt{2}\,[2\,(L-h_d)^2 + w_d\,h_d - 2\,e\,(L-h_d)]$

See CSA S16-14 Clause 17.7.2.4.

Design Example: Area of Concrete Pull-Out Pyramid, A_p



Idealized geometry: vertical rib walls



Find the area of the concrete pull-out pyramid for a single stud placed off-centre in a rib, in accordance with CSA S16-14 Clause 17.7.2.4, for the following configuration:

$$L > \frac{w_d}{4} + h_d \quad \text{and} \quad \frac{3}{4} w_d < L \le \frac{3}{4} w_d + h_d$$

For simplicity, rib walls are assumed to be vertical, with w_d = average rib width. For other deck and stud configurations, see the previous pages.

Surface	Area
abc + acd + ade + aeb	$4\sqrt{2}\,(L - h_d)^2$
cfgd	$\frac{\sqrt{2}}{2}\left[\frac{3}{2} w_d + 2(L - h_d)\right]\left(h_d - L + \frac{3}{4} w_d\right)$
2 × hikc	$2\sqrt{2}\,h_d\left(L - h_d + \frac{w_d}{4}\right)$
2 × cjf	$\sqrt{2}\left(h_d - L + \frac{3}{4} w_d\right)^2$
2 × jklf	$2\sqrt{2}\left(h_d - L + \frac{3}{4} w_d\right)\left(L - \frac{3}{4} w_d\right)$

Total area: $A_p = \sqrt{2}\left[2(L - h_d)^2 + \frac{1}{2} h_d w_d + \frac{3}{2} L w_d\right]$

Design Example: Factored Resistance of a Shear Stud

Given

Find the factored resistance of a shear stud placed off-centre in ribs perpendicular to the beam, for the following configuration:

Steel deck: $h_d = 75$ mm, $w_d = 180$ mm (average rib width)

Steel stud: diameter = 19 mm, $L = 150$ mm, $F_u = 450$ MPa, $\phi_{sc} = 0.80$

Concrete slab: $f'_c = 25$ MPa, $\gamma_c = 2\,350$ kg/m^3, $\rho = 1.0$ (normal-density concrete)

Assume a spandrel beam condition with edge distance, $e = 65$ mm.

Solution

Area of concrete pull-out pyramid, S16-14 Clause 17.7.2.4:

$L = 150 > w_d/4 + h_d = 120$ mm, $3\,w_d/4 = 135 < L = 150 < 3\,w_d/4 + h_d = 210$ mm.

See Table 5-7b, case (e).

$A_p = \sqrt{2}\,[2\,(L-h_d)^2 + h_d\,w_d/2 + 3\,L\,w_d/2] = 82\,700$ mm^2 (for an interior condition)

Edge distance, $e = 65 < L - h_d = 75$ mm, case (e) (ii)

Subtract: $\sqrt{2}\,[w_d\,(L-e) + (L-h_d-e)\,(L-h_d-w_d/4)] = 22\,100$ mm^2

$A_p = 82\,700 - 22\,100 = 60\,600$ mm^2 ($\approx 60\,700$ mm^2, from Table 5-6a)

$A_{sc} = \pi(\,19/2)^2 = 283.5$ mm^2

$E_c = (3\,300\sqrt{f'_c} + 6\,900)\,(\gamma_c/2\,300)^{1.5} = 24\,200$ MPa, S16-14 Clause 3.1

Clause 17.7.2.2, $q_{rs} = 0.50\,\phi_{sc}\,A_{sc}\sqrt{f'_c\,E_c} = 88.2$ kN $< \phi_{sc}\,A_{sc}\,F_u = 102$ kN

Clause 17.7.2.4(a), $q_{rr} = 0.35\,\phi_{sc}\,\rho\,A_p\sqrt{f'_c} = 84.8$ kN $< q_{rs} = 88.2$ kN

Factored shear resistance: $q_{rr} = 84.8$ kN (≈ 84.9 kN, from Table 5-6a)

Design Example: Composite Beam

Given:

Select a simply-supported composite beam to span 12 m and carry a uniformly distributed specified live load of 18 kN/m and a dead load of 12 kN/m. Beams are spaced at 3 m on centre and support a 75 mm steel deck (ribs perpendicular to the beam) with a 65 mm cover slab of 25 MPa normal density concrete. Calculations are based on F_y = 345 MPa for ASTM A992 and A572 Grade 50 steels. Live load deflections are limited to L/300.

Solution:

Total factored load $= (1.25 \times 12) + (1.50 \times 18) = 42.0$ kN/m

Therefore $M_f = 42.0 \times 12^2 / 8 = 756$ kN·m and $V_f = 42.0 \times 12 / 2 = 252$ kN

Compute minimum I_{reqd} for deflection limit L/300 using Figure 5-2 and Table 5-8.

Total specified live load, $W = 18 \times 12 = 216$ kN

$B_d = 1.0$ simple span UDL (Table 5-8)

$C_d = 2.8 \times 10^6$ mm^4 / kN for 12 m span and $L / \Delta = 300$ (Figure 5-2)

$I_{reqd} = W \times C_d \times B_d$

$= (216 \times 2.8 \times 10^6 \times 1.0)\ 1.15 = 696 \times 10^6$ mm^4 (with 15% allowance for creep)

Effective Width (S16-14 Clause 17.4.1)

a) $0.25L = 0.25 \times 12\,000$ mm $= 3\,000$ mm

b) beam spacing $= 3$ m $= 3\,000$ mm

Therefore, effective width $= 3\,000$ mm

Beam Selection

From the Composite Beams – Trial Selection Tables for 75 mm steel deck with 65 mm cover slab and b_1 = 3 000 mm, a suitable shape is a W460x74 with M_{rc} for 40% shear connection = 783 kN·m > 756 kN·m

$V_r = 843$ kN > 252 kN

$I_t = 1\,100 \times 10^6$ mm^4

For 40% shear connection, $I_e = I_s + 0.85\,p^{0.25}\,(I_t - I_s)$ (Clause 17.3.1(a))

$= 332 + 0.85\,(0.4)^{0.25}\,(1\,100 - 332) = 851 \times 10^6$ mm^4 $> 696 \times 10^6$ mm^4

$Q_r = 2\,570$ kN ; $S_t = 2\,350 \times 10^3$ mm^3; $M_r = 512$ kN·m ; $L_u = 2\,530$ mm

Check Steel Beam Under Construction Loads (Clause 17.12)

Clause 17.12 requires that the steel section alone must be capable of supporting all factored loads applied before concrete hardens. In this case the steel deck will provide lateral support to the compression flange of the beam.

Thus $M_r = 512$ kN·m applies.

Assuming dead load due to deck-slab and steel beam as 8 kN/m and construction live load as 2.5 kN/m, the total factored load applied before the concrete hardens is

$(1.25\times 8)+(1.5\times 2.5) = 13.8$ kN/m

$M_f = 13.8\times 12^2/8 = 248$ kN·m < 512 kN·m

Check Unshored Beam Tension Flange (Clause 17.11)

Assume that the load applied before concrete strength reaches $0.75 f'_c$ is the specified dead load (8 kN/m), and that the remaining dead load ($12-8 = 4$ kN/m) and the specified live load acts on the composite section.

Stress in tension flange due to specified load acting on steel beam alone:

S_x of steel beam $= 1\,460\times 10^3$ mm^3

$$f_1 = \frac{M_1}{S_x} = \frac{8\times 12\,000^2}{8\times 1\,460\times 10^3} = 98.6 \text{ MPa}$$

Stress in tension flange due to specified live and superimposed dead loads acting on composite section:

$$f_2 = \frac{M_2}{S_t} = \frac{(18+4)\times 12\,000^2}{8\times 2\,350\times 10^3} = 169 \text{ MPa}$$

$$f_1 + f_2 = 98.6 + 169 = 268 \text{ MPa} < 345 \text{ MPa}$$

Shear Connectors

Q_r (100% connection) = 2 570 kN

Assume ¾ inch (19 mm) diameter studs, length $L = 115$ mm.

Minimum flange thickness $= 19/2.5 = 7.6$ mm < 14.5 mm (Clause 17.6.5)

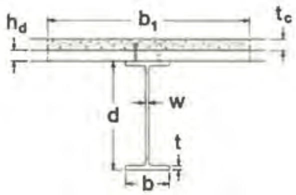
From Table 5-6a, for ¾-inch diameter studs, $h_d = 75$ mm, $w_d/h_d = 2.0$, $f'_c = 25$ MPa, $\gamma_c = 2\,350$ kg/m^3, factored shear resistance per stud, $q_{rr} = 68.7$ kN

Number of studs required:

$$= \frac{2\times Q_r \times (\%\,shear\ connection\,/\,100)}{q_{rr}} = \frac{2\times 2\,570\times(40/100)}{68.7} = 29.9 \quad \text{Use 30 studs.}$$

Since there are no concentrated loads, the studs can be spaced uniformly along the full length of the beam as permitted by the deck flutes.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 65 mm Slab
ϕ = 0.90, ϕ_c = 0.65



ASTM A992
A572 Grade 50
f'_c = 25 MPa
γ_c = 2350 kg/m^3

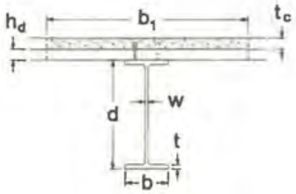
Steel section	b_1	Composite: M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite: Steel section data	Unbraced condition: L'	M$_r$'	L'	M$_r$'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W1000x249	7 000	5 430	5 200	4 670	6 010	12 000	13 800	8 990	M_r 3 510	4 000	3 440	14 000	831
W40x167	5 000	5 180	4 860	4 370	4 290	11 100	13 500	8 140	V_r 3 220	6 000	2 780	16 000	694
b = 300	3 000	4 690	4 390	4 020	2 570	9 640	12 900	7 080	L_u 3 740	8 000	1 940	18 000	596
t = 26	1 000	3 910	3 770	3 620	858	7 060	11 600	5 680	I_x 4 810	10 000	1 360	20 000	523
d = 980									S_x 9 820	12 000	1 030	22 000	466
W1000x222	7 000	4 880	4 680	4 170	6 010	10 700	12 200	8 010	M_r 3 040	4 000	2 940	14 000	634
W40x149	5 000	4 650	4 360	3 870	4 290	9 870	11 900	7 250	V_r 3 000	6 000	2 310	16 000	527
b = 300	3 000	4 190	3 900	3 530	2 570	8 600	11 400	6 260	L_u 3 590	8 000	1 520	18 000	451
t = 21.1	1 000	3 420	3 290	3 140	858	6 240	10 200	4 930	I_x 4 080	10 000	1 050	20 000	394
d = 970									S_x 8 410	12 000	794	22 000	350
W920x238	7 000	4 920	4 720	4 280	6 010	10 300	12 600	7 730	M_r 3 170	4 000	3 140	14 000	800
W36x160	5 000	4 700	4 450	4 010	4 290	9 540	12 300	7 000	V_r 3 090	6 000	2 590	16 000	668
b = 305	3 000	4 290	4 030	3 690	2 570	8 300	11 800	6 070	L_u 3 890	8 000	1 870	18 000	573
t = 25.9	1 000	3 590	3 470	3 330	858	6 050	10 600	4 840	I_x 4 060	10 000	1 310	20 000	502
d = 915									S_x 8 870	12 000	996	22 000	447
W920x223	7 000	4 660	4 470	4 050	6 010	9 730	11 800	7 320	M_r 2 960	4 500	2 800	12 000	881
W36x150	5 000	4 440	4 210	3 790	4 290	9 010	11 500	6 630	V_r 2 970	5 000	2 670	14 000	705
b = 304	3 000	4 060	3 800	3 480	2 570	7 860	11 100	5 730	L_u 3 830	6 000	2 380	16 000	587
t = 23.9	1 000	3 370	3 250	3 120	858	5 710	9 900	4 530	I_x 3 760	8 000	1 680	18 000	502
d = 911									S_x 8 260	10 000	1 170	20 000	439
W920x201	7 000	4 230	4 050	3 660	6 010	8 730	10 500	6 610	M_r 2 590	4 500	2 420	12 000	705
W36x135	5 000	4 020	3 810	3 410	4 290	8 110	10 300	5 980	V_r 2 710	5 000	2 300	14 000	560
b = 304	3 000	3 660	3 420	3 100	2 570	7 090	9 860	5 150	L_u 3 720	6 000	2 030	16 000	463
t = 20.1	1 000	3 000	2 880	2 750	858	5 130	8 810	4 000	I_x 3 250	8 000	1 360	18 000	394
d = 903									S_x 7 190	10 000	940	20 000	343
W840x210	7 000	4 160	3 970	3 620	6 010	8 210	10 500	6 200	M_r 2 620	4 500	2 460	12 000	792
W33x141	5 000	3 940	3 760	3 390	4 290	7 610	10 300	5 610	V_r 2 670	5 000	2 350	14 000	639
b = 293	3 000	3 630	3 400	3 100	2 570	6 650	9 910	4 840	L_u 3 770	6 000	2 090	16 000	535
t = 24.4	1 000	3 010	2 890	2 770	858	4 820	8 880	3 790	I_x 3 110	8 000	1 470	18 000	461
d = 846									S_x 7 340	10 000	1 040	20 000	404
W840x193	7 000	3 850	3 680	3 350	6 010	7 540	9 610	5 730	M_r 2 370	4 500	2 200	12 000	666
W33x130	5 000	3 650	3 480	3 130	4 290	7 010	9 420	5 180	V_r 2 530	5 000	2 090	14 000	534
b = 292	3 000	3 350	3 140	2 850	2 570	6 140	9 060	4 450	L_u 3 690	6 000	1 850	16 000	445
t = 21.7	1 000	2 750	2 640	2 520	858	4 440	8 110	3 450	I_x 2 780	8 000	1 260	18 000	382
d = 840									S_x 6 630	10 000	877	20 000	334
W840x176	7 000	3 550	3 380	3 080	6 010	6 870	8 690	5 260	M_r 2 110	4 500	1 950	12 000	551
W33x118	5 000	3 350	3 200	2 860	4 290	6 410	8 520	4 760	V_r 2 300	5 000	1 840	14 000	439
b = 292	3 000	3 080	2 880	2 590	2 570	5 630	8 210	4 080	L_u 3 610	6 000	1 610	16 000	364
t = 18.8	1 000	2 500	2 390	2 270	858	4 070	7 340	3 110	I_x 2 460	8 000	1 060	18 000	311
d = 835									S_x 5 900	10 000	731	20 000	271

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 65 mm Slab
ϕ = 0.90, ϕ_c = 0.65



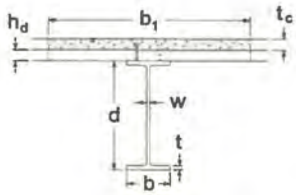
ASTM A992
A572 Grade 50
f'_c = 25 MPa
γ_c = 2350 kg/m^3

Steel section	b_1	Composite: M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite: Steel section data	Unbraced condition: L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W760x185	5 000	3 230	3 090	2 780	4 290	5 790	8 390	4 270	M_r 2 080	4 000	1 980	12 000	576
W30x124	4 000	3 120	2 960	2 660	3 430	5 480	8 260	3 990	V_r 2 340	5 000	1 780	14 000	470
b = 267	3 000	2 970	2 790	2 530	2 570	5 070	8 070	3 660	L_u 3 450	6 000	1 550	16 000	397
t = 23.6	2 000	2 750	2 580	2 380	1 720	4 500	7 770	3 270	I_x 2 230	8 000	1 040	18 000	344
d = 766	1 000	2 440	2 340	2 220	858	3 650	7 220	2 800	S_x 5 820	10 000	743	20 000	304
W760x173	5 000	3 060	2 920	2 620	4 290	5 450	7 860	4 040	M_r 1 930	4 000	1 830	12 000	506
W30x116	4 000	2 950	2 800	2 510	3 430	5 170	7 740	3 770	V_r 2 250	5 000	1 630	14 000	411
b = 267	3 000	2 810	2 630	2 380	2 570	4 790	7 570	3 460	L_u 3 410	6 000	1 410	16 000	346
t = 21.6	2 000	2 590	2 430	2 230	1 720	4 260	7 290	3 080	I_x 2 060	8 000	924	18 000	299
d = 762	1 000	2 290	2 190	2 070	858	3 450	6 770	2 620	S_x 5 400	10 000	657	20 000	264
W760x161	5 000	2 850	2 720	2 440	4 290	5 050	7 230	3 760	M_r 1 760	4 000	1 650	12 000	429
W30x108	4 000	2 750	2 610	2 330	3 430	4 800	7 130	3 510	V_r 2 140	5 000	1 460	14 000	347
b = 266	3 000	2 620	2 450	2 200	2 570	4 460	6 970	3 210	L_u 3 330	6 000	1 250	16 000	291
t = 19.3	2 000	2 410	2 250	2 060	1 720	3 970	6 720	2 860	I_x 1 860	8 000	793	18 000	251
d = 758	1 000	2 110	2 010	1 900	858	3 210	6 240	2 410	S_x 4 900	10 000	560	20 000	220
W760x147	5 000	2 640	2 520	2 260	4 290	4 650	6 600	3 480	M_r 1 580	4 000	1 470	12 000	358
W30x99	4 000	2 540	2 410	2 150	3 430	4 420	6 500	3 250	V_r 2 040	5 000	1 290	14 000	288
b = 265	3 000	2 420	2 270	2 020	2 570	4 110	6 370	2 970	L_u 3 260	6 000	1 090	16 000	241
t = 17	2 000	2 230	2 070	1 880	1 720	3 670	6 140	2 630	I_x 1 660	8 000	671	18 000	207
d = 753	1 000	1 940	1 840	1 730	858	2 960	5 700	2 200	S_x 4 410	10 000	470	20 000	181
W760x134	5 000	2 430	2 310	2 090	4 290	4 270	6 000	3 230	M_r 1 440	4 000	1 330	12 000	308
W30x90	4 000	2 340	2 230	1 990	3 430	4 080	5 920	3 020	V_r 1 650	5 000	1 160	14 000	246
b = 264	3 000	2 230	2 100	1 870	2 570	3 810	5 810	2 760	L_u 3 230	6 000	967	16 000	205
t = 15.5	2 000	2 060	1 920	1 730	1 720	3 400	5 610	2 440	I_x 1 500	8 000	587	18 000	175
d = 750	1 000	1 780	1 690	1 580	858	2 750	5 220	2 030	S_x 4 010	10 000	408	20 000	153
W690x192	5 000	3 080	2 940	2 660	4 290	5 160	8 130	3 790	M_r 2 010	4 000	1 910	12 000	634
W27x129	4 000	2 970	2 830	2 550	3 430	4 880	8 010	3 530	V_r 2 230	5 000	1 730	14 000	525
b = 254	3 000	2 840	2 670	2 430	2 570	4 510	7 820	3 240	L_u 3 440	6 000	1 540	16 000	449
t = 27.9	2 000	2 630	2 480	2 290	1 720	3 990	7 530	2 900	I_x 1 980	8 000	1 090	18 000	392
d = 702	1 000	2 350	2 250	2 150	858	3 230	6 980	2 480	S_x 5 640	10 000	802	20 000	349
W690x170	5 000	2 770	2 640	2 390	4 290	4 590	7 180	3 400	M_r 1 750	4 000	1 650	12 000	497
W27x114	4 000	2 660	2 530	2 280	3 430	4 350	7 070	3 170	V_r 2 060	5 000	1 480	14 000	408
b = 256	3 000	2 540	2 390	2 160	2 570	4 030	6 920	2 900	L_u 3 380	6 000	1 290	16 000	347
t = 23.6	2 000	2 350	2 210	2 030	1 720	3 580	6 660	2 580	I_x 1 700	8 000	875	18 000	302
d = 693	1 000	2 080	1 990	1 880	858	2 890	6 180	2 190	S_x 4 900	10 000	634	20 000	268
W690x152	5 000	2 520	2 390	2 170	4 290	4 170	6 450	3 120	M_r 1 550	4 000	1 460	12 000	406
W27x102	4 000	2 420	2 300	2 070	3 430	3 960	6 360	2 910	V_r 1 850	5 000	1 290	14 000	332
b = 254	3 000	2 310	2 180	1 960	2 570	3 690	6 230	2 660	L_u 3 320	6 000	1 110	16 000	281
t = 21.1	2 000	2 140	2 010	1 830	1 720	3 280	6 010	2 360	I_x 1 510	8 000	728	18 000	244
d = 688	1 000	1 880	1 790	1 690	858	2 650	5 590	1 990	S_x 4 380	10 000	523	20 000	216

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm $\qquad$ F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 65 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



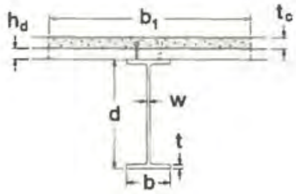
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		L' mm	M_r' kN·m	L' mm	M_r' kN·m
W690x140	5 000	2 350	2 220	2 020	4 290	3 850	5 910	2 900	M_r 1 410	4 000	1 320	10 000	447
W27x94	4 000	2 250	2 140	1 930	3 430	3 670	5 830	2 700	V_r 1 740	5 000	1 160	12 000	345
b = 254	3 000	2 140	2 020	1 820	2 570	3 420	5 710	2 470	L_u 3 270	6 000	987	14 000	280
t = 18.9	2 000	1 990	1 860	1 690	1 720	3 050	5 520	2 190	I_x 1 360	7 000	778	16 000	236
d = 684	1 000	1 740	1 650	1 550	858	2 460	5 140	1 830	S_x 3 980	8 000	628	18 000	204
W690x125	5 000	2 140	2 020	1 840	4 290	3 460	5 270	2 630	M_r 1 250	4 000	1 140	10 000	362
W27x84	4 000	2 040	1 940	1 750	3 430	3 300	5 200	2 450	V_r 1 610	5 000	999	12 000	277
b = 253	3 000	1 940	1 840	1 640	2 570	3 090	5 100	2 240	L_u 3 190	6 000	834	14 000	223
t = 16.3	2 000	1 800	1 680	1 520	1 720	2 770	4 940	1 970	I_x 1 180	7 000	640	16 000	187
d = 678	1 000	1 560	1 480	1 380	858	2 230	4 590	1 630	S_x 3 500	8 000	513	18 000	161
W610x174	5 000	2 570	2 440	2 240	4 290	3 930	6 860	2 900	M_r 1 660	4 500	1 660	10 000	924
W24x117	4 000	2 460	2 350	2 150	3 430	3 730	6 760	2 710	V_r 1 770	5 000	1 610	12 000	709
b = 325	3 000	2 350	2 240	2 040	2 570	3 450	6 610	2 480	L_u 4 480	6 000	1 490	14 000	574
t = 21.6	2 000	2 200	2 090	1 930	1 720	3 060	6 380	2 210	I_x 1 470	7 000	1 370	16 000	482
d = 616	1 000	1 970	1 890	1 800	858	2 470	5 940	1 880	S_x 4 780	8 000	1 230	18 000	415
W610x155	5 000	2 320	2 200	2 020	4 290	3 540	6 110	2 640	M_r 1 470	4 500	1 460	10 000	762
W24x104	4 000	2 220	2 120	1 940	3 430	3 370	6 020	2 470	V_r 1 590	5 000	1 410	12 000	579
b = 324	3 000	2 120	2 020	1 840	2 570	3 130	5 900	2 260	L_u 4 400	6 000	1 300	14 000	465
t = 19	2 000	1 990	1 880	1 730	1 720	2 790	5 700	2 000	I_x 1 290	7 000	1 180	16 000	388
d = 611	1 000	1 770	1 690	1 600	858	2 250	5 320	1 690	S_x 4 220	8 000	1 050	18 000	333
W610x140	5 000	2 170	2 040	1 850	4 290	3 260	5 480	2 440	M_r 1 290	4 000	1 170	10 000	422
W24x94	4 000	2 060	1 960	1 770	3 430	3 100	5 400	2 270	V_r 1 660	5 000	1 030	12 000	334
b = 230	3 000	1 960	1 850	1 670	2 570	2 890	5 290	2 070	L_u 3 070	6 000	874	14 000	277
t = 22.2	2 000	1 820	1 710	1 550	1 720	2 570	5 110	1 830	I_x 1 120	7 000	695	16 000	237
d = 617	1 000	1 590	1 510	1 420	858	2 070	4 740	1 520	S_x 3 630	8 000	573	18 000	207
W610x125	5 000	1 970	1 850	1 680	4 290	2 940	4 890	2 220	M_r 1 140	4 000	1 020	10 000	342
W24x84	4 000	1 870	1 770	1 600	3 430	2 800	4 830	2 080	V_r 1 490	5 000	889	12 000	269
b = 229	3 000	1 770	1 680	1 510	2 570	2 620	4 740	1 890	L_u 3 020	6 000	733	14 000	222
t = 19.6	2 000	1 650	1 550	1 400	1 720	2 340	4 580	1 670	I_x 985	7 000	575	16 000	189
d = 612	1 000	1 440	1 360	1 270	858	1 890	4 260	1 370	S_x 3 220	8 000	470	18 000	165
W610x113	5 000	1 830	1 710	1 550	4 290	2 680	4 430	2 050	M_r 1 020	4 000	906	10 000	282
W24x76	4 000	1 730	1 630	1 470	3 430	2 560	4 370	1 910	V_r 1 400	5 000	775	12 000	220
b = 228	3 000	1 630	1 550	1 380	2 570	2 400	4 290	1 740	L_u 2 950	6 000	617	14 000	180
t = 17.3	2 000	1 510	1 420	1 280	1 720	2 150	4 160	1 530	I_x 875	7 000	481	16 000	153
d = 608	1 000	1 310	1 240	1 150	858	1 740	3 870	1 250	S_x 2 880	8 000	391	18 000	133
W610x101	5 000	1 650	1 540	1 390	4 020	2 410	3 950	1 860	M_r 900	4 000	787	10 000	228
W24x68	4 000	1 580	1 480	1 340	3 430	2 310	3 910	1 740	V_r 1 300	5 000	664	12 000	176
b = 228	3 000	1 480	1 410	1 250	2 570	2 170	3 840	1 590	L_u 2 890	6 000	512	14 000	144
t = 14.9	2 000	1 380	1 290	1 150	1 720	1 960	3 720	1 390	I_x 764	7 000	396	16 000	121
d = 603	1 000	1 190	1 120	1 030	858	1 580	3 480	1 130	S_x 2 530	8 000	320	18 000	105

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm $F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 65 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

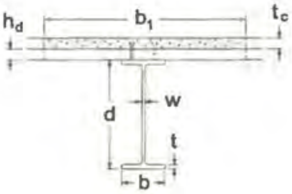
Steel section	b_1	Composite: M_{rc} (kN·m) for % shear connection			Composite: Q_r (kN)	Composite: I_t 10^6	Composite: S_t 10^3	Composite: I_{ts} 10^6	Non-composite: Steel section data	Non-composite: Unbraced condition L'	Unbraced condition M_r'	Unbraced condition L'	Unbraced condition M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W610x92	4 000	1 470	1 380	1 230	3 430	2 090	3 480	1 580	M_r 779	3 000	683	8 000	183
W24x62	3 000	1 370	1 290	1 140	2 570	1 970	3 420	1 440	V_r 1 350	4 000	540	10 000	135
b = 179	2 000	1 260	1 170	1 030	1 720	1 780	3 320	1 260	L_u 2 180	5 000	376	12 000	107
t = 15	1 000	1 070	997	908	858	1 440	3 090	1 010	I_x 646	6 000	281	14 000	88.9
d = 603	500	933	889	840	429	1 140	2 820	845	S_x 2 140	7 000	222	16 000	76.1
W610x82	4 000	1 320	1 230	1 100	3 240	1 870	3 090	1 430	M_r 683	3 000	587	8 000	145
W24x55	3 000	1 240	1 170	1 030	2 570	1 770	3 040	1 310	V_r 1 170	4 000	448	10 000	106
b = 178	2 000	1 140	1 060	929	1 720	1 600	2 950	1 140	L_u 2 110	5 000	304	12 000	83.4
t = 12.8	1 000	966	895	809	858	1 300	2 750	910	I_x 560	6 000	225	14 000	68.9
d = 599	500	833	790	742	429	1 030	2 520	755	S_x 1 870	7 000	177	16 000	58.7
W530x138	4 000	1 850	1 750	1 570	3 430	2 520	4 850	1 830	M_r 1 120	3 000	1 110	8 000	515
W21x93	3 000	1 750	1 650	1 470	2 570	2 340	4 750	1 660	V_r 1 650	4 000	1 000	10 000	390
b = 214	2 000	1 610	1 510	1 360	1 720	2 080	4 570	1 460	L_u 2 930	5 000	884	12 000	314
t = 23.6	1 000	1 400	1 330	1 240	858	1 660	4 220	1 200	I_x 861	6 000	759	14 000	263
d = 549	500	1 270	1 230	1 180	429	1 330	3 870	1 040	S_x 3 140	7 000	616	16 000	227
W530x123	4 000	1 690	1 580	1 420	3 430	2 280	4 340	1 670	M_r 997	3 000	984	8 000	421
W21x83	3 000	1 580	1 500	1 340	2 570	2 130	4 250	1 520	V_r 1 460	4 000	879	10 000	316
b = 212	2 000	1 460	1 370	1 230	1 720	1 900	4 110	1 330	L_u 2 860	5 000	762	12 000	253
t = 21.2	1 000	1 270	1 200	1 120	858	1 520	3 810	1 090	I_x 761	6 000	631	14 000	211
d = 544	500	1 140	1 100	1 050	429	1 210	3 490	937	S_x 2 800	7 000	505	16 000	182
W530x109	4 000	1 530	1 430	1 290	3 430	2 050	3 860	1 520	M_r 879	3 000	862	8 000	342
W21x73	3 000	1 430	1 350	1 210	2 570	1 920	3 780	1 390	V_r 1 280	4 000	764	10 000	254
b = 211	2 000	1 320	1 240	1 110	1 720	1 720	3 660	1 210	L_u 2 810	5 000	652	12 000	202
t = 18.8	1 000	1 140	1 080	997	858	1 380	3 410	984	I_x 667	6 000	520	14 000	168
d = 539	500	1 020	979	934	429	1 100	3 130	839	S_x 2 480	7 000	413	16 000	144
W530x101	4 000	1 440	1 350	1 220	3 430	1 920	3 590	1 440	M_r 814	3 000	794	8 000	301
W21x68	3 000	1 350	1 270	1 140	2 570	1 810	3 530	1 310	V_r 1 200	4 000	699	10 000	222
b = 210	2 000	1 240	1 170	1 040	1 720	1 620	3 420	1 150	L_u 2 770	5 000	591	12 000	176
t = 17.4	1 000	1 080	1 010	932	858	1 310	3 190	928	I_x 617	6 000	462	14 000	146
d = 537	500	953	914	870	429	1 040	2 930	787	S_x 2 300	7 000	365	16 000	125
W530x92	4 000	1 340	1 250	1 120	3 430	1 760	3 270	1 340	M_r 733	3 000	711	8 000	253
W21x62	3 000	1 250	1 170	1 050	2 570	1 660	3 220	1 220	V_r 1 110	4 000	621	9 000	214
b = 209	2 000	1 140	1 080	961	1 720	1 500	3 120	1 060	L_u 2 720	5 000	516	10 000	185
t = 15.6	1 000	992	929	851	858	1 210	2 920	855	I_x 552	6 000	393	12 000	146
d = 533	500	872	833	789	429	965	2 680	719	S_x 2 070	7 000	309	14 000	120
W530x82	4 000	1 210	1 120	1 000	3 250	1 570	2 900	1 210	M_r 640	3 000	616	8 000	203
W21x55	3 000	1 130	1 060	950	2 570	1 490	2 850	1 100	V_r 1 030	4 000	531	9 000	170
b = 209	2 000	1 030	974	863	1 720	1 350	2 770	964	L_u 2 660	5 000	433	10 000	147
t = 13.3	1 000	892	832	756	858	1 100	2 600	769	I_x 477	6 000	320	12 000	115
d = 528	500	776	739	695	429	872	2 390	639	S_x 1 810	7 000	249	14 000	94.0

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

$F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 65 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

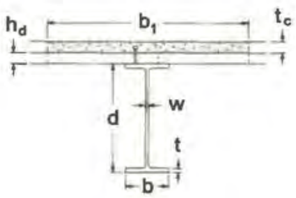
Steel section	b_1	Composite: M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite: Steel section data	Unbraced condition: L′	M_r'	L′	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W530x74	4 000	1 110	1 030	908	2 960	1 430	2 610	1 110	M_r 562	3 000	474	8 000	123
W21x50	3 000	1 060	987	873	2 570	1 360	2 560	1 010	V_r 1 050	4 000	357	9 000	105
b = 166	2 000	959	898	785	1 720	1 240	2 490	883	L_u 2 040	5 000	247	10 000	91.7
t = 13.6	1 000	815	754	678	858	1 010	2 330	698	I_x 411	6 000	186	12 000	73.2
d = 529	500	698	660	617	429	797	2 140	572	S_x 1 550	7 000	148	14 000	61.0
W530x66	4 000	982	903	793	2 600	1 270	2 280	994	M_r 484	3 000	398	8 000	94.9
W21x44	3 000	959	890	787	2 570	1 200	2 250	911	V_r 927	4 000	284	9 000	80.6
b = 165	2 000	863	809	704	1 720	1 100	2 190	796	L_u 1 980	5 000	195	10 000	70.0
t = 11.4	1 000	732	674	600	858	908	2 060	627	I_x 351	6 000	145	12 000	55.5
d = 525	500	619	582	540	429	718	1 890	508	S_x 1 340	7 000	115	14 000	46.0
W460x158	4 000	1 830	1 720	1 570	3 430	2 270	5 060	1 630	M_r 1 170	4 500	1 150	9 000	794
W18x106	3 000	1 720	1 640	1 490	2 570	2 100	4 940	1 480	V_r 1 460	5 000	1 110	10 000	696
b = 284	2 000	1 610	1 520	1 390	1 720	1 850	4 760	1 300	L_u 4 200	6 000	1 040	11 000	617
t = 23.9	1 000	1 430	1 360	1 290	858	1 470	4 390	1 080	I_x 796	7 000	955	12 000	555
d = 476	500	1 310	1 270	1 230	429	1 190	4 040	945	S_x 3 350	8 000	875	14 000	462
W460x144	4 000	1 700	1 600	1 460	3 430	2 110	4 660	1 520	M_r 1 070	4 500	1 050	9 000	693
W18x97	3 000	1 600	1 520	1 380	2 570	1 950	4 560	1 380	V_r 1 320	5 000	1 010	10 000	602
b = 283	2 000	1 490	1 410	1 290	1 720	1 730	4 390	1 210	L_u 4 130	6 000	936	11 000	533
t = 22.1	1 000	1 320	1 260	1 190	858	1 380	4 070	1 000	I_x 726	7 000	858	12 000	478
d = 472	500	1 200	1 170	1 130	429	1 110	3 740	872	S_x 3 080	8 000	779	14 000	396
W460x128	4 000	1 550	1 450	1 320	3 430	1 900	4 150	1 390	M_r 947	4 500	917	9 000	566
W18x86	3 000	1 440	1 370	1 250	2 570	1 770	4 070	1 260	V_r 1 170	5 000	884	10 000	489
b = 282	2 000	1 340	1 270	1 160	1 720	1 570	3 930	1 100	L_u 4 040	6 000	812	11 000	431
t = 19.6	1 000	1 190	1 130	1 060	858	1 260	3 650	903	I_x 637	7 000	736	12 000	385
d = 467	500	1 080	1 040	1 000	429	1 010	3 360	780	S_x 2 730	8 000	658	14 000	318
W460x113	4 000	1 400	1 300	1 180	3 430	1 700	3 670	1 260	M_r 829	4 500	796	9 000	458
W18x76	3 000	1 300	1 230	1 120	2 570	1 590	3 600	1 150	V_r 1 020	5 000	765	10 000	394
b = 280	2 000	1 200	1 140	1 040	1 720	1 420	3 490	1 000	L_u 3 950	6 000	696	11 000	345
t = 17.3	1 000	1 060	1 010	940	858	1 140	3 250	814	I_x 556	7 000	623	12 000	307
d = 463	500	958	923	884	429	912	3 000	696	S_x 2 400	8 000	545	14 000	252
W460x106	4 000	1 350	1 250	1 120	3 430	1 600	3 380	1 180	M_r 742	3 000	719	8 000	308
W18x71	3 000	1 250	1 170	1 040	2 570	1 500	3 310	1 070	V_r 1 210	4 000	637	9 000	266
b = 194	2 000	1 140	1 070	956	1 720	1 340	3 200	934	L_u 2 690	5 000	549	10 000	235
t = 20.6	1 000	985	926	853	858	1 070	2 970	747	I_x 488	6 000	450	11 000	210
d = 469	500	872	836	796	429	845	2 710	629	S_x 2 080	7 000	366	12 000	190
W460x97	4 000	1 260	1 160	1 040	3 430	1 480	3 100	1 110	M_r 677	3 000	652	8 000	264
W18x65	3 000	1 160	1 080	971	2 570	1 390	3 040	1 010	V_r 1 090	4 000	574	9 000	227
b = 193	2 000	1 060	994	887	1 720	1 250	2 950	876	L_u 2 650	5 000	488	10 000	200
t = 19	1 000	914	858	788	858	1 000	2 740	698	I_x 445	6 000	389	11 000	178
d = 466	500	806	771	731	429	792	2 510	584	S_x 1 910	7 000	314	12 000	161

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

$F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 65 mm Slab
$\phi = 0.90, \phi_c = 0.65$



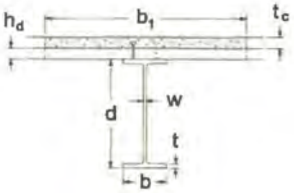
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W460x89	3 000	1 090	1 020	911	2 570	1 300	2 820	949	M_r 624	3 000	598	8 000	231
W18x60	2 000	988	932	832	1 720	1 170	2 740	826	V_r 996	4 000	523	9 000	198
b = 192	1 500	933	875	785	1 290	1 080	2 670	748	L_u 2 620	5 000	439	10 000	174
t = 17.7	1 000	857	803	734	858	946	2 560	656	I_x 409	6 000	343	11 000	155
d = 463	500	752	718	679	429	746	2 340	546	S_x 1 770	7 000	276	12 000	140
W460x82	3 000	1 020	948	848	2 570	1 200	2 590	886	M_r 568	3 000	540	8 000	195
W18x55	2 000	920	867	772	1 720	1 090	2 520	772	V_r 933	4 000	466	9 000	166
b = 191	1 500	867	813	727	1 290	1 000	2 460	699	L_u 2 560	5 000	384	10 000	146
t = 16	1 000	795	744	676	858	883	2 360	611	I_x 370	6 000	292	11 000	129
d = 460	500	694	660	622	429	696	2 160	504	S_x 1 610	7 000	234	12 000	116
W460x74	3 000	947	876	783	2 570	1 100	2 350	822	M_r 512	3 000	484	8 000	164
W18x50	2 000	849	798	712	1 720	1 000	2 290	717	V_r 843	4 000	414	9 000	140
b = 190	1 500	797	750	668	1 290	928	2 240	649	L_u 2 530	5 000	332	10 000	122
t = 14.5	1 000	733	685	620	858	819	2 150	566	I_x 332	6 000	249	11 000	108
d = 457	500	636	604	566	429	647	1 980	463	S_x 1 460	7 000	198	12 000	96.8
W460x68	3 000	898	829	734	2 570	1 030	2 160	772	M_r 463	3 000	390	8 000	112
W18x46	2 000	801	751	662	1 720	940	2 110	673	V_r 856	4 000	301	9 000	96.7
b = 154	1 500	750	701	619	1 290	870	2 060	608	L_u 2 010	5 000	213	10 000	85.2
t = 15.4	1 000	684	635	570	858	769	1 980	528	I_x 297	6 000	164	11 000	76.1
d = 459	500	586	554	516	429	606	1 810	428	S_x 1 290	7 000	133	12 000	68.9
W460x60	3 000	795	729	644	2 350	907	1 890	693	M_r 397	3 000	329	8 000	86.6
W18x40	2 000	718	670	593	1 720	834	1 840	606	V_r 746	4 000	242	9 000	74.3
b = 153	1 500	668	627	552	1 290	777	1 800	549	L_u 1 970	5 000	169	10 000	65.2
t = 13.3	1 000	611	567	505	858	691	1 740	476	I_x 255	6 000	129	11 000	58.1
d = 455	500	521	489	452	429	547	1 600	382	S_x 1 120	7 000	104	12 000	52.4
W460x52	3 000	697	636	557	2 060	792	1 630	614	M_r 338	3 000	269	8 000	63.6
W18x35	2 000	647	600	528	1 720	733	1 600	539	V_r 680	4 000	185	9 000	54.3
b = 152	1 500	598	560	488	1 290	685	1 560	488	L_u 1 890	5 000	128	10 000	47.4
t = 10.8	1 000	544	503	442	858	612	1 510	422	I_x 212	6 000	96.2	11 000	42.0
d = 450	500	458	427	390	429	486	1 390	334	S_x 942	7 000	76.7	12 000	37.8
W410x149	3 000	1 520	1 430	1 300	2 570	1 720	4 370	1 200	M_r 1 010	4 500	983	8 000	760
W16x100	2 000	1 400	1 330	1 210	1 720	1 520	4 210	1 050	V_r 1 320	5 000	952	9 000	696
b = 265	1 500	1 330	1 260	1 170	1 290	1 380	4 070	962	L_u 4 080	5 500	921	10 000	621
t = 25	1 000	1 240	1 190	1 120	858	1 200	3 870	861	I_x 618	6 000	889	11 000	554
d = 431	500	1 130	1 100	1 060	429	959	3 540	747	S_x 2 870	7 000	825	12 000	501
W410x132	3 000	1 370	1 290	1 170	2 570	1 550	3 890	1 090	M_r 885	4 500	853	8 000	635
W16x89	2 000	1 260	1 190	1 090	1 720	1 380	3 750	952	V_r 1 160	5 000	823	9 000	565
b = 263	1 500	1 200	1 130	1 040	1 290	1 250	3 640	868	L_u 3 940	5 500	792	10 000	495
t = 22.2	1 000	1 110	1 060	990	858	1 090	3 460	773	I_x 538	6 000	761	11 000	440
d = 425	500	1 010	974	937	429	865	3 170	664	S_x 2 530	7 000	698	12 000	397

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 65 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

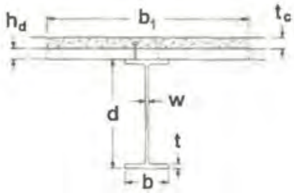
Steel section	b_1	Composite M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite Steel section data	Unbraced condition L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W410x114	3 000	1 220	1 140	1 030	2 570	1 380	3 390	982	M_r 764	4 500	726	8 000	513
W16x77	2 000	1 110	1 060	959	1 720	1 230	3 280	854	V_r 998	5 000	698	9 000	438
b = 261	1 500	1 060	1 000	915	1 290	1 120	3 190	777	L_u 3 810	5 500	668	10 000	382
t = 19.3	1 000	982	931	866	858	978	3 050	688	I_x 461	6 000	638	11 000	338
d = 420	500	883	851	814	429	774	2 790	584	S_x 2 200	7 000	576	12 000	304
W410x100	3 000	1 090	1 010	919	2 570	1 220	2 970	885	M_r 661	4 500	623	8 000	411
W16x67	2 000	987	935	851	1 720	1 100	2 880	770	V_r 850	5 000	596	9 000	348
b = 260	1 500	934	887	809	1 290	1 010	2 810	699	L_u 3 730	5 500	568	10 000	302
t = 16.9	1 000	871	824	763	858	882	2 690	616	I_x 398	6 000	539	11 000	266
d = 415	500	778	747	712	429	697	2 470	517	S_x 1 920	7 000	479	12 000	238
W410x85	3 000	975	901	802	2 570	1 060	2 500	775	M_r 534	3 000	507	8 000	205
W16x57	2 000	873	820	729	1 720	960	2 430	672	V_r 931	4 000	443	9 000	177
b = 181	1 500	820	768	686	1 290	882	2 360	606	L_u 2 530	5 000	375	10 000	157
t = 18.2	1 000	751	702	638	858	772	2 260	528	I_x 315	6 000	297	11 000	140
d = 417	500	654	623	587	429	604	2 060	433	S_x 1 510	7 000	242	12 000	127
W410x74	3 000	888	817	724	2 570	953	2 220	705	M_r 469	3 000	440	8 000	163
W16x50	2 000	789	739	657	1 720	866	2 150	612	V_r 821	4 000	379	9 000	140
b = 180	1 500	738	693	616	1 290	799	2 100	553	L_u 2 470	5 000	312	10 000	123
t = 16	1 000	676	631	570	858	703	2 020	480	I_x 275	6 000	239	11 000	110
d = 413	500	586	555	520	429	551	1 850	390	S_x 1 330	7 000	194	12 000	99.7
W410x67	3 000	824	755	666	2 570	868	2 000	651	M_r 422	3 000	392	8 000	135
W16x45	2 000	728	678	604	1 720	793	1 950	567	V_r 739	4 000	333	9 000	116
b = 179	1 500	677	636	565	1 290	734	1 900	511	L_u 2 420	5 000	264	10 000	102
t = 14.4	1 000	621	579	521	858	649	1 830	443	I_x 245	6 000	201	11 000	90.4
d = 410	500	535	506	471	429	510	1 680	357	S_x 1 200	7 000	161	12 000	81.6
W410x60	3 000	738	673	591	2 350	779	1 780	594	M_r 369	3 000	341	8 000	109
W16x40	2 000	662	614	547	1 720	716	1 730	519	V_r 642	4 000	286	9 000	93.1
b = 178	1 500	612	575	511	1 290	666	1 700	469	L_u 2 390	5 000	218	10 000	81.3
t = 12.8	1 000	560	524	469	858	592	1 640	407	I_x 216	6 000	165	11 000	72.1
d = 407	500	482	454	420	429	468	1 510	325	S_x 1 060	7 000	131	12 000	64.9
W410x54	3 000	665	603	527	2 110	698	1 580	538	M_r 326	3 000	295	8 000	86.0
W16x36	2 000	609	563	498	1 720	645	1 550	472	V_r 619	4 000	241	9 000	73.1
b = 177	1 500	561	525	463	1 290	602	1 510	427	L_u 2 310	5 000	176	10 000	63.5
t = 10.9	1 000	510	476	421	858	537	1 460	369	I_x 186	6 000	132	11 000	56.2
d = 403	500	435	407	374	429	425	1 350	292	S_x 923	7 000	104	12 000	50.4
W410x46	3 000	582	525	455	1 830	614	1 370	481	M_r 274	2 000	265	7 000	61.7
W16x31	2 000	553	507	445	1 720	570	1 340	424	V_r 578	3 000	210	8 000	51.8
b = 140	1 500	505	469	411	1 290	535	1 310	384	L_u 1 790	4 000	142	9 000	44.6
t = 11.2	1 000	455	423	371	858	480	1 270	331	I_x 156	5 000	99.9	10 000	39.2
d = 403	500	383	357	323	429	382	1 180	260	S_x 772	6 000	76.4	11 000	35.0

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 65 mm Slab
$\phi = 0.90, \phi_c = 0.65$



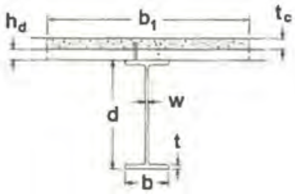
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350\ kg/m^3$

Steel section	b_1 (mm)	Composite: M_{rc} (kN·m) for % shear connection 100%	70%	40%	Q_r (kN) 100%	I_t 10^6 mm^4	S_t 10^3 mm^3	I_{ts} 10^6 mm^4	Non-composite: Steel section data	Unbraced condition: L' mm	M_r' kN·m	L' mm	M_r' kN·m
W410x39	3 000	496	445	383	1 550	522	1 150	417	M_r 227	2 000	216	7 000	44.0
W16x26	2 000	481	437	380	1 550	488	1 130	370	V_r 480	3 000	166	8 000	36.5
b = 140	1 500	447	412	360	1 290	461	1 110	336	L_u 1 730	4 000	105	9 000	31.2
t = 8.8	1 000	399	370	321	858	416	1 080	290	I_x 126	5 000	73.0	10 000	27.3
d = 399	500	333	307	275	429	335	999	225	S_x 634	6 000	55.1	11 000	24.2
W360x79	3 000	830	759	668	2 570	806	2 140	588	M_r 444	3 500	425	7 000	267
W14x53	2 000	731	681	611	1 720	729	2 080	509	V_r 682	4 000	404	8 000	225
b = 205	1 500	680	640	574	1 290	670	2 030	458	L_u 3 010	4 500	383	9 000	194
t = 16.8	1 000	625	587	533	858	586	1 940	396	I_x 226	5 000	361	10 000	171
d = 354	500	546	519	487	429	456	1 780	321	S_x 1 280	6 000	317	11 000	153
W360x72	3 000	771	702	613	2 570	733	1 940	542	M_r 397	3 500	377	7 000	222
W14x48	2 000	674	625	559	1 720	667	1 880	470	V_r 617	4 000	357	8 000	186
b = 204	1 500	624	586	525	1 290	615	1 840	423	L_u 2 940	4 500	336	9 000	160
t = 15.1	1 000	571	537	485	858	540	1 770	365	I_x 201	5 000	315	10 000	141
d = 350	500	497	471	439	429	421	1 620	294	S_x 1 150	6 000	272	11 000	126
W360x64	3 000	712	644	558	2 530	664	1 740	498	M_r 354	3 500	332	7 000	183
W14x43	2 000	620	572	510	1 720	607	1 690	433	V_r 548	4 000	313	8 000	153
b = 203	1 500	571	534	478	1 290	562	1 660	390	L_u 2 870	4 500	293	9 000	131
t = 13.5	1 000	520	489	440	858	496	1 590	336	I_x 178	5 000	273	10 000	115
d = 347	500	451	426	395	429	389	1 470	268	S_x 1 030	6 000	228	11 000	102
W360x57	3 000	651	587	508	2 240	622	1 560	473	M_r 314	3 000	289	7 000	119
W14x38	2 000	584	537	475	1 720	571	1 520	412	V_r 580	3 500	267	8 000	99.7
b = 172	1 500	535	498	442	1 290	531	1 490	371	L_u 2 360	4 000	244	9 000	85.9
t = 13.1	1 000	484	453	404	858	471	1 440	320	I_x 160	5 000	192	10 000	75.6
d = 358	500	415	390	359	429	370	1 320	252	S_x 896	6 000	147	11 000	67.5
W360x51	3 000	585	525	452	2 000	560	1 400	433	M_r 277	3 000	252	7 000	96.9
W14x34	2 000	539	493	433	1 720	518	1 360	379	V_r 524	3 500	232	8 000	80.9
b = 171	1 500	491	455	403	1 290	483	1 340	342	L_u 2 320	4 000	210	9 000	69.4
t = 11.6	1 000	441	413	366	858	431	1 290	294	I_x 141	5 000	159	10 000	60.8
d = 355	500	377	353	323	429	341	1 190	231	S_x 796	6 000	121	11 000	54.1
W360x45	3 000	522	467	401	1 780	501	1 240	392	M_r 242	3 000	217	7 000	76.4
W14x30	2 000	498	452	394	1 720	465	1 210	345	V_r 498	3 500	197	8 000	63.3
b = 171	1 500	450	415	365	1 290	436	1 190	312	L_u 2 260	4 000	176	9 000	54.0
t = 9.8	1 000	401	375	330	858	391	1 150	268	I_x 122	5 000	128	10 000	47.1
d = 352	500	340	317	287	429	311	1 070	209	S_x 691	6 000	96.0	11 000	41.8
W360x39	3 000	459	408	349	1 550	442	1 080	352	M_r 206	2 000	193	6 000	54.1
W14x26	2 000	444	401	346	1 550	413	1 050	311	V_r 470	2 500	172	7 000	44.2
b = 128	1 500	411	376	328	1 290	388	1 030	281	L_u 1 660	3 000	148	8 000	37.4
t = 10.7	1 000	362	337	293	858	351	1 000	242	I_x 102	4 000	97.0	9 000	32.5
d = 353	500	303	280	251	429	280	930	187	S_x 580	5 000	69.7	10 000	28.7

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm $\qquad F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 65 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

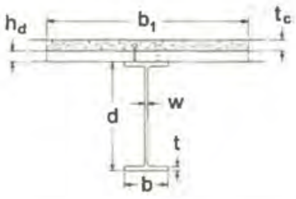
Steel section	b_1	Composite: M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite: Steel section data	Unbraced condition: L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W360x33	2 500	382	339	289	1 290	363	892	288	M_r 168	2 000	155	6 000	38.0
W14x22	2 000	376	336	288	1 290	350	881	270	V_r 396	2 500	135	7 000	30.8
b = 127	1 500	364	331	286	1 290	332	867	246	L_u 1 600	3 000	113	8 000	25.8
t = 8.5	1 000	317	293	253	858	302	842	212	I_x 82.6	4 000	70.2	9 000	22.3
d = 349	500	262	241	213	429	245	787	163	S_x 473	5 000	49.6	10 000	19.6
W310x74	2 500	682	622	546	2 150	610	1 840	434	M_r 366	3 500	354	6 000	274
W12x50	2 000	633	583	519	1 720	576	1 800	399	V_r 597	4 000	339	7 000	240
b = 205	1 500	582	543	487	1 290	529	1 760	358	L_u 3 100	4 500	323	8 000	204
t = 16.3	1 000	529	498	450	858	462	1 680	307	I_x 164	5 000	307	9 000	177
d = 310	500	461	437	408	429	356	1 540	244	S_x 1 060	5 500	291	10 000	156
W310x67	2 500	631	572	498	2 150	552	1 650	398	M_r 326	3 500	312	6 000	234
W12x45	2 000	583	534	473	1 720	522	1 620	367	V_r 533	4 000	297	7 000	198
b = 204	1 500	533	495	443	1 290	482	1 580	329	L_u 3 020	4 500	282	8 000	167
t = 14.6	1 000	481	453	408	858	423	1 520	282	I_x 144	5 000	266	9 000	144
d = 306	500	418	395	367	429	327	1 390	222	S_x 942	5 500	250	10 000	127
W310x60	2 500	585	528	455	2 150	500	1 480	366	M_r 290	3 500	275	6 000	199
W12x40	2 000	537	490	431	1 720	475	1 450	338	V_r 466	4 000	261	7 000	163
b = 203	1 500	488	452	403	1 290	440	1 420	304	L_u 2 960	4 500	246	8 000	137
t = 13.1	1 000	438	412	370	858	389	1 370	260	I_x 128	5 000	231	9 000	118
d = 303	500	379	358	330	429	303	1 260	204	S_x 842	5 500	215	10 000	104
W310x52	2 500	553	497	426	2 070	477	1 340	355	M_r 260	3 000	240	6 000	130
W12x35	2 000	512	465	406	1 720	455	1 320	329	V_r 494	3 500	223	7 000	106
b = 167	1 500	464	427	378	1 290	423	1 290	295	L_u 2 380	4 000	206	8 000	89.4
t = 13.2	1 000	413	387	344	858	376	1 240	253	I_x 118	4 500	187	9 000	77.4
d = 317	500	354	332	304	429	294	1 140	197	S_x 747	5 000	167	10 000	68.4
W310x45	2 500	476	425	362	1 770	413	1 140	315	M_r 220	3 000	200	6 000	98.2
W12x30	2 000	461	416	358	1 720	396	1 130	292	V_r 423	3 500	184	7 000	79.3
b = 166	1 500	414	378	333	1 290	371	1 110	263	L_u 2 310	4 000	167	8 000	66.5
t = 11.2	1 000	365	340	302	858	332	1 070	226	I_x 99.2	4 500	150	9 000	57.3
d = 313	500	310	290	263	429	262	993	174	S_x 634	5 000	128	10 000	50.4
W310x39	2 500	417	369	314	1 530	364	997	282	M_r 189	3 000	170	6 000	77.7
W12x26	2 000	408	365	312	1 530	349	985	263	V_r 368	3 500	155	7 000	62.2
b = 165	1 500	376	341	298	1 290	329	968	238	L_u 2 260	4 000	139	8 000	51.8
t = 9.7	1 000	328	304	269	858	297	938	205	I_x 85.1	4 500	121	9 000	44.3
d = 310	500	276	258	232	429	237	874	157	S_x 549	5 000	103	10 000	38.8
W250x67	2 500	571	512	437	2 150	443	1 510	314	M_r 280	3 500	275	6 000	223
W10x45	2 000	522	474	413	1 720	418	1 490	288	V_r 469	4 000	265	6 500	212
b = 204	1 500	472	435	387	1 290	384	1 450	256	L_u 3 260	4 500	254	7 000	202
t = 15.7	1 000	421	395	355	858	335	1 390	217	I_x 104	5 000	244	7 500	192
d = 257	500	363	343	317	429	255	1 260	168	S_x 806	5 500	233	8 000	180

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

$F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 65 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



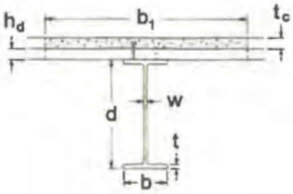
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

Steel section	b_1	Composite M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite Steel section data	Unbraced condition L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W250x58	2 500	521	464	391	2 150	388	1 310	280	M_r 239	3 500	232	6 000	181
W10x39	2 000	473	426	367	1 720	368	1 290	257	V_r 413	4 000	222	6 500	171
b = 203	1 500	425	388	342	1 290	340	1 260	230	L_u 3 130	4 500	212	7 000	161
t = 13.5	1 000	374	349	312	858	298	1 210	195	I_x 87.3	5 000	202	7 500	148
d = 252	500	320	301	276	429	229	1 110	149	S_x 693	5 500	192	8 000	137
W250x45	2 500	437	385	322	1 780	334	1 050	251	M_r 187	3 000	167	5 500	101
W10x30	2 000	421	375	317	1 720	319	1 030	232	V_r 414	3 500	155	6 000	90.6
b = 148	1 500	373	338	293	1 290	298	1 010	208	L_u 2 170	4 000	142	6 500	82.2
t = 13	1 000	324	299	263	858	265	975	177	I_x 71.1	4 500	129	7 000	75.2
d = 266	500	270	252	227	429	207	897	134	S_x 534	5 000	114	7 500	69.3
W250x39	2 500	379	332	276	1 530	291	904	223	M_r 159	3 000	140	5 500	77.5
W10x26	2 000	370	327	274	1 530	279	892	207	V_r 354	3 500	128	6 000	69.2
b = 147	1 500	338	304	260	1 290	262	875	186	L_u 2 110	4 000	115	6 500	62.5
t = 11.2	1 000	290	266	233	858	235	847	159	I_x 60.1	4 500	102	7 000	57.0
d = 262	500	239	222	199	429	186	784	120	S_x 459	5 000	88.0	7 500	52.4
W250x33	2 500	323	281	232	1 290	248	766	194	M_r 132	3 000	112	5 500	55.6
W10x22	2 000	317	278	231	1 290	239	756	181	V_r 323	3 500	100	6 000	49.4
b = 146	1 500	305	272	229	1 290	225	742	164	L_u 2 020	4 000	88.4	6 500	44.4
t = 9.1	1 000	258	235	204	858	204	720	140	I_x 48.9	4 500	74.1	7 000	40.3
d = 258	500	209	193	171	429	163	670	106	S_x 379	5 000	63.6	7 500	36.9
W200x42	2 500	359	309	250	1 650	232	867	173	M_r 138	3 000	133	5 500	99.6
W8x28	2 000	348	304	248	1 650	221	853	160	V_r 302	3 500	126	6 000	92.9
b = 166	1 500	307	272	228	1 290	206	835	142	L_u 2 610	4 000	120	6 500	84.6
t = 11.8	1 000	258	234	203	858	183	805	120	I_x 40.9	4 500	113	7 000	77.5
d = 205	500	208	194	173	429	142	740	88.0	S_x 399	5 000	106	7 500	71.6
W200x36	2 500	311	266	214	1 420	202	749	154	M_r 118	3 000	112	5 500	79.3
W8x24	2 000	303	262	213	1 420	193	738	142	V_r 255	3 500	105	6 000	71.3
b = 165	1 500	281	247	204	1 290	181	723	128	L_u 2 510	4 000	99.0	6 500	64.6
t = 10.2	1 000	233	210	180	858	162	699	108	I_x 34.4	4 500	92.5	7 000	59.0
d = 201	500	184	171	152	429	127	647	79.2	S_x 342	5 000	85.9	7 500	54.4
W200x31	2 500	281	240	193	1 240	188	669	146	M_r 104	2 000	104	4 500	65.2
W8x21	2 000	275	237	192	1 240	181	659	136	V_r 275	2 500	96.7	5 000	57.0
b = 134	1 500	265	232	190	1 240	170	646	123	L_u 1 980	3 000	89.3	5 500	50.6
t = 10.2	1 000	222	198	169	858	154	626	104	I_x 31.4	3 500	81.7	6 000	45.6
d = 210	500	172	159	139	429	122	581	76.7	S_x 299	4 000	74.0	6 500	41.5
W200x27	2 500	239	203	163	1 050	161	569	128	M_r 86.6	2 000	85.3	4 500	47.5
W8x18	2 000	235	201	162	1 050	155	561	120	V_r 246	2 500	78.7	5 000	41.2
b = 133	1 500	228	198	161	1 050	147	550	109	L_u 1 890	3 000	71.5	5 500	36.4
t = 8.4	1 000	201	178	149	858	134	534	92.8	I_x 25.8	3 500	64.1	6 000	32.6
d = 207	500	153	140	121	429	108	499	68.6	S_x 249	4 000	56.0	6 500	29.6

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm $\quad F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 75 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

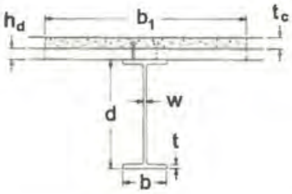
Steel section	b_1	Composite: M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite: Steel section data	Unbraced condition: L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W1000x249	7 000	5 580	5 350	4 830	6 930	12 500	14 000	9 450	M_r 3 510	4 000	3 440	14 000	831
W40x167	5 000	5 310	5 030	4 500	4 950	11 600	13 700	8 550	V_r 3 220	6 000	2 780	16 000	694
b = 300	3 000	4 840	4 520	4 110	2 970	10 100	13 200	7 390	L_u 3 740	8 000	1 940	18 000	596
t = 26	1 000	3 980	3 830	3 660	990	7 370	11 800	5 820	I_x 4 810	10 000	1 360	20 000	523
d = 980									S_x 9 820	12 000	1 030	22 000	466
W1000x222	7 000	5 030	4 820	4 330	6 930	11 100	12 300	8 430	M_r 3 040	4 000	2 940	14 000	634
W40x149	5 000	4 780	4 520	4 010	4 950	10 300	12 100	7 620	V_r 3 000	6 000	2 310	16 000	527
b = 300	3 000	4 330	4 030	3 630	2 970	9 040	11 600	6 550	L_u 3 590	8 000	1 520	18 000	451
t = 21.1	1 000	3 500	3 350	3 180	990	6 530	10 400	5 060	I_x 4 080	10 000	1 050	20 000	394
d = 970									S_x 8 410	12 000	794	22 000	350
W920x238	7 000	5 070	4 850	4 430	6 930	10 800	12 800	8 140	M_r 3 170	4 000	3 140	14 000	800
W36x160	5 000	4 810	4 590	4 130	4 950	9 980	12 500	7 360	V_r 3 090	6 000	2 590	16 000	668
b = 305	3 000	4 420	4 150	3 780	2 970	8 730	12 000	6 350	L_u 3 890	8 000	1 870	18 000	573
t = 25.9	1 000	3 660	3 520	3 370	990	6 330	10 800	4 960	I_x 4 060	10 000	1 310	20 000	502
d = 915									S_x 8 870	12 000	996	22 000	447
W920x223	7 000	4 800	4 590	4 190	6 930	10 100	12 000	7 710	M_r 2 960	4 500	2 800	12 000	881
W36x150	5 000	4 550	4 350	3 910	4 950	9 430	11 700	6 970	V_r 2 970	5 000	2 670	14 000	705
b = 304	3 000	4 190	3 920	3 560	2 970	8 260	11 300	6 000	L_u 3 830	6 000	2 380	16 000	587
t = 23.9	1 000	3 440	3 300	3 150	990	5 980	10 100	4 650	I_x 3 760	8 000	1 680	18 000	502
d = 911									S_x 8 260	10 000	1 170	20 000	439
W920x201	7 000	4 370	4 160	3 800	6 930	9 080	10 600	6 960	M_r 2 590	4 500	2 420	12 000	705
W36x135	5 000	4 120	3 940	3 520	4 950	8 480	10 400	6 300	V_r 2 710	5 000	2 300	14 000	560
b = 304	3 000	3 790	3 540	3 190	2 970	7 460	10 100	5 400	L_u 3 720	6 000	2 030	16 000	463
t = 20.1	1 000	3 070	2 930	2 780	990	5 380	8 990	4 120	I_x 3 250	8 000	1 360	18 000	394
d = 903									S_x 7 190	10 000	940	20 000	343
W840x210	7 000	4 300	4 090	3 750	6 930	8 550	10 700	6 530	M_r 2 620	4 500	2 460	12 000	792
W33x141	5 000	4 050	3 880	3 500	4 950	7 970	10 500	5 910	V_r 2 670	5 000	2 350	14 000	639
b = 293	3 000	3 740	3 510	3 180	2 970	7 000	10 100	5 070	L_u 3 770	6 000	2 090	16 000	535
t = 24.4	1 000	3 070	2 940	2 800	990	5 060	9 060	3 900	I_x 3 110	8 000	1 470	18 000	461
d = 846									S_x 7 340	10 000	1 040	20 000	404
W840x193	7 000	3 990	3 790	3 480	6 930	7 840	9 760	6 030	M_r 2 370	4 500	2 200	12 000	666
W33x130	5 000	3 750	3 590	3 240	4 950	7 330	9 580	5 460	V_r 2 530	5 000	2 090	14 000	534
b = 292	3 000	3 460	3 250	2 930	2 970	6 460	9 240	4 680	L_u 3 690	6 000	1 850	16 000	445
t = 21.7	1 000	2 820	2 690	2 550	990	4 660	8 280	3 550	I_x 2 780	8 000	1 260	18 000	382
d = 840									S_x 6 630	10 000	877	20 000	334
W840x176	7 000	3 690	3 490	3 200	6 930	7 140	8 830	5 540	M_r 2 110	4 500	1 950	12 000	551
W33x118	5 000	3 450	3 300	2 970	4 950	6 690	8 670	5 020	V_r 2 300	5 000	1 840	14 000	439
b = 292	3 000	3 180	2 980	2 670	2 970	5 920	8 370	4 290	L_u 3 610	6 000	1 610	16 000	364
t = 18.8	1 000	2 560	2 440	2 300	990	4 280	7 500	3 220	I_x 2 460	8 000	1 060	18 000	311
d = 835									S_x 5 900	10 000	731	20 000	271

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

$F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 75 mm Slab
$\phi = 0.90, \phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

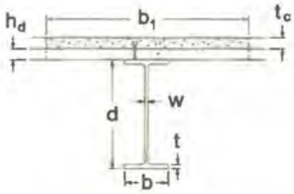
Steel section	b_1	Composite M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite Steel section data	Unbraced condition L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W760x185	5 000	3 340	3 180	2 880	4 950	6 060	8 540	4 510	M_r 2 080	4 000	1 980	12 000	576
W30x124	4 000	3 210	3 060	2 750	3 960	5 750	8 420	4 210	V_r 2 340	5 000	1 780	14 000	470
b = 267	3 000	3 070	2 890	2 600	2 970	5 340	8 240	3 850	L_u 3 450	6 000	1 550	16 000	397
t = 23.6	2 000	2 830	2 660	2 430	1 980	4 750	7 940	3 420	I_x 2 230	8 000	1 040	18 000	344
d = 766	1 000	2 500	2 380	2 250	990	3 840	7 380	2 890	S_x 5 820	10 000	743	20 000	304
W760x173	5 000	3 160	3 010	2 720	4 950	5 700	8 000	4 270	M_r 1 930	4 000	1 830	12 000	506
W30x116	4 000	3 040	2 900	2 590	3 960	5 420	7 890	3 980	V_r 2 250	5 000	1 630	14 000	411
b = 267	3 000	2 900	2 730	2 450	2 970	5 040	7 720	3 640	L_u 3 410	6 000	1 410	16 000	346
t = 21.6	2 000	2 680	2 510	2 280	1 980	4 490	7 450	3 230	I_x 2 060	8 000	924	18 000	299
d = 762	1 000	2 340	2 230	2 100	990	3 630	6 920	2 710	S_x 5 400	10 000	657	20 000	264
W760x161	5 000	2 960	2 810	2 540	4 950	5 280	7 360	3 980	M_r 1 760	4 000	1 650	12 000	429
W30x108	4 000	2 830	2 700	2 410	3 960	5 030	7 260	3 710	V_r 2 140	5 000	1 460	14 000	347
b = 266	3 000	2 700	2 540	2 270	2 970	4 690	7 110	3 390	L_u 3 330	6 000	1 250	16 000	291
t = 19.3	2 000	2 490	2 330	2 110	1 980	4 190	6 870	3 000	I_x 1 860	8 000	793	18 000	251
d = 758	1 000	2 170	2 060	1 930	990	3 380	6 380	2 500	S_x 4 900	10 000	560	20 000	220
W760x147	5 000	2 740	2 600	2 350	4 950	4 850	6 710	3 680	M_r 1 580	4 000	1 470	12 000	358
W30x99	4 000	2 630	2 500	2 230	3 960	4 630	6 630	3 430	V_r 2 040	5 000	1 290	14 000	288
b = 265	3 000	2 500	2 350	2 090	2 970	4 320	6 490	3 130	L_u 3 260	6 000	1 090	16 000	241
t = 17	2 000	2 310	2 150	1 930	1 980	3 870	6 270	2 760	I_x 1 660	8 000	671	18 000	207
d = 753	1 000	1 990	1 880	1 760	990	3 120	5 830	2 290	S_x 4 410	10 000	470	20 000	181
W760x134	5 000	2 530	2 400	2 170	4 950	4 460	6 110	3 420	M_r 1 440	4 000	1 330	12 000	308
W30x90	4 000	2 420	2 300	2 060	3 960	4 260	6 030	3 190	V_r 1 650	5 000	1 160	14 000	246
b = 264	3 000	2 290	2 180	1 930	2 970	4 000	5 920	2 920	L_u 3 230	6 000	967	16 000	205
t = 15.5	2 000	2 130	1 990	1 780	1 980	3 590	5 730	2 570	I_x 1 500	8 000	587	18 000	175
d = 750	1 000	1 840	1 730	1 610	990	2 910	5 340	2 120	S_x 4 010	10 000	408	20 000	153
W690x192	5 000	3 190	3 030	2 760	4 950	5 410	8 290	4 000	M_r 2 010	4 000	1 910	12 000	634
W27x129	4 000	3 060	2 920	2 630	3 960	5 130	8 170	3 730	V_r 2 230	5 000	1 730	14 000	525
b = 254	3 000	2 920	2 760	2 500	2 970	4 750	7 990	3 410	L_u 3 440	6 000	1 540	16 000	449
t = 27.9	2 000	2 710	2 550	2 340	1 980	4 220	7 700	3 030	I_x 1 980	8 000	1 090	18 000	392
d = 702	1 000	2 400	2 290	2 170	990	3 400	7 140	2 560	S_x 5 640	10 000	802	20 000	349
W690x170	5 000	2 880	2 720	2 470	4 950	4 810	7 310	3 590	M_r 1 750	4 000	1 650	12 000	497
W27x114	4 000	2 750	2 620	2 360	3 960	4 570	7 210	3 350	V_r 2 060	5 000	1 480	14 000	408
b = 256	3 000	2 620	2 480	2 230	2 970	4 250	7 060	3 060	L_u 3 380	6 000	1 290	16 000	347
t = 23.6	2 000	2 430	2 280	2 080	1 980	3 790	6 810	2 710	I_x 1 700	8 000	875	18 000	302
d = 693	1 000	2 130	2 030	1 910	990	3 050	6 330	2 260	S_x 4 900	10 000	634	20 000	268
W690x152	5 000	2 620	2 480	2 260	4 950	4 360	6 570	3 300	M_r 1 550	4 000	1 460	12 000	406
W27x102	4 000	2 500	2 380	2 150	3 960	4 160	6 480	3 080	V_r 1 850	5 000	1 290	14 000	332
b = 254	3 000	2 370	2 260	2 020	2 970	3 880	6 350	2 810	L_u 3 320	6 000	1 110	16 000	281
t = 21.1	2 000	2 210	2 070	1 880	1 980	3 470	6 150	2 480	I_x 1 510	8 000	728	18 000	244
d = 688	1 000	1 930	1 830	1 720	990	2 800	5 720	2 060	S_x 4 380	10 000	523	20 000	216

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 75 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



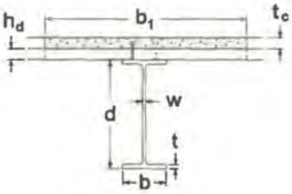
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W690x140	5 000	2 450	2 310	2 100	4 950	4 020	6 020	3 060	M_r 1 410	4 000	1 320	10 000	447
W27x94	4 000	2 330	2 210	2 000	3 960	3 840	5 940	2 860	V_r 1 740	5 000	1 160	12 000	345
b = 254	3 000	2 200	2 100	1 880	2 970	3 600	5 830	2 610	L_u 3 270	6 000	987	14 000	280
t = 18.9	2 000	2 050	1 920	1 740	1 980	3 220	5 640	2 300	I_x 1 360	7 000	778	16 000	236
d = 684	1 000	1 790	1 690	1 570	990	2 600	5 260	1 900	S_x 3 980	8 000	628	18 000	204
W690x125	5 000	2 240	2 110	1 910	4 950	3 610	5 370	2 780	M_r 1 250	4 000	1 140	10 000	362
W27x84	4 000	2 130	2 010	1 820	3 960	3 460	5 300	2 600	V_r 1 610	5 000	999	12 000	277
b = 253	3 000	2 000	1 910	1 700	2 970	3 250	5 210	2 370	L_u 3 190	6 000	834	14 000	223
t = 16.3	2 000	1 870	1 750	1 560	1 980	2 920	5 050	2 080	I_x 1 180	7 000	640	16 000	187
d = 678	1 000	1 610	1 520	1 410	990	2 360	4 710	1 700	S_x 3 500	8 000	513	18 000	161
W610x174	5 000	2 670	2 520	2 310	4 950	4 130	6 990	3 080	M_r 1 660	4 500	1 660	10 000	924
W24x117	4 000	2 550	2 420	2 220	3 960	3 920	6 890	2 870	V_r 1 770	5 000	1 610	12 000	709
b = 325	3 000	2 420	2 310	2 100	2 970	3 640	6 750	2 620	L_u 4 480	6 000	1 490	14 000	574
t = 21.6	2 000	2 270	2 150	1 970	1 980	3 240	6 520	2 320	I_x 1 470	7 000	1 370	16 000	482
d = 616	1 000	2 020	1 930	1 820	990	2 610	6 080	1 950	S_x 4 780	8 000	1 230	18 000	415
W610x155	5 000	2 420	2 280	2 090	4 950	3 710	6 220	2 800	M_r 1 470	4 500	1 460	10 000	762
W24x104	4 000	2 310	2 190	2 000	3 960	3 540	6 140	2 610	V_r 1 590	5 000	1 410	12 000	579
b = 324	3 000	2 180	2 080	1 900	2 970	3 300	6 030	2 390	L_u 4 400	6 000	1 300	14 000	465
t = 19	2 000	2 050	1 940	1 770	1 980	2 950	5 830	2 110	I_x 1 290	7 000	1 180	16 000	388
d = 611	1 000	1 810	1 730	1 620	990	2 380	5 440	1 750	S_x 4 220	8 000	1 050	18 000	333
W610x140	5 000	2 270	2 120	1 920	4 950	3 410	5 590	2 580	M_r 1 290	4 000	1 170	10 000	422
W24x94	4 000	2 150	2 030	1 830	3 960	3 260	5 510	2 410	V_r 1 660	5 000	1 030	12 000	334
b = 230	3 000	2 020	1 920	1 720	2 970	3 040	5 410	2 200	L_u 3 070	6 000	874	14 000	277
t = 22.2	2 000	1 880	1 760	1 590	1 980	2 720	5 230	1 930	I_x 1 120	7 000	695	16 000	237
d = 617	1 000	1 640	1 550	1 440	990	2 190	4 860	1 580	S_x 3 630	8 000	573	18 000	207
W610x125	5 000	2 070	1 930	1 750	4 950	3 070	4 990	2 360	M_r 1 140	4 000	1 020	10 000	342
W24x84	4 000	1 960	1 840	1 670	3 960	2 940	4 930	2 200	V_r 1 490	5 000	889	12 000	269
b = 229	3 000	1 830	1 740	1 560	2 970	2 760	4 840	2 010	L_u 3 020	6 000	733	14 000	222
t = 19.6	2 000	1 700	1 600	1 440	1 980	2 480	4 690	1 760	I_x 985	7 000	575	16 000	189
d = 612	1 000	1 480	1 400	1 290	990	2 000	4 370	1 430	S_x 3 220	8 000	470	18 000	165
W610x113	5 000	1 880	1 750	1 580	4 490	2 800	4 510	2 170	M_r 1 020	4 000	906	10 000	282
W24x76	4 000	1 810	1 700	1 530	3 960	2 680	4 460	2 030	V_r 1 400	5 000	775	12 000	220
b = 228	3 000	1 690	1 600	1 440	2 970	2 520	4 380	1 850	L_u 2 950	6 000	617	14 000	180
t = 17.3	2 000	1 560	1 470	1 320	1 980	2 280	4 250	1 620	I_x 875	7 000	481	16 000	153
d = 608	1 000	1 360	1 280	1 170	990	1 850	3 970	1 310	S_x 2 880	8 000	391	18 000	133
W610x101	5 000	1 690	1 570	1 410	4 020	2 510	4 030	1 970	M_r 900	4 000	787	10 000	228
W24x68	4 000	1 660	1 550	1 400	3 960	2 420	3 980	1 850	V_r 1 300	5 000	664	12 000	176
b = 228	3 000	1 540	1 460	1 310	2 970	2 280	3 920	1 690	L_u 2 890	6 000	512	14 000	144
t = 14.9	2 000	1 420	1 340	1 190	1 980	2 070	3 810	1 480	I_x 764	7 000	396	16 000	121
d = 603	1 000	1 230	1 150	1 050	990	1 680	3 560	1 190	S_x 2 530	8 000	320	18 000	105

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 75 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



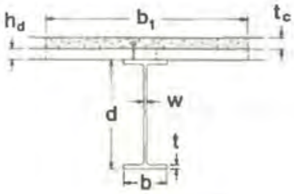
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

Steel section	b_1 (mm)	Composite: M_{rc} (kN·m) for % shear connection: 100%	70%	40%	Q_r (kN) 100%	I_t 10^6 mm^4	S_t 10^3 mm^3	I_{ts} 10^6 mm^4	Non-composite: Steel section data	Unbraced condition: L' (mm)	M_r' (kN·m)	L' (mm)	M_r' (kN·m)
W610x92	4 000	1 520	1 420	1 260	3 650	2 180	3 560	1 680	M_r 779	3 000	683	8 000	183
W24x62	3 000	1 430	1 350	1 190	2 970	2 070	3 500	1 530	V_r 1 350	4 000	540	10 000	135
b = 179	2 000	1 310	1 220	1 070	1 980	1 880	3 400	1 340	L_u 2 180	5 000	376	12 000	107
t = 15	1 000	1 110	1 030	932	990	1 530	3 170	1 060	I_x 646	6 000	281	14 000	88.9
d = 603	500	958	909	853	495	1 210	2 900	878	S_x 2 140	7 000	222	16 000	76.1
W610x82	4 000	1 360	1 260	1 110	3 240	1 950	3 150	1 520	M_r 683	3 000	587	8 000	145
W24x55	3 000	1 300	1 220	1 080	2 970	1 850	3 100	1 390	V_r 1 170	4 000	448	10 000	106
b = 178	2 000	1 190	1 110	969	1 980	1 690	3 020	1 220	L_u 2 110	5 000	304	12 000	83.4
t = 12.8	1 000	1 010	929	832	990	1 390	2 830	962	I_x 560	6 000	225	14 000	68.9
d = 599	500	857	809	754	495	1 100	2 590	787	S_x 1 870	7 000	177	16 000	58.7
W530x138	4 000	1 940	1 820	1 630	3 960	2 650	4 960	1 950	M_r 1 120	3 000	1 110	8 000	515
W21x93	3 000	1 810	1 710	1 530	2 970	2 470	4 860	1 770	V_r 1 650	4 000	1 000	10 000	390
b = 214	2 000	1 670	1 560	1 400	1 980	2 210	4 690	1 540	L_u 2 930	5 000	884	12 000	314
t = 23.6	1 000	1 450	1 360	1 270	990	1 760	4 340	1 250	I_x 861	6 000	759	14 000	263
d = 549	500	1 290	1 240	1 190	495	1 400	3 970	1 070	S_x 3 140	7 000	616	16 000	227
W530x123	4 000	1 770	1 650	1 480	3 960	2 400	4 440	1 780	M_r 997	3 000	984	8 000	421
W21x83	3 000	1 650	1 550	1 390	2 970	2 250	4 350	1 620	V_r 1 460	4 000	879	10 000	316
b = 212	2 000	1 520	1 420	1 270	1 980	2 010	4 210	1 410	L_u 2 860	5 000	762	12 000	253
t = 21.2	1 000	1 310	1 230	1 140	990	1 610	3 910	1 140	I_x 761	6 000	631	14 000	211
d = 544	500	1 160	1 120	1 060	495	1 280	3 580	967	S_x 2 800	7 000	505	16 000	182
W530x109	4 000	1 610	1 500	1 340	3 960	2 150	3 940	1 620	M_r 879	3 000	862	8 000	342
W21x73	3 000	1 490	1 400	1 260	2 970	2 020	3 870	1 480	V_r 1 280	4 000	764	10 000	254
b = 211	2 000	1 360	1 290	1 150	1 980	1 820	3 750	1 290	L_u 2 810	5 000	652	12 000	202
t = 18.8	1 000	1 180	1 110	1 020	990	1 470	3 500	1 030	I_x 667	6 000	520	14 000	168
d = 539	500	1 040	997	946	495	1 170	3 210	868	S_x 2 480	7 000	413	16 000	144
W530x101	4 000	1 520	1 410	1 270	3 960	2 020	3 670	1 530	M_r 814	3 000	794	8 000	301
W21x68	3 000	1 410	1 320	1 190	2 970	1 900	3 610	1 400	V_r 1 200	4 000	699	10 000	222
b = 210	2 000	1 280	1 210	1 080	1 980	1 720	3 500	1 220	L_u 2 770	5 000	591	12 000	176
t = 17.4	1 000	1 110	1 040	954	990	1 390	3 280	977	I_x 617	6 000	462	14 000	146
d = 537	500	976	932	881	495	1 110	3 010	816	S_x 2 300	7 000	365	16 000	125
W530x92	4 000	1 400	1 290	1 150	3 660	1 850	3 340	1 420	M_r 733	3 000	711	8 000	253
W21x62	3 000	1 310	1 220	1 100	2 970	1 750	3 290	1 300	V_r 1 110	4 000	621	9 000	214
b = 209	2 000	1 190	1 120	997	1 980	1 590	3 200	1 130	L_u 2 720	5 000	516	10 000	185
t = 15.6	1 000	1 030	960	872	990	1 290	3 000	901	I_x 552	6 000	393	12 000	146
d = 533	500	895	851	801	495	1 020	2 750	747	S_x 2 070	7 000	309	14 000	120
W530x82	4 000	1 240	1 150	1 020	3 250	1 640	2 960	1 280	M_r 640	3 000	616	8 000	203
W21x55	3 000	1 190	1 110	993	2 970	1 560	2 910	1 170	V_r 1 030	4 000	531	9 000	170
b = 209	2 000	1 070	1 010	898	1 980	1 430	2 840	1 030	L_u 2 660	5 000	433	10 000	147
t = 13.3	1 000	926	863	777	990	1 170	2 670	813	I_x 477	6 000	320	12 000	115
d = 528	500	799	756	707	495	927	2 450	666	S_x 1 810	7 000	249	14 000	94.0

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm $F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 75 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

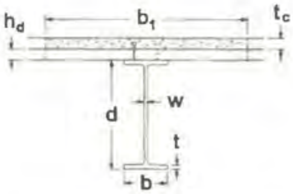
Steel section	b_1	Composite M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite Steel section data	Unbraced condition L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W530x74	4 000	1 140	1 050	920	2 960	1 500	2 660	1 180	M_r 562	3 000	474	8 000	123
W21x50	3 000	1 120	1 030	916	2 960	1 430	2 620	1 080	V_r 1 050	4 000	357	9 000	105
b = 166	2 000	1 000	939	821	1 980	1 310	2 550	942	L_u 2 040	5 000	247	10 000	91.7
t = 13.6	1 000	850	785	699	990	1 080	2 400	741	I_x 411	6 000	186	12 000	73.2
d = 529	500	721	678	628	495	849	2 200	599	S_x 1 550	7 000	148	14 000	61.0
W530x66	4 000	1 010	921	803	2 600	1 320	2 330	1 050	M_r 484	3 000	398	8 000	94.9
W21x44	3 000	987	910	800	2 600	1 260	2 300	970	V_r 927	4 000	284	9 000	80.6
b = 165	2 000	903	846	738	1 980	1 160	2 240	850	L_u 1 980	5 000	195	10 000	70.0
t = 11.4	1 000	764	704	620	990	967	2 110	667	I_x 351	6 000	145	12 000	55.5
d = 525	500	641	600	551	495	767	1 940	533	S_x 1 340	7 000	115	14 000	46.0
W460x158	4 000	1 920	1 790	1 630	3 960	2 400	5 180	1 730	M_r 1 170	4 500	1 150	9 000	794
W18x106	3 000	1 790	1 690	1 540	2 970	2 230	5 070	1 570	V_r 1 460	5 000	1 110	10 000	696
b = 284	2 000	1 660	1 570	1 430	1 980	1 970	4 890	1 370	L_u 4 200	6 000	1 040	11 000	617
t = 23.9	1 000	1 460	1 390	1 310	990	1 570	4 520	1 120	I_x 796	7 000	955	12 000	555
d = 476	500	1 330	1 290	1 240	495	1 250	4 140	971	S_x 3 350	8 000	875	14 000	462
W460x144	4 000	1 790	1 670	1 510	3 960	2 230	4 770	1 620	M_r 1 070	4 500	1 050	9 000	693
W18x97	3 000	1 660	1 570	1 430	2 970	2 070	4 670	1 470	V_r 1 320	5 000	1 010	10 000	602
b = 283	2 000	1 530	1 460	1 330	1 980	1 840	4 510	1 290	L_u 4 130	6 000	936	11 000	533
t = 22.1	1 000	1 360	1 290	1 210	990	1 470	4 180	1 050	I_x 726	7 000	858	12 000	478
d = 472	500	1 230	1 190	1 140	495	1 170	3 840	898	S_x 3 080	8 000	779	14 000	396
W460x128	4 000	1 630	1 510	1 360	3 960	2 000	4 250	1 480	M_r 947	4 500	917	9 000	566
W18x86	3 000	1 510	1 420	1 290	2 970	1 870	4 170	1 340	V_r 1 170	5 000	884	10 000	489
b = 282	2 000	1 380	1 310	1 190	1 980	1 670	4 030	1 170	L_u 4 040	6 000	812	11 000	431
t = 19.6	1 000	1 220	1 160	1 080	990	1 340	3 750	946	I_x 637	7 000	736	12 000	385
d = 467	500	1 100	1 060	1 010	495	1 060	3 450	805	S_x 2 730	8 000	658	14 000	318
W460x113	4 000	1 480	1 370	1 230	3 960	1 790	3 750	1 340	M_r 829	4 500	796	9 000	458
W18x76	3 000	1 360	1 280	1 160	2 970	1 680	3 690	1 220	V_r 1 020	5 000	765	10 000	394
b = 280	2 000	1 240	1 180	1 070	1 980	1 510	3 580	1 070	L_u 3 950	6 000	696	11 000	345
t = 17.3	1 000	1 100	1 040	959	990	1 220	3 340	855	I_x 556	7 000	623	12 000	307
d = 463	500	978	940	895	495	966	3 080	720	S_x 2 400	8 000	545	14 000	252
W460x106	4 000	1 430	1 320	1 170	3 960	1 690	3 460	1 260	M_r 742	3 000	719	8 000	308
W18x71	3 000	1 310	1 220	1 090	2 970	1 590	3 400	1 150	V_r 1 210	4 000	637	9 000	266
b = 194	2 000	1 180	1 110	991	1 980	1 430	3 290	996	L_u 2 690	5 000	549	10 000	235
t = 20.6	1 000	1 020	955	873	990	1 140	3 060	789	I_x 488	6 000	450	11 000	210
d = 469	500	893	853	807	495	898	2 790	654	S_x 2 080	7 000	366	12 000	190
W460x97	4 000	1 320	1 220	1 070	3 820	1 560	3 170	1 180	M_r 677	3 000	652	8 000	264
W18x65	3 000	1 220	1 130	1 010	2 970	1 470	3 120	1 080	V_r 1 090	4 000	574	9 000	227
b = 193	2 000	1 100	1 040	921	1 980	1 330	3 030	935	L_u 2 650	5 000	488	10 000	200
t = 19	1 000	948	887	807	990	1 070	2 820	738	I_x 445	6 000	389	11 000	178
d = 466	500	827	787	742	495	842	2 580	608	S_x 1 910	7 000	314	12 000	161

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

$F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 75 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

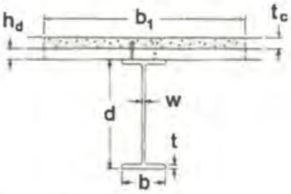
Steel section	b_1	Composite: M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite: Steel section data	Unbraced condition: L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W460x89	3 000	1 150	1 070	951	2 970	1 370	2 890	1 010	M_r 624	3 000	598	8 000	231
W18x60	2 000	1 030	970	864	1 980	1 240	2 810	882	V_r 996	4 000	523	9 000	198
b = 192	1 500	967	910	812	1 490	1 150	2 740	797	L_u 2 620	5 000	439	10 000	174
t = 17.7	1 000	888	831	754	990	1 010	2 630	695	I_x 409	6 000	343	11 000	155
d = 463	500	772	734	689	495	795	2 410	569	S_x 1 770	7 000	276	12 000	140
W460x82	3 000	1 080	997	886	2 970	1 270	2 650	946	M_r 568	3 000	540	8 000	195
W18x55	2 000	961	902	804	1 980	1 160	2 580	824	V_r 933	4 000	466	9 000	166
b = 191	1 500	899	846	753	1 490	1 070	2 520	745	L_u 2 560	5 000	384	10 000	146
t = 16	1 000	826	771	696	990	943	2 420	648	I_x 370	6 000	292	11 000	129
d = 460	500	714	676	632	495	743	2 230	527	S_x 1 610	7 000	234	12 000	116
W460x74	3 000	1 000	922	816	2 930	1 160	2 410	877	M_r 512	3 000	484	8 000	164
W18x50	2 000	889	832	742	1 980	1 060	2 350	766	V_r 843	4 000	414	9 000	140
b = 190	1 500	829	781	694	1 490	987	2 300	693	L_u 2 530	5 000	332	10 000	122
t = 14.5	1 000	761	711	638	990	874	2 210	602	I_x 332	6 000	249	11 000	108
d = 457	500	656	619	576	495	690	2 040	485	S_x 1 460	7 000	198	12 000	96.8
W460x68	3 000	936	857	754	2 710	1 080	2 220	824	M_r 463	3 000	390	8 000	112
W18x46	2 000	842	784	693	1 980	994	2 160	720	V_r 856	4 000	301	9 000	96.7
b = 154	1 500	781	733	644	1 490	925	2 110	650	L_u 2 010	5 000	213	10 000	85.2
t = 15.4	1 000	713	662	588	990	821	2 030	563	I_x 297	6 000	164	11 000	76.1
d = 459	500	606	570	526	495	648	1 870	450	S_x 1 290	7 000	133	12 000	68.9
W460x60	3 000	819	746	653	2 350	953	1 930	739	M_r 397	3 000	329	8 000	86.6
W18x40	2 000	758	702	621	1 980	882	1 890	649	V_r 746	4 000	242	9 000	74.3
b = 153	1 500	699	655	576	1 490	824	1 850	587	L_u 1 970	5 000	169	10 000	65.2
t = 13.3	1 000	636	592	523	990	737	1 780	508	I_x 255	6 000	129	11 000	58.1
d = 455	500	539	504	462	495	585	1 650	403	S_x 1 120	7 000	104	12 000	52.4
W460x52	3 000	718	651	565	2 060	832	1 680	654	M_r 338	3 000	269	8 000	63.6
W18x35	2 000	686	632	555	1 980	773	1 640	577	V_r 680	4 000	185	9 000	54.3
b = 152	1 500	628	586	512	1 490	726	1 610	522	L_u 1 890	5 000	128	10 000	47.4
t = 10.8	1 000	568	527	460	990	652	1 550	451	I_x 212	6 000	96.2	11 000	42.0
d = 450	500	476	442	400	495	521	1 440	354	S_x 942	7 000	76.7	12 000	37.8
W410x149	3 000	1 580	1 490	1 350	2 970	1 830	4 490	1 290	M_r 1 010	4 500	983	8 000	760
W16x100	2 000	1 450	1 370	1 250	1 980	1 620	4 330	1 120	V_r 1 320	5 000	952	9 000	696
b = 265	1 500	1 380	1 300	1 190	1 490	1 480	4 190	1 020	L_u 4 080	5 500	921	10 000	621
t = 25	1 000	1 280	1 210	1 130	990	1 280	3 990	902	I_x 618	6 000	889	11 000	554
d = 431	500	1 150	1 120	1 070	495	1 010	3 640	771	S_x 2 870	7 000	825	12 000	501
W410x132	3 000	1 430	1 340	1 210	2 970	1 650	3 990	1 170	M_r 885	4 500	853	8 000	635
W16x89	2 000	1 300	1 230	1 120	1 980	1 470	3 850	1 010	V_r 1 160	5 000	823	9 000	565
b = 263	1 500	1 230	1 170	1 070	1 490	1 340	3 740	920	L_u 3 940	5 500	792	10 000	495
t = 22.2	1 000	1 150	1 090	1 010	990	1 160	3 570	812	I_x 538	6 000	761	11 000	440
d = 425	500	1 030	990	947	495	917	3 260	686	S_x 2 530	7 000	698	12 000	397

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 75 mm Slab
ϕ = 0.90, ϕ_c = 0.65



ASTM A992
A572 Grade 50
f'_c = 25 MPa
γ_c = 2350 kg/m³

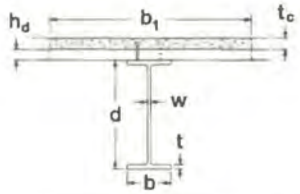
Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm⁴	mm³	mm⁴		mm	kN·m	mm	kN·m
W410x114	3 000	1 280	1 190	1 070	2 970	1 460	3 480	1 050	M_r 764	4 500	726	8 000	513
W16x77	2 000	1 150	1 090	990	1 980	1 310	3 370	911	V_r 998	5 000	698	9 000	438
b = 261	1 500	1 090	1 030	940	1 490	1 200	3 280	825	L_u 3 810	5 500	668	10 000	382
t = 19.3	1 000	1 010	958	885	990	1 050	3 140	725	I_x 461	6 000	638	11 000	338
d = 420	500	902	866	824	495	823	2 870	605	S_x 2 200	7 000	576	12 000	304
W410x100	3 000	1 150	1 060	954	2 970	1 300	3 050	947	M_r 661	4 500	623	8 000	411
W16x67	2 000	1 030	969	880	1 980	1 170	2 960	823	V_r 850	5 000	596	9 000	348
b = 260	1 500	966	918	834	1 490	1 080	2 890	744	L_u 3 730	5 500	568	10 000	302
t = 16.9	1 000	898	850	781	990	944	2 770	651	I_x 398	6 000	539	11 000	266
d = 415	500	797	763	721	495	742	2 540	538	S_x 1 920	7 000	479	12 000	238
W410x85	3 000	1 030	951	839	2 970	1 130	2 570	830	M_r 534	3 000	507	8 000	205
W16x57	2 000	915	855	759	1 980	1 020	2 500	720	V_r 931	4 000	443	9 000	177
b = 181	1 500	852	800	711	1 490	942	2 430	649	L_u 2 530	5 000	375	10 000	157
t = 18.2	1 000	780	728	656	990	827	2 330	562	I_x 315	6 000	297	11 000	140
d = 417	500	673	638	596	495	646	2 130	453	S_x 1 510	7 000	242	12 000	127
W410x74	3 000	945	865	758	2 960	1 010	2 270	755	M_r 469	3 000	440	8 000	163
W16x50	2 000	830	772	686	1 980	920	2 210	657	V_r 821	4 000	379	9 000	140
b = 180	1 500	769	722	640	1 490	852	2 160	592	L_u 2 470	5 000	312	10 000	123
t = 16	1 000	703	656	588	990	752	2 080	512	I_x 275	6 000	239	11 000	110
d = 413	500	604	570	530	495	590	1 910	410	S_x 1 330	7 000	194	12 000	99.7
W410x67	3 000	858	780	681	2 670	916	2 050	696	M_r 422	3 000	392	8 000	135
W16x45	2 000	768	711	632	1 980	841	2 000	608	V_r 739	4 000	333	9 000	116
b = 179	1 500	708	664	588	1 490	782	1 960	548	L_u 2 420	5 000	264	10 000	102
t = 14.4	1 000	645	603	538	990	694	1 890	473	I_x 245	6 000	201	11 000	90.4
d = 410	500	553	520	481	495	546	1 740	376	S_x 1 200	7 000	161	12 000	81.6
W410x60	3 000	762	690	600	2 350	821	1 820	634	M_r 369	3 000	341	8 000	109
W16x40	2 000	701	646	573	1 980	758	1 780	556	V_r 642	4 000	286	9 000	93.1
b = 178	1 500	643	600	533	1 490	708	1 740	503	L_u 2 390	5 000	218	10 000	81.3
t = 12.8	1 000	582	546	486	990	632	1 680	435	I_x 216	6 000	165	11 000	72.1
d = 407	500	499	469	430	495	501	1 560	344	S_x 1 060	7 000	131	12 000	64.9
W410x54	3 000	686	618	536	2 110	735	1 620	575	M_r 326	3 000	295	8 000	86.0
W16x36	2 000	648	595	523	1 980	682	1 590	506	V_r 619	4 000	241	9 000	73.1
b = 177	1 500	591	549	485	1 490	639	1 560	458	L_u 2 310	5 000	176	10 000	63.5
t = 10.9	1 000	531	497	438	990	573	1 510	395	I_x 186	6 000	132	11 000	56.2
d = 403	500	451	421	383	495	456	1 400	310	S_x 923	7 000	104	12 000	50.4
W410x46	3 000	600	538	463	1 830	646	1 410	513	M_r 274	2 000	265	7 000	61.7
W16x31	2 000	579	527	459	1 830	603	1 370	454	V_r 578	3 000	210	8 000	51.8
b = 140	1 500	535	493	432	1 490	567	1 350	412	L_u 1 790	4 000	142	9 000	44.6
t = 11.2	1 000	476	444	387	990	512	1 310	356	I_x 156	5 000	99.9	10 000	39.2
d = 403	500	400	370	333	495	411	1 210	276	S_x 772	6 000	76.4	11 000	35.0

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm⁴, S_x - 10^3 mm³, b - mm, t - mm, d - mm

F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 75 mm Slab
$\phi = 0.90, \phi_c = 0.65$



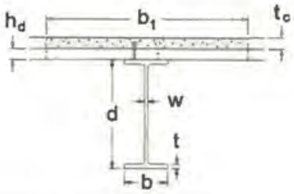
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		L' mm	M_r' kN·m	L' mm	M_r' kN·m
W410x39	3 000	511	456	389	1 550	549	1 190	445	M_r 227	2 000	216	7 000	44.0
W16x26	2 000	496	448	387	1 550	515	1 160	396	V_r 480	3 000	166	8 000	36.5
b = 140	1 500	476	436	380	1 490	488	1 140	361	L_u 1 730	4 000	105	9 000	31.2
t = 8.8	1 000	419	390	337	990	444	1 110	312	I_x 126	5 000	73.0	10 000	27.3
d = 399	500	348	321	284	495	359	1 030	240	S_x 634	6 000	55.1	11 000	24.2
W360x79	3 000	889	808	700	2 970	856	2 200	633	M_r 444	3 500	425	7 000	267
W14x53	2 000	772	714	637	1 980	778	2 140	548	V_r 682	4 000	404	8 000	225
b = 205	1 500	711	666	596	1 490	718	2 090	492	L_u 3 010	4 500	383	9 000	194
t = 16.8	1 000	648	610	549	990	630	2 010	424	I_x 226	5 000	361	10 000	171
d = 354	500	563	533	496	495	490	1 840	338	S_x 1 280	6 000	317	11 000	153
W360x72	3 000	818	739	637	2 830	777	1 990	583	M_r 397	3 500	377	7 000	222
W14x48	2 000	715	658	584	1 980	710	1 940	506	V_r 617	4 000	357	8 000	186
b = 204	1 500	655	611	546	1 490	658	1 900	455	L_u 2 940	4 500	336	9 000	160
t = 15.1	1 000	593	558	501	990	581	1 820	391	I_x 201	5 000	315	10 000	141
d = 350	500	513	484	448	495	453	1 680	310	S_x 1 150	6 000	272	11 000	126
W360x64	3 000	737	662	568	2 530	703	1 790	535	M_r 354	3 500	332	7 000	183
W14x43	2 000	660	605	533	1 980	646	1 740	466	V_r 548	4 000	313	8 000	153
b = 203	1 500	601	558	498	1 490	601	1 700	420	L_u 2 870	4 500	293	9 000	131
t = 13.5	1 000	540	509	455	990	533	1 640	361	I_x 178	5 000	273	10 000	115
d = 347	500	467	439	404	495	418	1 520	284	S_x 1 030	6 000	228	11 000	102
W360x57	3 000	674	603	517	2 240	657	1 610	507	M_r 314	3 000	289	7 000	119
W14x38	2 000	623	569	498	1 980	607	1 570	444	V_r 580	3 500	267	8 000	99.7
b = 172	1 500	565	523	463	1 490	567	1 530	400	L_u 2 360	4 000	244	9 000	85.9
t = 13.1	1 000	505	473	419	990	506	1 480	344	I_x 160	5 000	192	10 000	75.6
d = 358	500	431	403	368	495	399	1 370	268	S_x 896	6 000	147	11 000	67.5
W360x51	3 000	605	539	460	2 000	592	1 440	463	M_r 277	3 000	252	7 000	96.9
W14x34	2 000	578	525	455	1 980	549	1 400	407	V_r 524	3 500	232	8 000	80.9
b = 171	1 500	521	479	423	1 490	515	1 380	368	L_u 2 320	4 000	210	9 000	69.4
t = 11.6	1 000	462	432	382	990	462	1 330	317	I_x 141	5 000	159	10 000	60.8
d = 355	500	392	366	332	495	367	1 230	245	S_x 796	6 000	121	11 000	54.1
W360x45	3 000	540	479	408	1 780	529	1 280	420	M_r 242	3 000	217	7 000	76.4
W14x30	2 000	520	469	405	1 780	493	1 250	371	V_r 498	3 500	197	8 000	63.3
b = 171	1 500	480	439	384	1 490	464	1 220	336	L_u 2 260	4 000	176	9 000	54.0
t = 9.8	1 000	422	393	344	990	418	1 190	289	I_x 122	5 000	128	10 000	47.1
d = 352	500	354	329	295	495	335	1 100	223	S_x 691	6 000	96.0	11 000	41.8
W360x39	3 000	475	419	355	1 550	466	1 110	376	M_r 206	2 000	193	6 000	54.1
W14x26	2 000	460	412	352	1 550	437	1 080	334	V_r 470	2 500	172	7 000	44.2
b = 128	1 500	440	400	346	1 490	413	1 060	303	L_u 1 660	3 000	148	8 000	37.4
t = 10.7	1 000	382	354	308	990	375	1 030	261	I_x 102	4 000	97.0	9 000	32.5
d = 353	500	317	292	259	495	302	962	200	S_x 580	5 000	69.7	10 000	28.7

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm $\quad$ F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 75 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

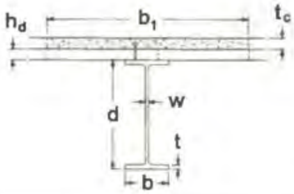
Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W360x33	2 500	395	349	294	1 290	383	920	308	M_r 168	2 000	155	6 000	38.0
W14x22	2 000	388	345	293	1 290	370	908	289	V_r 396	2 500	135	7 000	30.8
b = 127	1 500	378	340	292	1 290	352	893	264	L_u 1 600	3 000	113	8 000	25.8
t = 8.5	1 000	337	309	267	990	322	868	229	I_x 82.6	4 000	70.2	9 000	22.3
d = 349	500	275	253	221	495	264	814	175	S_x 473	5 000	49.6	10 000	19.6
W310x74	2 500	731	663	573	2 480	651	1 890	469	M_r 366	3 500	354	6 000	274
W12x50	2 000	673	616	543	1 980	616	1 860	432	V_r 597	4 000	339	7 000	240
b = 205	1 500	613	568	507	1 490	569	1 820	387	L_u 3 100	4 500	323	8 000	204
t = 16.3	1 000	550	518	465	990	499	1 740	330	I_x 164	5 000	307	9 000	177
d = 310	500	476	450	416	495	385	1 590	259	S_x 1 060	5 500	291	10 000	156
W310x67	2 500	680	613	524	2 480	588	1 700	430	M_r 326	3 500	312	6 000	234
W12x45	2 000	623	567	495	1 980	559	1 670	397	V_r 533	4 000	297	7 000	198
b = 204	1 500	563	520	462	1 490	518	1 630	355	L_u 3 020	4 500	282	8 000	167
t = 14.6	1 000	502	472	422	990	456	1 570	303	I_x 144	5 000	266	9 000	144
d = 306	500	432	407	375	495	354	1 440	236	S_x 942	5 500	250	10 000	127
W310x60	2 500	622	557	474	2 340	533	1 520	396	M_r 290	3 500	275	6 000	199
W12x40	2 000	577	522	452	1 980	507	1 500	366	V_r 466	4 000	261	7 000	163
b = 203	1 500	519	476	422	1 490	472	1 470	329	L_u 2 960	4 500	246	8 000	137
t = 13.1	1 000	459	429	384	990	419	1 420	281	I_x 128	5 000	231	9 000	118
d = 303	500	393	369	338	495	327	1 310	217	S_x 842	5 500	215	10 000	104
W310x52	2 500	574	512	434	2 070	507	1 380	383	M_r 260	3 000	240	6 000	130
W12x35	2 000	551	497	427	1 980	484	1 360	355	V_r 494	3 500	223	7 000	106
b = 167	1 500	494	452	397	1 490	453	1 330	319	L_u 2 380	4 000	206	8 000	89.4
t = 13.2	1 000	434	405	359	990	404	1 290	273	I_x 118	4 500	187	9 000	77.4
d = 317	500	368	344	312	495	318	1 190	210	S_x 747	5 000	167	10 000	68.4
W310x45	2 500	494	437	369	1 770	439	1 180	338	M_r 220	3 000	200	6 000	98.2
W12x30	2 000	482	431	367	1 770	421	1 170	315	V_r 423	3 500	184	7 000	79.3
b = 166	1 500	443	402	350	1 490	396	1 140	285	L_u 2 310	4 000	167	8 000	66.5
t = 11.2	1 000	385	357	315	990	356	1 110	244	I_x 99.2	4 500	150	9 000	57.3
d = 313	500	323	301	271	495	284	1 030	187	S_x 634	5 000	128	10 000	50.4
W310x39	2 500	432	380	320	1 530	386	1 030	303	M_r 189	3 000	170	6 000	77.7
W12x26	2 000	423	376	318	1 530	371	1 020	283	V_r 368	3 500	155	7 000	62.2
b = 165	1 500	405	365	313	1 490	351	998	257	L_u 2 260	4 000	139	8 000	51.8
t = 9.7	1 000	348	320	282	990	318	969	221	I_x 85.1	4 500	121	9 000	44.3
d = 310	500	288	269	240	495	256	905	169	S_x 549	5 000	103	10 000	38.8
W250x67	2 500	620	552	464	2 480	475	1 570	342	M_r 280	3 500	275	6 000	223
W10x45	2 000	562	506	434	1 980	450	1 540	314	V_r 469	4 000	265	6 500	212
b = 204	1 500	503	460	404	1 490	415	1 500	279	L_u 3 260	4 500	254	7 000	202
t = 15.7	1 000	442	412	368	990	364	1 440	236	I_x 104	5 000	244	7 500	192
d = 257	500	376	354	325	495	278	1 310	180	S_x 806	5 500	233	8 000	180

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

$F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 75 mm Slab
$\phi = 0.90, \phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

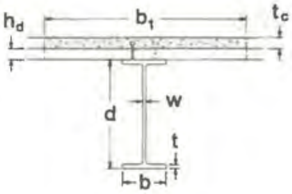
Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		L' mm	M_r' kN·m	L' mm	M_r' kN·m
W250x58	2 500	555	491	408	2 300	416	1 360	305	M_r 239	3 500	232	6 000	181
W10x39	2 000	513	458	388	1 980	395	1 340	281	V_r 413	4 000	222	6 500	171
b = 203	1 500	455	413	359	1 490	367	1 310	250	L_u 3 130	4 500	212	7 000	161
t = 13.5	1 000	395	366	325	990	324	1 260	212	I_x 87.3	5 000	202	7 500	148
d = 252	500	332	312	284	495	249	1 150	160	S_x 693	5 500	192	8 000	137
W250x45	2 500	455	398	329	1 780	357	1 090	272	M_r 187	3 000	167	5 500	101
W10x30	2 000	443	392	327	1 780	342	1 070	252	V_r 414	3 500	155	6 000	90.6
b = 148	1 500	403	362	309	1 490	320	1 050	226	L_u 2 170	4 000	142	6 500	82.2
t = 13	1 000	344	316	276	990	287	1 010	192	I_x 71.1	4 500	129	7 000	75.2
d = 266	500	282	262	235	495	226	934	144	S_x 534	5 000	114	7 500	69.3
W250x39	2 500	394	342	282	1 530	311	938	241	M_r 159	3 000	140	5 500	77.5
W10x26	2 000	385	338	280	1 530	298	925	225	V_r 354	3 500	128	6 000	69.2
b = 147	1 500	367	328	276	1 490	281	906	203	L_u 2 110	4 000	115	6 500	62.5
t = 11.2	1 000	310	282	245	990	254	878	173	I_x 60.1	4 500	102	7 000	57.0
d = 262	500	250	233	206	495	202	816	130	S_x 459	5 000	88.0	7 500	52.4
W250x33	2 500	336	290	237	1 290	265	796	210	M_r 132	3 000	112	5 500	55.6
W10x22	2 000	329	287	236	1 290	255	784	196	V_r 323	3 500	100	6 000	49.4
b = 146	1 500	319	281	235	1 290	241	769	178	L_u 2 020	4 000	88.4	6 500	44.4
t = 9.1	1 000	278	251	215	990	220	746	153	I_x 48.9	4 500	74.1	7 000	40.3
d = 258	500	219	203	178	495	178	697	115	S_x 379	5 000	63.6	7 500	36.9
W200x42	2 500	375	321	257	1 650	250	905	189	M_r 138	3 000	133	5 500	99.6
W8x28	2 000	365	316	255	1 650	239	890	175	V_r 302	3 500	126	6 000	92.9
b = 166	1 500	336	296	244	1 490	224	870	157	L_u 2 610	4 000	120	6 500	84.6
t = 11.8	1 000	278	250	215	990	200	840	132	I_x 40.9	4 500	113	7 000	77.5
d = 205	500	218	203	180	495	156	774	96.4	S_x 399	5 000	106	7 500	71.6
W200x36	2 500	325	276	219	1 420	218	783	168	M_r 118	3 000	112	5 500	79.3
W8x24	2 000	317	272	218	1 420	209	771	156	V_r 255	3 500	105	6 000	71.3
b = 165	1 500	305	266	216	1 420	196	754	140	L_u 2 510	4 000	99.0	6 500	64.6
t = 10.2	1 000	253	226	191	990	177	729	119	I_x 34.4	4 500	92.5	7 000	59.0
d = 201	500	194	180	158	495	140	677	87.0	S_x 342	5 000	85.9	7 500	54.4
W200x31	2 500	293	248	198	1 240	203	699	159	M_r 104	2 000	104	4 500	65.2
W8x21	2 000	287	245	197	1 240	195	688	148	V_r 275	2 500	96.7	5 000	57.0
b = 134	1 500	278	241	195	1 240	184	673	134	L_u 1 980	3 000	89.3	5 500	50.6
t = 10.2	1 000	241	214	179	990	167	652	115	I_x 31.4	3 500	81.7	6 000	45.6
d = 210	500	183	168	146	495	134	607	84.3	S_x 299	4 000	74.0	6 500	41.5
W200x27	2 500	250	210	167	1 050	174	595	139	M_r 86.6	2 000	85.3	4 500	47.5
W8x18	2 000	246	208	166	1 050	167	586	130	V_r 246	2 500	78.7	5 000	41.2
b = 133	1 500	239	205	165	1 050	159	574	119	L_u 1 890	3 000	71.5	5 500	36.4
t = 8.4	1 000	220	194	160	990	145	556	102	I_x 25.8	3 500	64.1	6 000	32.6
d = 207	500	163	149	128	495	118	521	75.6	S_x 249	4 000	56.0	6 500	29.6

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 90 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



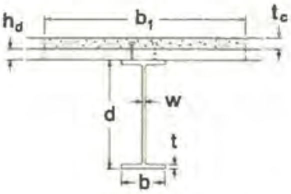
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

Steel section	b_1	M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
		Composite							**Non-composite**	L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W1000x249	7 000	5 810	5 550	5 070	8 320	13 200	14 300	10 100	M_r 3 510	4 000	3 440	14 000	831
W40x167	5 000	5 490	5 260	4 700	5 940	12 300	14 000	9 140	V_r 3 220	6 000	2 780	16 000	694
b = 300	3 000	5 050	4 720	4 250	3 560	10 800	13 500	7 850	L_u 3 740	8 000	1 940	18 000	596
t = 26	1 000	4 090	3 920	3 710	1 190	7 830	12 100	6 030	I_x 4 810	10 000	1 360	20 000	523
d = 980									S_x 9 820	12 000	1 030	22 000	466
W1000x222	7 000	5 260	5 010	4 560	8 320	11 700	12 600	9 020	M_r 3 040	4 000	2 940	14 000	634
W40x149	5 000	4 950	4 730	4 210	5 940	10 900	12 400	8 160	V_r 3 000	6 000	2 310	16 000	527
b = 300	3 000	4 530	4 220	3 760	3 560	9 650	11 900	6 980	L_u 3 590	8 000	1 520	18 000	451
t = 21.1	1 000	3 610	3 430	3 230	1 190	6 960	10 700	5 270	I_x 4 080	10 000	1 050	20 000	394
d = 970									S_x 8 410	12 000	794	22 000	350
W920x238	7 000	5 300	5 050	4 630	8 320	11 300	13 000	8 710	M_r 3 170	4 000	3 140	14 000	800
W36x160	5 000	4 990	4 780	4 320	5 940	10 600	12 800	7 880	V_r 3 090	6 000	2 590	16 000	668
b = 305	3 000	4 600	4 330	3 910	3 560	9 320	12 300	6 760	L_u 3 890	8 000	1 870	18 000	573
t = 25.9	1 000	3 760	3 600	3 420	1 190	6 740	11 000	5 150	I_x 4 060	10 000	1 310	20 000	502
d = 915									S_x 8 870	12 000	996	22 000	447
W920x223	7 000	5 030	4 790	4 390	8 320	10 700	12 200	8 250	M_r 2 960	4 500	2 800	12 000	881
W36x150	5 000	4 730	4 530	4 090	5 940	9 990	12 000	7 470	V_r 2 970	5 000	2 670	14 000	705
b = 304	3 000	4 360	4 100	3 690	3 560	8 820	11 600	6 390	L_u 3 830	6 000	2 380	16 000	587
t = 23.9	1 000	3 540	3 380	3 200	1 190	6 380	10 400	4 840	I_x 3 760	8 000	1 680	18 000	502
d = 911									S_x 8 260	10 000	1 170	20 000	439
W920x201	7 000	4 560	4 330	3 950	7 950	9 560	10 900	7 450	M_r 2 590	4 500	2 420	12 000	705
W36x135	5 000	4 290	4 110	3 700	5 940	8 970	10 700	6 750	V_r 2 710	5 000	2 300	14 000	560
b = 304	3 000	3 950	3 710	3 310	3 560	7 960	10 300	5 770	L_u 3 720	6 000	2 030	16 000	463
t = 20.1	1 000	3 170	3 010	2 830	1 190	5 750	9 250	4 300	I_x 3 250	8 000	1 360	18 000	394
d = 903									S_x 7 190	10 000	940	20 000	343
W840x210	7 000	4 520	4 290	3 930	8 320	9 010	10 900	6 990	M_r 2 620	4 500	2 460	12 000	792
W33x141	5 000	4 220	4 030	3 660	5 940	8 450	10 700	6 330	V_r 2 670	5 000	2 350	14 000	639
b = 293	3 000	3 890	3 670	3 300	3 560	7 470	10 400	5 420	L_u 3 770	6 000	2 090	16 000	535
t = 24.4	1 000	3 170	3 020	2 850	1 190	5 400	9 320	4 060	I_x 3 110	8 000	1 470	18 000	461
d = 846									S_x 7 340	10 000	1 040	20 000	404
W840x193	7 000	4 160	3 930	3 590	7 660	8 260	9 980	6 460	M_r 2 370	4 500	2 200	12 000	666
W33x130	5 000	3 920	3 740	3 390	5 940	7 760	9 810	5 860	V_r 2 530	5 000	2 090	14 000	534
b = 292	3 000	3 600	3 400	3 040	3 560	6 900	9 480	5 000	L_u 3 690	6 000	1 850	16 000	445
t = 21.7	1 000	2 910	2 760	2 600	1 190	4 990	8 520	3 710	I_x 2 780	8 000	1 260	18 000	382
d = 840									S_x 6 630	10 000	877	20 000	334
W840x176	7 000	3 790	3 570	3 240	6 960	7 510	9 020	5 930	M_r 2 110	4 500	1 950	12 000	551
W33x118	5 000	3 620	3 440	3 120	5 940	7 080	8 870	5 380	V_r 2 300	5 000	1 840	14 000	439
b = 292	3 000	3 300	3 120	2 780	3 560	6 320	8 590	4 600	L_u 3 610	6 000	1 610	16 000	364
t = 18.8	1 000	2 650	2 510	2 340	1 190	4 580	7 730	3 370	I_x 2 460	8 000	1 060	18 000	311
d = 835									S_x 5 900	10 000	731	20 000	271

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm $\quad F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 90 mm Slab
$\phi = 0.90, \phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W760x185	5 000	3 510	3 330	3 020	5 940	6 420	8 750	4 850	M_r 2 080	4 000	1 980	12 000	576
W30x124	4 000	3 350	3 200	2 880	4 750	6 120	8 630	4 520	V_r 2 340	5 000	1 780	14 000	470
b = 267	3 000	3 190	3 020	2 710	3 560	5 710	8 460	4 130	L_u 3 450	6 000	1 550	16 000	397
t = 23.6	2 000	2 960	2 770	2 510	2 380	5 110	8 170	3 650	I_x 2 230	8 000	1 040	18 000	344
d = 766	1 000	2 580	2 450	2 290	1 190	4 120	7 600	3 030	S_x 5 820	10 000	743	20 000	304
W760x173	5 000	3 330	3 150	2 860	5 940	6 040	8 200	4 590	M_r 1 930	4 000	1 830	12 000	506
W30x116	4 000	3 170	3 020	2 720	4 750	5 770	8 090	4 280	V_r 2 250	5 000	1 630	14 000	411
b = 267	3 000	3 010	2 860	2 550	3 560	5 390	7 930	3 910	L_u 3 410	6 000	1 410	16 000	346
t = 21.6	2 000	2 800	2 620	2 360	2 380	4 830	7 670	3 450	I_x 2 060	8 000	924	18 000	299
d = 762	1 000	2 430	2 300	2 150	1 190	3 900	7 140	2 850	S_x 5 400	10 000	657	20 000	264
W760x161	5 000	3 120	2 940	2 670	5 940	5 590	7 540	4 280	M_r 1 760	4 000	1 650	12 000	429
W30x108	4 000	2 970	2 820	2 540	4 750	5 350	7 450	3 990	V_r 2 140	5 000	1 460	14 000	347
b = 266	3 000	2 810	2 670	2 380	3 560	5 010	7 300	3 650	L_u 3 330	6 000	1 250	16 000	291
t = 19.3	2 000	2 610	2 440	2 190	2 380	4 500	7 070	3 210	I_x 1 860	8 000	793	18 000	251
d = 758	1 000	2 250	2 120	1 970	1 190	3 630	6 580	2 640	S_x 4 900	10 000	560	20 000	220
W760x147	5 000	2 890	2 730	2 470	5 820	5 130	6 880	3 950	M_r 1 580	4 000	1 470	12 000	358
W30x99	4 000	2 760	2 620	2 350	4 750	4 920	6 800	3 700	V_r 2 040	5 000	1 290	14 000	288
b = 265	3 000	2 600	2 470	2 190	3 560	4 620	6 670	3 370	L_u 3 260	6 000	1 090	16 000	241
t = 17	2 000	2 420	2 250	2 010	2 380	4 160	6 460	2 960	I_x 1 660	8 000	671	18 000	207
d = 753	1 000	2 070	1 950	1 800	1 190	3 360	6 020	2 420	S_x 4 410	10 000	470	20 000	181
W760x134	5 000	2 640	2 470	2 240	5 270	4 710	6 260	3 670	M_r 1 440	4 000	1 330	12 000	308
W30x90	4 000	2 550	2 410	2 180	4 750	4 520	6 180	3 440	V_r 1 650	5 000	1 160	14 000	246
b = 264	3 000	2 400	2 280	2 030	3 560	4 260	6 080	3 140	L_u 3 230	6 000	967	16 000	205
t = 15.5	2 000	2 230	2 090	1 850	2 380	3 850	5 900	2 760	I_x 1 500	8 000	587	18 000	175
d = 750	1 000	1 910	1 790	1 650	1 190	3 130	5 510	2 240	S_x 4 010	10 000	408	20 000	153
W690x192	5 000	3 360	3 180	2 890	5 940	5 750	8 510	4 310	M_r 2 010	4 000	1 910	12 000	634
W27x129	4 000	3 200	3 040	2 760	4 750	5 480	8 390	4 020	V_r 2 230	5 000	1 730	14 000	525
b = 254	3 000	3 030	2 890	2 600	3 560	5 100	8 210	3 670	L_u 3 440	6 000	1 540	16 000	449
t = 27.9	2 000	2 830	2 660	2 420	2 380	4 540	7 930	3 230	I_x 1 980	8 000	1 090	18 000	392
d = 702	1 000	2 480	2 360	2 210	1 190	3 660	7 370	2 690	S_x 5 640	10 000	802	20 000	349
W690x170	5 000	3 040	2 860	2 600	5 940	5 110	7 510	3 870	M_r 1 750	4 000	1 650	12 000	497
W27x114	4 000	2 890	2 740	2 480	4 750	4 870	7 410	3 610	V_r 2 060	5 000	1 480	14 000	408
b = 256	3 000	2 720	2 590	2 330	3 560	4 550	7 260	3 290	L_u 3 380	6 000	1 290	16 000	347
t = 23.6	2 000	2 540	2 380	2 150	2 380	4 080	7 020	2 900	I_x 1 700	8 000	875	18 000	302
d = 693	1 000	2 210	2 090	1 950	1 190	3 280	6 530	2 380	S_x 4 900	10 000	634	20 000	268
W690x152	5 000	2 780	2 610	2 370	5 940	4 620	6 740	3 550	M_r 1 550	4 000	1 460	12 000	406
W27x102	4 000	2 630	2 490	2 260	4 750	4 430	6 650	3 320	V_r 1 850	5 000	1 290	14 000	332
b = 254	3 000	2 480	2 360	2 120	3 560	4 150	6 530	3 030	L_u 3 320	6 000	1 110	16 000	281
t = 21.1	2 000	2 310	2 170	1 950	2 380	3 730	6 330	2 660	I_x 1 510	8 000	728	18 000	244
d = 688	1 000	2 010	1 890	1 760	1 190	3 020	5 910	2 180	S_x 4 380	10 000	523	20 000	216

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

$F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 90 mm Slab
ϕ = 0.90, ϕ_c = 0.65

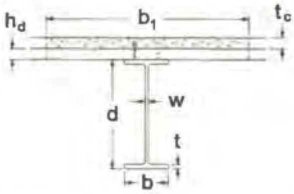
ASTM A992
A572 Grade 50
f'_c = 25 MPa
γ_c = 2350 kg/m^3

Steel section	b_1	Composite: M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite: Steel section data	Unbraced condition: L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W690x140	5 000	2 570	2 410	2 180	5 530	4 260	6 170	3 300	M_r 1 410	4 000	1 320	10 000	447
W27x94	4 000	2 460	2 320	2 100	4 750	4 090	6 100	3 080	V_r 1 740	5 000	1 160	12 000	345
b = 254	3 000	2 310	2 190	1 970	3 560	3 840	5 990	2 820	L_u 3 270	6 000	987	14 000	280
t = 18.9	2 000	2 140	2 020	1 810	2 380	3 460	5 810	2 470	I_x 1 360	7 000	778	16 000	236
d = 684	1 000	1 860	1 750	1 610	1 190	2 810	5 430	2 010	S_x 3 980	8 000	628	18 000	204
W690x125	5 000	2 320	2 160	1 940	4 970	3 820	5 510	2 990	M_r 1 250	4 000	1 140	10 000	362
W27x84	4 000	2 250	2 120	1 920	4 750	3 670	5 440	2 800	V_r 1 610	5 000	999	12 000	277
b = 253	3 000	2 100	2 000	1 790	3 560	3 460	5 350	2 560	L_u 3 190	6 000	834	14 000	223
t = 16.3	2 000	1 950	1 830	1 630	2 380	3 140	5 200	2 240	I_x 1 180	7 000	640	16 000	187
d = 678	1 000	1 680	1 580	1 440	1 190	2 550	4 860	1 810	S_x 3 500	8 000	513	18 000	161
W610x174	5 000	2 830	2 660	2 420	5 940	4 390	7 180	3 320	M_r 1 660	4 500	1 660	10 000	924
W24x117	4 000	2 680	2 540	2 320	4 750	4 190	7 090	3 100	V_r 1 770	5 000	1 610	12 000	709
b = 325	3 000	2 520	2 410	2 190	3 560	3 910	6 950	2 820	L_u 4 480	6 000	1 490	14 000	574
t = 21.6	2 000	2 360	2 240	2 040	2 380	3 500	6 730	2 480	I_x 1 470	7 000	1 370	16 000	482
d = 616	1 000	2 090	1 980	1 860	1 190	2 810	6 270	2 050	S_x 4 780	8 000	1 230	18 000	415
W610x155	5 000	2 580	2 420	2 190	5 940	3 950	6 390	3 020	M_r 1 470	4 500	1 460	10 000	762
W24x104	4 000	2 440	2 300	2 100	4 750	3 780	6 310	2 820	V_r 1 590	5 000	1 410	12 000	579
b = 324	3 000	2 280	2 170	1 980	3 560	3 540	6 200	2 570	L_u 4 400	6 000	1 300	14 000	465
t = 19	2 000	2 120	2 020	1 830	2 380	3 180	6 010	2 260	I_x 1 290	7 000	1 180	16 000	388
d = 611	1 000	1 880	1 780	1 660	1 190	2 570	5 620	1 850	S_x 4 220	8 000	1 050	18 000	333
W610x140	5 000	2 390	2 230	2 000	5 540	3 620	5 740	2 790	M_r 1 290	4 000	1 170	10 000	422
W24x94	4 000	2 280	2 140	1 930	4 750	3 470	5 670	2 610	V_r 1 660	5 000	1 030	12 000	334
b = 230	3 000	2 120	2 010	1 810	3 560	3 260	5 570	2 380	L_u 3 070	6 000	874	14 000	277
t = 22.2	2 000	1 960	1 850	1 660	2 380	2 930	5 400	2 080	I_x 1 120	7 000	695	16 000	237
d = 617	1 000	1 700	1 610	1 480	1 190	2 370	5 030	1 680	S_x 3 630	8 000	573	18 000	207
W610x125	5 000	2 140	1 990	1 780	4 950	3 260	5 130	2 540	M_r 1 140	4 000	1 020	10 000	342
W24x84	4 000	2 080	1 950	1 760	4 750	3 130	5 070	2 380	V_r 1 490	5 000	889	12 000	269
b = 229	3 000	1 930	1 830	1 650	3 560	2 950	4 980	2 170	L_u 3 020	6 000	733	14 000	222
t = 19.6	2 000	1 780	1 680	1 500	2 380	2 670	4 840	1 900	I_x 985	7 000	575	16 000	189
d = 612	1 000	1 550	1 450	1 330	1 190	2 170	4 520	1 530	S_x 3 220	8 000	470	18 000	165
W610x113	5 000	1 950	1 800	1 610	4 490	2 960	4 640	2 340	M_r 1 020	4 000	906	10 000	282
W24x76	4 000	1 910	1 780	1 600	4 490	2 850	4 590	2 190	V_r 1 400	5 000	775	12 000	220
b = 228	3 000	1 790	1 680	1 510	3 560	2 700	4 510	2 000	L_u 2 950	6 000	617	14 000	180
t = 17.3	2 000	1 630	1 550	1 380	2 380	2 450	4 390	1 750	I_x 875	7 000	481	16 000	153
d = 608	1 000	1 420	1 330	1 210	1 190	2 000	4 110	1 400	S_x 2 880	8 000	391	18 000	133
W610x101	5 000	1 750	1 610	1 430	4 020	2 660	4 140	2 120	M_r 900	4 000	787	10 000	228
W24x68	4 000	1 720	1 600	1 430	4 020	2 570	4 100	1 990	V_r 1 300	5 000	664	12 000	176
b = 228	3 000	1 640	1 540	1 380	3 560	2 430	4 030	1 830	L_u 2 890	6 000	512	14 000	144
t = 14.9	2 000	1 490	1 410	1 250	2 380	2 220	3 930	1 600	I_x 764	7 000	396	16 000	121
d = 603	1 000	1 290	1 200	1 090	1 190	1 820	3 690	1 270	S_x 2 530	8 000	320	18 000	105

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm $\quad$ F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 90 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



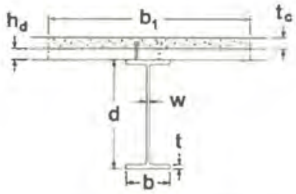
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m³

Steel section	b_1	Composite: M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite: Steel section data	Unbraced condition: L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm⁴	mm³	mm⁴		mm	kN·m	mm	kN·m
W610x92	4 000	1 580	1 450	1 280	3 650	2 320	3 660	1 810	M_r 779	3 000	683	8 000	183
W24x62	3 000	1 530	1 430	1 270	3 560	2 200	3 600	1 660	V_r 1 350	4 000	540	10 000	135
b = 179	2 000	1 380	1 300	1 130	2 380	2 020	3 510	1 450	L_u 2 180	5 000	376	12 000	107
t = 15	1 000	1 170	1 080	967	1 190	1 660	3 290	1 140	I_x 646	6 000	281	14 000	88.9
d = 603	500	996	939	872	594	1 310	3 010	929	S_x 2 140	7 000	222	16 000	76.1
W610x82	4 000	1 400	1 290	1 130	3 240	2 070	3 240	1 640	M_r 683	3 000	587	8 000	145
W24x55	3 000	1 370	1 270	1 120	3 240	1 970	3 200	1 510	V_r 1 170	4 000	448	10 000	106
b = 178	2 000	1 250	1 180	1 030	2 380	1 820	3 110	1 320	L_u 2 110	5 000	304	12 000	83.4
t = 12.8	1 000	1 060	980	867	1 190	1 500	2 930	1 040	I_x 560	6 000	225	14 000	68.9
d = 599	500	895	839	773	594	1 190	2 690	835	S_x 1 870	7 000	177	16 000	58.7
W530x138	4 000	2 070	1 930	1 720	4 750	2 840	5 120	2 120	M_r 1 120	3 000	1 110	8 000	515
W21x93	3 000	1 920	1 800	1 610	3 560	2 660	5 020	1 920	V_r 1 650	4 000	1 000	10 000	390
b = 214	2 000	1 750	1 650	1 470	2 380	2 390	4 850	1 670	L_u 2 930	5 000	884	12 000	314
t = 23.6	1 000	1 510	1 420	1 300	1 190	1 910	4 500	1 340	I_x 861	6 000	759	14 000	263
d = 549	500	1 330	1 270	1 210	594	1 510	4 110	1 120	S_x 3 140	7 000	616	16 000	227
W530x123	4 000	1 900	1 760	1 570	4 750	2 560	4 580	1 940	M_r 997	3 000	984	8 000	421
W21x83	3 000	1 750	1 640	1 470	3 560	2 410	4 490	1 760	V_r 1 460	4 000	879	10 000	316
b = 212	2 000	1 590	1 500	1 330	2 380	2 180	4 360	1 530	L_u 2 860	5 000	762	12 000	253
t = 21.2	1 000	1 370	1 280	1 170	1 190	1 760	4 060	1 220	I_x 761	6 000	631	14 000	211
d = 544	500	1 200	1 150	1 080	594	1 390	3 720	1 010	S_x 2 800	7 000	505	16 000	182
W530x109	4 000	1 700	1 570	1 390	4 310	2 290	4 060	1 760	M_r 879	3 000	862	8 000	342
W21x73	3 000	1 590	1 480	1 330	3 560	2 170	3 990	1 600	V_r 1 280	4 000	764	10 000	254
b = 211	2 000	1 430	1 360	1 210	2 380	1 970	3 880	1 400	L_u 2 810	5 000	652	12 000	202
t = 18.8	1 000	1 240	1 160	1 050	1 190	1 600	3 630	1 110	I_x 667	6 000	520	14 000	168
d = 539	500	1 080	1 030	963	594	1 260	3 330	914	S_x 2 480	7 000	413	16 000	144
W530x101	4 000	1 590	1 460	1 290	4 010	2 150	3 780	1 660	M_r 814	3 000	794	8 000	301
W21x68	3 000	1 500	1 400	1 250	3 560	2 040	3 720	1 520	V_r 1 200	4 000	699	10 000	222
b = 210	2 000	1 350	1 280	1 140	2 380	1 860	3 620	1 330	L_u 2 770	5 000	591	12 000	176
t = 17.4	1 000	1 170	1 090	987	1 190	1 520	3 400	1 050	I_x 617	6 000	462	14 000	146
d = 537	500	1 010	960	899	594	1 200	3 120	860	S_x 2 300	7 000	365	16 000	125
W530x92	4 000	1 450	1 330	1 170	3 660	1 970	3 440	1 540	M_r 733	3 000	711	8 000	253
W21x62	3 000	1 400	1 300	1 160	3 560	1 870	3 390	1 410	V_r 1 110	4 000	621	9 000	214
b = 209	2 000	1 250	1 180	1 050	2 380	1 710	3 300	1 230	L_u 2 720	5 000	516	10 000	185
t = 15.6	1 000	1 080	1 010	905	1 190	1 400	3 110	972	I_x 552	6 000	393	12 000	146
d = 533	500	929	879	818	594	1 110	2 860	790	S_x 2 070	7 000	309	14 000	120
W530x82	4 000	1 290	1 180	1 040	3 250	1 750	3 050	1 390	M_r 640	3 000	616	8 000	203
W21x55	3 000	1 260	1 160	1 030	3 250	1 670	3 000	1 270	V_r 1 030	4 000	531	9 000	170
b = 209	2 000	1 140	1 070	951	2 380	1 530	2 930	1 120	L_u 2 660	5 000	433	10 000	147
t = 13.3	1 000	975	908	809	1 190	1 270	2 760	879	I_x 477	6 000	320	12 000	115
d = 528	500	832	783	724	594	1 010	2 550	708	S_x 1 810	7 000	249	14 000	94.0

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm⁴, S_x - 10^3 mm³, b - mm, t - mm, d - mm $\qquad F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 90 mm Slab
ϕ = 0.90, ϕ_c = 0.65



ASTM A992
A572 Grade 50
f'_c = 25 MPa
γ_c = 2350 kg/m^3

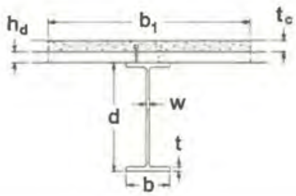
Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W530x74	4 000	1 190	1 080	938	2 960	1 600	2 750	1 270	M_r 562	3 000	474	8 000	123
W21x50	3 000	1 160	1 070	933	2 960	1 520	2 710	1 170	V_r 1 050	4 000	357	9 000	105
b = 166	2 000	1 070	996	874	2 380	1 400	2 640	1 030	L_u 2 040	5 000	247	10 000	91.7
t = 13.6	1 000	899	830	731	1 190	1 170	2 490	804	I_x 411	6 000	186	12 000	73.2
d = 529	500	754	705	646	594	926	2 290	639	S_x 1 550	7 000	148	14 000	61.0
W530x66	4 000	1 050	948	819	2 600	1 410	2 410	1 140	M_r 484	3 000	398	8 000	94.9
W21x44	3 000	1 030	938	815	2 600	1 350	2 380	1 050	V_r 927	4 000	284	9 000	80.6
b = 165	2 000	967	900	789	2 380	1 250	2 320	927	L_u 1 980	5 000	195	10 000	70.0
t = 11.4	1 000	810	747	652	1 190	1 050	2 190	726	I_x 351	6 000	145	12 000	55.5
d = 525	500	674	626	568	594	837	2 030	572	S_x 1 340	7 000	115	14 000	46.0
W460x158	4 000	2 050	1 900	1 710	4 750	2 590	5 360	1 890	M_r 1 170	4 500	1 150	9 000	794
W18x106	3 000	1 890	1 780	1 610	3 560	2 410	5 250	1 710	V_r 1 460	5 000	1 110	10 000	696
b = 284	2 000	1 730	1 640	1 490	2 380	2 150	5 070	1 490	L_u 4 200	6 000	1 040	11 000	617
t = 23.9	1 000	1 520	1 440	1 340	1 190	1 710	4 690	1 200	I_x 796	7 000	955	12 000	555
d = 476	500	1 360	1 310	1 260	594	1 350	4 290	1 010	S_x 3 350	8 000	875	14 000	462
W460x144	4 000	1 920	1 780	1 590	4 750	2 390	4 930	1 770	M_r 1 070	4 500	1 050	9 000	693
W18x97	3 000	1 760	1 650	1 500	3 560	2 240	4 830	1 610	V_r 1 320	5 000	1 010	10 000	602
b = 283	2 000	1 600	1 520	1 380	2 380	2 000	4 670	1 390	L_u 4 130	6 000	936	11 000	533
t = 22.1	1 000	1 410	1 340	1 240	1 190	1 600	4 350	1 120	I_x 726	7 000	858	12 000	478
d = 472	500	1 260	1 210	1 160	594	1 260	3 980	938	S_x 3 080	8 000	779	14 000	396
W460x128	4 000	1 760	1 620	1 440	4 750	2 150	4 390	1 610	M_r 947	4 500	917	9 000	566
W18x86	3 000	1 610	1 500	1 350	3 560	2 020	4 310	1 470	V_r 1 170	5 000	884	10 000	489
b = 282	2 000	1 450	1 370	1 250	2 380	1 820	4 180	1 270	L_u 4 040	6 000	812	11 000	431
t = 19.6	1 000	1 270	1 200	1 110	1 190	1 460	3 900	1 010	I_x 637	7 000	736	12 000	385
d = 467	500	1 130	1 080	1 030	594	1 150	3 580	844	S_x 2 730	8 000	658	14 000	318
W460x113	4 000	1 580	1 450	1 280	4 470	1 920	3 880	1 460	M_r 829	4 500	796	9 000	458
W18x76	3 000	1 460	1 360	1 220	3 560	1 810	3 810	1 330	V_r 1 020	5 000	765	10 000	394
b = 280	2 000	1 310	1 230	1 120	2 380	1 640	3 700	1 160	L_u 3 950	6 000	696	11 000	345
t = 17.3	1 000	1 140	1 080	989	1 190	1 330	3 470	919	I_x 556	7 000	623	12 000	307
d = 463	500	1 010	965	911	594	1 050	3 190	758	S_x 2 400	8 000	545	14 000	252
W460x106	4 000	1 510	1 380	1 200	4 180	1 810	3 580	1 380	M_r 742	3 000	719	8 000	308
W18x71	3 000	1 410	1 300	1 150	3 560	1 710	3 520	1 260	V_r 1 210	4 000	637	9 000	266
b = 194	2 000	1 250	1 180	1 040	2 380	1 550	3 410	1 090	L_u 2 690	5 000	549	10 000	235
t = 20.6	1 000	1 070	1 000	904	1 190	1 250	3 190	852	I_x 488	6 000	450	11 000	210
d = 469	500	925	879	823	594	978	2 910	692	S_x 2 080	7 000	366	12 000	190
W460x97	4 000	1 380	1 260	1 100	3 820	1 670	3 280	1 290	M_r 677	3 000	652	8 000	264
W18x65	3 000	1 320	1 220	1 070	3 560	1 580	3 230	1 180	V_r 1 090	4 000	574	9 000	227
b = 193	2 000	1 170	1 090	972	2 380	1 440	3 140	1 020	L_u 2 650	5 000	488	10 000	200
t = 19	1 000	995	930	837	1 190	1 170	2 940	799	I_x 445	6 000	389	11 000	178
d = 466	500	858	813	758	594	919	2 690	645	S_x 1 910	7 000	314	12 000	161

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 90 mm Slab
$\phi = 0.90, \phi_c = 0.65$



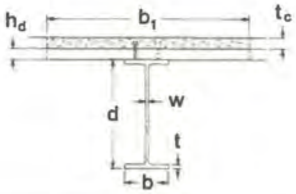
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W460x89	3 000	1 240	1 140	1 010	3 530	1 470	2 990	1 110	M_r 624	3 000	598	8 000	231
W18x60	2 000	1 100	1 020	913	2 380	1 350	2 910	964	V_r 996	4 000	523	9 000	198
b = 192	1 500	1 020	960	853	1 780	1 250	2 850	870	L_u 2 620	5 000	439	10 000	174
t = 17.7	1 000	933	873	783	1 190	1 100	2 740	754	I_x 409	6 000	343	11 000	155
d = 463	500	803	759	705	594	867	2 510	605	S_x 1 770	7 000	276	12 000	140
W460x82	3 000	1 150	1 050	922	3 240	1 360	2 750	1 030	M_r 568	3 000	540	8 000	195
W18x55	2 000	1 030	957	851	2 380	1 250	2 680	902	V_r 933	4 000	466	9 000	166
b = 191	1 500	950	894	793	1 780	1 160	2 620	814	L_u 2 560	5 000	384	10 000	146
t = 16	1 000	868	812	725	1 190	1 030	2 520	704	I_x 370	6 000	292	11 000	129
d = 460	500	744	701	648	594	812	2 320	562	S_x 1 610	7 000	234	12 000	116
W460x74	3 000	1 050	953	833	2 930	1 240	2 490	956	M_r 512	3 000	484	8 000	164
W18x50	2 000	954	886	787	2 380	1 150	2 430	838	V_r 843	4 000	414	9 000	140
b = 190	1 500	879	825	732	1 780	1 070	2 380	757	L_u 2 530	5 000	332	10 000	122
t = 14.5	1 000	800	750	667	1 190	953	2 300	655	I_x 332	6 000	249	11 000	108
d = 457	500	685	644	592	594	755	2 120	519	S_x 1 460	7 000	198	12 000	96.8
W460x68	3 000	976	886	770	2 710	1 160	2 300	898	M_r 463	3 000	390	8 000	112
W18x46	2 000	906	838	739	2 380	1 070	2 240	788	V_r 856	4 000	301	9 000	96.7
b = 154	1 500	831	777	683	1 780	1 000	2 190	712	L_u 2 010	5 000	213	10 000	85.2
t = 15.4	1 000	752	701	617	1 190	896	2 120	614	I_x 297	6 000	164	11 000	76.1
d = 459	500	635	594	542	594	710	1 950	483	S_x 1 290	7 000	133	12 000	68.9
W460x60	3 000	854	771	667	2 350	1 020	2 000	804	M_r 397	3 000	329	8 000	86.6
W18x40	2 000	819	754	662	2 350	949	1 960	710	V_r 746	4 000	242	9 000	74.3
b = 153	1 500	748	696	613	1 780	891	1 920	643	L_u 1 970	5 000	169	10 000	65.2
t = 13.3	1 000	671	628	551	1 190	802	1 860	556	I_x 255	6 000	129	11 000	58.1
d = 455	500	567	528	478	594	641	1 720	434	S_x 1 120	7 000	104	12 000	52.4
W460x52	3 000	749	672	577	2 060	889	1 740	711	M_r 338	3 000	269	8 000	63.6
W18x35	2 000	722	659	573	2 060	831	1 700	631	V_r 680	4 000	185	9 000	54.3
b = 152	1 500	676	626	547	1 780	784	1 670	573	L_u 1 890	5 000	128	10 000	47.4
t = 10.8	1 000	601	562	488	1 190	709	1 610	495	I_x 212	6 000	96.2	11 000	42.0
d = 450	500	503	465	416	594	571	1 500	384	S_x 942	7 000	76.7	12 000	37.8
W410x149	3 000	1 680	1 570	1 410	3 560	1 990	4 660	1 410	M_r 1 010	4 500	983	8 000	760
W16x100	2 000	1 520	1 440	1 300	2 380	1 770	4 500	1 220	V_r 1 320	5 000	952	9 000	696
b = 265	1 500	1 430	1 360	1 240	1 780	1 620	4 370	1 100	L_u 4 080	5 500	921	10 000	621
t = 25	1 000	1 330	1 260	1 160	1 190	1 400	4 160	966	I_x 618	6 000	889	11 000	554
d = 431	500	1 180	1 140	1 090	594	1 100	3 780	807	S_x 2 870	7 000	825	12 000	501
W410x132	3 000	1 530	1 420	1 270	3 560	1 790	4 140	1 280	M_r 885	4 500	853	8 000	635
W16x89	2 000	1 370	1 290	1 170	2 380	1 600	4 000	1 110	V_r 1 160	5 000	823	9 000	565
b = 263	1 500	1 290	1 220	1 110	1 780	1 460	3 890	999	L_u 3 940	5 500	792	10 000	495
t = 22.2	1 000	1 190	1 130	1 040	1 190	1 280	3 720	873	I_x 538	6 000	761	11 000	440
d = 425	500	1 060	1 010	963	594	996	3 390	722	S_x 2 530	7 000	698	12 000	397

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 90 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

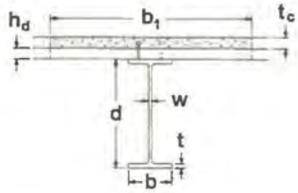
Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W410x114	3 000	1 380	1 270	1 130	3 560	1 580	3 610	1 150	M_r 764	4 500	726	8 000	513
W16x77	2 000	1 220	1 150	1 040	2 380	1 430	3 500	997	V_r 998	5 000	698	9 000	438
b = 261	1 500	1 140	1 080	980	1 780	1 310	3 410	899	L_u 3 810	5 500	668	10 000	382
t = 19.3	1 000	1 060	998	913	1 190	1 150	3 270	782	I_x 461	6 000	638	11 000	338
d = 420	500	931	890	839	594	896	2 990	639	S_x 2 200	7 000	576	12 000	304
W410x100	3 000	1 240	1 140	1 010	3 560	1 400	3 160	1 040	M_r 661	4 500	623	8 000	411
W16x67	2 000	1 090	1 020	924	2 380	1 270	3 070	901	V_r 850	5 000	596	9 000	348
b = 260	1 500	1 020	961	871	1 780	1 170	3 000	813	L_u 3 730	5 500	568	10 000	302
t = 16.9	1 000	936	887	808	1 190	1 030	2 880	705	I_x 398	6 000	539	11 000	266
d = 415	500	825	786	737	594	810	2 650	571	S_x 1 920	7 000	479	12 000	238
W410x85	3 000	1 110	1 010	882	3 360	1 210	2 670	911	M_r 534	3 000	507	8 000	205
W16x57	2 000	982	910	805	2 380	1 110	2 590	791	V_r 931	4 000	443	9 000	177
b = 181	1 500	903	847	749	1 780	1 030	2 530	712	L_u 2 530	5 000	375	10 000	157
t = 18.2	1 000	821	767	685	1 190	908	2 430	613	I_x 315	6 000	297	11 000	140
d = 417	500	702	662	612	594	709	2 230	485	S_x 1 510	7 000	242	12 000	127
W410x74	3 000	990	896	775	2 960	1 080	2 360	827	M_r 469	3 000	440	8 000	163
W16x50	2 000	895	826	729	2 380	997	2 300	722	V_r 821	4 000	379	9 000	140
b = 180	1 500	819	765	677	1 780	928	2 250	650	L_u 2 470	5 000	312	10 000	123
t = 16	1 000	740	693	616	1 190	824	2 170	559	I_x 275	6 000	239	11 000	110
d = 413	500	632	593	545	594	648	1 990	440	S_x 1 330	7 000	194	12 000	99.7
W410x67	3 000	898	808	697	2 670	986	2 130	762	M_r 422	3 000	392	8 000	135
W16x45	2 000	832	765	672	2 380	910	2 080	668	V_r 739	4 000	333	9 000	116
b = 179	1 500	757	705	624	1 780	851	2 040	603	L_u 2 420	5 000	264	10 000	102
t = 14.4	1 000	680	638	565	1 190	759	1 960	519	I_x 245	6 000	201	11 000	90.4
d = 410	500	580	543	495	594	600	1 810	406	S_x 1 200	7 000	161	12 000	81.6
W410x60	3 000	797	714	614	2 350	882	1 890	693	M_r 369	3 000	341	8 000	109
W16x40	2 000	762	697	608	2 350	819	1 850	611	V_r 642	4 000	286	9 000	93.1
b = 178	1 500	691	640	566	1 780	769	1 810	553	L_u 2 390	5 000	218	10 000	81.3
t = 12.8	1 000	615	578	511	1 190	691	1 750	477	I_x 216	6 000	165	11 000	72.1
d = 407	500	524	490	444	594	551	1 630	372	S_x 1 060	7 000	131	12 000	64.9
W410x54	3 000	718	640	548	2 110	790	1 690	628	M_r 326	3 000	295	8 000	86.0
W16x36	2 000	690	626	544	2 110	736	1 650	555	V_r 619	4 000	241	9 000	73.1
b = 177	1 500	639	589	517	1 780	693	1 620	504	L_u 2 310	5 000	176	10 000	63.5
t = 10.9	1 000	564	529	464	1 190	626	1 570	434	I_x 186	6 000	132	11 000	56.2
d = 403	500	476	443	398	594	502	1 460	336	S_x 923	7 000	104	12 000	50.4
W410x46	3 000	628	557	474	1 830	693	1 460	560	M_r 274	2 000	265	7 000	61.7
W16x31	2 000	607	547	470	1 830	649	1 430	498	V_r 578	3 000	210	8 000	51.8
b = 140	1 500	582	533	464	1 780	614	1 400	453	L_u 1 790	4 000	142	9 000	44.6
t = 11.2	1 000	509	474	412	1 190	558	1 360	392	I_x 156	5 000	99.9	10 000	39.2
d = 403	500	423	391	347	594	452	1 270	302	S_x 772	6 000	76.4	11 000	35.0

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

$F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 90 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

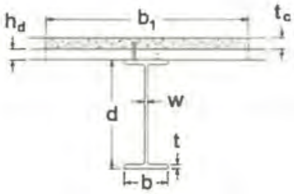
Steel section	b_1	Composite M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite Steel section data	Unbraced condition L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W410x39	3 000	535	472	398	1 550	590	1 240	484	M_r 227	2 000	216	7 000	44.0
W16x26	2 000	519	465	396	1 550	555	1 210	434	V_r 480	3 000	166	8 000	36.5
b = 140	1 500	504	457	394	1 550	527	1 190	397	L_u 1 730	4 000	105	9 000	31.2
t = 8.8	1 000	451	417	361	1 190	483	1 150	344	I_x 126	5 000	73.0	10 000	27.3
d = 399	500	371	341	298	594	395	1 080	264	S_x 634	6 000	55.1	11 000	24.2
W360x79	3 000	948	852	728	3 130	928	2 300	697	M_r 444	3 500	425	7 000	267
W14x53	2 000	838	768	676	2 380	849	2 230	605	V_r 682	4 000	404	8 000	225
b = 205	1 500	761	707	630	1 780	787	2 180	543	L_u 3 010	4 500	383	9 000	194
t = 16.8	1 000	682	643	575	1 190	695	2 100	465	I_x 226	5 000	361	10 000	171
d = 354	500	588	554	510	594	541	1 930	364	S_x 1 280	6 000	317	11 000	153
W360x72	3 000	860	769	654	2 830	842	2 080	642	M_r 397	3 500	377	7 000	222
W14x48	2 000	779	711	621	2 380	774	2 020	559	V_r 617	4 000	357	8 000	186
b = 204	1 500	704	651	578	1 780	721	1 980	503	L_u 2 940	4 500	336	9 000	160
t = 15.1	1 000	627	589	525	1 190	640	1 910	431	I_x 201	5 000	315	10 000	141
d = 350	500	537	505	462	594	501	1 760	335	S_x 1 150	6 000	272	11 000	126
W360x64	3 000	775	688	583	2 530	761	1 860	588	M_r 354	3 500	332	7 000	183
W14x43	2 000	724	658	568	2 380	703	1 820	515	V_r 548	4 000	313	8 000	153
b = 203	1 500	650	598	529	1 780	657	1 780	464	L_u 2 870	4 500	293	9 000	131
t = 13.5	1 000	574	538	479	1 190	586	1 720	398	I_x 178	5 000	273	10 000	115
d = 347	500	490	459	418	594	462	1 590	308	S_x 1 030	6 000	228	11 000	102
W360x57	3 000	707	626	530	2 240	710	1 680	557	M_r 314	3 000	289	7 000	119
W14x38	2 000	675	611	525	2 240	659	1 630	490	V_r 580	3 500	267	8 000	99.7
b = 172	1 500	614	563	493	1 780	618	1 600	442	L_u 2 360	4 000	244	9 000	85.9
t = 13.1	1 000	538	503	443	1 190	555	1 550	380	I_x 160	5 000	192	10 000	75.6
d = 358	500	454	423	382	594	441	1 430	292	S_x 896	6 000	147	11 000	67.5
W360x51	3 000	635	560	472	2 000	639	1 500	508	M_r 277	3 000	252	7 000	96.9
W14x34	2 000	609	547	468	2 000	596	1 460	449	V_r 524	3 500	232	8 000	80.9
b = 171	1 500	569	519	452	1 780	561	1 430	407	L_u 2 320	4 000	210	9 000	69.4
t = 11.6	1 000	495	460	405	1 190	506	1 390	350	I_x 141	5 000	159	10 000	60.8
d = 355	500	414	385	345	594	406	1 290	268	S_x 796	6 000	121	11 000	54.1
W360x45	3 000	567	498	419	1 780	571	1 340	459	M_r 242	3 000	217	7 000	76.4
W14x30	2 000	547	488	416	1 780	534	1 300	409	V_r 498	3 500	197	8 000	63.3
b = 171	1 500	527	478	412	1 780	505	1 280	371	L_u 2 260	4 000	176	9 000	54.0
t = 9.8	1 000	454	420	367	1 190	458	1 240	320	I_x 122	5 000	128	10 000	47.1
d = 352	500	376	348	309	594	370	1 160	245	S_x 691	6 000	96.0	11 000	41.8
W360x39	3 000	498	435	364	1 550	504	1 160	411	M_r 206	2 000	193	6 000	54.1
W14x26	2 000	483	428	362	1 550	473	1 130	368	V_r 470	2 500	172	7 000	44.2
b = 128	1 500	468	421	359	1 550	448	1 110	335	L_u 1 660	3 000	148	8 000	37.4
t = 10.7	1 000	415	381	330	1 190	410	1 080	290	I_x 102	4 000	97.0	9 000	32.5
d = 353	500	337	311	272	594	334	1 010	221	S_x 580	5 000	69.7	10 000	28.7

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

$F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 90 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

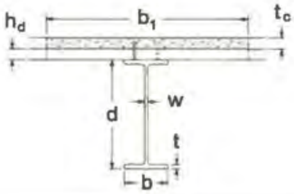
Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W360x33	2 500	414	362	302	1 290	415	964	337	M_r 168	2 000	155	6 000	38.0
W14x22	2 000	408	359	301	1 290	401	951	318	V_r 396	2 500	135	7 000	30.8
b = 127	1 500	397	354	299	1 290	382	933	292	L_u 1 600	3 000	113	8 000	25.8
t = 8.5	1 000	368	336	289	1 190	351	907	254	I_x 82.6	4 000	70.2	9 000	22.3
d = 349	500	294	271	234	594	291	853	194	S_x 473	5 000	49.6	10 000	19.6
W310x74	2 500	807	725	614	2 930	712	1 980	521	M_r 366	3 500	354	6 000	274
W12x50	2 000	738	670	578	2 380	676	1 950	481	V_r 597	4 000	339	7 000	240
b = 205	1 500	663	609	538	1 780	627	1 900	430	L_u 3 100	4 500	323	8 000	204
t = 16.3	1 000	585	548	488	1 190	553	1 830	366	I_x 164	5 000	307	9 000	177
d = 310	500	498	469	429	594	428	1 680	281	S_x 1 060	5 500	291	10 000	156
W310x67	2 500	730	651	548	2 620	643	1 780	478	M_r 326	3 500	312	6 000	234
W12x45	2 000	686	620	530	2 380	612	1 750	441	V_r 533	4 000	297	7 000	198
b = 204	1 500	612	560	491	1 780	570	1 710	396	L_u 3 020	4 500	282	8 000	167
t = 14.6	1 000	536	500	445	1 190	506	1 650	337	I_x 144	5 000	266	9 000	144
d = 306	500	453	426	388	594	394	1 520	257	S_x 942	5 500	250	10 000	127
W310x60	2 500	657	582	488	2 340	581	1 600	439	M_r 290	3 500	275	6 000	199
W12x40	2 000	637	572	484	2 340	555	1 570	407	V_r 466	4 000	261	7 000	163
b = 203	1 500	567	516	449	1 780	519	1 540	366	L_u 2 960	4 500	246	8 000	137
t = 13.1	1 000	492	456	406	1 190	463	1 490	312	I_x 128	5 000	231	9 000	118
d = 303	500	413	388	351	594	364	1 370	238	S_x 842	5 500	215	10 000	104
W310x52	2 500	605	534	447	2 070	552	1 440	423	M_r 260	3 000	240	6 000	130
W12x35	2 000	589	526	444	2 070	528	1 420	394	V_r 494	3 500	223	7 000	106
b = 167	1 500	542	491	424	1 780	496	1 390	355	L_u 2 380	4 000	206	8 000	89.4
t = 13.2	1 000	467	432	381	1 190	446	1 350	304	I_x 118	4 500	187	9 000	77.4
d = 317	500	388	362	325	594	354	1 250	231	S_x 747	5 000	167	10 000	68.4
W310x45	2 500	520	456	380	1 770	477	1 240	373	M_r 220	3 000	200	6 000	98.2
W12x30	2 000	509	450	378	1 770	458	1 220	349	V_r 423	3 500	184	7 000	79.3
b = 166	1 500	489	440	375	1 770	433	1 200	316	L_u 2 310	4 000	167	8 000	66.5
t = 11.2	1 000	417	383	336	1 190	392	1 160	272	I_x 99.2	4 500	150	9 000	57.3
d = 313	500	341	319	284	594	315	1 080	206	S_x 634	5 000	128	10 000	50.4
W310x39	2 500	455	396	329	1 530	419	1 080	334	M_r 189	3 000	170	6 000	77.7
W12x26	2 000	446	392	327	1 530	404	1 070	313	V_r 368	3 500	155	7 000	62.2
b = 165	1 500	431	384	325	1 530	383	1 050	285	L_u 2 260	4 000	139	8 000	51.8
t = 9.7	1 000	380	346	301	1 190	349	1 010	247	I_x 85.1	4 500	121	9 000	44.3
d = 310	500	305	285	252	594	284	951	187	S_x 549	5 000	103	10 000	38.8
W250x67	2 500	673	593	489	2 660	524	1 650	383	M_r 280	3 500	275	6 000	223
W10x45	2 000	626	559	469	2 380	498	1 620	353	V_r 469	4 000	265	6 500	212
b = 204	1 500	552	500	431	1 780	461	1 580	314	L_u 3 260	4 500	254	7 000	202
t = 15.7	1 000	475	439	389	1 190	407	1 520	265	I_x 104	5 000	244	7 500	192
d = 257	500	395	372	337	594	313	1 390	198	S_x 806	5 500	233	8 000	180

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

$F_y = 345$ MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 90 mm Slab
$\phi = 0.90, \phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 2350$ kg/m^3

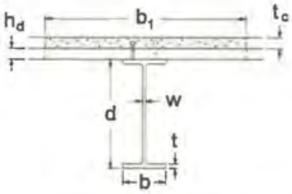
Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W250x58	2 500	590	515	422	2 300	458	1 440	341	M_r 239	3 500	232	6 000	181
W10x39	2 000	570	505	419	2 300	437	1 410	315	V_r 413	4 000	222	6 500	171
b = 203	1 500	503	452	385	1 780	407	1 380	282	L_u 3 130	4 500	212	7 000	161
t = 13.5	1 000	428	393	345	1 190	362	1 330	238	I_x 87.3	5 000	202	7 500	148
d = 252	500	350	328	295	594	281	1 220	177	S_x 693	5 500	192	8 000	137
W250x45	2 500	482	417	340	1 780	392	1 150	303	M_r 187	3 000	167	5 500	101
W10x30	2 000	470	411	338	1 780	376	1 130	282	V_r 414	3 500	155	6 000	90.6
b = 148	1 500	450	401	335	1 780	353	1 100	254	L_u 2 170	4 000	142	6 500	82.2
t = 13	1 000	377	342	296	1 190	318	1 070	216	I_x 71.1	4 500	129	7 000	75.2
d = 266	500	300	279	246	594	253	988	161	S_x 534	5 000	114	7 500	69.3
W250x39	2 500	417	358	291	1 530	341	992	268	M_r 159	3 000	140	5 500	77.5
W10x26	2 000	408	354	289	1 530	328	976	251	V_r 354	3 500	128	6 000	69.2
b = 147	1 500	394	347	287	1 530	309	955	227	L_u 2 110	4 000	115	6 500	62.5
t = 11.2	1 000	342	309	264	1 190	281	925	195	I_x 60.1	4 500	102	7 000	57.0
d = 262	500	267	248	218	594	227	862	145	S_x 459	5 000	88.0	7 500	52.4
W250x33	2 500	355	303	245	1 290	291	842	233	M_r 132	3 000	112	5 500	55.6
W10x22	2 000	349	300	244	1 290	281	830	219	V_r 323	3 500	100	6 000	49.4
b = 146	1 500	338	295	242	1 290	266	812	200	L_u 2 020	4 000	88.4	6 500	44.4
t = 9.1	1 000	310	277	233	1 190	243	787	172	I_x 48.9	4 500	74.1	7 000	40.3
d = 258	500	236	218	189	594	199	737	129	S_x 379	5 000	63.6	7 500	36.9
W200x42	2 500	400	338	266	1 650	279	966	213	M_r 138	3 000	133	5 500	99.6
W8x28	2 000	390	333	265	1 650	267	948	198	V_r 302	3 500	126	6 000	92.9
b = 166	1 500	372	325	262	1 650	250	925	178	L_u 2 610	4 000	120	6 500	84.6
t = 11.8	1 000	311	277	232	1 190	225	892	151	I_x 40.9	4 500	113	7 000	77.5
d = 205	500	235	217	190	594	178	826	110	S_x 399	5 000	106	7 500	71.6
W200x36	2 500	346	291	228	1 420	243	836	189	M_r 118	3 000	112	5 500	79.3
W8x24	2 000	339	287	227	1 420	233	823	176	V_r 255	3 500	105	6 000	71.3
b = 165	1 500	326	281	225	1 420	219	803	159	L_u 2 510	4 000	99.0	6 500	64.6
t = 10.2	1 000	285	252	208	1 190	198	775	136	I_x 34.4	4 500	92.5	7 000	59.0
d = 201	500	211	193	168	594	159	721	99.2	S_x 342	5 000	85.9	7 500	54.4
W200x31	2 500	312	261	205	1 240	226	746	178	M_r 104	2 000	104	4 500	65.2
W8x21	2 000	306	259	204	1 240	217	734	167	V_r 275	2 500	96.7	5 000	57.0
b = 134	1 500	296	254	203	1 240	205	717	152	L_u 1 980	3 000	89.3	5 500	50.6
t = 10.2	1 000	273	240	197	1 190	187	693	131	I_x 31.4	3 500	81.7	6 000	45.6
d = 210	500	199	182	156	594	152	646	96.2	S_x 299	4 000	74.0	6 500	41.5
W200x27	2 500	266	222	173	1 050	194	635	155	M_r 86.6	2 000	85.3	4 500	47.5
W8x18	2 000	261	219	173	1 050	187	626	146	V_r 246	2 500	78.7	5 000	41.2
b = 133	1 500	254	216	171	1 050	177	613	134	L_u 1 890	3 000	71.5	5 500	36.4
t = 8.4	1 000	240	209	169	1 050	162	592	116	I_x 25.8	3 500	64.1	6 000	32.6
d = 207	500	179	162	138	594	134	554	86.4	S_x 249	4 000	56.0	6 500	29.6

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

F_y = 345 MPa

Sections highlighted in yellow are commonly used sizes and are generally readily available.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 85 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



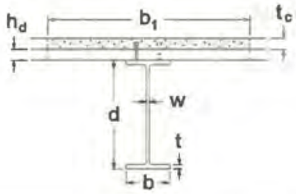
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 1850$ kg/m^3

Steel section	b_1	Composite: M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite: Steel section data	Unbraced condition: L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W1000x249	7 000	5 730	5 490	4 990	7 860	12 000	13 900	8 890	M_r 3 510	4 000	3 440	14 000	831
W40x167	5 000	5 430	5 190	4 640	5 610	11 000	13 600	8 040	V_r 3 220	6 000	2 780	16 000	694
b = 300	3 000	4 980	4 660	4 210	3 370	9 550	13 000	6 990	L_u 3 740	8 000	1 940	18 000	596
t = 26	1 000	4 060	3 890	3 700	1 120	6 970	11 600	5 640	I_x 4 810	10 000	1 360	20 000	523
d = 980									S_x 9 820	12 000	1 030	22 000	466
W1000x222	7 000	5 180	4 950	4 480	7 860	10 700	12 300	7 930	M_r 3 040	4 000	2 940	14 000	634
W40x149	5 000	4 890	4 670	4 140	5 610	9 840	12 000	7 160	V_r 3 000	6 000	2 310	16 000	527
b = 300	3 000	4 470	4 160	3 720	3 370	8 530	11 400	6 180	L_u 3 590	8 000	1 520	18 000	451
t = 21.1	1 000	3 570	3 400	3 210	1 120	6 160	10 200	4 890	I_x 4 080	10 000	1 050	20 000	394
d = 970									S_x 8 410	12 000	794	22 000	350
W920x238	7 000	5 220	4 980	4 570	7 860	10 300	12 700	7 660	M_r 3 170	4 000	3 140	14 000	800
W36x160	5 000	4 930	4 720	4 260	5 610	9 510	12 400	6 920	V_r 3 090	6 000	2 590	16 000	668
b = 305	3 000	4 550	4 270	3 870	3 370	8 230	11 800	6 000	L_u 3 890	8 000	1 870	18 000	573
t = 25.9	1 000	3 730	3 570	3 400	1 120	5 980	10 500	4 800	I_x 4 060	10 000	1 310	20 000	502
d = 915									S_x 8 870	12 000	996	22 000	447
W920x223	7 000	4 960	4 720	4 330	7 860	9 760	11 900	7 260	M_r 2 960	4 500	2 800	12 000	881
W36x150	5 000	4 670	4 470	4 030	5 610	9 000	11 600	6 550	V_r 2 970	5 000	2 670	14 000	705
b = 304	3 000	4 300	4 040	3 650	3 370	7 800	11 100	5 660	L_u 3 830	6 000	2 380	16 000	587
t = 23.9	1 000	3 510	3 360	3 180	1 120	5 650	9 890	4 490	I_x 3 760	8 000	1 680	18 000	502
d = 911									S_x 8 260	10 000	1 170	20 000	439
W920x201	7 000	4 510	4 290	3 930	7 860	8 770	10 600	6 560	M_r 2 590	4 500	2 420	12 000	705
W36x135	5 000	4 230	4 060	3 640	5 610	8 110	10 300	5 910	V_r 2 710	5 000	2 300	14 000	560
b = 304	3 000	3 900	3 650	3 270	3 370	7 050	9 910	5 080	L_u 3 720	6 000	2 030	16 000	463
t = 20.1	1 000	3 130	2 980	2 810	1 120	5 070	8 800	3 970	I_x 3 250	8 000	1 360	18 000	394
d = 903									S_x 7 190	10 000	940	20 000	343
W840x210	7 000	4 450	4 220	3 870	7 860	8 250	10 600	6 150	M_r 2 620	4 500	2 460	12 000	792
W33x141	5 000	4 170	3 980	3 610	5 610	7 620	10 400	5 550	V_r 2 670	5 000	2 350	14 000	639
b = 293	3 000	3 840	3 620	3 260	3 370	6 610	9 970	4 780	L_u 3 770	6 000	2 090	16 000	535
t = 24.4	1 000	3 130	2 990	2 830	1 120	4 770	8 880	3 760	I_x 3 110	8 000	1 470	18 000	461
d = 846									S_x 7 340	10 000	1 040	20 000	404
W840x193	7 000	4 120	3 900	3 570	7 660	7 590	9 720	5 690	M_r 2 370	4 500	2 200	12 000	666
W33x130	5 000	3 860	3 690	3 340	5 610	7 020	9 510	5 130	V_r 2 530	5 000	2 090	14 000	534
b = 292	3 000	3 550	3 350	3 010	3 370	6 110	9 120	4 400	L_u 3 690	6 000	1 850	16 000	445
t = 21.7	1 000	2 880	2 740	2 580	1 120	4 390	8 110	3 420	I_x 2 780	8 000	1 260	18 000	382
d = 840									S_x 6 630	10 000	877	20 000	334
W840x176	7 000	3 760	3 540	3 230	6 960	6 930	8 800	5 230	M_r 2 110	4 500	1 950	12 000	551
W33x118	5 000	3 560	3 390	3 070	5 610	6 430	8 610	4 710	V_r 2 300	5 000	1 840	14 000	439
b = 292	3 000	3 260	3 080	2 750	3 370	5 620	8 260	4 030	L_u 3 610	6 000	1 610	16 000	364
t = 18.8	1 000	2 620	2 480	2 330	1 120	4 020	7 350	3 090	I_x 2 460	8 000	1 060	18 000	311
d = 835									S_x 5 900	10 000	731	20 000	271

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm F_y = 345 MPa

This table may also be used with a concrete density of 2000 kg/m^3.   Readily available sizes are shown in yellow.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 85 mm Slab
ϕ = 0.90, ϕ_c = 0.65



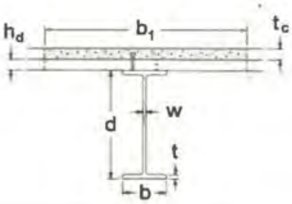
ASTM A992
A572 Grade 50
f'_c = 25 MPa
γ_c = 1850 kg/m^3

Steel section	b_1	Composite M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite Steel section data	Unbraced condition L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W760x185	5 000	3 450	3 280	2 980	5 610	5 810	8 480	4 240	M_r 2 080	4 000	1 980	12 000	576
W30x124	4 000	3 300	3 150	2 830	4 490	5 490	8 340	3 950	V_r 2 340	5 000	1 780	14 000	470
b = 267	3 000	3 150	2 980	2 670	3 370	5 060	8 130	3 620	L_u 3 450	6 000	1 550	16 000	397
t = 23.6	2 000	2 920	2 740	2 490	2 240	4 470	7 810	3 240	I_x 2 230	8 000	1 040	18 000	344
d = 766	1 000	2 550	2 430	2 280	1 120	3 610	7 230	2 780	S_x 5 820	10 000	743	20 000	304
W760x173	5 000	3 270	3 100	2 820	5 610	5 480	7 950	4 010	M_r 1 930	4 000	1 830	12 000	506
W30x116	4 000	3 130	2 980	2 680	4 490	5 180	7 820	3 740	V_r 2 250	5 000	1 630	14 000	411
b = 267	3 000	2 980	2 820	2 520	3 370	4 780	7 630	3 420	L_u 3 410	6 000	1 410	16 000	346
t = 21.6	2 000	2 760	2 580	2 340	2 240	4 230	7 330	3 050	I_x 2 060	8 000	924	18 000	299
d = 762	1 000	2 400	2 280	2 130	1 120	3 410	6 780	2 600	S_x 5 400	10 000	657	20 000	264
W760x161	5 000	3 060	2 900	2 630	5 610	5 090	7 320	3 740	M_r 1 760	4 000	1 650	12 000	429
W30x108	4 000	2 920	2 780	2 500	4 490	4 820	7 200	3 480	V_r 2 140	5 000	1 460	14 000	347
b = 266	3 000	2 770	2 630	2 340	3 370	4 460	7 030	3 180	L_u 3 330	6 000	1 250	16 000	291
t = 19.3	2 000	2 570	2 400	2 160	2 240	3 950	6 760	2 830	I_x 1 860	8 000	793	18 000	251
d = 758	1 000	2 220	2 100	1 960	1 120	3 170	6 250	2 390	S_x 4 900	10 000	560	20 000	220
W760x147	5 000	2 850	2 690	2 440	5 610	4 680	6 680	3 460	M_r 1 580	4 000	1 470	12 000	358
W30x99	4 000	2 710	2 580	2 310	4 490	4 440	6 580	3 220	V_r 2 040	5 000	1 290	14 000	288
b = 265	3 000	2 570	2 440	2 160	3 370	4 120	6 430	2 940	L_u 3 260	6 000	1 090	16 000	241
t = 17	2 000	2 380	2 220	1 980	2 240	3 650	6 180	2 600	I_x 1 660	8 000	671	18 000	207
d = 753	1 000	2 050	1 930	1 780	1 120	2 930	5 710	2 180	S_x 4 410	10 000	470	20 000	181
W760x134	5 000	2 610	2 460	2 220	5 270	4 320	6 090	3 220	M_r 1 440	4 000	1 330	12 000	308
W30x90	4 000	2 500	2 370	2 140	4 490	4 100	6 000	3 000	V_r 1 650	5 000	1 160	14 000	246
b = 264	3 000	2 360	2 250	2 000	3 370	3 810	5 870	2 740	L_u 3 230	6 000	967	16 000	205
t = 15.5	2 000	2 200	2 050	1 830	2 240	3 390	5 650	2 410	I_x 1 500	8 000	587	18 000	175
d = 750	1 000	1 890	1 770	1 630	1 120	2 730	5 230	2 010	S_x 4 010	10 000	408	20 000	153
W690x192	5 000	3 300	3 130	2 850	5 610	5 190	8 240	3 760	M_r 2 010	4 000	1 910	12 000	634
W27x129	4 000	3 150	3 000	2 720	4 490	4 890	8 090	3 500	V_r 2 230	5 000	1 730	14 000	525
b = 254	3 000	3 000	2 850	2 560	3 370	4 500	7 890	3 210	L_u 3 440	6 000	1 540	16 000	449
t = 27.9	2 000	2 790	2 620	2 390	2 240	3 970	7 570	2 870	I_x 1 980	8 000	1 090	18 000	392
d = 702	1 000	2 450	2 340	2 200	1 120	3 200	7 000	2 460	S_x 5 640	10 000	802	20 000	349
W690x170	5 000	2 980	2 820	2 560	5 610	4 630	7 280	3 380	M_r 1 750	4 000	1 650	12 000	497
W27x114	4 000	2 840	2 700	2 440	4 490	4 370	7 160	3 140	V_r 2 060	5 000	1 480	14 000	408
b = 256	3 000	2 690	2 560	2 290	3 370	4 040	6 980	2 870	L_u 3 380	6 000	1 290	16 000	347
t = 23.6	2 000	2 500	2 350	2 130	2 240	3 570	6 710	2 550	I_x 1 700	8 000	875	18 000	302
d = 693	1 000	2 180	2 070	1 940	1 120	2 860	6 200	2 170	S_x 4 900	10 000	634	20 000	268
W690x152	5 000	2 730	2 570	2 330	5 610	4 210	6 540	3 100	M_r 1 550	4 000	1 460	12 000	406
W27x102	4 000	2 590	2 450	2 220	4 490	3 990	6 440	2 890	V_r 1 850	5 000	1 290	14 000	332
b = 254	3 000	2 440	2 330	2 090	3 370	3 690	6 290	2 640	L_u 3 320	6 000	1 110	16 000	281
t = 21.1	2 000	2 280	2 140	1 930	2 240	3 270	6 060	2 340	I_x 1 510	8 000	728	18 000	244
d = 688	1 000	1 980	1 870	1 740	1 120	2 630	5 610	1 970	S_x 4 380	10 000	523	20 000	216

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm F_y = 345 MPa

This table may also be used with a concrete density of 2000 kg/m^3.   Readily available sizes are shown in yellow.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 85 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



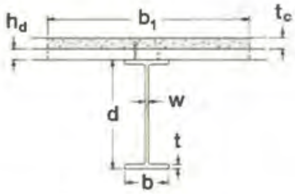
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 1850$ kg/m^3

Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W690x140	5 000	2 540	2 390	2 170	5 530	3 890	6 000	2 890	M_r 1 410	4 000	1 320	10 000	447
W27x94	4 000	2 420	2 280	2 070	4 490	3 700	5 910	2 690	V_r 1 740	5 000	1 160	12 000	345
b = 254	3 000	2 270	2 160	1 940	3 370	3 430	5 780	2 450	L_u 3 270	6 000	987	14 000	280
t = 18.9	2 000	2 110	1 990	1 780	2 240	3 050	5 570	2 170	I_x 1 360	7 000	778	16 000	236
d = 684	1 000	1 840	1 730	1 600	1 120	2 440	5 160	1 810	S_x 3 980	8 000	628	18 000	204
W690x125	5 000	2 290	2 140	1 930	4 970	3 510	5 360	2 620	M_r 1 250	4 000	1 140	10 000	362
W27x84	4 000	2 210	2 080	1 880	4 490	3 340	5 280	2 440	V_r 1 610	5 000	999	12 000	277
b = 253	3 000	2 070	1 970	1 760	3 370	3 100	5 170	2 220	L_u 3 190	6 000	834	14 000	223
t = 16.3	2 000	1 920	1 810	1 610	2 240	2 760	4 980	1 960	I_x 1 180	7 000	640	16 000	187
d = 678	1 000	1 660	1 560	1 430	1 120	2 220	4 610	1 620	S_x 3 500	8 000	513	18 000	161
W610x174	5 000	2 780	2 610	2 390	5 610	3 970	6 960	2 890	M_r 1 660	4 500	1 660	10 000	924
W24x117	4 000	2 640	2 500	2 290	4 490	3 750	6 850	2 690	V_r 1 770	5 000	1 610	12 000	709
b = 325	3 000	2 490	2 380	2 160	3 370	3 460	6 690	2 460	L_u 4 480	6 000	1 490	14 000	574
t = 21.6	2 000	2 330	2 210	2 020	2 240	3 060	6 430	2 190	I_x 1 470	7 000	1 370	16 000	482
d = 616	1 000	2 060	1 960	1 850	1 120	2 460	5 960	1 870	S_x 4 780	8 000	1 230	18 000	415
W610x155	5 000	2 530	2 370	2 150	5 610	3 590	6 200	2 640	M_r 1 470	4 500	1 460	10 000	762
W24x104	4 000	2 390	2 260	2 070	4 490	3 400	6 110	2 460	V_r 1 590	5 000	1 410	12 000	579
b = 324	3 000	2 250	2 140	1 950	3 370	3 140	5 980	2 240	L_u 4 400	6 000	1 300	14 000	465
t = 19	2 000	2 100	1 990	1 810	2 240	2 780	5 760	1 990	I_x 1 290	7 000	1 180	16 000	388
d = 611	1 000	1 860	1 760	1 650	1 120	2 240	5 340	1 680	S_x 4 220	8 000	1 050	18 000	333
W610x140	5 000	2 360	2 210	1 990	5 540	3 300	5 570	2 430	M_r 1 290	4 000	1 170	10 000	422
W24x94	4 000	2 230	2 100	1 900	4 490	3 130	5 490	2 260	V_r 1 660	5 000	1 030	12 000	334
b = 230	3 000	2 090	1 980	1 780	3 370	2 900	5 360	2 060	L_u 3 070	6 000	874	14 000	277
t = 22.2	2 000	1 930	1 820	1 640	2 240	2 570	5 160	1 810	I_x 1 120	7 000	695	16 000	237
d = 617	1 000	1 680	1 590	1 470	1 120	2 050	4 770	1 510	S_x 3 630	8 000	573	18 000	207
W610x125	5 000	2 120	1 970	1 770	4 950	2 990	4 980	2 220	M_r 1 140	4 000	1 020	10 000	342
W24x84	4 000	2 040	1 910	1 730	4 490	2 840	4 910	2 070	V_r 1 490	5 000	889	12 000	269
b = 229	3 000	1 900	1 800	1 620	3 370	2 640	4 810	1 880	L_u 3 020	6 000	733	14 000	222
t = 19.6	2 000	1 750	1 660	1 480	2 240	2 350	4 630	1 650	I_x 985	7 000	575	16 000	189
d = 612	1 000	1 530	1 430	1 320	1 120	1 880	4 290	1 360	S_x 3 220	8 000	470	18 000	165
W610x113	5 000	1 930	1 790	1 600	4 490	2 730	4 510	2 050	M_r 1 020	4 000	906	10 000	282
W24x76	4 000	1 890	1 770	1 590	4 490	2 600	4 450	1 910	V_r 1 400	5 000	775	12 000	220
b = 228	3 000	1 760	1 650	1 490	3 370	2 420	4 360	1 740	L_u 2 950	6 000	617	14 000	180
t = 17.3	2 000	1 610	1 520	1 360	2 240	2 160	4 210	1 520	I_x 875	7 000	481	16 000	153
d = 608	1 000	1 400	1 310	1 200	1 120	1 730	3 900	1 240	S_x 2 880	8 000	391	18 000	133
W610x101	5 000	1 730	1 600	1 420	4 020	2 460	4 030	1 870	M_r 900	4 000	787	10 000	228
W24x68	4 000	1 700	1 580	1 420	4 020	2 350	3 980	1 740	V_r 1 300	5 000	664	12 000	176
b = 228	3 000	1 610	1 510	1 360	3 370	2 200	3 900	1 580	L_u 2 890	6 000	512	14 000	144
t = 14.9	2 000	1 470	1 390	1 230	2 240	1 970	3 770	1 380	I_x 764	7 000	396	16 000	121
d = 603	1 000	1 270	1 190	1 080	1 120	1 580	3 500	1 120	S_x 2 530	8 000	320	18 000	105

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm $\quad$ F_y = 345 MPa

This table may also be used with a concrete density of 2000 kg/m^3. Readily available sizes are shown in yellow.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 85 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



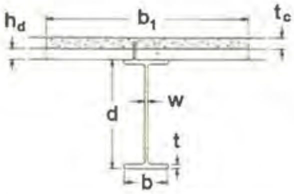
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 1850$ kg/m^3

Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W610x92	4 000	1 560	1 440	1 270	3 650	2 130	3 550	1 580	M_r 779	3 000	683	8 000	183
W24x62	3 000	1 500	1 400	1 240	3 370	1 990	3 480	1 440	V_r 1 350	4 000	540	10 000	135
b = 179	2 000	1 360	1 270	1 110	2 240	1 790	3 360	1 250	L_u 2 180	5 000	376	12 000	107
t = 15	1 000	1 150	1 070	955	1 120	1 430	3 110	999	I_x 646	6 000	281	14 000	88.9
d = 603	500	983	929	865	561	1 130	2 830	839	S_x 2 140	7 000	222	16 000	76.1
W610x82	4 000	1 390	1 280	1 120	3 240	1 910	3 150	1 440	M_r 683	3 000	587	8 000	145
W24x55	3 000	1 360	1 260	1 120	3 240	1 790	3 090	1 310	V_r 1 170	4 000	448	10 000	106
b = 178	2 000	1 230	1 160	1 010	2 240	1 620	2 990	1 140	L_u 2 110	5 000	304	12 000	83.4
t = 12.8	1 000	1 040	963	855	1 120	1 300	2 780	902	I_x 560	6 000	225	14 000	68.9
d = 599	500	882	829	767	561	1 030	2 530	749	S_x 1 870	7 000	177	16 000	58.7
W530x138	4 000	2 030	1 890	1 690	4 490	2 560	4 940	1 830	M_r 1 120	3 000	1 110	8 000	515
W21x93	3 000	1 880	1 770	1 580	3 370	2 360	4 820	1 650	V_r 1 650	4 000	1 000	10 000	390
b = 214	2 000	1 730	1 620	1 450	2 240	2 080	4 630	1 450	L_u 2 930	5 000	884	12 000	314
t = 23.6	1 000	1 490	1 400	1 290	1 120	1 650	4 250	1 190	I_x 861	6 000	759	14 000	263
d = 549	500	1 320	1 260	1 200	561	1 320	3 880	1 040	S_x 3 140	7 000	616	16 000	227
W530x123	4 000	1 850	1 720	1 540	4 490	2 320	4 430	1 670	M_r 997	3 000	984	8 000	421
W21x83	3 000	1 710	1 610	1 440	3 370	2 150	4 330	1 520	V_r 1 460	4 000	879	10 000	316
b = 212	2 000	1 560	1 470	1 310	2 240	1 910	4 160	1 320	L_u 2 860	5 000	762	12 000	253
t = 21.2	1 000	1 350	1 270	1 160	1 120	1 510	3 840	1 080	I_x 761	6 000	631	14 000	211
d = 544	500	1 190	1 140	1 080	561	1 210	3 500	932	S_x 2 800	7 000	505	16 000	182
W530x109	4 000	1 680	1 550	1 380	4 310	2 090	3 930	1 530	M_r 879	3 000	862	8 000	342
W21x73	3 000	1 550	1 450	1 300	3 370	1 940	3 850	1 380	V_r 1 280	4 000	764	10 000	254
b = 211	2 000	1 410	1 330	1 190	2 240	1 730	3 720	1 210	L_u 2 810	5 000	652	12 000	202
t = 18.8	1 000	1 220	1 140	1 040	1 120	1 380	3 440	977	I_x 667	6 000	520	14 000	168
d = 539	500	1 070	1 020	957	561	1 100	3 140	834	S_x 2 480	7 000	413	16 000	144
W530x101	4 000	1 570	1 450	1 290	4 010	1 960	3 670	1 450	M_r 814	3 000	794	8 000	301
W21x68	3 000	1 470	1 370	1 230	3 370	1 830	3 600	1 310	V_r 1 200	4 000	699	10 000	222
b = 210	2 000	1 330	1 260	1 120	2 240	1 640	3 470	1 140	L_u 2 770	5 000	591	12 000	176
t = 17.4	1 000	1 150	1 080	975	1 120	1 310	3 220	922	I_x 617	6 000	462	14 000	146
d = 537	500	1 000	951	893	561	1 040	2 940	782	S_x 2 300	7 000	365	16 000	125
W530x92	4 000	1 430	1 320	1 170	3 660	1 800	3 340	1 340	M_r 733	3 000	711	8 000	253
W21x62	3 000	1 370	1 280	1 140	3 370	1 690	3 280	1 220	V_r 1 110	4 000	621	9 000	214
b = 209	2 000	1 230	1 160	1 030	2 240	1 510	3 170	1 060	L_u 2 720	5 000	516	10 000	185
t = 15.6	1 000	1 060	991	894	1 120	1 210	2 950	849	I_x 552	6 000	393	12 000	146
d = 533	500	917	869	812	561	960	2 690	714	S_x 2 070	7 000	309	14 000	120
W530x82	4 000	1 280	1 170	1 030	3 250	1 610	2 960	1 210	M_r 640	3 000	616	8 000	203
W21x55	3 000	1 240	1 150	1 030	3 250	1 510	2 910	1 100	V_r 1 030	4 000	531	9 000	170
b = 209	2 000	1 120	1 050	933	2 240	1 370	2 820	961	L_u 2 660	5 000	433	10 000	147
t = 13.3	1 000	959	893	798	1 120	1 100	2 630	764	I_x 477	6 000	320	12 000	115
d = 528	500	821	774	718	561	868	2 400	635	S_x 1 810	7 000	249	14 000	94.0

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm $\quad$ F_y = 345 MPa

This table may also be used with a concrete density of 2000 kg/m^3. $\quad$ Readily available sizes are shown in yellow.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 85 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 1850$ kg/m^3

Steel section	b_1	Composite: M_{rc} (kN·m) for % shear connection			Composite: Q_r (kN)	Composite: I_t 10^6	Composite: S_t 10^3	Composite: I_{ts} 10^6	Non-composite: Steel section data	Non-composite: Unbraced condition L'	Unbraced condition M_r'	Unbraced condition L'	Unbraced condition M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W530x74	4 000	1 170	1 070	932	2 960	1 470	2 670	1 120	M_r 562	3 000	474	8 000	123
W21x50	3 000	1 140	1 060	927	2 960	1 390	2 620	1 020	V_r 1 050	4 000	357	9 000	105
b = 166	2 000	1 040	977	856	2 240	1 250	2 540	881	L_u 2 040	5 000	247	10 000	91.7
t = 13.6	1 000	883	815	720	1 120	1 010	2 360	694	I_x 411	6 000	186	12 000	73.2
d = 529	500	743	696	640	561	793	2 150	568	S_x 1 550	7 000	148	14 000	61.0
W530x66	4 000	1 030	939	813	2 600	1 300	2 340	1 000	M_r 484	3 000	398	8 000	94.9
W21x44	3 000	1 010	929	810	2 600	1 230	2 300	916	V_r 927	4 000	284	9 000	80.6
b = 165	2 000	945	882	772	2 240	1 120	2 230	796	L_u 1 980	5 000	195	10 000	70.0
t = 11.4	1 000	795	733	641	1 120	913	2 090	623	I_x 351	6 000	145	12 000	55.5
d = 525	500	663	617	562	561	716	1 900	504	S_x 1 340	7 000	115	14 000	46.0
W460x158	4 000	2 000	1 870	1 680	4 490	2 310	5 160	1 630	M_r 1 170	4 500	1 150	9 000	794
W18x106	3 000	1 860	1 750	1 590	3 370	2 120	5 030	1 470	V_r 1 460	5 000	1 110	10 000	696
b = 284	2 000	1 700	1 620	1 470	2 240	1 860	4 820	1 290	L_u 4 200	6 000	1 040	11 000	617
t = 23.9	1 000	1 500	1 420	1 330	1 120	1 470	4 430	1 070	I_x 796	7 000	955	12 000	555
d = 476	500	1 350	1 300	1 250	561	1 180	4 050	941	S_x 3 350	8 000	875	14 000	462
W460x144	4 000	1 870	1 740	1 560	4 490	2 140	4 750	1 520	M_r 1 070	4 500	1 050	9 000	693
W18x97	3 000	1 730	1 620	1 470	3 370	1 980	4 640	1 380	V_r 1 320	5 000	1 010	10 000	602
b = 283	2 000	1 580	1 500	1 360	2 240	1 740	4 460	1 210	L_u 4 130	6 000	936	11 000	533
t = 22.1	1 000	1 390	1 320	1 230	1 120	1 380	4 100	994	I_x 726	7 000	858	12 000	478
d = 472	500	1 250	1 200	1 150	561	1 100	3 760	868	S_x 3 080	8 000	779	14 000	396
W460x128	4 000	1 710	1 580	1 410	4 490	1 940	4 240	1 390	M_r 947	4 500	917	9 000	566
W18x86	3 000	1 570	1 470	1 330	3 370	1 800	4 150	1 260	V_r 1 170	5 000	884	10 000	489
b = 282	2 000	1 430	1 360	1 230	2 240	1 590	3 990	1 100	L_u 4 040	6 000	812	11 000	431
t = 19.6	1 000	1 260	1 190	1 100	1 120	1 260	3 690	898	I_x 637	7 000	736	12 000	385
d = 467	500	1 120	1 080	1 020	561	1 000	3 380	776	S_x 2 730	8 000	658	14 000	318
W460x113	4 000	1 560	1 440	1 270	4 470	1 740	3 750	1 270	M_r 829	4 500	796	9 000	458
W18x76	3 000	1 430	1 330	1 200	3 370	1 620	3 680	1 150	V_r 1 020	5 000	765	10 000	394
b = 280	2 000	1 280	1 220	1 100	2 240	1 440	3 550	999	L_u 3 950	6 000	696	11 000	345
t = 17.3	1 000	1 130	1 070	979	1 120	1 140	3 290	809	I_x 556	7 000	623	12 000	307
d = 463	500	999	956	905	561	908	3 020	692	S_x 2 400	8 000	545	14 000	252
W460x106	4 000	1 480	1 360	1 200	4 180	1 640	3 460	1 190	M_r 742	3 000	719	8 000	308
W18x71	3 000	1 370	1 270	1 130	3 370	1 530	3 390	1 080	V_r 1 210	4 000	637	9 000	266
b = 194	2 000	1 230	1 160	1 030	2 240	1 360	3 260	931	L_u 2 690	5 000	549	10 000	235
t = 20.6	1 000	1 050	985	893	1 120	1 070	3 010	743	I_x 488	6 000	450	11 000	210
d = 469	500	915	870	818	561	841	2 730	626	S_x 2 080	7 000	366	12 000	190
W460x97	4 000	1 360	1 240	1 090	3 820	1 520	3 180	1 120	M_r 677	3 000	652	8 000	264
W18x65	3 000	1 280	1 190	1 050	3 370	1 420	3 110	1 010	V_r 1 090	4 000	574	9 000	227
b = 193	2 000	1 140	1 070	955	2 240	1 270	3 000	874	L_u 2 650	5 000	488	10 000	200
t = 19	1 000	980	916	827	1 120	1 010	2 780	694	I_x 445	6 000	389	11 000	178
d = 466	500	848	804	753	561	788	2 530	581	S_x 1 910	7 000	314	12 000	161

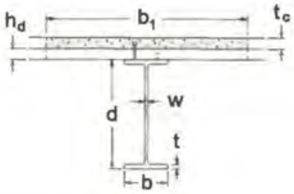
Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

$F_y = 345$ MPa

This table may also be used with a concrete density of 2000 kg/m^3.

Readily available sizes are shown in yellow.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 85 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



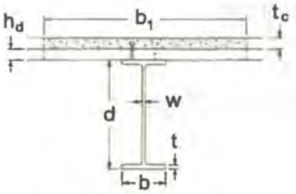
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 1850$ kg/m^3

Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W460x89	3 000	1 210	1 120	990	3 370	1 330	2 890	952	M_r 624	3 000	598	8 000	231
W18x60	2 000	1 070	1 010	897	2 240	1 190	2 790	825	V_r 996	4 000	523	9 000	198
b = 192	1 500	1 000	943	839	1 680	1 090	2 710	746	L_u 2 620	5 000	439	10 000	174
t = 17.7	1 000	919	859	773	1 120	949	2 590	653	I_x 409	6 000	343	11 000	155
d = 463	500	793	751	700	561	743	2 360	543	S_x 1 770	7 000	276	12 000	140
W460x82	3 000	1 130	1 040	915	3 240	1 230	2 650	891	M_r 568	3 000	540	8 000	195
W18x55	2 000	1 000	938	835	2 240	1 110	2 570	772	V_r 933	4 000	466	9 000	166
b = 191	1 500	933	879	779	1 680	1 020	2 500	697	L_u 2 560	5 000	384	10 000	146
t = 16	1 000	854	798	715	1 120	888	2 390	608	I_x 370	6 000	292	11 000	129
d = 460	500	734	693	643	561	695	2 180	501	S_x 1 610	7 000	234	12 000	116
W460x74	3 000	1 030	942	827	2 930	1 130	2 410	827	M_r 512	3 000	484	8 000	164
W18x50	2 000	932	867	772	2 240	1 020	2 340	718	V_r 843	4 000	414	9 000	140
b = 190	1 500	861	811	719	1 680	941	2 280	648	L_u 2 530	5 000	332	10 000	122
t = 14.5	1 000	787	737	657	1 120	824	2 180	564	I_x 332	6 000	249	11 000	108
d = 457	500	675	635	586	561	645	1 990	461	S_x 1 460	7 000	198	12 000	96.8
W460x68	3 000	963	876	765	2 710	1 060	2 220	778	M_r 463	3 000	390	8 000	112
W18x46	2 000	884	820	723	2 240	959	2 150	674	V_r 856	4 000	301	9 000	96.7
b = 154	1 500	814	763	670	1 680	884	2 100	607	L_u 2 010	5 000	213	10 000	85.2
t = 15.4	1 000	739	688	608	1 120	775	2 010	526	I_x 297	6 000	164	11 000	76.1
d = 459	500	626	586	536	561	605	1 830	425	S_x 1 290	7 000	133	12 000	68.9
W460x60	3 000	842	762	663	2 350	935	1 940	700	M_r 397	3 000	329	8 000	86.6
W18x40	2 000	799	737	649	2 240	854	1 880	609	V_r 746	4 000	242	9 000	74.3
b = 153	1 500	731	682	600	1 680	791	1 840	549	L_u 1 970	5 000	169	10 000	65.2
t = 13.3	1 000	660	616	541	1 120	698	1 770	474	I_x 255	6 000	129	11 000	58.1
d = 455	500	558	520	472	561	547	1 620	380	S_x 1 120	7 000	104	12 000	52.4
W460x52	3 000	739	665	573	2 060	819	1 680	622	M_r 338	3 000	269	8 000	63.6
W18x35	2 000	712	652	569	2 060	752	1 640	543	V_r 680	4 000	185	9 000	54.3
b = 152	1 500	660	612	536	1 680	699	1 600	489	L_u 1 890	5 000	128	10 000	47.4
t = 10.8	1 000	590	550	478	1 120	620	1 540	421	I_x 212	6 000	96.2	11 000	42.0
d = 450	500	494	457	411	561	487	1 410	333	S_x 942	7 000	76.7	12 000	37.8
W410x149	3 000	1 650	1 540	1 390	3 370	1 750	4 460	1 200	M_r 1 010	4 500	983	8 000	760
W16x100	2 000	1 500	1 420	1 280	2 240	1 530	4 270	1 050	V_r 1 320	5 000	952	9 000	696
b = 265	1 500	1 420	1 340	1 220	1 680	1 390	4 130	958	L_u 4 080	5 500	921	10 000	621
t = 25	1 000	1 310	1 240	1 150	1 120	1 200	3 910	857	I_x 618	6 000	889	11 000	554
d = 431	500	1 170	1 130	1 080	561	955	3 560	745	S_x 2 870	7 000	825	12 000	501
W410x132	3 000	1 500	1 390	1 250	3 370	1 580	3 970	1 090	M_r 885	4 500	853	8 000	635
W16x89	2 000	1 350	1 270	1 150	2 240	1 390	3 810	950	V_r 1 160	5 000	823	9 000	565
b = 263	1 500	1 270	1 200	1 090	1 680	1 260	3 690	865	L_u 3 940	5 500	792	10 000	495
t = 22.2	1 000	1 180	1 110	1 030	1 120	1 090	3 500	769	I_x 538	6 000	761	11 000	440
d = 425	500	1 050	1 010	957	561	862	3 190	661	S_x 2 530	7 000	698	12 000	397

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm $F_y = 345$ MPa

This table may also be used with a concrete density of 2000 kg/m^3. Readily available sizes are shown in yellow.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 85 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 1850$ kg/m^3

Steel section	b_1	Composite: M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite: Steel section data	Unbraced condition: L′	M_r'	L′	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W410x114	3 000	1 340	1 240	1 110	3 370	1 410	3 470	984	M_r 764	4 500	726	8 000	513
W16x77	2 000	1 200	1 130	1 020	2 240	1 250	3 340	853	V_r 998	5 000	698	9 000	438
b = 261	1 500	1 120	1 070	966	1 680	1 130	3 240	774	L_u 3 810	5 500	668	10 000	382
t = 19.3	1 000	1 040	985	904	1 120	981	3 090	685	I_x 461	6 000	638	11 000	338
d = 420	500	922	882	834	561	772	2 810	581	S_x 2 200	7 000	576	12 000	304
W410x100	3 000	1 210	1 120	989	3 370	1 250	3 040	889	M_r 661	4 500	623	8 000	411
W16x67	2 000	1 070	1 000	910	2 240	1 120	2 940	770	V_r 850	5 000	596	9 000	348
b = 260	1 500	999	947	858	1 680	1 020	2 860	698	L_u 3 730	5 500	568	10 000	302
t = 16.9	1 000	924	875	799	1 120	886	2 730	614	I_x 398	6 000	539	11 000	266
d = 415	500	815	778	731	561	696	2 490	515	S_x 1 920	7 000	479	12 000	238
W410x85	3 000	1 100	1 000	875	3 360	1 090	2 570	781	M_r 534	3 000	507	8 000	205
W16x57	2 000	959	891	790	2 240	979	2 480	673	V_r 931	4 000	443	9 000	177
b = 181	1 500	886	832	736	1 680	895	2 410	606	L_u 2 530	5 000	375	10 000	157
t = 18.2	1 000	808	754	675	1 120	778	2 300	526	I_x 315	6 000	297	11 000	140
d = 417	500	692	654	606	561	603	2 080	431	S_x 1 510	7 000	242	12 000	127
W410x74	3 000	975	886	770	2 960	982	2 280	712	M_r 469	3 000	440	8 000	163
W16x50	2 000	873	808	715	2 240	885	2 210	615	V_r 821	4 000	379	9 000	140
b = 180	1 500	802	751	665	1 680	812	2 150	553	L_u 2 470	5 000	312	10 000	123
t = 16	1 000	728	681	606	1 120	709	2 050	479	I_x 275	6 000	239	11 000	110
d = 413	500	622	585	540	561	551	1 870	388	S_x 1 330	7 000	194	12 000	99.7
W410x67	3 000	884	799	692	2 670	897	2 060	658	M_r 422	3 000	392	8 000	135
W16x45	2 000	810	746	659	2 240	813	2 000	570	V_r 739	4 000	333	9 000	116
b = 179	1 500	741	691	612	1 680	748	1 950	512	L_u 2 420	5 000	264	10 000	102
t = 14.4	1 000	668	627	556	1 120	656	1 870	443	I_x 245	6 000	201	11 000	90.4
d = 410	500	571	535	490	561	510	1 700	356	S_x 1 200	7 000	161	12 000	81.6
W410x60	3 000	786	706	609	2 350	807	1 830	602	M_r 369	3 000	341	8 000	109
W16x40	2 000	743	681	598	2 240	736	1 780	523	V_r 642	4 000	286	9 000	93.1
b = 178	1 500	675	626	555	1 680	681	1 740	471	L_u 2 390	5 000	218	10 000	81.3
t = 12.8	1 000	604	568	503	1 120	600	1 670	406	I_x 216	6 000	165	11 000	72.1
d = 407	500	516	483	440	561	469	1 530	324	S_x 1 060	7 000	131	12 000	64.9
W410x54	3 000	707	633	544	2 110	725	1 630	547	M_r 326	3 000	295	8 000	86.0
W16x36	2 000	679	619	540	2 110	664	1 590	476	V_r 619	4 000	241	9 000	73.1
b = 177	1 500	623	575	507	1 680	617	1 550	429	L_u 2 310	5 000	176	10 000	63.5
t = 10.9	1 000	553	518	455	1 120	546	1 490	369	I_x 186	6 000	132	11 000	56.2
d = 403	500	468	435	393	561	427	1 370	291	S_x 923	7 000	104	12 000	50.4
W410x46	3 000	619	551	470	1 830	639	1 410	491	M_r 274	2 000	265	7 000	61.7
W16x31	2 000	597	540	467	1 830	589	1 380	429	V_r 578	3 000	210	8 000	51.8
b = 140	1 500	566	519	453	1 680	550	1 350	386	L_u 1 790	4 000	142	9 000	44.6
t = 11.2	1 000	497	464	404	1 120	489	1 300	332	I_x 156	5 000	99.9	10 000	39.2
d = 403	500	416	384	342	561	385	1 190	259	S_x 772	6 000	76.4	11 000	35.0

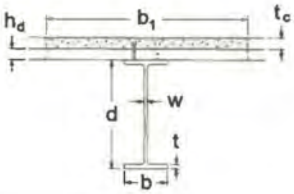
Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm

$F_y = 345$ MPa

This table may also be used with a concrete density of 2000 kg/m^3.

Readily available sizes are shown in yellow.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 85 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



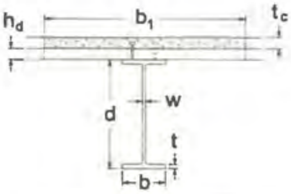
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 1850$ kg/m^3

Steel section	b_1	Composite M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Non-composite Steel section data	Unbraced condition L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W410x39	3 000	527	467	395	1 550	546	1 200	427	M_r 227	2 000	216	7 000	44.0
W16x26	2 000	512	459	393	1 550	507	1 170	376	V_r 480	3 000	166	8 000	36.5
b = 140	1 500	496	452	390	1 550	475	1 140	339	L_u 1 730	4 000	105	9 000	31.2
t = 8.8	1 000	440	408	353	1 120	426	1 100	291	I_x 126	5 000	73.0	10 000	27.3
d = 399	500	363	334	293	561	338	1 020	224	S_x 634	6 000	55.1	11 000	24.2
W360x79	3 000	932	841	722	3 130	835	2 210	596	M_r 444	3 500	425	7 000	267
W14x53	2 000	815	750	663	2 240	749	2 140	512	V_r 682	4 000	404	8 000	225
b = 205	1 500	744	693	619	1 680	684	2 080	459	L_u 3 010	4 500	383	9 000	194
t = 16.8	1 000	671	632	566	1 120	594	1 980	396	I_x 226	5 000	361	10 000	171
d = 354	500	579	547	505	561	458	1 800	320	S_x 1 280	6 000	317	11 000	153
W360x72	3 000	846	759	648	2 830	761	2 000	551	M_r 397	3 500	377	7 000	222
W14x48	2 000	757	693	608	2 240	686	1 940	474	V_r 617	4 000	357	8 000	186
b = 204	1 500	687	637	567	1 680	629	1 890	425	L_u 2 940	4 500	336	9 000	160
t = 15.1	1 000	615	579	517	1 120	549	1 810	366	I_x 201	5 000	315	10 000	141
d = 350	500	529	498	457	561	423	1 640	293	S_x 1 150	6 000	272	11 000	126
W360x64	3 000	762	680	578	2 530	691	1 800	507	M_r 354	3 500	332	7 000	183
W14x43	2 000	702	639	556	2 240	626	1 740	438	V_r 548	4 000	313	8 000	153
b = 203	1 500	633	585	519	1 680	577	1 700	392	L_u 2 870	4 500	293	9 000	131
t = 13.5	1 000	562	529	471	1 120	505	1 630	337	I_x 178	5 000	273	10 000	115
d = 347	500	482	453	413	561	391	1 490	268	S_x 1 030	6 000	228	11 000	102
W360x57	3 000	696	619	526	2 240	648	1 620	482	M_r 314	3 000	289	7 000	119
W14x38	2 000	664	603	521	2 240	591	1 570	417	V_r 580	3 500	267	8 000	99.7
b = 172	1 500	597	549	483	1 680	546	1 530	374	L_u 2 360	4 000	244	9 000	85.9
t = 13.1	1 000	527	493	435	1 120	481	1 470	320	I_x 160	5 000	192	10 000	75.6
d = 358	500	446	417	377	561	373	1 340	252	S_x 896	6 000	147	11 000	67.5
W360x51	3 000	625	553	468	2 000	585	1 450	442	M_r 277	3 000	252	7 000	96.9
W14x34	2 000	599	540	464	2 000	537	1 410	384	V_r 524	3 500	232	8 000	80.9
b = 171	1 500	552	505	442	1 680	498	1 380	345	L_u 2 320	4 000	210	9 000	69.4
t = 11.6	1 000	483	450	397	1 120	441	1 320	295	I_x 141	5 000	159	10 000	60.8
d = 355	500	407	379	341	561	344	1 210	230	S_x 796	6 000	121	11 000	54.1
W360x45	3 000	558	492	415	1 780	525	1 290	402	M_r 242	3 000	217	7 000	76.4
W14x30	2 000	538	482	412	1 780	483	1 250	351	V_r 498	3 500	197	8 000	63.3
b = 171	1 500	511	465	403	1 680	451	1 230	315	L_u 2 260	4 000	176	9 000	54.0
t = 9.8	1 000	443	411	360	1 120	401	1 180	270	I_x 122	5 000	128	10 000	47.1
d = 352	500	369	342	304	561	314	1 090	209	S_x 691	6 000	96.0	11 000	41.8
W360x39	3 000	490	430	361	1 550	465	1 120	361	M_r 206	2 000	193	6 000	54.1
W14x26	2 000	475	423	359	1 550	430	1 090	317	V_r 470	2 500	172	7 000	44.2
b = 128	1 500	460	415	356	1 550	403	1 070	285	L_u 1 660	3 000	148	8 000	37.4
t = 10.7	1 000	404	372	322	1 120	360	1 030	244	I_x 102	4 000	97.0	9 000	32.5
d = 353	500	331	305	268	561	284	949	187	S_x 580	5 000	69.7	10 000	28.7

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm F_y = 345 MPa

This table may also be used with a concrete density of 2000 kg/m^3.   Readily available sizes are shown in yellow.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 85 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



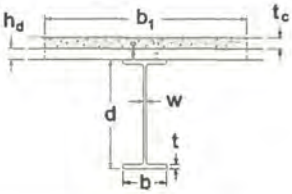
ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 1850$ kg/m^3

Steel section	b_1	Composite							Non-composite				
		M_{rc} (kN·m) for % shear connection			Q_r (kN)	I_t 10^6	S_t 10^3	I_{ts} 10^6	Steel section data	Unbraced condition			
										L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W360x33	2 500	408	358	299	1 290	382	928	297	M_r 168	2 000	155	6 000	38.0
W14x22	2 000	401	354	298	1 290	367	915	276	V_r 396	2 500	135	7 000	30.8
b = 127	1 500	391	349	297	1 290	345	897	250	L_u 1 600	3 000	113	8 000	25.8
t = 8.5	1 000	358	327	282	1 120	312	868	214	I_x 82.6	4 000	70.2	9 000	22.3
d = 349	500	288	265	230	561	249	805	163	S_x 473	5 000	49.6	10 000	19.6
W310x74	2 500	783	706	601	2 810	634	1 900	441	M_r 366	3 500	354	6 000	274
W12x50	2 000	716	651	566	2 240	595	1 860	404	V_r 597	4 000	339	7 000	240
b = 205	1 500	646	595	527	1 680	544	1 810	360	L_u 3 100	4 500	323	8 000	204
t = 16.3	1 000	573	538	481	1 120	471	1 730	308	I_x 164	5 000	307	9 000	177
d = 310	500	491	462	425	561	359	1 560	244	S_x 1 060	5 500	291	10 000	156
W310x67	2 500	717	642	543	2 620	575	1 710	405	M_r 326	3 500	312	6 000	234
W12x45	2 000	665	602	518	2 240	542	1 680	372	V_r 533	4 000	297	7 000	198
b = 204	1 500	596	547	482	1 680	497	1 630	332	L_u 3 020	4 500	282	8 000	167
t = 14.6	1 000	524	490	437	1 120	432	1 560	283	I_x 144	5 000	266	9 000	144
d = 306	500	446	420	383	561	330	1 410	222	S_x 942	5 500	250	10 000	127
W310x60	2 500	646	574	483	2 340	523	1 530	374	M_r 290	3 500	275	6 000	199
W12x40	2 000	618	557	475	2 240	494	1 510	344	V_r 466	4 000	261	7 000	163
b = 203	1 500	550	503	440	1 680	455	1 470	307	L_u 2 960	4 500	246	8 000	137
t = 13.1	1 000	480	447	398	1 120	398	1 410	262	I_x 128	5 000	231	9 000	118
d = 303	500	406	382	347	561	306	1 280	204	S_x 842	5 500	215	10 000	104
W310x52	2 500	594	526	443	2 070	499	1 390	363	M_r 260	3 000	240	6 000	130
W12x35	2 000	578	518	440	2 070	473	1 360	334	V_r 494	3 500	223	7 000	106
b = 167	1 500	525	478	415	1 680	438	1 330	299	L_u 2 380	4 000	206	8 000	89.4
t = 13.2	1 000	456	422	373	1 120	386	1 280	254	I_x 118	4 500	187	9 000	77.4
d = 317	500	381	356	321	561	298	1 170	197	S_x 747	5 000	167	10 000	68.4
W310x45	2 500	512	450	376	1 770	434	1 190	323	M_r 220	3 000	200	6 000	98.2
W12x30	2 000	500	444	374	1 770	414	1 170	298	V_r 423	3 500	184	7 000	79.3
b = 166	1 500	474	428	367	1 680	385	1 150	267	L_u 2 310	4 000	167	8 000	66.5
t = 11.2	1 000	406	374	329	1 120	342	1 100	228	I_x 99.2	4 500	150	9 000	57.3
d = 313	500	335	313	279	561	266	1 020	175	S_x 634	5 000	128	10 000	50.4
W310x39	2 500	447	391	326	1 530	383	1 040	290	M_r 189	3 000	170	6 000	77.7
W12x26	2 000	438	386	324	1 530	366	1 020	269	V_r 368	3 500	155	7 000	62.2
b = 165	1 500	423	379	322	1 530	343	1 000	242	L_u 2 260	4 000	139	8 000	51.8
t = 9.7	1 000	369	337	295	1 120	307	969	207	I_x 85.1	4 500	121	9 000	44.3
d = 310	500	299	280	248	561	242	896	158	S_x 549	5 000	103	10 000	38.8
W250x67	2 500	659	584	484	2 660	465	1 580	322	M_r 280	3 500	275	6 000	223
W10x45	2 000	604	541	457	2 240	437	1 550	294	V_r 469	4 000	265	6 500	212
b = 204	1 500	535	486	422	1 680	399	1 500	260	L_u 3 260	4 500	254	7 000	202
t = 15.7	1 000	464	430	382	1 120	344	1 430	219	I_x 104	5 000	244	7 500	192
d = 257	500	389	366	333	561	259	1 290	169	S_x 806	5 500	233	8 000	180

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm $F_y = 345$ MPa

This table may also be used with a concrete density of 2000 kg/m^3. Readily available sizes are shown in yellow.

COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 85 mm Slab
$\phi = 0.90$, $\phi_c = 0.65$



ASTM A992
A572 Grade 50
$f'_c = 25$ MPa
$\gamma_c = 1850$ kg/m^3

Steel section	b_1	Composite: M_{rc} (kN·m) for % shear connection			Q_r	I_t	S_t	I_{ts}	Non-composite: Steel section data	Unbraced condition			
					(kN)	10^6	10^3	10^6		L'	M_r'	L'	M_r'
	mm	100%	70%	40%	100%	mm^4	mm^3	mm^4		mm	kN·m	mm	kN·m
W250x58	2 500	578	507	417	2 300	409	1 370	288	M_r 239	3 500	232	6 000	181
W10x39	2 000	554	493	411	2 240	386	1 350	264	V_r 413	4 000	222	6 500	171
b = 203	1 500	487	439	376	1 680	354	1 310	234	L_u 3 130	4 500	212	7 000	161
t = 13.5	1 000	417	383	339	1 120	308	1 250	197	I_x 87.3	5 000	202	7 500	148
d = 252	500	344	323	291	561	233	1 130	150	S_x 693	5 500	192	8 000	137
W250x45	2 500	473	410	336	1 780	354	1 100	259	M_r 187	3 000	167	5 500	101
W10x30	2 000	461	404	334	1 780	336	1 080	238	V_r 414	3 500	155	6 000	90.6
b = 148	1 500	434	388	326	1 680	312	1 050	212	L_u 2 170	4 000	142	6 500	82.2
t = 13	1 000	366	333	289	1 120	275	1 010	179	I_x 71.1	4 500	129	7 000	75.2
d = 266	500	294	273	242	561	212	922	134	S_x 534	5 000	114	7 500	69.3
W250x39	2 500	409	353	288	1 530	309	949	231	M_r 159	3 000	140	5 500	77.5
W10x26	2 000	401	349	286	1 530	295	934	214	V_r 354	3 500	128	6 000	69.2
b = 147	1 500	386	341	284	1 530	275	913	191	L_u 2 110	4 000	115	6 500	62.5
t = 11.2	1 000	331	300	258	1 120	245	879	162	I_x 60.1	4 500	102	7 000	57.0
d = 262	500	261	243	214	561	190	808	121	S_x 459	5 000	88.0	7 500	52.4
W250x33	2 500	349	299	242	1 290	265	806	202	M_r 132	3 000	112	5 500	55.6
W10x22	2 000	342	296	241	1 290	254	793	188	V_r 323	3 500	100	6 000	49.4
b = 146	1 500	332	290	240	1 290	238	776	169	L_u 2 020	4 000	88.4	6 500	44.4
t = 9.1	1 000	299	268	227	1 120	213	749	143	I_x 48.9	4 500	74.1	7 000	40.3
d = 258	500	230	213	185	561	168	692	107	S_x 379	5 000	63.6	7 500	36.9
W200x42	2 500	392	332	263	1 650	250	918	181	M_r 138	3 000	133	5 500	99.6
W8x28	2 000	381	327	262	1 650	237	901	166	V_r 302	3 500	126	6 000	92.9
b = 166	1 500	364	319	259	1 650	219	878	147	L_u 2 610	4 000	120	6 500	84.6
t = 11.8	1 000	300	268	226	1 120	193	843	123	I_x 40.9	4 500	113	7 000	77.5
d = 205	500	229	212	187	561	147	767	89.2	S_x 399	5 000	106	7 500	71.6
W200x36	2 500	339	286	225	1 420	218	796	162	M_r 118	3 000	112	5 500	79.3
W8x24	2 000	332	282	224	1 420	208	782	149	V_r 255	3 500	105	6 000	71.3
b = 165	1 500	319	276	222	1 420	193	763	133	L_u 2 510	4 000	99.0	6 500	64.6
t = 10.2	1 000	274	243	202	1 120	171	734	111	I_x 34.4	4 500	92.5	7 000	59.0
d = 201	500	205	189	165	561	132	673	80.5	S_x 342	5 000	85.9	7 500	54.4
W200x31	2 500	306	257	203	1 240	204	711	154	M_r 104	2 000	104	4 500	65.2
W8x21	2 000	300	254	202	1 240	195	698	143	V_r 275	2 500	96.7	5 000	57.0
b = 134	1 500	290	249	200	1 240	182	682	128	L_u 1 980	3 000	89.3	5 500	50.6
t = 10.2	1 000	262	231	191	1 120	163	657	107	I_x 31.4	3 500	81.7	6 000	45.6
d = 210	500	193	177	153	561	127	605	78.0	S_x 299	4 000	74.0	6 500	41.5
W200x27	2 500	260	218	171	1 050	176	607	135	M_r 86.6	2 000	85.3	4 500	47.5
W8x18	2 000	256	216	170	1 050	168	596	126	V_r 246	2 500	78.7	5 000	41.2
b = 133	1 500	249	212	169	1 050	158	582	113	L_u 1 890	3 000	71.5	5 500	36.4
t = 8.4	1 000	235	206	167	1 050	142	562	96.1	I_x 25.8	3 500	64.1	6 000	32.6
d = 207	500	173	157	134	561	113	521	70.0	S_x 249	4 000	56.0	6 500	29.6

Units: M_r - kN·m, V_r - kN, L_u - mm, I_x - 10^6 mm^4, S_x - 10^3 mm^3, b - mm, t - mm, d - mm $\qquad F_y$ = 345 MPa

This table may also be used with a concrete density of 2000 kg/m^3. Readily available sizes are shown in yellow.

DEFLECTION OF FLEXURAL MEMBERS

The CSA S16-14 Standard considers deflection to be a serviceability limit state which must be accounted for in the design of flexural members. Annex D of S16-14, "Recommended maximum values for deflection for specified design live, snow and wind loads", provides some guidance to designers. Deflections tend to be more significant with longer clear spans, shallower members and with the use of high-strength steels. Deflection calculations are based on specified loads.

Three methods for dealing with deflection of prismatic beams are summarized below:

1. Compute the required minimum moment of inertia to satisfy the deflection constraint, prior to selection of the beam size.

   I_{reqd} = $W C_d B_d$, where

   I_{reqd} = required value of moment of inertia (10^6 mm^4)

   W = specified load value as described in Table 5-8 (kN)

   For distributed loading, W is the total applied load in kN. If there are multiple spans, W is the total applied load on a *single* span.

   For point loads, W is the value of a *single* point load in kN. For example, if point loads are applied at the quarter points (number of spaces, $n = 4$), the total load applied on a given span is $(n - 1)\,W = 3\,W$.

   C_d = value of deflection constant obtained from Figure 5-2 for the appropriate span L and span/deflection limit L / Δ (10^6 mm^4 / kN)

   B_d = a number to relate the actual load and support condition to a uniformly distributed load (UDL) on a simply-supported beam, Table 5-8. Values of B_d are computed for the maximum deflection within the span. For a uniformly distributed load, $B_d = 1.0$.

   The actual deflection of a beam can be computed as:

   Δ = $(I_{reqd} / I)\,\Delta_m$, where

   Δ = actual deflection (mm)

   I = moment of inertia of beam (10^6 mm^4)

   Δ_m = maximum deflection permitted (mm)

   I_{reqd} = moment of inertia required to meet Δ_m (10^6 mm^4).

2. Compute deflections using the formulas for deflection of beams included in the *Beam Diagrams and Formulas* provided in Part 5 of this Handbook.

3. The *Beam Load Tables* for W-shapes in Part 5 list approximate deflections for the various steel sections and spans, based on the tabulated uniformly distributed total factored loads at an assumed stress of 240 MPa for steels with a yield stress of 345 or 350 MPa. Deflections (for live load only or for total load) caused by stress levels that are different from those assumed can be determined by multiplying the tabulated deflection with the ratio of actual stress to assumed stress (240 MPa). Also see *Vertical Deflection* in the section on *Factored Resistance of Beams* in Part 5.

Examples

Given:

A W410x85 section has been chosen for a simply supported non-composite beam spanning 10 m and subjected to a uniformly distributed specified load of 15 kN/m live and 7 kN/m dead. Check for live load deflection assuming the beam is laterally supported, ASTM A992 steel, and deflection is limited to $L / 300 = 33$ mm.

Solutions:

Method 1

From Table 5-8:

$B_d = 1.0$ (simple-span UDL)

Using the graph (Figure 5-2, upper left):

$C_d = 1.95 \times 10^6 \text{ mm}^4 / \text{kN}$ (for $L / \Delta = 300$ and $L = 10$ m)

Or using the formula given on the same figure:

$C_d = \gamma L^2 / 15\,360 = 300 \times 10^2 / 15\,360 = 1.95 \times 10^6 \text{ mm}^4 / \text{kN}$

$I_{reqd} = W C_d B_d = (15 \times 10) \times 1.95 \times 10^6 \times 1.0 = 293 \times 10^6 \text{ mm}^4$

For W410x85, $I_x = 315 \times 10^6 \text{ mm}^4$

Actual deflection, $\Delta = (I_{reqd} / I)\, \Delta_m = (293 / 315)\, 33 = 31$ mm

Method 2

From *Beam Diagrams and Formulas* in Part 5:

$$\Delta = \frac{5 w L^4}{384 E I}, \text{ where}$$

$I = 315 \times 10^6 \text{ mm}^4$, $E = 200\,000$ MPa

Therefore $$\Delta = \frac{5 \times 15 \times (10 \times 10^3)^4}{384 \times 200\,000 \times 315 \times 10^6} = 31 \text{ mm}$$

Method 3

From the *Beam Load Tables* in Part 5, approximate deflection for W410x85 beam, span 10 m, loaded to a stress of 240 MPa = 61 mm

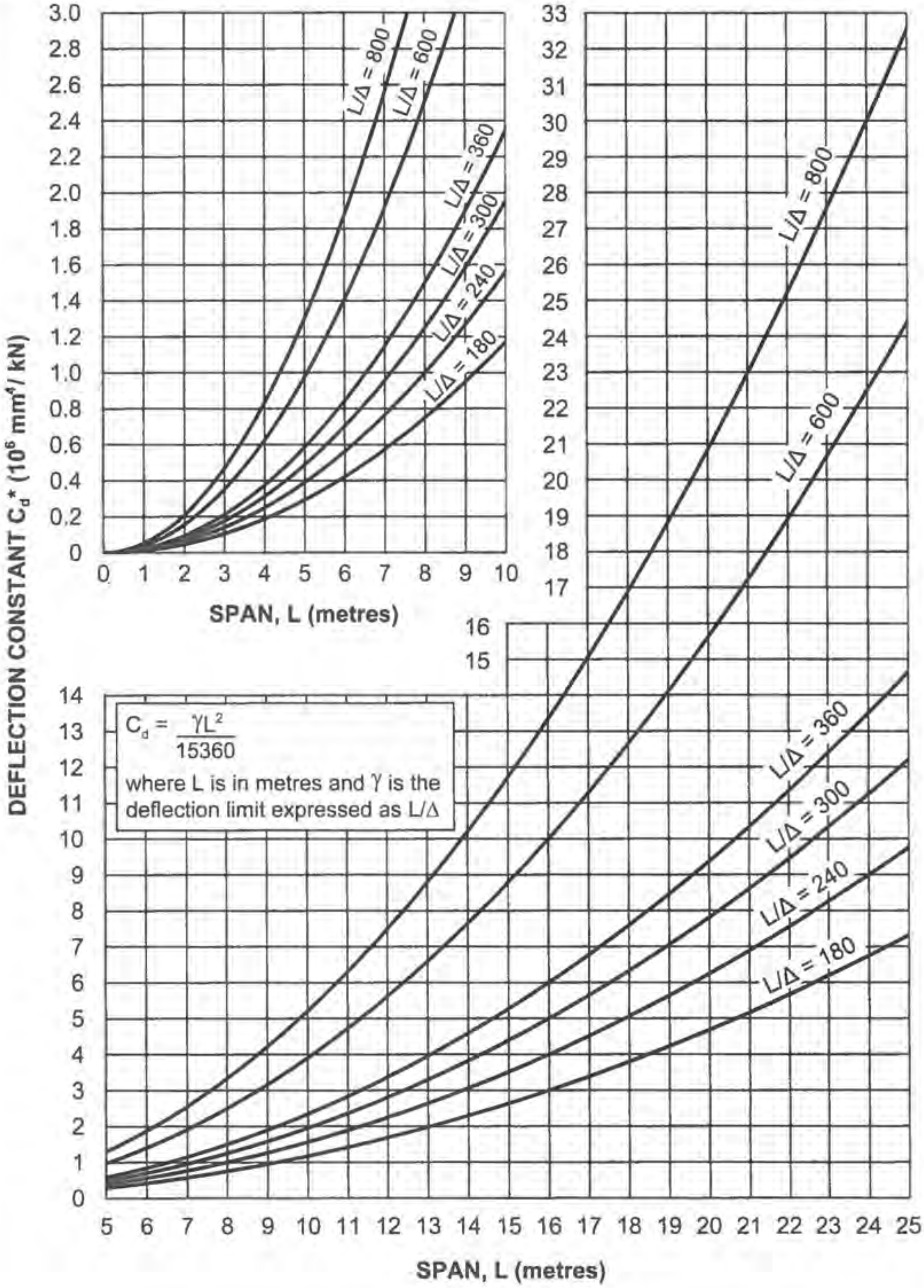
Stress due to live load is:

$$\frac{M}{S} = \frac{W L}{8 S} = \frac{(15 \times 10 \times 10^3)(10 \times 10^3)}{8 \times 1510 \times 10^3} = 124 \text{ MPa}$$

Live load deflection is $(124 / 240)\, 61 = 32$ mm

DEFLECTION CONSTANT C_d

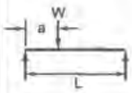
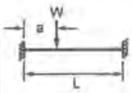
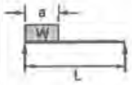
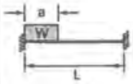
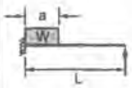
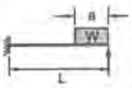
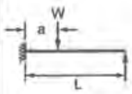
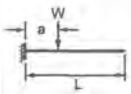
Figure 5-2

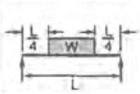
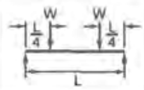
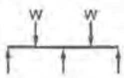
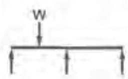
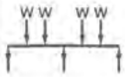
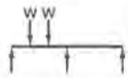
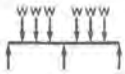
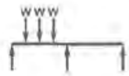
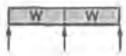
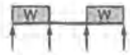
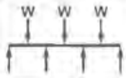
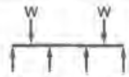


* C_d computed for simple-span UDL. For other conditions see Table 5-8 for values of B_d.

Table 5-8

Values of B_d for Various Loadings & Support Conditions

LOADING CONDITION	a/L	B_d	LOADING CONDITION	a/L	B_d
	1.0	0.000		1.0	0.000
	0.8	0.927		0.8	0.155
	0.6	1.52		0.6	0.366
	0.5	1.60		0.5	0.400
	0.4	1.52		0.4	0.366
	0.2	0.927		0.2	0.155
	1.0	1.00		1.0	0.200
	0.8	1.13		0.8	0.237
	0.6	1.10		0.6	0.233
	0.5	1.01		0.5	0.206
	0.4	0.869		0.4	0.163
	0.2	0.477		0.2	0.0576
	1.0	0.415		1.0	0.415
	0.8	0.456		0.8	0.503
	0.6	0.393		0.6	0.539
	0.5	0.325		0.5	0.520
	0.4	0.243		0.4	0.467
	0.2	0.0794		0.2	0.271
	1.0	0.000		1.0	25.6
	0.8	0.517		0.8	18.0
	0.6	0.752		0.6	11.1
	0.5	0.716		0.5	8.00
	0.4	0.590		0.4	5.32
	0.2	0.219		0.2	1.43

LOADING CONDITION	B_d	LOADING CONDITION	B_d
	1.00		0.200
	0.415		9.60
	1.43		2.24
	1.00		1.28
	0.716		1.15
	1.17		1.93
	1.60		2.69
	0.416		0.703
	0.529		0.760
	0.886		1.24

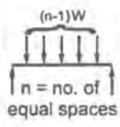
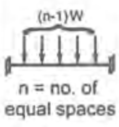
$I_{required} = W\, C_d\, B_d$

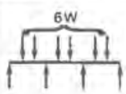
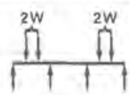
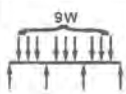
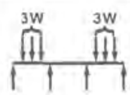
Where: I 10^6 mm^4

W kN

C_d from graph (Figure 5-2)

B_d from this table = 1.0 for single span, UDL

LOADING CONDITION	n	B_d	LOADING CONDITION	n	B_d
 n = no. of equal spaces	2	1.60	 n = no. of equal spaces	2	0.400
	3	2.73		3	0.593
	4	3.80		4	0.800
	5	4.84		5	0.998
	6	5.87		6	1.20
	7	6.89		7	1.40

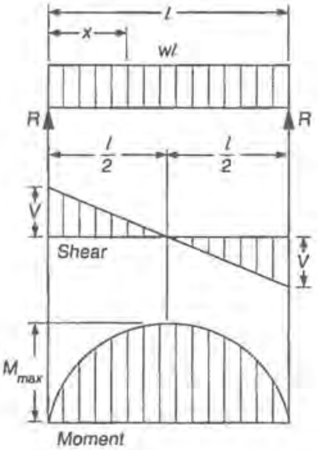
LOADING CONDITION	B_d	LOADING CONDITION	B_d
	1.47		2.09
	2.04		2.91

B_d is calculated at the position of maximum deflection. For multiple spans, each span length is L.

BEAM DIAGRAMS AND FORMULAS

Equivalent Tabular Load is the uniformly distributed factored load given in the Beam Load Tables.

1. SIMPLE BEAM — UNIFORMLY DISTRIBUTED LOAD



Equivalent Tabular Load .... $= wl$

$R = V$ .... $= \dfrac{wl}{2}$

V_x .... $= w\left(\dfrac{l}{2} - x\right)$

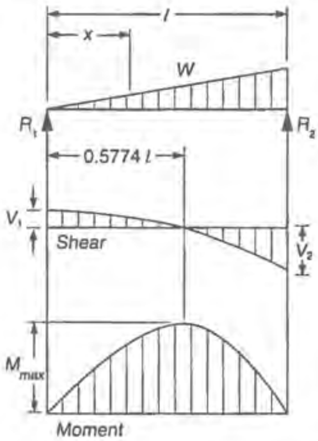
M max. (at center) .... $= \dfrac{wl^2}{8}$

M_x .... $= \dfrac{wx}{2}(l - x)$

Δ max. (at center) .... $= \dfrac{5wl^4}{384\ EI}$

Δ_x .... $= \dfrac{wx}{24\ EI}\left(l^3 - 2lx^2 + x^3\right)$

2. SIMPLE BEAM — LOAD INCREASING UNIFORMLY TO ONE END



Equivalent Tabular Load .... $= \dfrac{16W}{9\sqrt{3}} = 1.0264W$

$R_1 = V_1$ .... $= \dfrac{W}{3}$

$R_2 = V_2 = V$ max. .... $= \dfrac{2W}{3}$

V_x .... $= \dfrac{W}{3} - \dfrac{Wx^2}{l^2}$

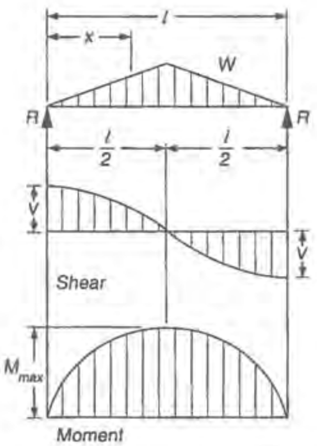
M max. $\left(\text{at } x = \dfrac{l}{\sqrt{3}} = 0.5774l\right)$ .... $= \dfrac{2Wl}{9\sqrt{3}} = .1283\ Wl$

M_x .... $= \dfrac{Wx}{3l^2}\left(l^2 - x^2\right)$

Δ max. $\left(\text{at } x = l\sqrt{1 - \sqrt{\dfrac{8}{15}}} = .5193\ l\right)$ .... $= .01304\ \dfrac{Wl^3}{EI}$

Δ_x .... $= \dfrac{Wx}{180\ EIl^2}\left(3x^4 - 10l^2x^2 + 7l^4\right)$

3. SIMPLE BEAM — LOAD INCREASING UNIFORMLY TO CENTER



Equivalent Tabular Load .... $= \dfrac{4W}{3}$

$R = V$ .... $= \dfrac{W}{2}$

$V_x\left(\text{when } x < \dfrac{l}{2}\right)$ .... $= \dfrac{W}{2l^2}\left(l^2 - 4x^2\right)$

M max. (at center) .... $= \dfrac{Wl}{6}$

$M_x\left(\text{when } x < \dfrac{l}{2}\right)$ .... $= Wx\left(\dfrac{1}{2} - \dfrac{2x^2}{3l^2}\right)$

Δ max. (at center) .... $= \dfrac{Wl^3}{60\ EI}$

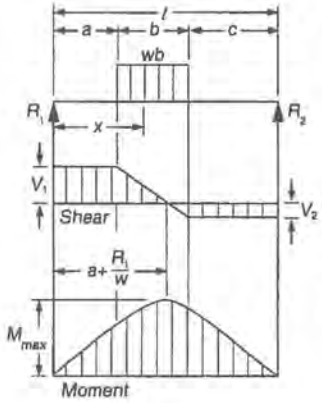
$\Delta_x\left(\text{when } x < \dfrac{l}{2}\right)$ .... $= \dfrac{Wx}{480\ EIl^2}\left(5l^2 - 4x^2\right)^2$

Note: For deflection calculations, use specified loads.

BEAM DIAGRAMS AND FORMULAS

Equivalent Tabular Load is the uniformly distributed factored load given in the Beam Load Tables.

4. SIMPLE BEAM — UNIFORM LOAD PARTIALLY DISTRIBUTED



$R_1 = V_1$ (max. when $a < c$) .... $= \dfrac{wb}{2l}(2c + b)$

$R_2 = V_2$ (max. when $a > c$) .... $= \dfrac{wb}{2l}(2a + b)$

V_x (when $x > a$ and $< (a + b)$) ... $= R_1 - w(x - a)$

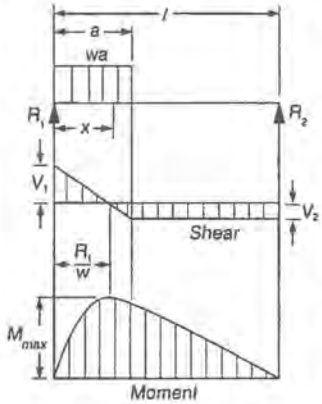
M max. $\left(\text{at } x = a + \dfrac{R_1}{w}\right)$ .... $= R_1\left(a + \dfrac{R_1}{2w}\right)$

M_x (when $x < a$) .... $= R_1 x$

M_x (when $x > a$ and $< (a + b)$) .... $= R_1 x - \dfrac{w}{2}(x - a)^2$

M_x (when $x > (a + b)$) .... $= R_2(l - x)$

5. SIMPLE BEAM — UNIFORM LOAD PARTIALLY DISTRIBUTED AT ONE END



$R_1 = V_1$ max. .... $= \dfrac{wa}{2l}(2l - a)$

$R_2 = V_2$ .... $= \dfrac{wa^2}{2l}$

V_x (when $x < a$) .... $= R_1 - wx$

M max. $\left(\text{at } x = \dfrac{R_1}{w}\right)$ .... $= \dfrac{R_1^2}{2w}$

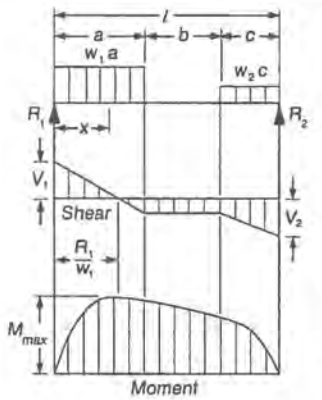
M_x (when $x < a$) .... $= R_1 x - \dfrac{wx^2}{2}$

M_x (when $x > a$) .... $= R_2(l - x)$

Δ_x (when $x < a$) .... $= \dfrac{wx}{24\,EIl}\left(a^2(2l - a)^2 - 2ax^2(2l - a) + lx^3\right)$

Δ_x (when $x > a$) .... $= \dfrac{wa^2(l - x)}{24\,EIl}(4xl - 2x^2 - a^2)$

6. SIMPLE BEAM — UNIFORM LOADS PARTIALLY DISTRIBUTED AT EACH END



$R_1 = V_1$ .... $= \dfrac{w_1 a(2l - a) + w_2 c^2}{2l}$

$R_2 = V_2$ .... $= \dfrac{w_2 c(2l - c) + w_1 a^2}{2l}$

V_x (when $x < a$) .... $= R_1 - w_1 x$

V_x (when $x > a$ and $< (a + b)$) .... $= R_1 - w_1 a$

V_x (when $x > (a + b)$) .... $= w_2(l - x) - R_2$

M max. $\left(\text{at } x = \dfrac{R_1}{w_1}, \text{ when } R_1 < w_1 a\right)$ .... $= \dfrac{R_1^2}{2w_1}$

M max. $\left(\text{at } x = l - \dfrac{R_2}{w_2}, \text{ when } R_2 < w_2 c\right) = \dfrac{R_2^2}{2w_2}$

M_x (when $x < a$) .... $= R_1 x - \dfrac{w_1 x^2}{2}$

M_x (when $x > a$ and $< (a + b)$) .... $= R_1 x - \dfrac{w_1 a}{2}(2x - a)$

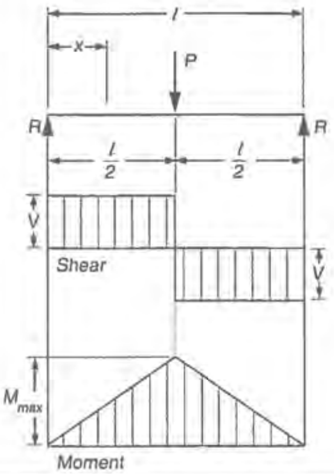
M_x (when $x > (a + b)$) .... $= R_2(l - x) - w_2\dfrac{(l - x)^2}{2}$

Note: For deflection calculations, use specified loads.

BEAM DIAGRAMS AND FORMULAS

Equivalent Tabular Load is the uniformly distributed factored load given in the Beam Load Tables.

7. SIMPLE BEAM — CONCENTRATED LOAD AT CENTER



Equivalent Tabular Load .... $= 2P$

$R = V$ .... $= \frac{P}{2}$

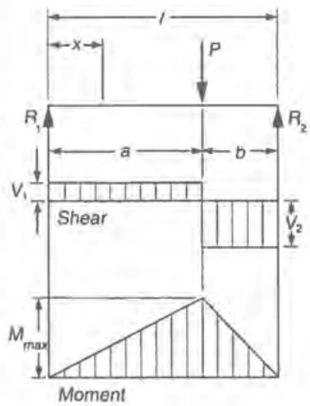
M max. (at point of load) .... $= \frac{Pl}{4}$

M_x (when $x < \frac{l}{2}$) .... $= \frac{Px}{2}$

Δ max. (at point of load) .... $= \frac{Pl^3}{48EI}$

Δ_x (when $x < \frac{l}{2}$) .... $= \frac{Px}{48EI}\left(3l^2 - 4x^2\right)$

8. SIMPLE BEAM — CONCENTRATED LOAD AT ANY POINT



Equivalent Tabular Load .... $= \frac{8Pab}{l^2}$

$R_1 = V_1$ (max. when $a < b$) .... $= \frac{Pb}{l}$

$R_2 = V_2$ (max. when $a > b$) .... $= \frac{Pa}{l}$

M max. (at point of load) .... $= \frac{Pab}{l}$

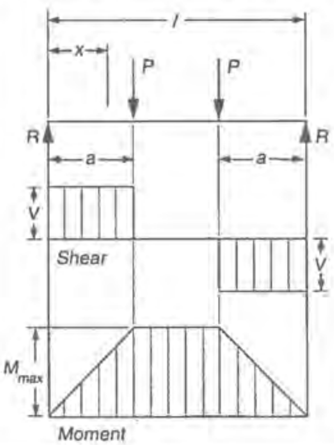
M_x (when $x < a$) .... $= \frac{Pbx}{l}$

Δ max. $\left(\text{at } x = \sqrt{\frac{a(a+2b)}{3}} \text{ when } a > b\right)$ ..... $= \frac{Pab(a+2b)\sqrt{3a(a+2b)}}{27\,EIl}$

Δ_a (at point of load) .... $= \frac{Pa^2b^2}{3\,EIl}$

Δ_x (when $x < a$) .... $= \frac{Pbx}{6\,EIl}\left(l^2 - b^2 - x^2\right)$

9. SIMPLE BEAM — TWO EQUAL CONCENTRATED LOADS SYMMETRICALLY PLACED



Equivalent Tabular Load .... $= \frac{8\,Pa}{l}$

$R = V$ .... $= P$

M max. (between loads) .... $= Pa$

M_x (when $x < a$) .... $= Px$

Δ max. (at center) .... $= \frac{Pa}{24\,EI}\left(3l^2 - 4a^2\right)$

Δ_x (when $x < a$) .... $= \frac{Px}{6\,EI}\left(3la - 3a^2 - x^2\right)$

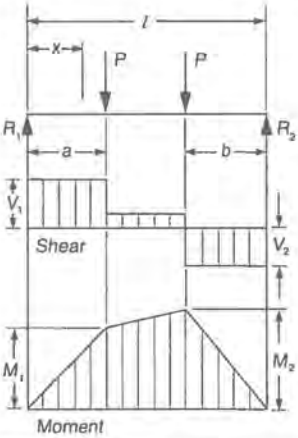
Δ_x (when $x > a$ and $< (l - a)$) .... $= \frac{Pa}{6\,EI}\left(3lx - 3x^2 - a^2\right)$

Note: For deflection calculations, use specified loads.

BEAM DIAGRAMS AND FORMULAS

Equivalent Tabular Load is the uniformly distributed factored load given in the Beam Load Tables.

10. SIMPLE BEAM — TWO EQUAL CONCENTRATED LOADS UNSYMMETRICALLY PLACED



$R_1 = V_1$ (max. when $a < b$) .... $= \frac{P}{l}(l - a + b)$

$R_2 = V_2$ (max. when $a > b$) .... $= \frac{P}{l}(l - b + a)$

V_x (when $x > a$ and $< (l - b)$) .... $= \frac{P}{l}(b - a)$

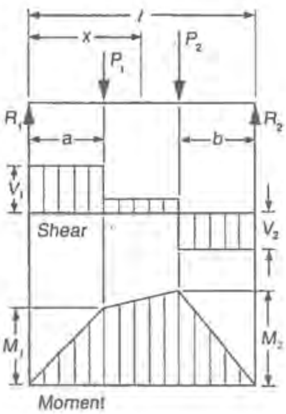
M_1 (max. when $a > b$) .... $= R_1 a$

M_2 (max. when $a < b$) .... $= R_2 b$

M_x (when $x < a$) .... $= R_1 x$

M_x (when $x > a$ and $< (l - b)$) .... $= R_1 x - P(x - a)$

11. SIMPLE BEAM — TWO UNEQUAL CONCENTRATED LOADS UNSYMMETRICALLY PLACED



$R_1 = V_1$ .... $= \frac{P_1(l - a) + P_2 b}{l}$

$R_2 = V_2$ .... $= \frac{P_1 a + P_2(l - b)}{l}$

V_x (when $x > a$ and $< (l - b)$) .... $= R_1 - P_1$

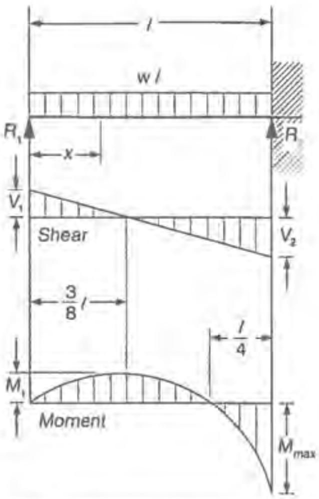
M_1 (max. when $R_1 < P_1$) .... $= R_1 a$

M_2 (max. when $R_2 < P_2$) .... $= R_2 b$

M_x (when $x < a$) .... $= R_1 x$

M_x (when $x > a$ and $< (l - b)$) .... $= R_1 x - P_1(x - a)$

12. BEAM FIXED AT ONE END, SUPPORTED AT OTHER — UNIFORMLY DISTRIBUTED LOAD



Equivalent Tabular Load .... $= wl$

$R_1 = V_1$ .... $= \frac{3wl}{8}$

$R_2 = V_2$ max. .... $= \frac{5wl}{8}$

V_x .... $= R_1 - wx$

M max. .... $= \frac{wl^2}{8}$

M_1 (at $x = \frac{3}{8}l$) .... $= \frac{9}{128}wl^2$

M_x .... $= R_1 x - \frac{wx^2}{2}$

Δ max. $\left(\text{at } x = \frac{l}{16}(1 + \sqrt{33}) = .4215l\right)$ .... $= \frac{wl^4}{185\,EI}$

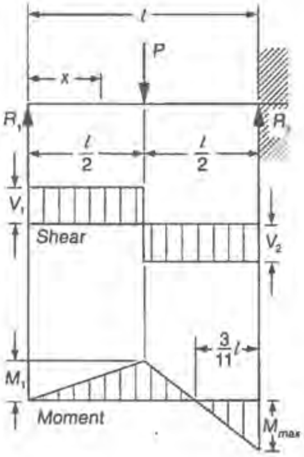
Δ_x .... $= \frac{wx}{48\,EI}(l^3 - 3lx^2 + 2x^3)$

Note: For deflection calculations, use specified loads.

BEAM DIAGRAMS AND FORMULAS

Equivalent Tabular Load is the uniformly distributed factored load given in the Beam Load Tables.

13. BEAM FIXED AT ONE END, SUPPORTED AT OTHER — CONCENTRATED LOAD AT CENTER



Equivalent Tabular Load .... $= \frac{3P}{2}$

$R_1 = V_1$ .... $= \frac{5P}{16}$

$R_2 = V_2$ max. .... $= \frac{11P}{16}$

M max. (at fixed end) .... $= \frac{3Pl}{16}$

M_1 (at point of load) .... $= \frac{5Pl}{32}$

M_x (when $x < \frac{l}{2}$) .... $= \frac{5Px}{16}$

M_x (when $x > \frac{l}{2}$) .... $= P\left(\frac{l}{2} - \frac{11x}{16}\right)$

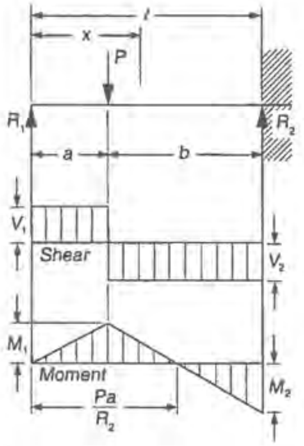
Δ max. $\left(\text{at } x = l\sqrt{\frac{1}{5}} = .4472l\right)$ .... $= \frac{Pl^3}{48\,EI\sqrt{5}} = .009317\,\frac{Pl^3}{EI}$

Δ_x (at point of load) .... $= \frac{7Pl^3}{768\,EI}$

$\Delta_x \left(\text{when } x < \frac{l}{2}\right)$ .... $= \frac{Px}{96\,EI}\left(3l^2 - 5x^2\right)$

$\Delta_x \left(\text{when } x > \frac{l}{2}\right)$ .... $= \frac{P}{96\,EI}\left(x - l\right)^2\left(11x - 2l\right)$

14. BEAM FIXED AT ONE END, SUPPORTED AT OTHER — CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1$ .... $= \frac{Pb^2}{2l^3}\,(a + 2l)$

$R_2 = V_2$ .... $= \frac{Pa}{2l^3}\,(3l^2 - a^2)$

M_1 (at point of load) .... $= R_1 a$

M_2 (at fixed end) .... $= \frac{Pab}{2l^2}\,(a + l)$

M_x (when $x < a$) .... $= R_1 x$

M_x (when $x > a$) .... $= R_1 x - P\,(x - a)$

Δ max. $\left(\text{when } a < .414l \text{ at } x = l\,\frac{l^2 + a^2}{3l^2 - a^2}\right) = \frac{Pa}{3EI}\,\frac{(l^2 - a^2)^3}{(3l^2 - a^2)^2}$

Δ max. $\left(\text{when } a > .414l \text{ at } x = l\sqrt{\frac{a}{2l + a}}\right) = \frac{Pab^2}{6\,EI}\sqrt{\frac{a}{2l + a}}$

Δ_a (at point of load) .... $= \frac{Pa^2b^3}{12\,EI\,l^3}\,(3l + a)$

Δ_x (when $x < a$) .... $= \frac{Pb^2x}{12\,EI\,l^3}\left(3al^2 - 2lx^2 - ax^2\right)$

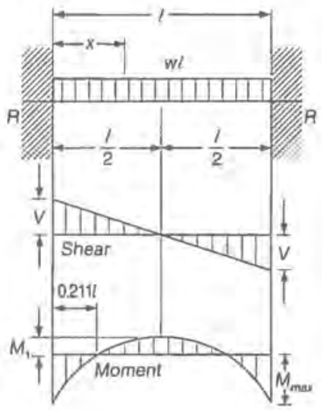
Δ_x (when $x > a$) .... $= \frac{Pa}{12\,EI\,l^3}\left(l - x\right)^2\left(3l^2x - a^2x - 2a^2l\right)$

Note: For deflection calculations, use specified loads.

BEAM DIAGRAMS AND FORMULAS

Equivalent Tabular Load is the uniformly distributed factored load given in the Beam Load Tables.

15. BEAM FIXED AT BOTH ENDS — UNIFORMLY DISTRIBUTED LOADS



Equivalent Tabular Load .... $= \frac{2wl}{3}$

$R = V$ .... $= \frac{wl}{2}$

V_x .... $= w\left(\frac{l}{2} - x\right)$

M max. (at ends) .... $= \frac{wl^2}{12}$

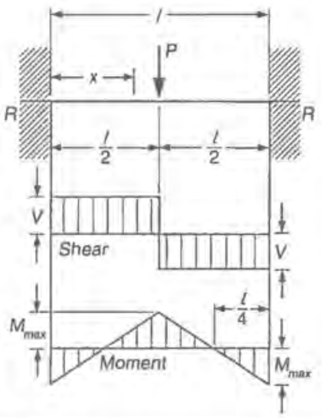
M_1 (at center) .... $= \frac{wl^2}{24}$

M_x .... $= \frac{w}{12}\left(6lx - l^2 - 6x^2\right)$

Δ max. (at center) .... $= \frac{wl^4}{384\ EI}$

Δ_x .... $= \frac{wx^2}{24\ EI}\left(l - x\right)^2$

16. BEAM FIXED AT BOTH ENDS — CONCENTRATED LOAD AT CENTER



Equivalent Tabular Load .... $= P$

$R = V$ .... $= \frac{P}{2}$

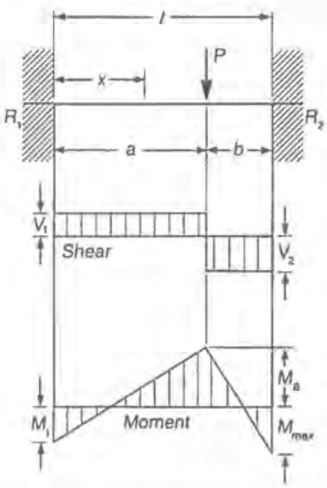
M max. (at center and ends) .... $= \frac{Pl}{8}$

$M_x\left(\text{when } x < \frac{l}{2}\right)$ .... $= \frac{P}{8}(4x - l)$

Δ max. (at center) .... $= \frac{Pl^3}{192\ EI}$

$\Delta_x\left(\text{when } x < \frac{l}{2}\right)$ .... $= \frac{Px^2}{48\ EI}(3l - 4x)$

17. BEAM FIXED AT BOTH ENDS — CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1$ (max. when $a < b$) .... $= \frac{Pb^2}{l^3}(3a + b)$

$R_2 = V_2$ (max. when $a > b$) .... $= \frac{Pa^2}{l^3}(a + 3b)$

M_1 (max. when $a < b$) .... $= \frac{Pab^2}{l^2}$

M_2 (max. when $a > b$) .... $= \frac{Pa^2b}{l^2}$

M_a (at point of load) .... $= \frac{2Pa^2b^2}{l^3}$

M_x (when $x < a$) .... $= R_1x - \frac{Pab^2}{l^2}$

Δ max. $\left(\text{when } a > b \text{ at } x = \frac{2al}{3a + b}\right)$ .... $= \frac{2Pa^3b^2}{3\ EI\ (3a + b)^2}$

Δ_a (at point of load) .... $= \frac{Pa^3b^3}{3\ EI\ l^3}$

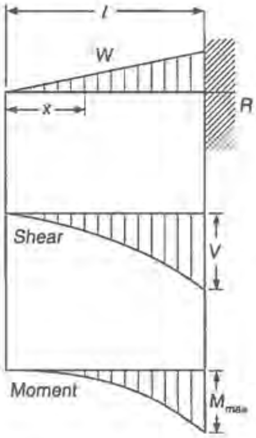
Δ_x (when $x < a$) .... $= \frac{Pb^2x^2}{6\ EI\ l^3}(3al - 3ax - bx)$

Note: For deflection calculations, use specified loads.

BEAM DIAGRAMS AND FORMULAS

Equivalent Tabular Load is the uniformly distributed factored load given in the Beam Load Tables.

18. CANTILEVER BEAM — LOAD INCREASING UNIFORMLY TO FIXED END



Equivalent Tabular Load .... $= \frac{8}{3} W$

$R = V$ .... $= W$

V_x .... $= W \frac{x^2}{l^2}$

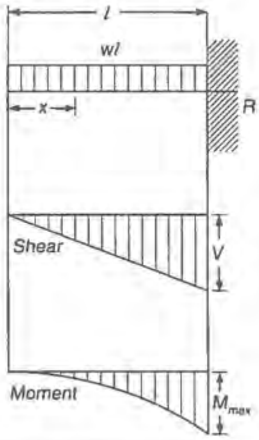
M max. (at fixed end) .... $= \frac{Wl}{3}$

M_x .... $= \frac{Wx^3}{3l^2}$

Δ max. (at free end) .... $= \frac{Wl^3}{15\ EI}$

Δ_x .... $= \frac{W}{60\ EI\ l^2}\left(x^5 - 5l^4x + 4l^5\right)$

19. CANTILEVER BEAM — UNIFORMLY DISTRIBUTED LOAD



Equivalent Tabular Load .... $= 4wl$

$R = V$ .... $= wl$

V_x .... $= wx$

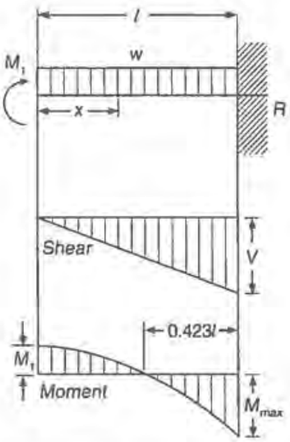
M max. (at fixed end) .... $= \frac{wl^2}{2}$

M_x .... $= \frac{wx^2}{2}$

Δ max. (at free end) .... $= \frac{wl^4}{8\ EI}$

Δ_x .... $= \frac{w}{24\ EI}\left(x^4 - 4l^3x + 3l^4\right)$

20. BEAM FIXED AT ONE END, FREE BUT GUIDED AT OTHER — UNIFORMLY DISTRIBUTED LOAD



Equivalent Tabular Load .... $= \frac{8}{3} wl$

$R = V$ .... $= wl$

V_x .... $= wx$

M max. (at fixed end) .... $= \frac{wl^2}{3}$

M_1 (at deflected end) .... $= \frac{wl^2}{6}$

M_x .... $= \frac{w}{6}\left(l^2 - 3x^2\right)$

Δ max. (at deflected end) .... $= \frac{wl^4}{24\ EI}$

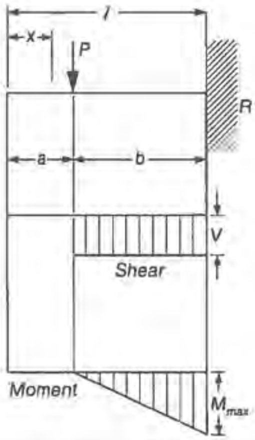
Δ_x .... $= \frac{w\left(l^2 - x^2\right)^2}{24\ EI}$

Note: For deflection calculations, use specified loads.

BEAM DIAGRAMS AND FORMULAS

Equivalent Tabular Load is the uniformly distributed factored load given in the Beam Load Tables.

21. CANTILEVER BEAM — CONCENTRATED LOAD AT ANY POINT



Equivalent Tabular Load .... $= \dfrac{8Pb}{l}$

$R = V$ (when $x < a$) .... $= P$

M max. (at fixed end) .... $= Pb$

M_x (when $x > a$) .... $= P(x - a)$

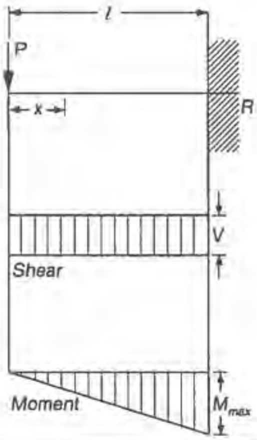
Δ max. (at free end) .... $= \dfrac{Pb^2}{6\,EI}(3l - b)$

Δ_a (at point of load) .... $= \dfrac{Pb^3}{3\,EI}$

Δ_x (when $x < a$) .... $= \dfrac{Pb^2}{6\,EI}(3l - 3x - b)$

Δ_x (when $x > a$) .... $= \dfrac{P(l - x)^2}{6\,EI}(3b - l + x)$

22. CANTILEVER BEAM — CONCENTRATED LOAD AT FREE END



Equivalent Tabular Load .... $= 8P$

$R = V$ .... $= P$

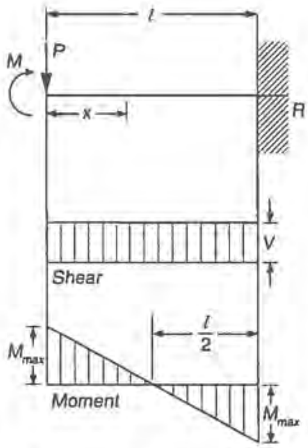
M max. (at fixed end) .... $= Pl$

M_x .... $= Px$

Δ max. (at free end) .... $= \dfrac{Pl^3}{3\,EI}$

Δ_x .... $= \dfrac{P}{6\,EI}\left(2l^3 - 3l^2x + x^3\right)$

23. BEAM FIXED AT ONE END, FREE BUT GUIDED AT OTHER — CONCENTRATED LOAD AT GUIDED END



Equivalent Tabular Load .... $= 4P$

$R = V$ .... $= P$

M max. (at both ends) .... $= \dfrac{Pl}{2}$

M_x .... $= P\left(\dfrac{l}{2} - x\right)$

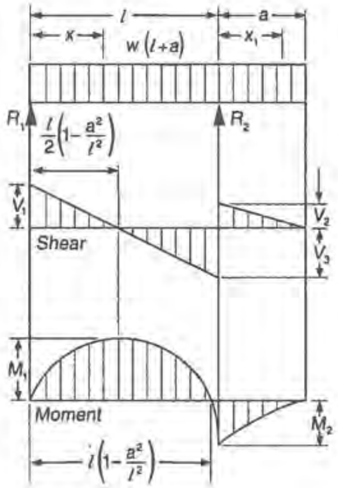
Δ max. (at deflected end) .... $= \dfrac{Pl^3}{12\,EI}$

Δ_x .... $= \dfrac{P(l - x)^2}{12\,EI}(l + 2x)$

Note: For deflection calculations, use specified loads.

Equivalent Tabular Load is the uniformly distributed factored load given in the Beam Load Tables.

24. BEAM OVERHANGING ONE SUPPORT — UNIFORMLY DISTRIBUTED LOAD



$R_1 = V_1$ .... $= \frac{w}{2l}\left(l^2 - a^2\right)$

$R_2 = V_2 + V_3$ .... $= \frac{w}{2l}\left(l + a\right)^2$

V_2 .... $= wa$

V_3 .... $= \frac{w}{2l}\left(l^2 + a^2\right)$

V_x (between supports) .... $= R_1 - wx$

V_{x_1} (for overhang) .... $= w\left(a - x_1\right)$

$M_1 \left(\text{at } x = \frac{l}{2}\left[1 - \frac{a^2}{l^2}\right]\right)$ .... $= \frac{w}{8l^2}\left(l+a\right)^2\left(l-a\right)^2$

$M_2 \left(\text{at } R_2\right)$ .... $= \frac{wa^2}{2}$

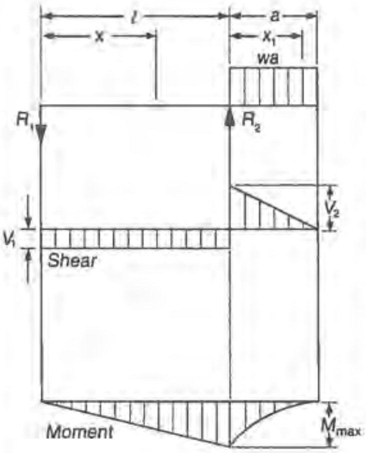
M_x (between supports) .... $= \frac{wx}{2l}\left(l^2 - a^2 - xl\right)$

M_{x_1} (for overhang) .... $= \frac{w}{2}\left(a - x_1\right)^2$

Δ_x (between supports) .... $= \frac{wx}{24\,EIl}\left(l^4 - 2l^2x^2 + lx^3 - 2a^2l^2 + 2a^2x^2\right)$

Δ_{x_1} (for overhang) .... $= \frac{wx_1}{24\,EI}\left(4a^2l - l^3 + 6a^2x_1 - 4ax_1^2 + x_1^3\right)$

25. BEAM OVERHANGING ONE SUPPORT — UNIFORMLY DISTRIBUTED LOAD ON OVERHANG



$R_1 = V_1$ .... $= \frac{wa^2}{2l}$

$R_2 = V_1 + V_2$ .... $= \frac{wa}{2l}\left(2l + a\right)$

V_2 .... $= wa$

V_{x_1} (for overhang) .... $= w\left(a - x_1\right)$

M max. $\left(\text{at } R_2\right)$ .... $= \frac{wa^2}{2}$

M_x (between supports) .... $= \frac{wa^2x}{2l}$

M_{x_1} (for overhang) .... $= \frac{w}{2}\left(a - x_1\right)^2$

Δ max. $\left(\text{between supports at } x = \frac{l}{\sqrt{3}}\right) = \frac{wa^2l^2}{18\sqrt{3}\,EI} = .03208\,\frac{wa^2l^2}{EI}$

Δ max. $\left(\text{for overhang at } x_1 = a\right)$......... $= \frac{wa^3}{24\,EI}\left(4l + 3a\right)$

Δ_x (between supports) .... $= \frac{wa^2x}{12\,EIl}\left(l^2 - x^2\right)$

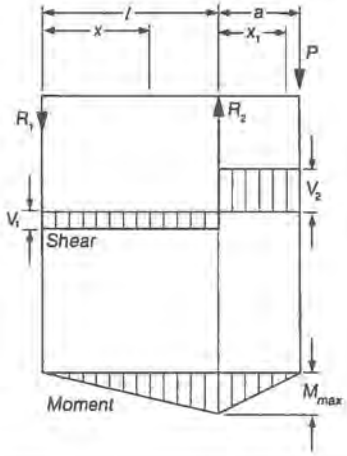
Δ_{x_1} (for overhang) .... $= \frac{wx_1}{24\,EI}\left(4a^2l + 6a^2x_1 - 4ax_1^2 + x_1^3\right)$

Note: For deflection calculations, use specified loads.

BEAM DIAGRAMS AND FORMULAS

Equivalent Tabular Load is the uniformly distributed factored load given in the Beam Load Tables.

26. BEAM OVERHANGING ONE SUPPORT — CONCENTRATED LOAD AT END OF OVERHANG



$R_1 = V_1 \dots\dots = \frac{Pa}{l}$

$R_2 = V_1 + V_2 \dots\dots = \frac{P}{l}(l + a)$

$V_2 \dots\dots = P$

M max. (at R_2) $\dots\dots = Pa$

M_x (between supports) $\dots\dots = \frac{Pax}{l}$

M_{x_1} (for overhang) $\dots\dots = P(a - x_1)$

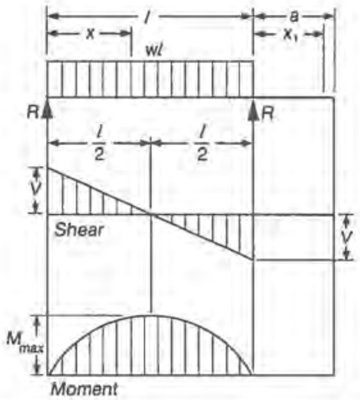
Δ max. $\left(\text{between supports at } x = \frac{l}{\sqrt{3}}\right) \dots\dots = \frac{Pal^2}{9\sqrt{3}\,EI} = .06415\,\frac{Pal^2}{EI}$

Δ max. (for overhang at $x_1 = a$) $\dots\dots = \frac{Pa^2}{3\,EI}(l + a)$

Δ_x (between supports) $\dots\dots = \frac{Pax}{6\,EIl}(l^2 - x^2)$

Δ_{x_1} (for overhang) $\dots\dots = \frac{Px_1}{6\,EI}(2al + 3ax_1 - x_1^2)$

27. BEAM OVERHANGING ONE SUPPORT — UNIFORMLY DISTRIBUTED LOAD BETWEEN SUPPORTS



Equivalent Tabular Load $\dots\dots = wl$

$R = V \dots\dots = \frac{wl}{2}$

$V_x \dots\dots = w\left(\frac{l}{2} - x\right)$

M max. (at center) $\dots\dots = \frac{wl^2}{8}$

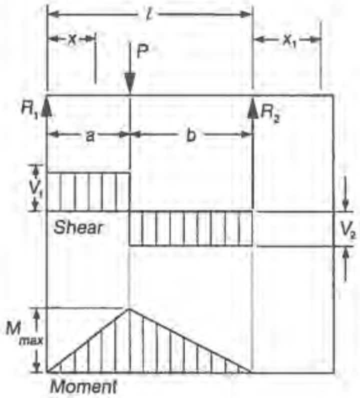
$M_x \dots\dots = \frac{wx}{2}(l - x)$

Δ max. (at center) $\dots\dots = \frac{5wl^4}{384\,EI}$

$\Delta_x \dots\dots = \frac{wx}{24\,EI}(l^3 - 2lx^2 + x^3)$

$\Delta_{x_1} \dots\dots = \frac{wl^3x_1}{24\,EI}$

28. BEAM OVERHANGING ONE SUPPORT — CONCENTRATED LOAD ANY POINT BETWEEN SUPPORTS



Equivalent Tabular Load $\dots\dots = \frac{8\,Pab}{l^2}$

$R_1 = V_1$ (max. when $a < b$) $\dots\dots = \frac{Pb}{l}$

$R_2 = V_2$ (max. when $a > b$) $\dots\dots = \frac{Pa}{l}$

M max. (at point of load) $\dots\dots = \frac{Pab}{l}$

M_x (when $x < a$) $\dots\dots = \frac{Pbx}{l}$

Δ max. $\left(\text{at } x = \sqrt{\frac{a(a+2b)}{3}} \text{ when } a > b\right) = \frac{Pab\,(a + 2b)\sqrt{3a\,(a + 2b)}}{27\,EI\,l}$

Δ_a (at point of load) $\dots\dots = \frac{Pa^2b^2}{3\,EIl}$

Δ_x (when $x < a$) $\dots\dots = \frac{Pbx}{6\,EIl}(l^2 - b^2 - x^2)$

Δ_x (when $x > a$) $\dots\dots = \frac{Pa\,(l - x)}{6\,EIl}(2lx - x^2 - a^2)$

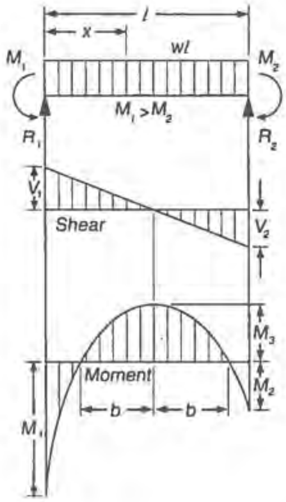
$\Delta_{x_1} \dots\dots = \frac{Pabx_1}{6\,EIl}(l + a)$

Note: For deflection calculations, use specified loads.

BEAM DIAGRAMS AND FORMULAS

Equivalent Tabular Load is the uniformly distributed factored load given in the Beam Load Tables.

29. BEAM — UNIFORMLY DISTRIBUTED LOAD AND VARIABLE END MOMENTS



$$R_1 = V_1 = \frac{wl}{2} + \frac{M_1 - M_2}{l}$$

$$R_2 = V_2 = \frac{wl}{2} - \frac{M_1 - M_2}{l}$$

$$V_x = w\left(\frac{l}{2} - x\right) + \frac{M_1 - M_2}{l}$$

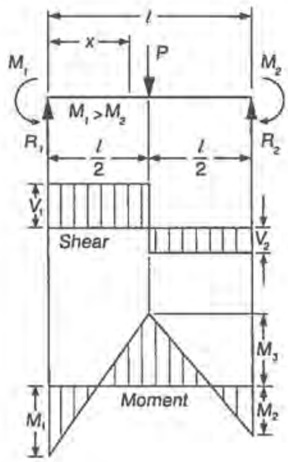
$$M_3\left(\text{at } x = \frac{l}{2} + \frac{M_1 - M_2}{wl}\right) = \frac{wl^2}{8} - \frac{M_1 + M_2}{2} + \frac{(M_1 - M_2)^2}{2wl^2}$$

$$M_x = \frac{wx}{2}(l - x) + \left(\frac{M_1 - M_2}{l}\right)x - M_1$$

$$b \text{ (To locate infection points)} = \sqrt{\frac{l^2}{4} - \left(\frac{M_1 + M_2}{w}\right) + \left(\frac{M_1 - M_2}{wl}\right)^2}$$

$$\Delta_x = \frac{wx}{24\,EI}\left[x^3 - \left(2l + \frac{4M_1}{wl} - \frac{4M_2}{wl}\right)x^2 + \frac{12M_1}{w}x + l^3 - \frac{8M_1 l}{w} - \frac{4M_2 l}{w}\right]$$

30. BEAM — CONCENTRATED LOAD AT CENTER AND VARIABLE END MOMENTS



$$R_1 = V_1 = \frac{P}{2} + \frac{M_1 - M_2}{l}$$

$$R_2 = V_2 = \frac{P}{2} - \frac{M_1 - M_2}{l}$$

$$M_3 \text{ (at center)} = \frac{Pl}{4} - \frac{M_1 + M_2}{2}$$

$$M_x\left(\text{when } x < \frac{l}{2}\right) = \left(\frac{P}{2} + \frac{M_1 - M_2}{l}\right)x - M_1$$

$$M_x\left(\text{when } x > \frac{l}{2}\right) = \frac{P}{2}(l - x) + \frac{(M_1 - M_2)x}{l} - M_1$$

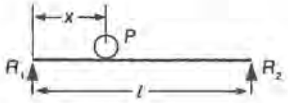
$$\Delta_x\left(\text{when } x < \frac{l}{2}\right) = \frac{Px}{48\,EI}\left(3l^2 - 4x^2 - \frac{8(l - x)}{Pl}\left[M_1(2l - x) + M_2(l + x)\right]\right)$$

Note: For deflection calculations, use specified loads.

BEAM DIAGRAMS AND FORMULAS

Equivalent Tabular Load is the uniformly distributed factored load given in the Beam Load Tables.

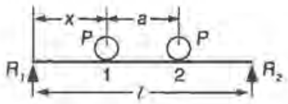
31. SIMPLE BEAM — ONE CONCENTRATED MOVING LOAD



R_1 max. = V_1 max. (at $x = 0$) .... = P

M max. $\left(\text{at point of load, when } x = \frac{l}{2}\right)$ .... = $\frac{Pl}{4}$

32. SIMPLE BEAM — TWO EQUAL CONCENTRATED MOVING LOADS

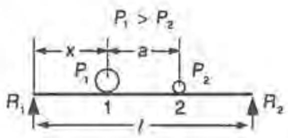


R_1 max. = V_1 max. (at $x = 0$) .... = $P\left(2 - \frac{a}{l}\right)$

M max.

- when $a < (2 - \sqrt{2})\,l = .586l$ under load 1 at $x = \frac{1}{2}\left(l - \frac{a}{2}\right)$ ...... = $\frac{P}{2l}\left(l - \frac{a}{2}\right)^2$
- when $a > (2 - \sqrt{2})\,l = .586l$ with one load at center of span (case 31) ...... = $\frac{Pl}{4}$

33. SIMPLE BEAM — TWO UNEQUAL CONCENTRATED MOVING LOADS

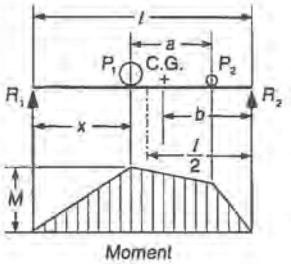


R_1 max. = V_1 max. (at $x = 0$) .... = $P_1 + P_2\frac{l - a}{l}$

M max.

- under P_1, at $x = \frac{1}{2}\left(l - \frac{P_2 a}{P_1 + P_2}\right)$ = $(P_1 + P_2)\frac{x^2}{l}$
- M max. may occur with larger load at center of span and other load off span (case 31) ..... = $\frac{P_1 l}{4}$

GENERAL RULES FOR SIMPLE BEAMS CARRYING MOVING CONCENTRATED LOADS



Moment

The maximum shear due to moving concentrated loads occurs at one support when one of the loads is at that support. With several moving loads, the location that will produce maximum shear must be determined by trial.

The maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support.

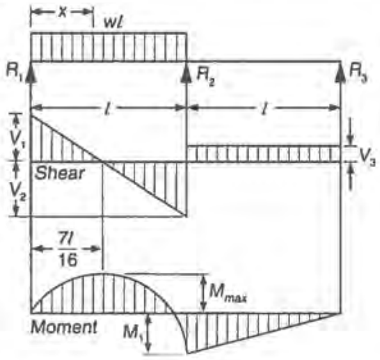
In the accompanying diagram, the maximum bending moment occurs under load P_1 when $x = b$. It should also be noted that this condition occurs when the center line of the span is midway between the center of gravity of loads and the nearest concentrated load.

Note: For deflection calculations, use specified loads.

BEAM DIAGRAMS AND FORMULAS

Equivalent Tabular Load is the uniformly distributed factored load given in the Beam Load Tables.

34. CONTINUOUS BEAM — TWO EQUAL SPANS — UNIFORM LOAD ON ONE SPAN



Equivalent Tabular Load .... $= \frac{49}{64} wl$

$R_1 = V_1$ .... $= \frac{7}{16} wl$

$R_2 = V_2 + V_3$ .... $= \frac{5}{8} wl$

$R_3 = V_3$ .... $= -\frac{1}{16} wl$

V_2 .... $= \frac{9}{16} wl$

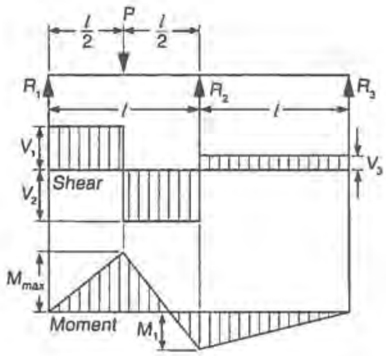
M max. $\left(\text{at } x = \frac{7}{16} l\right)$ .... $= \frac{49}{512} wl^2$

M_1 (at support R_2) .... $= \frac{1}{16} wl^2$

M_x (when $x < l$) .... $= \frac{wx}{16}(7l - 8x)$

Δ max. $\left(0.472\ l \text{ from } R_1\right)$ .... $= 0.0092\ wl^4/EI$

35. CONTINUOUS BEAM — TWO EQUAL SPANS — CONCENTRATED LOAD AT CENTER OF ONE SPAN



Equivalent Tabular Load .... $= \frac{13}{8} P$

$R_1 = V_1$ .... $= \frac{13}{32} P$

$R_2 = V_2 + V_3$ .... $= \frac{11}{16} P$

$R_3 = V_3$ .... $= -\frac{3}{32} P$

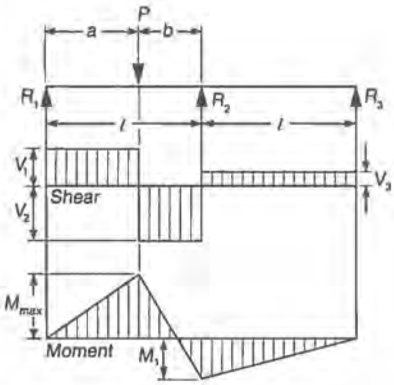
V_2 .... $= \frac{19}{32} P$

M max. (at point of load) .... $= \frac{13}{64} Pl$

M_1 (at support R_2) .... $= \frac{3}{32} Pl$

Δ max. $\left(0.480\ l \text{ from } R_1\right)$ .... $= 0.015\ Pl^3/EI$

36. CONTINUOUS BEAM — TWO EQUAL SPANS — CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1$ .... $= \frac{Pb}{4l^3}\left(4l^2 - a(l + a)\right)$

$R_2 = V_2 + V_3$ .... $= \frac{Pa}{2l^3}\left(2l^2 + b(l + a)\right)$

$R_3 = V_3$ .... $= -\frac{Pab}{4l^3}(l + a)$

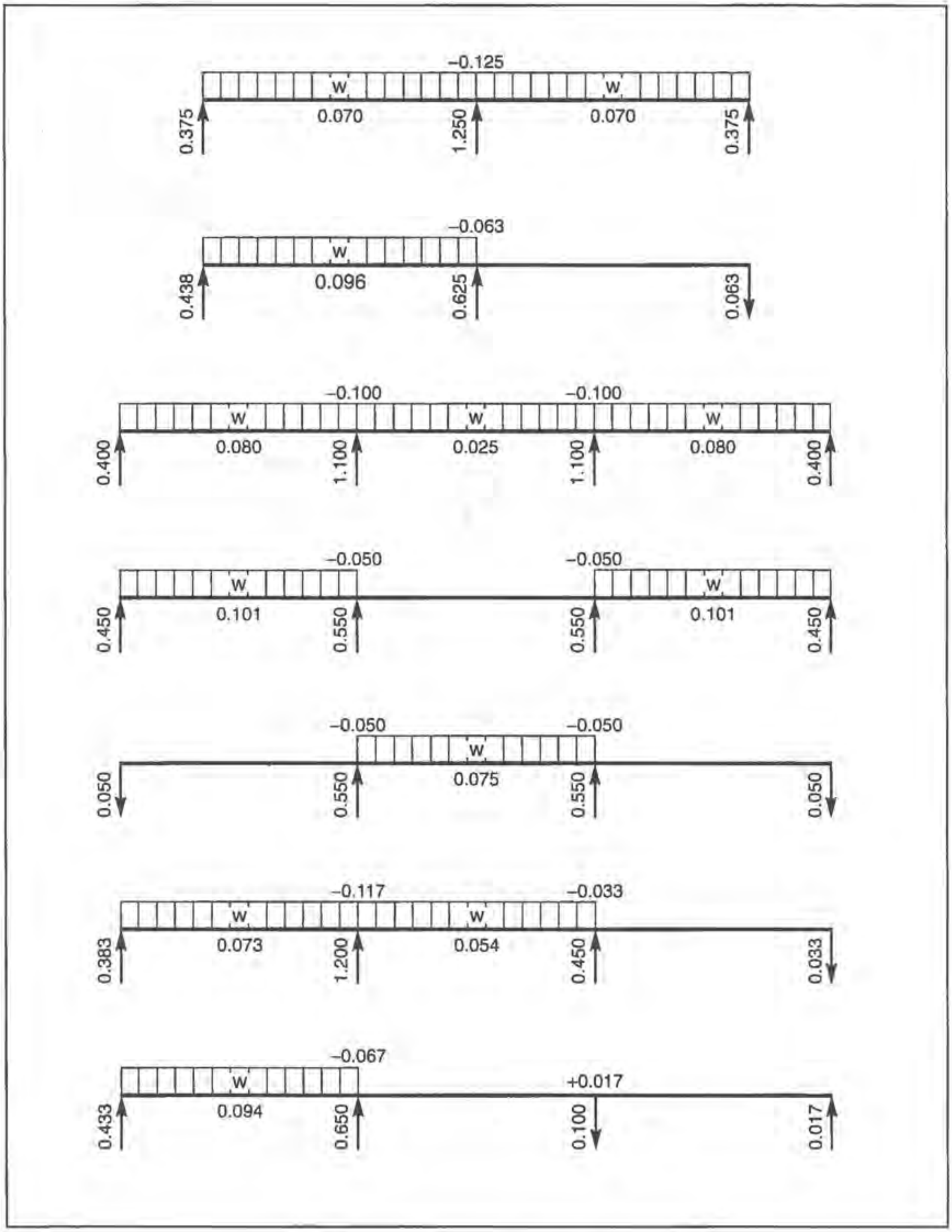
V_2 .... $= \frac{Pa}{4l^3}\left(4l^2 + b(l + a)\right)$

M max. (at point of load) .... $= \frac{Pab}{4l^3}\left(4l^2 - a(l + a)\right)$

M_1 (at support R_2) .... $= \frac{Pab}{4l^2}(l + a)$

Note: For deflection calculations, use specified loads.

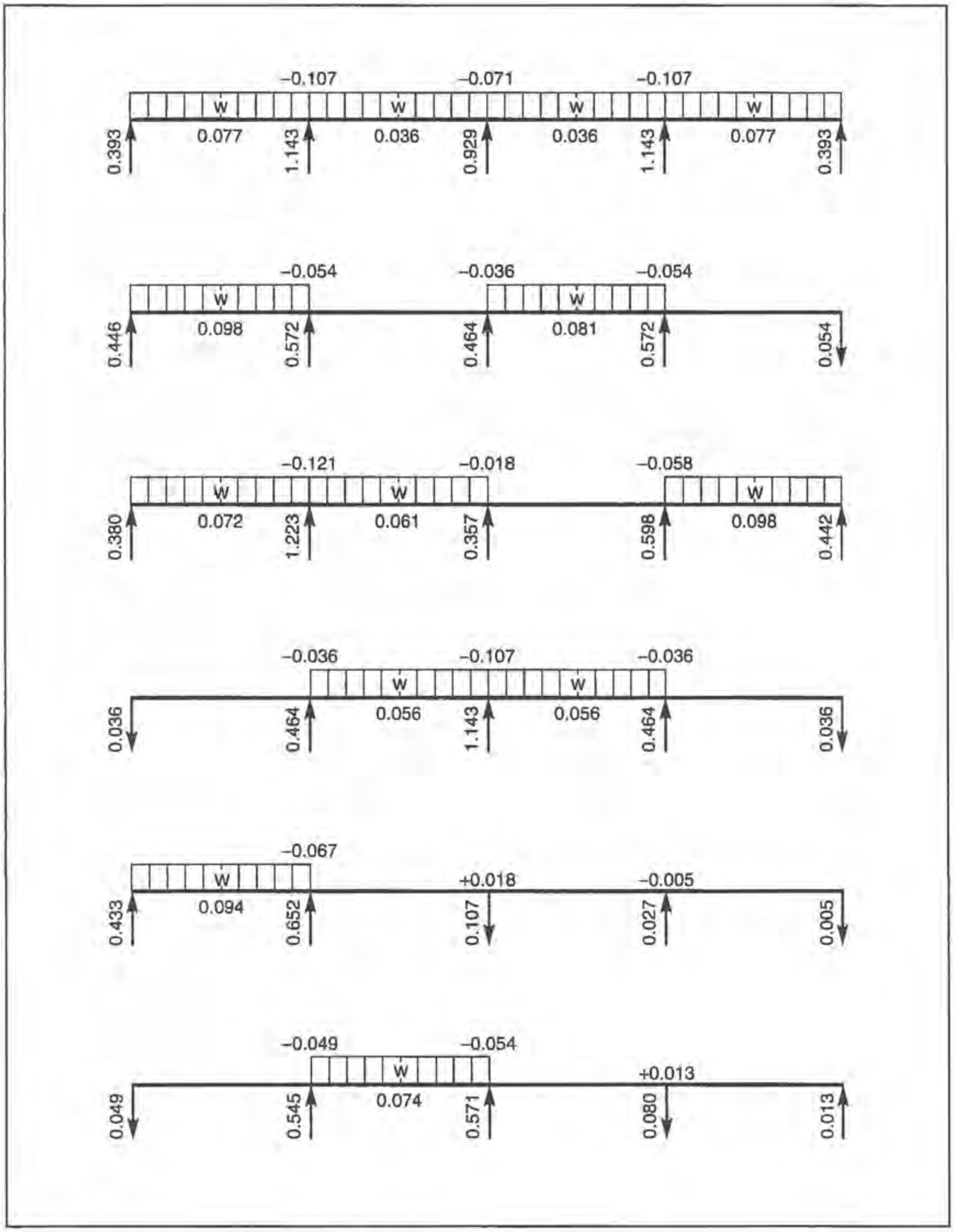
Moment = Coefficient × W × L
Reaction = Coefficient × W
Where: W = Total uniformly distributed load on one span
L = Length of one span



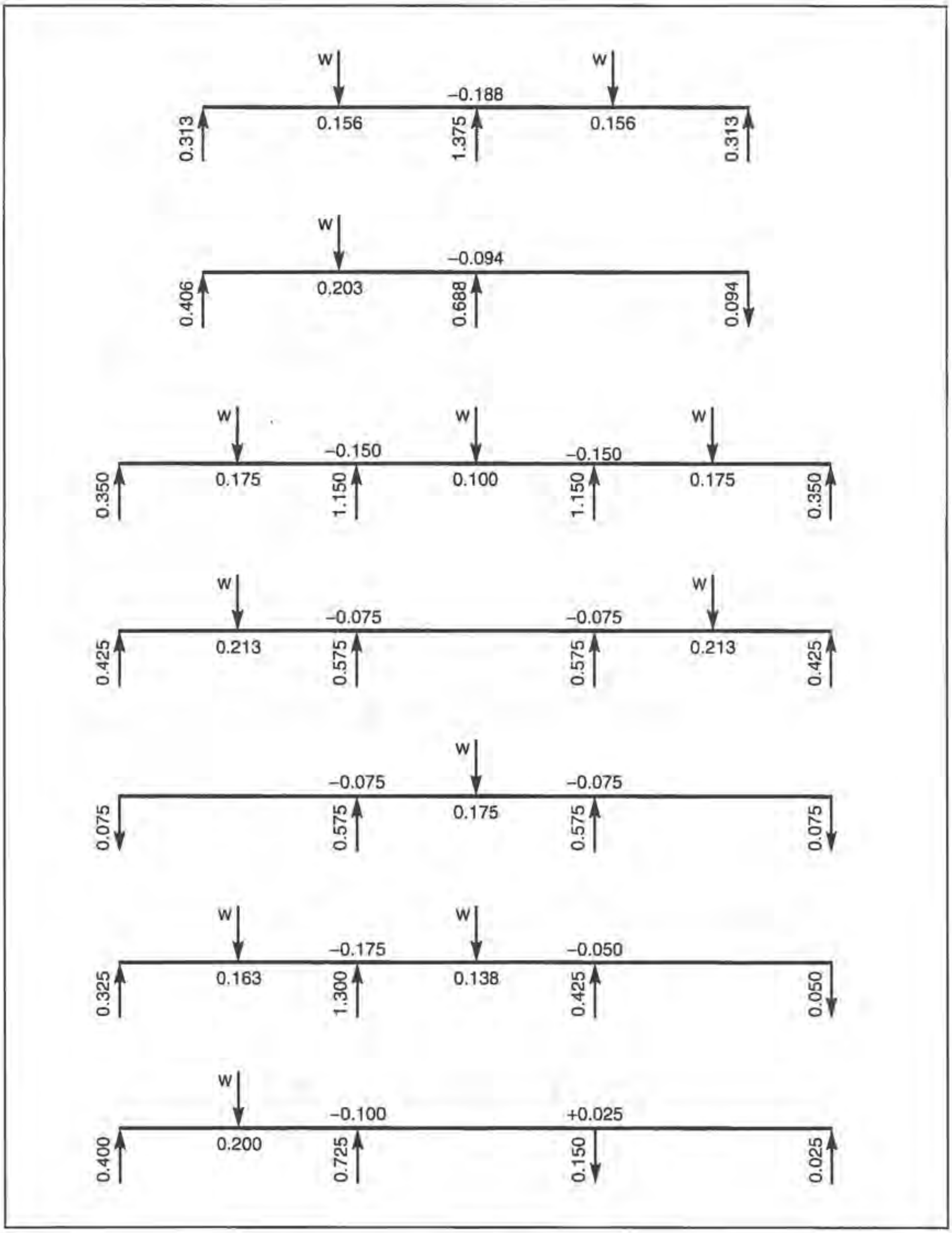
MOMENTS, REACTIONS
Equal Span Continuous Beams

UNIFORMLY DISTRIBUTED LOADS

Moment = Coefficient × W × L
Reaction = Coefficient × W
Where: W = Total uniformly distributed load on one span
L = Length of one span



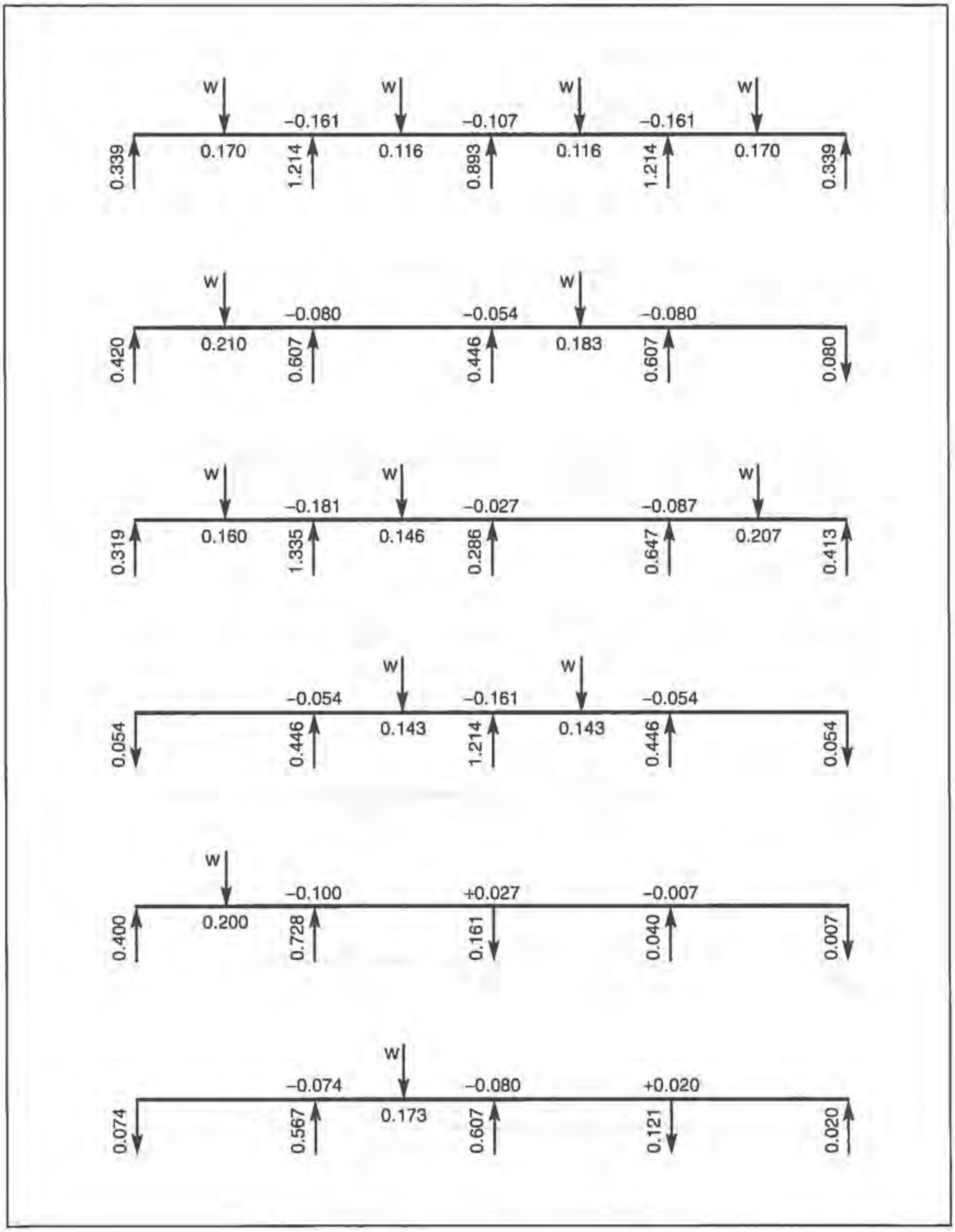
Moment = Coefficient × W × L
Reaction = Coefficient × W
Where: W = The concentrated load on one span
L = Length of one span



MOMENTS, REACTIONS
Equal Span Continuous Beams

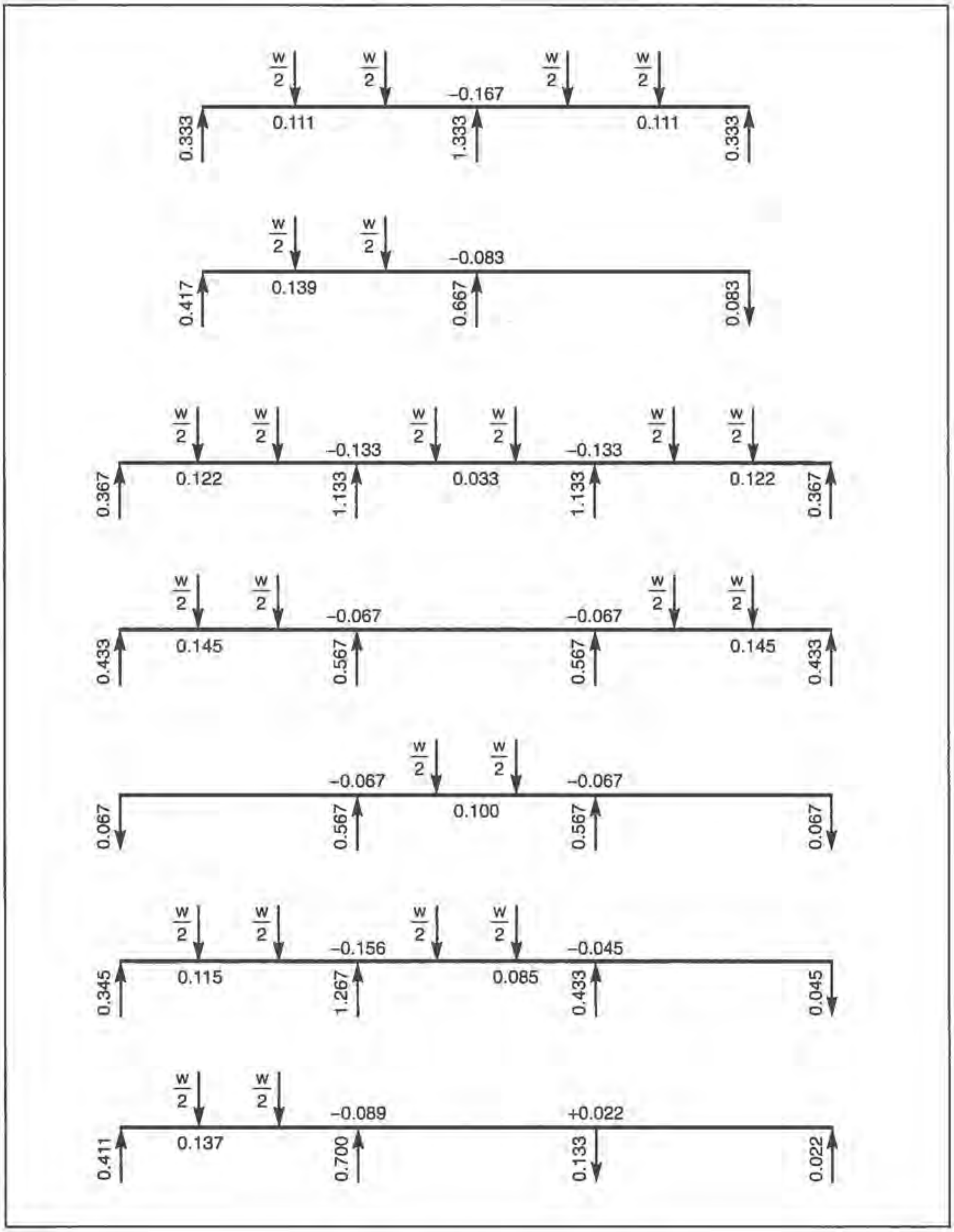
CENTRAL POINT LOADS

Moment = Coefficient × W × L
Reaction = Coefficient × W
Where: W = The concentrated load on one span
L = Length of one span



POINT LOADS AT THIRD POINTS OF SPAN

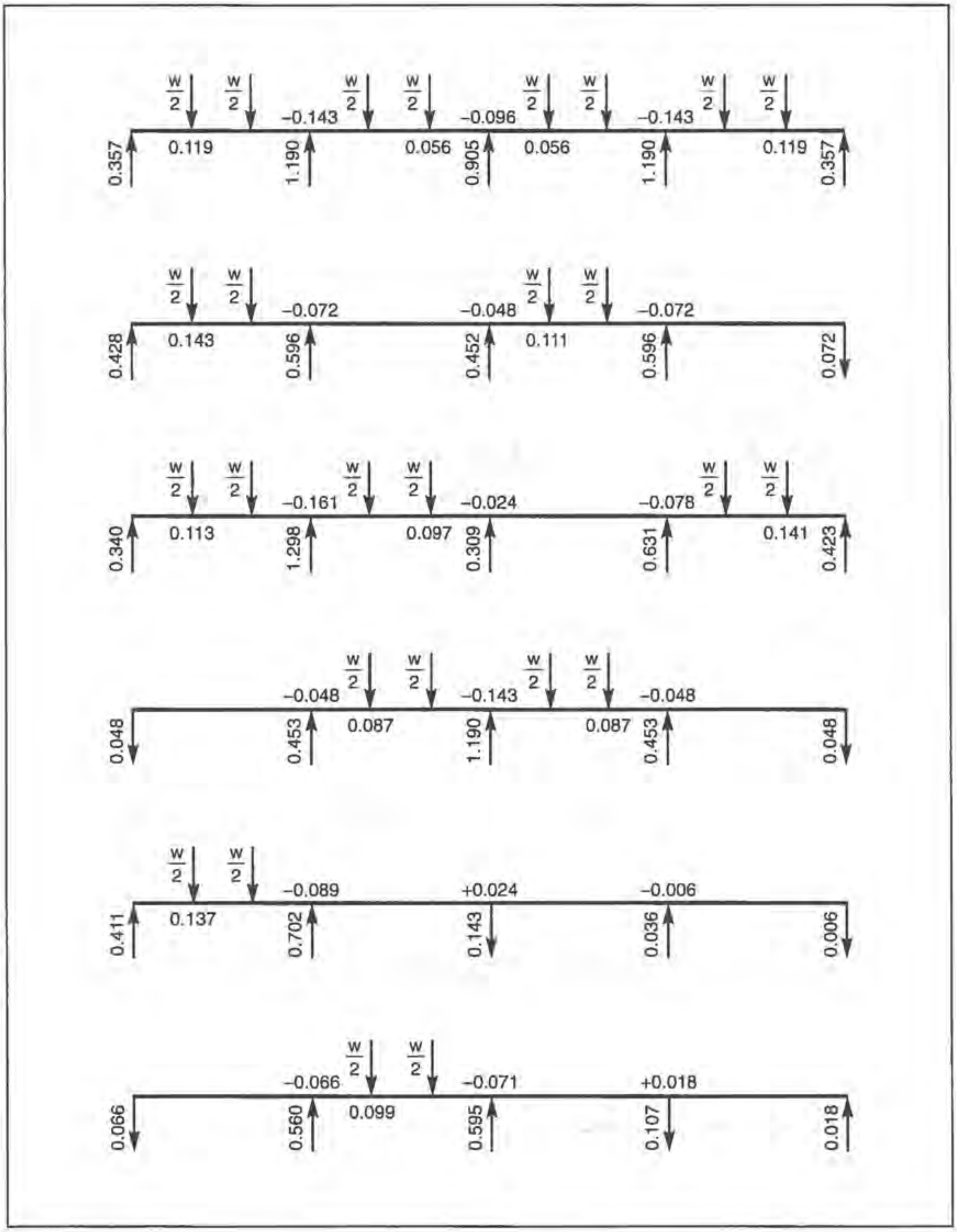
Moment = Coefficient × W × L
Reaction = Coefficient × W
Where: W = The total load on one span
L = Length of one span



MOMENTS, REACTIONS
Equal Span Continuous Beams

POINT LOADS AT THIRD POINTS OF SPAN

Moment = Coefficient × W × L
Reaction = Coefficient × W
Where: W = The total load on one span
L = Length of one span



PART SIX
PROPERTIES AND DIMENSIONS

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STRUCTURAL STEELS

General

Canadian structural steels are covered by two standards prepared by the Canadian Standards Association Technical Committee on Structural Steel, G40. These are CSA G40.20 and CSA G40.21. The information provided in this section is based on the current 2013 editions of both standards, and on the SI metric values, in keeping with Canadian design standards for steel structures.

CSA G40.20, "General Requirements for Rolled or Welded Structural Quality Steel" sets out the general requirements governing the delivery of structural quality steels. These requirements include: Definitions, Chemical Composition, Variations in Dimensions, Methods of Testing, Frequency of Testing, Heat Treatment, Repairs of Defects, Marking, etc.

CSA G40.21, "Structural Quality Steel" governs the chemical and mechanical properties of 7 types and 9 strength levels of structural steels for general construction and engineering purposes. All strength levels are not available in all types, and selection of the proper grade (type and strength level) is important for a particular application. G40.21-350A and G40.21-350AT are atmospheric corrosion-resistant steels normally used in bridge construction. For HSS sections, 350W is the normal grade used when produced to G40.21.

The 7 types covered in G40.21 are:

(a) **Type W – Weldable Steel.** Steels of this type meet specified strength requirements and are suitable for general welded construction where notch toughness at low temperatures is not a design requirement. Applications include buildings, compression members of bridges, etc. Steels within this type meeting more restrictive chemical and mechanical requirements[1] shall be designated WM. This designation meets the requirements of ASTM A992/A992M.

(b) **Type WT – Weldable Notch-Tough Steel.** Steels of this type meet specified strength and Charpy V-notch impact requirements and are suitable for welded construction where notch toughness at low temperature is a design requirement. The purchaser, in addition to specifying the grade, specifies the required category of steel that establishes the Charpy V-notch test temperature and energy level. Applications include primary tension members in bridges and similar elements. Steels within this type meeting more restrictive chemical and mechanical requirements[1] shall be designated WMT. This designation meets the requirements of ASTM A992/A992M with Charpy V-notch toughness.

(c) **Type R – Atmospheric Corrosion-Resistant Steel.** Steels of this type meet specified strength requirements. The atmospheric corrosion resistance of these steels in most environments is substantially better than that of carbon structural steels with or without a copper addition[2]. These steels are welded readily up to the maximum thickness covered by the G40.21 standard. Applications include unpainted siding, unpainted light structural members, etc., where notch toughness at low temperature is not a design requirement.

(d) **Type A – Atmospheric Corrosion-Resistant Weldable Steel.** Steels of this type meet specified strength requirements. The atmospheric corrosion resistance of these steels in most environments is substantially better than that of carbon structural steels with or without a copper addition[2]. These steels are suitable for welded construction where notch toughness at low temperature is not a design requirement. Applications include those similar to type W steel.

(e) **Type AT – Atmospheric Corrosion-Resistant Weldable Notch-Tough Steel**. Steels of this type meet specified strength and Charpy V-notch impact requirements. The atmospheric corrosion resistance of these steels in most environments is substantially better than that of carbon structural steels with or without a copper addition[2]. These steels are suitable for welded construction where notch toughness at low temperature is a design requirement. The purchaser, in addition to specifying the grade, specifies the required category of steel that establishes the Charpy V-notch test temperature and energy level. Applications include primary tension members in bridges and similar elements.

(f) **Type Q – Quenched and Tempered Low-Alloy Steel Plate.** Steels of this type meet specified strength requirements. While these steels are weldable, the welding and fabrication techniques are of fundamental importance to the properties of the plate, especially the heat-affected zone. Applications include bridges and similar structures.

(g) **Type QT – Quenched and Tempered Low-Alloy Notch-Tough Steel Plate.** Steels of this type meet specified strength and Charpy V-notch impact requirements. They provide good resistance to brittle fracture and are suitable for structures where notch toughness at low temperature is a design requirement. The purchaser, in addition to specifying the grade, specifies the required category of steel that establishes the Charpy V-notch test temperature and energy level. While these steels are weldable, the welding and fabrication techniques are of fundamental importance to the properties of the plate, especially the heat-affected zone. Applications include primary tension members in bridges and similar elements.

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1 See CSA G40.21 Tables 3, 6 and Clause 7.7.

2 For methods of estimating the atmospheric corrosion resistance of low-alloy steels, see CSA G40.21 Clause 7.6. When properly exposed to the atmosphere, these steels can be used bare (unpainted) for many applications.

Tables

Table 6-1, "Grades, Types, Strength Levels", gives the grade designation of the various types and strength levels of structural steels according to the requirements of CSA G40.21.

Availability of any grade and shape combination should be kept in mind when designing to ensure overall economy, since a specified product may not always be available in the tonnage and time frame contemplated. Local availability should always be checked.

Table 6-2, "Shape Size Groupings for Tensile Property Classification", summarizes the size groupings for C, MC and L shapes. Table 6-3, "Mechanical Properties Summary", summarizes the various grades, tensile strengths and yield strengths for plates, bars, welded shapes, rolled shapes, sheet piling, and hollow structural sections based on CSA G40.21.

Table 6-4, "Chemical Composition", summarizes the chemical requirements of various grades of steel covered by CSA G40.21. Table 6-5 specifies the "Steel Marking Colour Code" for material identification. Table 6-6 specifies the "Standard Impact Energy and Test Temperature" for the various grades, strength levels and categories of notch-tough steels.

The particular standards, CSA G40.20 and CSA G40.21, should be consulted for more details. Similar information about steel covered by ASTM standards should be consulted when appropriate.

Historical Remarks

When confronted with an unidentified structural steel, Clause 5.2.2 of CSA S16-14 requires that F_y be taken as 210 MPa and F_u as 380 MPa. This provides a minimum in the place of more precise information, such as coupon testing. The following tables list selected dates of publication and data from various CSA and ASTM structural steel standards and specifications, many of which preceded current standards.

For more information on ASTM specifications and properties and dimensions of iron and steel beams previously produced in the USA, consult the "AISC Rehabilitation and Retrofit Guide: A Reference for Historic Shapes and Specifications" published by the American Institute of Steel Construction. In that publication, the first date listed for both ASTM A7 and A9 is the year 1900. Between 1900 and 1909, medium steel in A7 and A9 had a tensile strength 5 ksi higher than that adopted in 1914. For CSA standards, consult original documents.

Historical Listing of Selected Structural Steels

CSA Standards

Designation	Date Published	Yield Strength		Tensile Strength (F_u)	
		ksi	MPa	ksi	MPa
A16	1924	½ F_u	½ F_u	55-65	380-450
S39	1935	30	210	55-65	380-450
S40	1935	33	230	60-72	410-500
G40.4	1950	33	230	60-72	410-500
G40.5	1950	33	230	60-72	410-500
G40.6	1950	45[1]	310	80-95	550-650
G40.8	1960	40[2]	280	65-85	450-590
G40.12	1964	44[3]	300	65	450
G40.21	1973	Replaced all previous Standards, see CISC Handbook			

[1] Silicon steel

[2] Yield reduces when thickness exceeds ⅝ inches (16 mm).

[3] Yield reduces when thickness exceeds 1½ inches (40 mm).

Rivet Steel

Designation	Date Published	Yield Strength		Tensile Strength (F_u)	
		ksi	MPa	ksi	MPa
G40.2	1950	28	190	52 - 62	360 - 430

ASTM Specifications

Designation	Date Published	Yield Strength		Tensile Strength (F_u)	
		ksi	MPa	ksi	MPa
A7 (bridges) A9 (buildings)	1914*	½ F_u	½ F_u	55-65	380-450
	1924	½ $F_u \geq 30$	½ $F_u \geq 210$	55-65	380-450
	1934	½ $F_u \geq 33$	½ $F_u \geq 230$	60-72	410-500
A373	1954	32	220	58-75	400-520
A242	1955	50[1]	350	70[1]	480
A36	1960	36	250	60-80	410-550
A440	1959	50[1]	350	70[1]	480
A441	1960	50[1]	350	70[1]	480
A572 grade 50	1966	50	345	65	450
A588	1968	50[1]	345	70[1]	485
A992	1998	50 min. to 65 max.	345 min. to 450 max.	65	450

* See text, Historical Remarks, above.

[1] Reduces with increasing thickness

GRADES, TYPES, STRENGTH LEVELS* Table 6-1

Type	Nominal Yield Strength, MPa								
	260	300	345 - 350	380	400	450	480	550	700
W	260W	300W	345WM, 350W	380W**	400W	450W	480W	550W	—
WT	260WT	300WT	345WMT, 350WT	380WT***	400WT	450WT	480WT	550WT	—
R	—	—	350R	—	—	—	—	—	—
A	—	—	350A	—	400A	—	480A	550A	—
AT	—	—	350AT	—	400AT	—	480AT	550AT	—
Q	—	—	—	—	—	—	—	—	700Q
QT	—	—	—	—	—	—	—	—	700QT

* See CSA G40.20/G40.21

** This grade is available in Hollow Structural Sections, angles and bars only.

*** This grade is available in Hollow Structural Sections only.

SHAPE SIZE GROUPINGS FOR TENSILE PROPERTY CLASSIFICATION* Table 6-2

Shape Type	Group 1	Group 2	Group 3
C Shapes	To 30.8 kg/m	Over 30.8 kg/m	—
MC Shapes	To 42.4 kg/m	Over 42.4 kg/m	—
L Shapes	To 13 mm	Over 13 to 19 mm	Over 19 mm

* See CSA G40.20/G40.21

Table 6-3 MECHANICAL PROPERTIES SUMMARY

CSA G40.20 / G40.21		Tensile Strength	Plates, Floor Plates, Bars, Sheet and Welded Shapes		Rolled Shapes and Sheet Piling		Hollow Structural Sections
			F_y (MPa) min.		Common Available Shape Size Group	.F_y (MPa) min.	F_y (MPa) min.
Type	Grade	F_u (MPa)	Thickness t ≤ 65 mm	Thickness[4] t > 65 mm		Groups 1 to 3	
W	260W	410-590	260	250	3	260	—
	300W	440-620[1]	300	280	3	300	300
	345WM[5]	≥ 450	345-450	345-450	2	345-450	—
	350W	450-650[2]	350	320	2	350	350
	380W[3]	480-650	380	350	2	380	380
	400W	520-690	400	370	1	400	400
	450W	550-725	450	420	—	—	—
	480W	590-790	480	450	1	480	480
	550W	620-860	550	520	—	—	550
WT	260WT	410-590	260	250	3	260	—
	300WT	440-620[6]	300	280	3	300	—
	345WMT[5]	≥ 450	345-450	345-450	3	345-450	—
	350WT	450-650[2,7]	350	320	3	350	350
	380WT	480-650	—	—	—	—	380
	400WT	520-690	400	370	2	400	400
	450WT	550-725	450	420	—	—	—
	480WT	590-790	480	450	1	480	480
	550WT	620-860	550	520	—	—	550
R	350R	480-650	350	—	1	350	—
A	350A	480-650	350	350	3	350	350
	400A	520-690	400	—	2	400	400
	480A	590-790	480	—	—	—	480
	550A	620-860	550	—	—	—	550
AT	350AT	480-650	350	350	3	350	350
	400AT	520-690	400	—	2	400	400
	480AT	590-790	480	—	—	—	480
	550AT	620-860	550	—	—	—	550
Q	700Q	760-895	700	620	—	—	—
QT	700QT	760-895	700	620	—	—	—

[1] 410-590 MPa for HSS
[2] 450-620 MPa for HSS
[3] Available in angles and bars only
[4] For thickness t > 100 mm, see CSA G40.21
[5] The maximum yield strength is 450 MPa, and the maximum yield-to-tensile strength ratio is 0.85. For structural shapes that are required to be tested from the web location, a maximum yield strength of 480 MPa and a maximum yield-to-tensile strength ratio of 0.87 are permitted.
[6] 450-620 MPa for rolled shapes and sheet piling
[7] 480-650 MPa for rolled shapes and sheet piling

CHEMICAL COMPOSITION - CSA G40.21[1] Table 6-4

CSA G40.21 Grade	Chemical Composition (Heat Analysis) Percent[2]								
	All percentages are maxima unless otherwise indicated.								
	C	Mn[3]	P	S	Si[4,5]	Other[6]	Cr	Ni	Cu[7]
260W	0.20[10]	0.50-1.50	0.04	0.05	0.40	0.15	—	—	—
300W[8]	0.22[10]	0.50-1.50	0.04	0.05	0.40	0.15	—	—	—
345WM	0.23[18]	0.50-1.60	0.035	0.045	0.10-0.40	0.15[19]	0.35	0.45	0.60
350W	0.23	0.50-1.50	0.04	0.05	0.40	0.15	—	—	—
380W[9]	0.23	0.50-1.50	0.04	0.05	0.40	0.15	—	—	—
400W	0.23[11]	0.50-1.50	0.04	0.05	0.40	0.15	—	—	—
450W	0.23	0.50-1.50	0.04	0.05	0.40	0.15	—	—	—
480W	0.26[11]	0.50-1.50	0.04	0.05	0.40	0.15[15]	—	—	—
550W	0.15	1.75[12]	0.04	0.05	0.40	0.15	—	—	—
260WT	0.20[10]	0.80-1.50	0.03	0.04	0.15-0.40	0.15	—	—	—
300WT	0.22[10]	0.80-1.50	0.03	0.04	0.15-0.40	0.15	—	—	—
345WMT	0.23[18]	0.80-1.50[12]	0.035	0.045	0.10-0.40	0.15[19]	0.35	0.45	0.60
350WT	0.22[10]	0.80-1.50[12]	0.03	0.04	0.15-0.40	0.15	—	—	—
380WT[9]	0.22	0.80-1.50	0.03	0.04	0.15-0.40	0.15	—	—	—
400WT	0.22[11]	0.80-1.60	0.03	0.04[14]	0.15-0.40	0.15	—	—	—
450WT	0.22	0.80-1.50[12]	0.03	0.04	0.15-0.40	0.15	—	—	—
480WT	0.26[11]	0.80-1.50[12]	0.03	0.04[14]	0.15-0.40	0.15[15]	—	—	—
550WT	0.15	1.75[12]	0.03	0.04[14]	0.15-0.40	0.15	—	—	—
350R	0.16	0.75	0.05-0.15	0.04	0.75	0.15	0.30-1.25[16]	0.90[16]	0.20-0.60[16]
350A	0.20	0.75-1.35[12]	0.03	0.04	0.15-0.50	0.15	0.70[17]	0.90[17]	0.20-0.60
400A	0.20	0.75-1.35[12]	0.03	0.04[14]	0.15-0.50	0.15	0.70[17]	0.90[17]	0.20-0.60
480A	0.20	1.00-1.60	0.025[13]	0.035[14]	0.15-0.50	0.15[15]	0.70[17]	0.25-0.50[17]	0.20-0.60
550A	0.15	1.75[12]	0.025[13]	0.035[14]	0.15-0.50	0.15	0.70[17]	0.25-0.50[17]	0.20-0.60
350AT	0.20	0.75-1.35[12]	0.03	0.04	0.15-0.50	0.15	0.70[17]	0.90[17]	0.20-0.60
400AT	0.20	0.75-1.35[12]	0.03	0.04[14]	0.15-0.50	0.15	0.70[17]	0.90[17]	0.20-0.60
480AT	0.20	1.00-1.60	0.025[13]	0.035[14]	0.15-0.50	0.15[15]	0.70[17]	0.25-0.50[17]	0.20-0.60
550AT	0.15	1.75[12]	0.025[13]	0.035[14]	0.15-0.50	0.15	0.70[17]	0.25-0.50[17]	0.20-0.60
700Q	0.20	1.50	0.03	0.04	0.15-0.40	—	Boron 0.0005-0.005		—
700QT	0.20	1.50	0.03	0.04	0.15-0.40	—	Boron 0.0005-0.005		—

Notes:

1. Consult CSA G40.20/G40.21 for full details. Usual deoxidation for all grades is fully killed.
2. Additional alloying elements may be used when approved.
3. For HSS Mn 0.50 - 1.50% for 350WT and 380WT, 1.65% for 400 yield, 1.75% for 480 yield and 1.85% for 550 yield steels. For HSS minimum limit for Mn shall be 0.30% provided that the ratio of Mn to C is not less than 2 to 1 and the ratio of Mn to S is not less than 20 to 1.
4. Si content of 0.15% to 0.40% is required for type W steel over 40 mm thickness, HSS of A or AT steel, or bar diameter except as required by Note 5.
5. By purchaser's request or producer's option, no minimum Si content is required provided that 0.015% acid-soluble Al or 0.02% total Al is used.
6. Includes grain-refining elements Cb, V, Al. Elements Cb and V may be used singly or in combination. See G40.20/G40.21 for qualifications. Al, when used, is not included in the summation. For HSS with 300 - 400 yield, 0.10%.
7. Copper content of 0.20% minimum may be specified.
8. For HSS 0.26% C and 0.30-1.20% Mn.
9. Only angles, bars, and HSS in 380W grade, and only HSS in 380WT grade.
10. For thicknesses over 100 mm, C may be 0.22% for 260W and 260WT grades, and 0.23% for 300W, 300WT and 350WT grades.
11. For HSS 0.20% C.
12. Mn may be increased. See G40.20/G40.21 for qualifications.
13. For HSS 0.03% P.
14. For HSS 0.03% S.
15. For HSS 0.12%
16. Cr + Ni + Cu ≥ 1.00%
17. Cr + Ni ≥ 0.40% and for HSS, 0.90% Ni max.
18. Carbon equivalent ≤ 0.47% for shapes with flange thickness > 50 mm and 0.45% for other shapes.
19. When steel is aluminum-killed, total aluminum ≥ 0.015%.
    N ≤ 0.015%. V ≤ 0.15%, Nb ≤ 0.05%, V + Nb ≤ 0.15%, Mo ≤ 0.15%. Consult CSA G40.20/G40.21 for full details.

Table 6-5 STEEL MARKING COLOUR CODE

Steel Grade	Primary Colour	Secondary Colour
260W	White	Green
300W	Green	Green
350W	Blue	Green
380W	Brown	Green
400W	Black	Green
480W	Yellow	Green
550W	Pink	Green
260WT	White	White
300WT	Green	White
350WT	Blue	White
380WT	Brown	White
400WT	Black	White
480WT	Yellow	White
550WT	Pink	White
350R	Blue	Blue
350A	Blue	Yellow
400A	Black	Yellow
480A	Yellow	Yellow
550A	Pink	Yellow
350AT	Blue	Brown
400AT	Black	Brown
480AT	Yellow	Brown
550AT	Pink	Brown
700Q	Red	Red
700QT	Red	Purple

In this Code, the following colour system applies:

Strength Level	Primary Colour	Type	Secondary Colour
260	White	W	Green
300	Green	WT	White
350	Blue	R	Blue
380	Brown	A	Yellow
400	Black	AT	Brown
480	Yellow	Q	Red
550	Pink	QT	Purple
700	Red		

STANDARD IMPACT ENERGY AND TEST TEMPERATURE FOR NOTCH-TOUGH STEELS

TABLE 6-6

Type	Grade	Category				
		1	2	3	4	5
WT	260, 300	20 J, 0°C	20 J, -20°C	20 J, -30°C	20 J, -45°C	Both energy and test temperature are specified by the purchaser.
	350, 380, 400, 450, 480, 550	27 J, 0°C	27 J, -20°C	27 J, -30°C	27 J, -45°C	
WMT	345	27 J, 0°C	27 J, -20°C	27 J, -30°C	27 J, -45°C	
AT	350, 400, 480, 550	27 J, 0°C	27 J, -20°C	27 J, -30°C	27 J, -45°C	
QT	700	34 J, 0°C	34 J, -20°C	34 J, -30°C	34 J, -45°C	

Units: Impact energy in Joules (1 J ≈ 0.738 ft·lb) and test temperature in degrees Celsius.

Notes: Charpy V-Notch, longitudinal specimens. See CSA G40.21-13 Clause 8.2.2.
See CSA S16-14 Annex L "Design to Prevent Brittle Fracture" for information on test and service temperatures.

MECHANICAL PROPERTIES — Table 6-7

Of Selected ASTM Steel Grades

Steel Grade: Rolled Shapes and HSS	Steel Grade: Plates and Bars	F_y (MPa)	F_u (MPa)
A36[1]	A36[2]	250	400 - 550
A500 Gr. C - Round		317[3]	427[3]
A500 Gr. C - Square and Rectangular		345	427[3]
A572 Gr. 50 (345) A913 Gr. 50 (345)	A572 Gr. 50 (345)[6]	345	450
A709M Gr. 345S **A992**		**345 - 450[4]**	**450[4]**
A1085[5]		345 - 485	450
A588	A709M Grades 345W[6], HPS 345W[6]	345	485
A913 Gr. 65 (450)		450	550
	A709M Gr. HPS 485W[6]	485	585 - 760
A913 Gr. 70 (485)		485	620

[1] Flange thickness ≤ 75 mm
[2] Plate thickness ≤ 200 mm
[3] Soft-converted from imperial units
[4] $F_y / F_u \leq 0.85$
[5] Heat treatment available as supplementary requirement S1
[6] Plate thickness ≤ 100 mm

STEEL GRADES FOR BUILDING CONSTRUCTION — Table 6-8

Relative Availability

Steel Grade		F_y MPa	W	C	L	HSS Square, Rectangular	HSS Round	HP
CSA	G40.21 350W	350				West of Quebec *		
CSA	G40.21 300W	300						
ASTM	A992	345						
ASTM	A572 Gr. 50	345						
ASTM	A913 Gr. 65	450	Heavy Sections					
ASTM	A500 Gr. C	345				East of Ontario		
ASTM	A500 Gr. C	317						

Grade preferred for relative availability

Other grades

* G40.21 350W Class C

CHEMICAL COMPOSITION[1] OF SELECTED ASTM STEEL GRADES

Table 6-9

ASTM Steel Grade	Chemical Composition (Heat Analysis) Percent								
	All percentages are maxima unless otherwise indicated.								
	C	Mn	P	S	Si	Other	Cr	Ni	Cu
A36 Shapes[2]	0.26	—[3]	0.04	0.05	0.40[4]	—	—	—	0.20[6]
A500 Gr. C	0.23[5]	1.35[5]	0.035	0.035	—	—	—	—	0.20[6]
A572 Gr. 50 (345)[7]	0.23[8]	1.35[9]	0.04	0.05	0.40[10]	—	—	—	—[6]
A913 Gr. 50 (345) A913 Gr. 65 (450) A913 Gr. 70 (485)	0.12 0.16 0.16	1.60 1.60 1.60	0.04 0.03 0.04	0.03 0.03 0.03	0.40 0.40 0.40	(11) (11) (11)	0.25 0.25 0.25	0.25 0.25 0.25	0.45 0.35 0.45
A992[12]	0.23	0.50-1.60[13]	0.035	0.045	0.40	(14)	0.35	0.45	0.60
A709M Gr. 345S[12]	0.23	0.50-1.60[13]	0.035	0.045	0.40	(14)	0.35	0.45	0.60
A588 Gr. A	0.19[5]	0.80-1.25[5]	0.04	0.05	0.30-0.65	V 0.02-0.10	0.40-0.65	0.40	0.25-0.40
A709M Gr. 345W[15] Type A	0.19[5]	0.80-1.25[5]	0.04	0.05	0.30-0.65	V 0.02-0.10	0.40-0.65	0.40	0.25-0.40
A588 Gr. B	0.20[5]	0.75-1.35[5]	0.04	0.05	0.15-0.50	V 0.01-0.10	0.40-0.70	0.50	0.20-0.40
A709M Gr. 345W[15] Type B	0.20[5]	0.75-1.35[5]	0.04	0.05	0.15-0.50	V 0.01-0.10	0.40-0.70	0.50	0.20-0.40
A709M Gr. HPS 345W	0.11	1.10-1.35[16]	0.02	0.006[17]	0.30-0.50	(18)	0.45-0.70	0.25-0.40	0.25-0.40
A709M Gr. HPS 485W	0.11	1.10-1.35[16]	0.02	0.006[17]	0.30-0.50	(18)	0.45-0.70	0.25-0.40	0.25-0.40
A1085	0.26[5]	1.35[5]	0.035	0.035	0.04	(19)	—	—	—

Notes: Where "—" appears in this table, there is no requirement.

1. Consult ASTM standards for full details.
2. For A36 plates and bars, refer to the A36 standard.
3. Mn content of 0.85-1.35% is required for shapes with flange thickness over 75 mm.
4. Si content of 0.15-0.40% is required for shapes with flange thickness over 75 mm.
5. For each reduction of 0.01 percentage point below the specified maximum for C, an increase of 0.06 percentage point above the specified maximum for Mn is permitted, up to a maximum of 1.50% by heat analysis.
6. Cu when specified shall have a minimum content of 0.20% by heat analysis.
7. Round bars up to and including 275 mm in diameter are permitted.
8. For each reduction of 0.01 percentage point below the specified maximum for C, an increase of 0.06 percentage point above the specified maximum for Mn is permitted, up to a maximum of 1.60% by heat analysis.
9. Mn, minimum, by heat analysis of 0.80% shall be required for all plates > 10 mm thick; a minimum of 0.50% shall be required for plates ≤ 10 mm thick, and for all other products. The Mn to C ratio shall not be less than 2 to 1.
10. Plates ≤ 40 mm thick, shapes with flange or leg thickness ≤ 75 mm, sheet piling, bars, zees, and rolled tees. Plates > 40 mm thick and shapes with flange thickness > 75 mm shall have a Si content of 0.15-0.40%. Bars > 40 mm in diameter, thickness, or distance between parallel faces shall be made by a killed steel practice.
11. Mo 0.07%; Nb 0.05%; V 0.06% gr. 50, 0.08% gr. 65, 0.09% gr. 70. Consult ASTM standard for full details.
12. In addition to the elements listed, test reports shall include, for information, the chemical analysis for tin. Where the amount of tin is < 0.02%, it shall be permissible for the analysis to be reported as "< 0.02%".
13. Provided that the ratio of Mn to S is ≥ 20 to 1, the minimum limit for Mn for shapes with flange or leg thickness ≤ 25 mm shall be 0.30%.
14. Mo 0.15%, Nb 0.05%, V 0.15%. Nb + V ≤ 0.15%. Consult ASTM standard for full details.
15. Types A and B for A709M Gr. 345W steel are equivalent to A588/A588M, Grades A and B, respectively.
16. Mn content for plates and bars ≤ 65 mm. Mn content of 1.10-1.50% is required for plates and bars > 65 mm.
17. The steel shall be calcium treated for sulfide shape control.
18. Mo 0.02-0.08%, Al 0.01-0.04%, V 0.04-0.08%, N 0.015%.
19. Acid soluble Al 0.015% minimum or total Al content 0.02% minimum.

STANDARD MILL PRACTICE

General

Rolled structural shapes are produced by passing hot blooms, billets or slabs of steel through a series of grooved rolls. Wear on the rolls can cause the dimensions of the finished product to vary slightly from the theoretical, published dimensions. Standard rolling tolerances have been established to make allowance for roll wear and other factors. These tolerances are contained in CSA Standard G40.20, "General Requirements for Rolled or Welded Structural Quality Steel".

Letter symbols for dimensions on sketches shown in this section are in accordance with CSA G40.20, ASTM A6, and mill catalogs.

Methods of increasing area and mass by spreading rolls

Most nominal size groups of rolled shapes contain several specific shapes, each of which is slightly different in mass, area and properties from other shapes in the same size group. Methods used to increase the area and mass, from the minimum nominal size, by spreading the rolls are described below:

For W Shapes (Fig. 6-1), the thickness of both flange and web is increased, resulting in an increase to the overall beam depth and flange width, with the distance between inside faces of flanges being unchanged.

For S Shapes and Channels (Fig. 6-2 and 6-3), the web thickness and flange width are increased by equal amounts, all other dimensions remaining unchanged.

For angles (Fig. 6-4) the thickness of each leg is increased an equal amount, resulting in a corresponding increase in leg length.

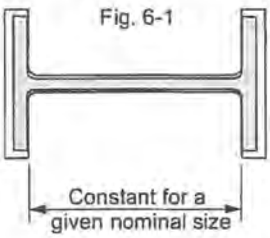


Fig. 6-1

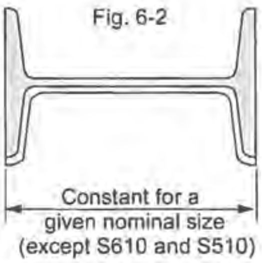


Fig. 6-2

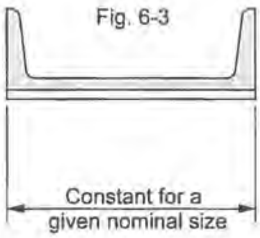


Fig. 6-3

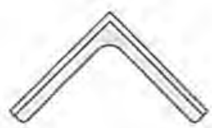


Fig. 6-4

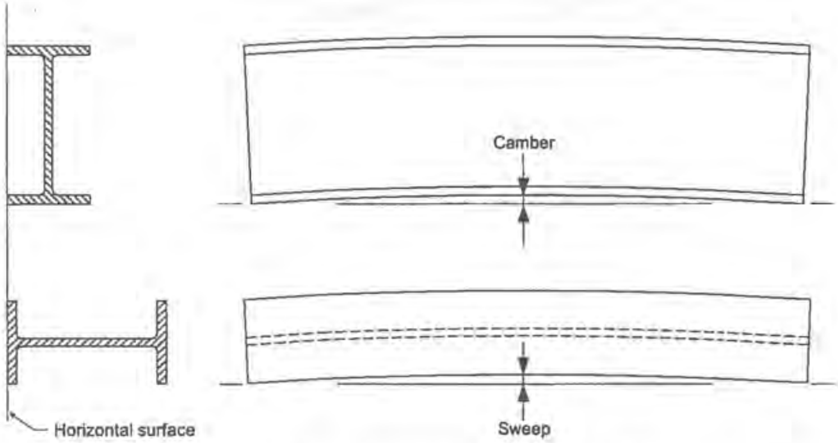
Tolerances

Tolerances are the permissible variations in the mass, cross-sectional area, length, depth, flange width, camber, sweep and other geometric properties of a rolled or welded section. A summary of the basic manufacturing tolerances, taken from CSA G40.20, are provided in the following tables. While these tables are provided for convenience, the actual Standard should be referred to for complete information.

Camber and Sweep

After a section is rolled, it is cold-straightened to meet the specified sweep and camber tolerances.

Camber is a deflection, approximating a simple regular curve, measured along the depth of a section. It is usually measured halfway between two specified points. The length for purposes of determining the "maximum permissible variation" is the distance between the two specified points.



Positions for measuring camber and sweep

Sweep is a deflection, similar to camber, measured along the width of the section.

The following table lists Permissible Variations in Straightness.

PERMISSIBLE VARIATIONS IN STRAIGHTNESS

Shape	Maximum Permissible Variation in Straightness, mm
W and HP shapes with flange width ≥ 150 mm [1] (camber and sweep) Welded beams or girders where there is no specified camber or sweep	L / 1000
W and HP shapes with flange width < 150 mm [1] (sweep)	L / 500
Welded beams or girders with specified camber	6 + L / 4000
W and HP shapes specified as columns, with flange width approximately equal to depth [1, 2] (camber and sweep) Welded columns and compression members in trusses	L ≤ 14 000 mm: L / 1000 ≤ 10 mm L > 14 000 mm: 10 + (L – 14 000) / 1000
S, M, C, MC, L, T shapes [1] (greatest cross-sectional dimension ≥ 75 mm)	Camber: L / 500 Sweep: Negotiable
Bars [1, 3]	6 mm in any 1500 mm and L / 250 [(4)]
S, M, C, MC, L, T bar-size shapes [1] (greatest cross-sectional dimension < 75 mm)	Camber: L / 250 Sweep: Negotiable

Notes:

[1] See ASTM A6 / A6M

[2] Applies only to: 200 mm-deep sections – 46 kg/m and heavier, 250 mm-deep sections – 73 kg/m and heavier, 310 mm-deep sections – 97 kg/m and heavier, and 360 mm-deep sections – 116 kg/m and heavier. For other sections specified as columns, tolerances are negotiable.

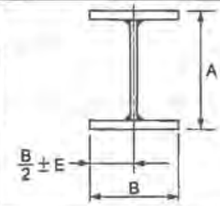
[3] Permitted variations do not apply to hot-rolled bars if any subsequent heating operation has been performed.

[4] Round to the nearest whole millimetre.

Sectional Dimensions

The permissible variations in sectional dimensions for welded shapes and rolled shapes are given in the following tables.

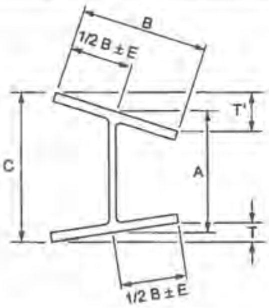
PERMISSIBLE VARIATIONS IN SECTIONAL DIMENSIONS OF WELDED STRUCTURAL SHAPES

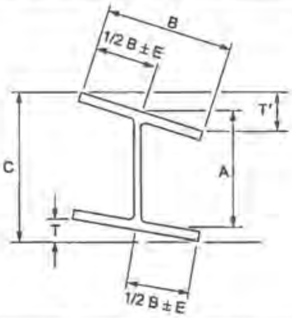
Nominal Depth, mm	Depth, A, mm		Width of flange, B, mm		Combined warpage and tilt,* mm	Web off-centre, E, mm	Web flatness **	Diagram
	Over	Under	Over	Under	Maximum	Maximum	Maximum	
900 and under	5	3	6	5	Greater of B / 100 or 6	6	A / 150	
Over 900 to 2000 incl.	5	5	6	5		6	A / 150	

* The combined warpage and tilt of the flange is measured from the toe of the flange to a line normal to the plane of the web through the intersection of the centreline of the web with the outside surface of the flange plate.

** The deviation from flatness of the web is measured in any length of the web equal to the total depth of the beam.

PERMISSIBLE VARIATIONS IN SECTIONAL DIMENSIONS OF W AND HP SHAPES





Nominal Depth, mm	Depth A, mm		Width of flange B, mm		Out-of-square T+T', mm	Web off-centre, E, mm	Max. overall depth at any cross-section, C, mm
	Over	Under	Over	Under	Not over	Not over	Over nominal
310 and under	4	3	6	5	6	5	6
Over 310	4	3	6	5	8	5 *	6

"A" is measured at the centreline of the web, "B" parallel to the flange, and "C" parallel to the web.

* Web off-centre tolerance is 8 mm for sections over 634 kg/m. See ASTM A6 / A6M.

PERMISSIBLE VARIATIONS IN LENGTH FOR W AND HP SHAPES

Nominal Depth, mm	Variations from Specified Length for Lengths Given, mm			
	9000 and under		Over 9000	
	Over	Under	Over	Under
Beams 610 mm and under	10	10	10 plus 1 for each additional 1000 mm or fraction thereof	10
Beams over 610 mm and all columns	13	13	13 plus 1 for each additional 1000 mm or fraction thereof	13

Notes: For W and HP shapes used as bearing piles, the length tolerance is +125 mm, -0 mm.

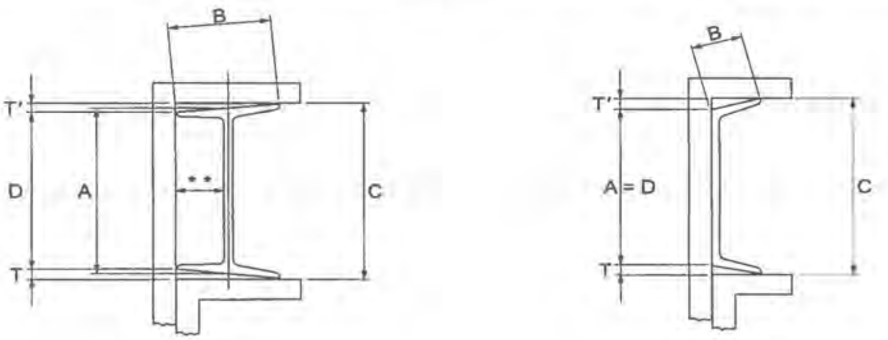
The permitted variations in end out-of-square for W and HP shapes shall be 0.016 mm per mm of depth, or per mm of flange width if the flange width is larger than the depth, rounded to the nearest mm. See ASTM A6 / A6M.

PERMISSIBLE VARIATIONS IN LENGTH FOR S, M, C, MC, L, AND T SHAPES

Nominal Size, mm (Greatest Cross-sectional Dimension)	Variations from Specified Length for Lengths Given, mm											
	1500 to 3000 excl.		3000 to 6000 excl.		6000 to 9000 incl.		Over 9000 to 12 000 incl.		Over 12 000 to 20 000 incl.		Over 20 000	
	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under
Under 75	16	0	25	0	38	0	51	0	64	0	—	—
75 and over	25	0	38	0	45	0	57	0	70	0	—	—

Note: Where "__" appears in this table, there is no requirement. See ASTM A6 / A6M.

PERMISSIBLE VARIATIONS IN SECTION DIMENSIONS FOR S, M, C AND MC SHAPES



(S Shape) (C Shape)

Shapes	Nominal Size, mm	Depth, A, mm		Flange Width, B, mm		Out-of-Square $\frac{T+T'}{B}$, mm / mm	Out-of-parallel $\frac{C-D}{B}$, mm / mm
		Over	Under	Over	Under		
S, M *	75 – 180 incl.	2	2	3	3	0.03	—
	Over 180 – 360 incl.	3	2	4	4	0.03	—
	Over 360 – 610 incl.	5	3	5	5	0.03	—
Channels (C and MC)	75 – 180 incl.	3	2	3	3	0.03	0.03
	Over 180 – 360 incl.	3	3	3	4	0.03	0.03
	Over 360	5	4	3	5	0.03	0.03

* Web off-centre tolerance is 5 mm.

** Back of square and centreline of web to be parallel when measuring out-of-square.

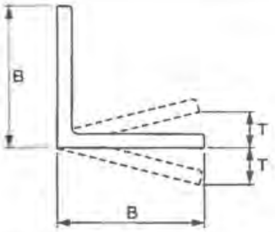
"A" is measured at centreline of web for beams and at back of web for channels.

Mass and Area Tolerances

Structural-size shapes – cross-sectional area or mass: ±2.5% from theoretical.

Tolerances for Angles

Permissible variations for cross-sectional dimensions of bar-size angles (defined as rolled angles having maximum cross-sectional dimensions less than 75 mm), differ from structural size angles, and both variations are given in the following table (see ASTM A6 / A6M).



Structural Size Angles				Bar-Size Angles**				
Specified Size*, mm	Length of Leg, B, mm		Out-of-Square T / B	Specified Size*, mm	Variations from Thickness Given, mm			Variations from Length of Leg Over and Under, mm
	Over	Under			5 and under	Over 5 to 10 incl.	Over 10	
Over 64 to 102 incl.	3	2	0.026	25 and Under	0.2	0.2	—	1
Over 102 to 152 incl.	3	3	0.026	Over 25 to 51 incl.	0.2	0.2	0.3	1
Over 152 to 203 incl.	5	3	0.026	Over 51 to 64 incl.	0.3	0.4	0.4	2
Over 203 to 254 incl.	6	6	0.026	Note: Where "__" appears in this table, there is no requirement.				
Over 254	6	10	0.026	* For unequal-leg angles, longer leg determines classification. ** Permissible out-of-square in either direction is 1.5 degrees.				

HOLLOW STRUCTURAL SECTIONS (HSS)

General

Production information and tolerances given below correspond to HSS produced in accordance with CSA G40.20/G40.21, unless noted otherwise.

Class

Class H means hollow sections made by:

(i) A seamless or furnace-butt-welded (continuous-welded) or automatic electric welding process hot-formed to final shape; or

(ii) A seamless or automatic electric welding process producing a continuous weld, and cold-formed to final shape, subsequently stress-relieved by heating to a temperature of 450°C or higher, followed by cooling in air.

Class C means HSS that are cold-formed from a section produced by a seamless process or by an automatic electric welding process producing a continuous weld.

Cross-Sectional Dimensions

Outside dimensions measured across the flats or diameter at positions at least 50 mm from either end of a piece, including an allowance for convexity or concavity, shall not vary from the specified dimensions of the section by more than the prescribed tolerances.

Largest Outside Dimension Across Flats or Diameter, mm	Tolerance*, mm
To 65	± 0.5
Over 65 - 90 incl.	± 0.8
Over 90 - 140 incl.	± 1.0
Over 140	± 1%

* Tolerance includes allowance for convexity or concavity. Tolerance may be increased by 50 percent when applied to the smaller dimension of rectangular sections whose ratio of cross-sectional dimensions is between 1.5 and 3, and by 100 percent when this ratio exceeds 3.

Corner Squareness

For rectangular sections, corners shall be square (90°) within ± 1° for hot-formed sections and ± 2° for cold-formed sections, with the average slope of the sides being the basis for determination.

Straightness Variation

Deviation from straightness in millimetres shall not exceed the total length in millimetres divided by 500.

Permissible Twist

Twist of a rectangular section, measured by holding down the side of one end of the section on a flat surface and noting the height above the surface of either corner at the opposite end of that side, shall not exceed the prescribed tolerances:

Largest Outside Dimension, mm	Maximum Twist per 1000 mm of Length, mm
To 40 incl.	1.3
Over 40 - 65 incl.	1.7
Over 65 - 105 incl.	2.1
Over 105 - 155 incl.	2.4
Over 155 - 205 incl.	2.8
Over 205	3.1

Cutting Tolerances

Tolerances on ordered cold-cut lengths are:

+ 12 and - 6 millimetres for lengths 7500 mm and under;
+ 18 and - 6 millimetres for lengths over 7500 mm.

Tolerances on ordered hot-cut lengths of hot rolled sections are:

± 25 millimetres for lengths 7500 mm and under;
± 50 millimetres for lengths over 7500 mm.

Mass Variation – CSA G40.20, ASTM A1085, and ASTM A500

For HSS produced to CSA G40.20 and to ASTM A1085 and based on a mass density of 7850 kg/m^3, the actual mass shall not deviate from the published mass by more than - 3.5 % or + 10 %. For HSS produced to ASTM A500, there is no restriction on mass variation.

Wall Thickness – CSA G40.20, ASTM A1085 and ASTM A500

For HSS produced to CSA G40.20 and to ASTM A1085, the tolerance on the wall thickness is not more than - 5 % or + 10 % from the nominal specified wall thickness, except for the weld seam. For ASTM A500, the tolerance is not more than ± 10 % from the nominal wall thickness, except for the weld seam.

Outside Corner Radius Tolerances for Square and Rectangular HSS

CSA G40.20

Wall Thickness mm	Maximum Outside Corner Radii, mm	
	Perimeter to 700 mm Incl.	Perimeter Over 700 mm
To 3 incl.	6	–
Over 3 - 4 incl.	8	–
Over 4 - 5 incl.	15	–
Over 5 - 6 incl.	18	18
Over 6 - 8 incl.	21	24
Over 8 - 10 incl.	27	30
Over 10 - 13 incl.	36	39
Over 13	–	3 x wall thickness

For HSS produced to ASTM A500, the radius of outside corners shall not exceed three times the specified wall thickness. For ASTM A1085, the outside corner radius shall meet the following requirements, where t is the wall thickness:

$t \leq 10.2$ mm, $1.6\,t \leq$ corner radius $\leq 3.0\,t$

$t > 10.2$ mm, $1.8\,t \leq$ corner radius $\leq 3.0\,t$

PRINCIPAL SOURCES OF STRUCTURAL STEEL SECTIONS

General

Standard Canadian and North American sections can be supplied by a number of steel mills in Canada and elsewhere. Principal sources for the various section sizes listed in this Handbook are indicated below.

In 2010, Essar Steel Algoma Inc. withdrew from the production of welded wide-flange (WWF) sections.

W-Shapes

In 1999, Algoma Steel Inc. (Essar Steel Algoma Inc.), the sole Canadian producer of W and HP-shapes for three decades, announced its withdrawal from the rolled shape market. W-shapes most commonly used in North America today are ASTM A992 products. Some of the very heavy sections are produced to ASTM A913.

Channel and Angle Sections

Most channels and angles listed in Part 6 are available from Canadian mills. Imported sizes are identified by an asterisk (*) in tables of Properties and Dimensions. In general, all sizes should be specified to the CSA G40.20/G40.21 material standards. Gerdau operates several North American mills that typically produce channels and angles certified to multiple grades, including CSA G40.21-350W and 300W, and ASTM 572 Grade 50.

Hollow Structural Sections

Both CSA G40.20/G40.21 and ASTM A500 HSS are produced in Canada. Jumbo HSS are the exceptions; they are identified as imports by an asterisk (*) in tables of Properties and Dimensions. A500 products are not a direct substitute for G40.21-350W HSS. In the section entitled *Hollow Structural Sections*, the text preceding the tables of Properties and Dimensions highlights the differences between these two products.

Principal Sources

Some of the more common sources (for Canada) of structural sections and other products are listed below. Producers' catalogs should be consulted for more information and details about other products produced. This list is a general guide and is not necessarily complete.

ArcelorMittal Canada (bars, sheet steel)
ArcelorMittal International Canada * (shapes, plate, bars, HSS)
Atlas Tube Canada ULC (HSS)
Essar Steel Algoma Inc. (plate, checkered floor plate, coil)
Evraz North America * (pipe, plate, coil)
Gerdau (angles, channels, bars)
Gerdau - Texas Steel Mill * (shapes)
Nucor Corporation * (plate, bars, sheet steel)
Nucor-Yamato Steel Company * (shapes)
SSAB Central Inc. (sheet steel, plate)
Steel Dynamics Inc. * (shapes, sheet steel)
Welded Tube of Canada (HSS, pipe)

** non-Canadian sources*

Note: Since not all of the above are members of CISC, please visit the CISC website (www.cisc-icca.ca) to view the current list of CISC mill and steel service centre members.

Availability

Section sizes are generally produced according to production (rolling) schedules. Steel producers and service centres carry various inventories, usually of the more commonly used sections, and serve as a buffer between production cycles to provide ready availability of material. The designer should consider material availability when specifying section sizes, particularly for the heavier mass per metre sizes in a nominal size range and for small quantities of the less commonly used sizes.

Because regional availability of steel products varies, information on the availability of particular sizes can be obtained from local steel fabricators, producers, and service centres. In order to provide approximate guidance on general availability, this Handbook adopts the following convention:

- I-shapes (all W, HP, S and M sections are imported): readily available sizes are highlighted in yellow.
- Other sections (the majority of channels, angles and HSS are produced in Canada): imported sizes are labelled with an asterisk (*).

Table 6-8 shows the primary and secondary grades for common steel shapes in terms of general availability and usage.

METRIC AND IMPERIAL SHAPES

General

In Canada, the official size designation for structural steel sections for purposes of design, detailing and ordering material is the metric (SI) designation. Canadian and North American sections may also be defined using imperial designations; however, all tables of properties and dimensions, and all design tables included elsewhere in this Handbook generally provide only metric properties and metric design information.

General requirements for rolled and welded shapes are specified in CSA Standard G40.20/21, which refers mostly to ASTM A6/A6M for the designation and dimensions of rolled shapes. Tables on the following pages list metric (SI) designations and corresponding imperial designations.

W, HP, S, M, C and MC Shapes

The metric designation is the nominal depth in millimetres times the nominal mass in kilograms per metre, and the corresponding imperial designation is expressed in inches × lb/ft.

Angles (L)

The metric size description given in this Handbook is expressed as leg lengths in whole millimetres and thickness in millimetres to two significant figures, while the imperial description is expressed as leg lengths in inches and thickness in fractional inches.

Hollow Structural Sections (HSS)

The metric size description of square, rectangular and round hollow structural sections is expressed as the outside dimensions in whole millimetres times the nominal wall thickness in millimetres to two significant figures. The imperial description consists of the outside dimensions in inches and the nominal wall thickness in decimal inches.

Welded Sections

Welded wide-flange (WWF) and welded reduced-flange (WRF) sections must be produced to CSA Standard G40.20/21, whereas welded three-plate sections are generally fabricated to the requirements of CSA Standard W59. The major producer of WWF and WRF sections discontinued production in 2010. Data for these sections are no longer provided in this Handbook.

METRIC SHAPES

Metric (SI) designations for rolled shapes in this Handbook generally comply with ASTM A6/A6M except for sections also listed in CSA Standard G312.3-M92 "Metric Dimensions for Structural Steel Shapes and Hollow Structural Sections". For a number of section sizes, the respective metric designations in the two standards are slightly different. In many cases, the principal difference involves a decimal digit in the nominal mass based on A6. These sections are listed in the comparison table below, with the imperial designation also provided for reference purposes. For other sections not listed, metric designations given in this Handbook are the same as in A6/A6M.

In the case of angles, the only difference between the respective metric size descriptions involves a decimal digit in the nominal leg thickness based on A6 for thicknesses greater than 9.5 mm. Since the leg widths are identical according to both standards, only the thicknesses are listed.

Handbook	A6/A6M	
Metric	Metric	Imperial
W Shapes		
W410x74	W410x75	W16x50
W410x54	W410x53	W16x36
W410x46	W410x46.1	W16x31
W410x39	W410x38.8	W16x26
W360x57	W360x58	W14x38
W360x45	W360x44.6	W14x30
W360x39	W360x39.0	W14x26
W360x33	W360x32.9	W14x22
W310x118	W310x117	W12x79
W310x45	W310x44.5	W12x30
W310x39	W310x38.7	W12x26
W310x33	W310x32.7	W12x22
W310x28	W310x28.3	W12x19
W310x24	W310x23.8	W12x16
W310x21	W310x21.0	W12x14
W250x49	W250x49.1	W10x33
W250x45	W250x44.8	W10x30
W250x39	W250x38.5	W10x26
W250x33	W250x32.7	W10x22
W250x28	W250x28.4	W10x19
W250x25	W250x25.3	W10x17
W250x22	W250x22.3	W10x15
W250x18	W250x17.9	W10x12

Handbook	A6/A6M	
Metric	Metric	Imperial
W Shapes (Cont'd)		
W200x46	W200x46.1	W8x31
W200x42	W200x41.7	W8x28
W200x36	W200x35.9	W8x24
W200x31	W200x31.3	W8x21
W200x27	W200x26.6	W8x18
W200x22	W200x22.5	W8x15
W200x19	W200x19.3	W8x13
W200x15	W200x15.0	W8x10
W150x37	W150x37.1	W6x25
W150x30	W150x29.8	W6x20
W150x24	W150x24.0	W6x16
W150x22	W150x22.5	W6x15
W150x18	W150x18.0	W6x12
W150x14	W150x13.5	W6x9
W150x13	W150x13.0	W6x8.5
W130x28	W130x28.1	W5x19
W130x24	W130x23.8	W5x16
W100x19	W100x19.3	W4x13

METRIC SHAPES (Cont'd)

Handbook	A6/A6M	
Metric	Metric	Imperial
HP Shapes		
HP310x94	HP310x93	HP12x63
HP200x54	HP200x53	HP8x36
S Shapes		
S510x98.2	S510x98	S20x66
S310x47	S310x47.3	S12x31.8
S250x38	S250x37.8	S10x25.4
S200x27	S200x27.4	S8x18.4
S150x26	S150x25.7	S6x17.25
S150x19	S150x18.6	S6x12.5
S100x11	S100x11.5	S4x7.7
S75x11	S75x11.2	S3x7.5
S75x8	S75x8.5	S3x5.7
C Shapes		
C380x50	C380x50.4	C15x33.9
C310x31	C310x30.8	C12x20.7
C250x23	C250x22.8	C10x15.3
C230x20	C230x19.9	C9x13.4
C200x28	C200x27.9	C8x18.75
C200x21	C200x20.5	C8x13.75
C200x17	C200x17.1	C8x11.5
C180x18	C180x18.2	C7x12.25
C180x15	C180x14.6	C7x9.8
C150x19	C150x19.3	C6x13
C150x16	C150x15.6	C6x10.5
C150x12	C150x12.2	C6x8.2
C130x10	C130x10.4	C5x6.7
C100x11	C100x10.8	C4x7.25
C100x9	C100x9.3	C4x6.25
C100x7	C100x6.7	C4x4.5
C75x9	C75x8.9	C3x6
C75x7	C75x7.4	C3x5
C75x6	C75x6.1	C3x4.1
C75x5	C75x5.2	C3x3.5

Handbook	A6/A6M	
mm	mm	in.
L Shapes - Leg Thicknesses > 9.5 mm		
35	34.9	$1^3/_8$
32	31.8	$1^1/_4$
29	28.6	$1^1/_8$
25	25.4	1
22	22.2	$^7/_8$
19	19.1/19.0	$^3/_4$
16	15.9	$^5/_8$
14	14.3	$^9/_{16}$
13	12.7	$^1/_2$
11	11.1	$^7/_{16}$

DESIGNATION TABLE FOR W SHAPES

Canadian (SI) Designation (mm x kg/m)	Imperial Designation (in. x lb./ft.)
W1100x499	W44x335
x433	x290
x390	x262
x343	x230
W1000x976	W40x655
x883	x593
x748	x503
x642	x431
x591	x397
x554	x372
x539	x362
x483	x324
x443	x297
x412	x277
x371	x249
x321	x215
x296	x199
W1000x584	W40x392
x494	x331
x486	x327
x438	x294
x415	x278
x393	x264
x350	x235
x314	x211
x272	x183
x249	x167
x222	x149
W920x1377	W36x925
x1269	x853
x1194	x802
x1077	x723
x970	x652
x787	x529
x725	x487
x656	x441
x588	x395
x537	x361
x491	x330
x449	x302
x420	x282
x390	x262
x368	x247
x344	x231
W920x381	W36x256
x345	x232
x313	x210
x289	x194
x271	x182
x253	x170
x238	x160
x223	x150
x201	x135

Canadian (SI) Designation (mm x kg/m)	Imperial Designation (in. x lb./ft.)
W840x576	W33x387
x527	x354
x473	x318
x433	x291
x392	x263
x359	x241
x329	x221
x299	x201
W840x251	W33x169
x226	x152
x210	x141
x193	x130
x176	x118
W760x582	W30x391
x531	x357
x484	x326
x434	x292
x389	x261
x350	x235
x314	x211
x284	x191
x257	x173
W760x220	W30x148
x196	x132
x185	x124
x173	x116
x161	x108
x147	x99
x134	x90
W690x802	W27x539
x548	x368
x500	x336
x457	x307
x419	x281
x384	x258
x350	x235
x323	x217
x289	x194
x265	x178
x240	x161
x217	x146
W690x192	W27x129
x170	x114
x152	x102
x140	x94
x125	x84

Canadian (SI) Designation (mm x kg/m)	Imperial Designation (in. x lb./ft.)
W610x551	W24x370
x498	x335
x455	x306
x415	x279
x372	x250
x341	x229
x307	x207
x285	x192
x262	x176
x241	x162
x217	x146
x195	x131
x174	x117
x155	x104
W610x153	W24x103
x140	x94
x125	x84
x113	x76
x101	x68
W610x92	W24x62
x82	x55
W530x409	W21x275
x369	x248
x332	x223
x300	x201
x272	x182
x248	x166
x219	x147
x196	x132
x182	x122
x165	x111
x150	x101
W530x138	W21x93
x123	x83
x109	x73
x101	x68
x92	x62
x82	x55
x72	x48
W530x85	W21x57
x74	x50
x66	x44

DESIGNATION TABLE FOR W SHAPES

Canadian (SI) Designation (mm x kg/m)	Imperial Designation (in. x lb./ft.)
W460x464	W18x311
x421	x283
x384	x258
x349	x234
x315	x211
x286	x192
x260	x175
x235	x158
x213	x143
x193	x130
x177	x119
x158	x106
x144	x97
x128	x86
x113	x76
W460x106	W18x71
x97	x65
x89	x60
x82	x55
x74	x50
W460x68	W18x46
x60	x40
x52	x35
W410x149	W16x100
x132	x89
x114	x77
x100	x67
W410x85	W16x57
x74	x50
x67	x45
x60	x40
x54	x36
W410x46	W16x31
x39	x26
W360x1299	W14x873
x1202	x808
x1086	x730
x990	x665
x900	x605
x818	x550
x744	x500
x677	x455
x634	x426
x592	x398
x551	x370
x509	x342
x463	x311
x421	x283
x382	x257
x347	x233
x314	x211
x287	x193
x262	x176
x237	x159
x216	x145

Canadian (SI) Designation (mm x kg/m)	Imperial Designation (in. x lb./ft.)
W360x196	W14x132
x179	x120
x162	x109
x147	x99
x134	x90
W360x122	W14x82
x110	x74
x101	x68
x91	x61
W360x79	W14x53
x72	x48
x64	x43
W360x57	W14x38
x51	x34
x45	x30
W360x39	W14x26
x33	x22
W310x500	W12x336
x454	x305
x415	x279
x375	x252
x342	x230
x313	x210
x283	x190
x253	x170
x226	x152
x202	x136
x179	x120
x158	x106
x143	x96
x129	x87
x118	x79
x107	x72
x97	x65
W310x86	W12x58
x79	x53
W310x74	W12x50
x67	x45
x60	x40
W310x52	W12x35
x45	x30
x39	x26
W310x33	W12x22
x28	x19
x24	x16
x21	x14

Canadian (SI) Designation (mm x kg/m)	Imperial Designation (in. x lb./ft.)
W250x167	W10x112
x149	x100
x131	x88
x115	x77
x101	x68
x89	x60
x80	x54
x73	x49
W250x67	W10x45
x58	x39
x49	x33
W250x45	W10x30
x39	x26
x33	x22
W250x28	W10x19
x25	x17
x22	x15
x18	x12
W200x100	W8x67
x86	x58
x71	x48
x59	x40
x52	x35
x46	x31
W200x42	W8x28
x36	x24
W200x31	W8x21
x27	x18
W200x22	W8x15
x19	x13
x15	x10
W150x37	W6x25
x30	x20
x22	x15
W150x24	W6x16
x18	x12
x14	x9
x13	x8.5
W130x28	W5x19
x24	x16
W100x19	W4x13

DESIGNATION TABLE FOR HP, M, S, C, MC SHAPES

Canadian (SI) Designation (mm x kg/m)	Imperial Designation (in. x lb./ft.)
HP460x304	HP18x204
x269	x181
x234	x157
x202	x135
HP410x272	HP16x183
x242	x162
x211	x141
x181	x121
x151	x101
x131	x88
HP360x174	HP14x117
x152	x102
x132	x89
x108	x73
HP310x132	HP12x89
x125	x84
x110	x74
x94	x63
x79	x53
HP250x85	HP10x57
x62	x42
HP200x54	HP8x36
M318x18.5	M12.5x12.4
x17.3	x11.6
M310x17.6	M12x11.8
x16.1	x10.8
x14.9	x10.0
M250x13.4	M10x9.0
x11.9	x8.0
x11.2	x7.5
M200x9.7	M8x6.5
x9.2	x6.2
M150x6.6	M6x4.4
x5.5	x3.7
M130x28.1	M5x18.9
M100x8.9	M4x6.0
x6.1	x4.08
M75x4.3	M3x2.9
S610x180	S24x121
x158	x106
S610x149	S24x100
x134	x90
x119	x80
S510x143	S20x96
x128	x86
S510x112	S20x75
x98.2	x66

Canadian (SI) Designation (mm x kg/m)	Imperial Designation (in. x lb./ft.)
S460x104	S18x70
x81.4	x54.7
S380x74	S15x50
x64	x42.9
S310x74	S12x50
x60.7	x40.8
S310x52	S12x35
x47	x31.8
S250x52	S10x35
x38	x25.4
S200x34	S8x23
x27	x18.4
S150x26	S6x17.25
x19	x12.5
S130x15	S5x10
S100x14.1	S4x9.5
x11	x7.7
S75x11	S3x7.5
x8	x5.7
C380x74	C15x50
x60	x40
x50	x33.9
C310x45	C12x30
x37	x25
x31	x20.7
C250x45	C10x30
x37	x25
x30	x20
x23	x15.3
C230x30	C9x20
x22	x15
x20	x13.4
C200x28	C8x18.75
x21	x13.75
x17	x11.5
C180x22	C7x14.75
x18	x12.25
x15	x9.8
C150x19	C6x13
x16	x10.5
x12	x8.2
C130x13	C5x9
x10	x6.7
C100x11	C4x7.25
x9	x6.25
x8	x5.4
x7	x4.5

Canadian (SI) Designation (mm x kg/m)	Imperial Designation (in. x lb./ft.)
C75x9	C3x6
x7	x5
x6	x4.1
x5	x3.5
MC460x86	MC18x58
x77.2	x51.9
x68.2	x45.8
x63.5	x42.7
MC330x74	MC13x50
x60	x40
x52	x35
x47.3	x31.8
MC310x74	MC12x50
x67	x45
x60	x40
x52	x35
x46	x31
MC310x21.3	MC12x14.3
MC310x15.8	MC12x10.6
MC250x61.2	MC10x41.1
x50	x33.6
x42.4	x28.5
MC250x37	MC10x25
x33	x22
MC250x12.5	MC10x8.4
x9.7	x6.5
MC230x37.8	MC9x25.4
x35.6	x23.9
MC200x33.9	MC8x22.8
x31.8	x21.4
MC200x29.8	MC8x20
x27.8	x18.7
MC200x12.6	MC8x8.5
MC180x33.8	MC7x22.7
x28.4	x19.1
MC150x26.8	MC6x18
x22.8	x15.3
MC150x24.3	MC6x16.3
x22.5	x15.1
MC150x17.9	MC6x12
MC150x10.4	MC6x7.0
x9.7	x6.5
MC100x20.5	MC4x13.8
MC75x10.6	MC3x7.1

ANGLES

Canadian (SI) Section (mm x mm x mm)	Imperial Section (in. x in. x in.)
L254x 254x 32	L10x 10x $1^1/_4$
x 29	x $1^1/_8$
x 25	x 1
x 22	x $^7/_8$
x 19	x $^3/_4$
L203x 203x 29	L8x 8x $1^1/_8$
x 25	x 1
x 22	x $^7/_8$
x 19	x $^3/_4$
x 16	x $^5/_8$
x 14	x $^9/_{16}$
x 13	x $^1/_2$
L203x 152x 25	L8x 6x 1
x 22	x $^7/_8$
x 19	x $^3/_4$
x 16	x $^5/_8$
x 14	x $^9/_{16}$
x 13	x $^1/_2$
x 11	x $^7/_{16}$
L203x 102x 25	L8x 4x 1
x 22	x $^7/_8$
x 19	x $^3/_4$
x 16	x $^5/_8$
x 14	x $^9/_{16}$
x 13	x $^1/_2$
x 11	x $^7/_{16}$
L178x 102x 19	L7x 4x $^3/_4$
x 16	x $^5/_8$
x 13	x $^1/_2$
x 11	x $^7/_{16}$
x 9.5	x $^3/_8$
L152x 152x 25	L6x 6x 1
x 22	x $^7/_8$
x 19	x $^3/_4$
x 16	x $^5/_8$
x 14	x $^9/_{16}$
x 13	x $^1/_2$
x 11	x $^7/_{16}$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
L152x 102x 22	L6x 4x $^7/_8$
x 19	x $^3/_4$
x 16	x $^5/_8$
x 14	x $^9/_{16}$
x 13	x $^1/_2$
x 11	x $^7/_{16}$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
L152x 89x 13	L6x $3^1/_2$ x $^1/_2$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$

Canadian (SI) Section (mm x mm x mm)	Imperial Section (in. x in. x in.)
L127x 127x 22	L5x 5x $^7/_8$
x 19	x $^3/_4$
x 16	x $^5/_8$
x 13	x $^1/_2$
x 11	x $^7/_{16}$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
L127x 89x 19	L5x $3^1/_2$ x $^3/_4$
x 16	x $^5/_8$
x 13	x $^1/_2$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
L127x 76x 13	L5x 3x $^1/_2$
x 11	x $^7/_{16}$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
L102x 102x 19	L4x 4x $^3/_4$
x 16	x $^5/_8$
x 13	x $^1/_2$
x 11	x $^7/_{16}$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
L102x 89x 13	L4x $3^1/_2$ x $^1/_2$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
L102x 76x 16	L4x 3x $^5/_8$
x 13	x $^1/_2$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
L89x 89x 13	L$3^1/_2$ x $3^1/_2$ x $^1/_2$
x 11	x $^7/_{16}$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
L89x 76x 13	L$3^1/_2$ x 3x $^1/_2$
x 11	x $^7/_{16}$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
L89x 64x 13	L$3^1/_2$ x $2^1/_2$ x $^1/_2$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
L76x 76x 13	L3x 3x $^1/_2$
x 11	x $^7/_{16}$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
x 4.8	x $^3/_{16}$

Canadian (SI) Section (mm x mm x mm)	Imperial Section (in. x in. x in.)
L76x 64x 13	L3x $2^1/_2$ x $^1/_2$
x 11	x $^7/_{16}$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
x 4.8	x $^3/_{16}$
L76x 51x 13	L3x 2x $^1/_2$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
x 4.8	x $^3/_{16}$
L64x 64x 13	L$2^1/_2$ x $2^1/_2$ x $^1/_2$
x 9.5	x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
x 4.8	x $^3/_{16}$
L64x 51x 9.5	L$2^1/_2$ x 2x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
x 4.8	x $^3/_{16}$
L64x 38x 6.4	L$2^1/_2$ x $1^1/_2$ x $^1/_4$
x 4.8	x $^3/_{16}$
L51x 51x 9.5	L2x 2x $^3/_8$
x 7.9	x $^5/_{16}$
x 6.4	x $^1/_4$
x 4.8	x $^3/_{16}$
x 3.2	x $^1/_8$
L51x 38x 6.4	L2x $1^1/_2$ x $^1/_4$
x 4.8	x $^3/_{16}$
x 3.2	x $^1/_8$
L44x 44x 6.4	L$1^3/_4$ x $1^3/_4$ x $^1/_4$
x 4.8	x $^3/_{16}$
x 3.2	x $^1/_8$
L38x 38x 6.4	L$1^1/_2$ x $1^1/_2$ x $^1/_4$
x 4.8	x $^3/_{16}$
x 4.0	x $^5/_{32}$
x 3.2	x $^1/_8$
L32x 32x 6.4	L$1^1/_4$ x $1^1/_4$ x $^1/_4$
x 4.8	x $^3/_{16}$
x 3.2	x $^1/_8$
L25x 25x 6.4	L1x 1x $^1/_4$
x 4.8	x $^3/_{16}$
x 3.2	x $^1/_8$
L19x 19x 3.2	L$^3/_4$ x $^3/_4$ x $^1/_8$

SQUARE HSS

Canadian (SI) Section (mm x mm x mm)	Imperial Section (in. x in. x in.)	CSA G40.21	ASTM A500
559x 559x 19	22x 22x 0.750	✓	
508x 508x 22	20x 20x 0.875	✓	
x 19	x 0.750	✓	
x 16	x 0.625	✓	
x 13	x 0.500	✓	
457x 457x 22	18x 18x 0.875	✓	
x 19	x 0.750	✓	
x 16	x 0.625	✓	
x 13	x 0.500	✓	
406x 406x 22	16x 16x 0.875	✓	
x 19	x 0.750	✓	
x 16	x 0.625	✓	✓
x 13	x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
356x 356x 16	14x 14x 0.625	✓	✓
x 13	x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
305x 305x 16	12x 12x 0.625	✓	✓
x 13	x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
254x 254x 16	10x 10x 0.625	✓	✓
x 13	x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
203x 203x 16	8x 8x 0.625	✓	✓
x 13	x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
178x 178x 16	7x 7x 0.625	✓	✓
x 13	x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
152x 152x 13	6x 6x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
127x 127x 13	5x 5x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓

Canadian (SI) Section (mm x mm x mm)	Imperial Section (in. x in. x in.)	CSA G40.21	ASTM A500
102x 102x 13	4x 4x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
89x 89x 9.5	3.5x 3.5x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
76x 76x 9.5	3x 3x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
64x 64x 6.4	2.5x 2.5x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
51x 51x 6.4	2x 2x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
38x 38x 4.8	1.5x 1.5x 0.188	✓	✓
x 3.2	x 0.125	✓	✓

RECTANGULAR HSS

Canadian (SI) Section (mm x mm x mm)	Imperial Section (in. x in. x in.)	CSA G40.21	ASTM A500
305x 203x 16	12x 8x 0.625	✓	✓
x 13	x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
305x 152x 16	12x 6x 0.625	✓	✓
x 13	x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
254x 203x 16	10x 8x 0.625	✓	✓
x 13	x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
254x 152x 16	10x 6x 0.625	✓	✓
x 13	x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
203x 152x 16	8x 6x 0.625	✓	✓
x 13	x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
203x 102x 13	8x 4x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
178x 127x 13	7x 5x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
152x 102x 13	6x 4x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
152x 76x 13	6x 3x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
127x 76x 13	5x 3x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
102x 76x 9.5	4x 3x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
102x 51x 9.5	4x 2x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
89x 64x 6.4	3.5x 2.5x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
76x 51x 7.9	3x 2x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
76x 38x 6.4	3x 1.5x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
64x 38x 6.4	2.5x 1.5x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
51x 25x 4.8	2x 1x 0.188	✓	✓
x 3.2	x 0.125	✓	✓

ROUND HSS

Canadian (SI) Section (mm x mm x mm)	Imperial Section (in. x in. x in.)	CSA G40.21	ASTM A500
508x 13	20x 0.500		✓
x 9.5	x 0.375		✓
x 6.4	x 0.250		✓
457x 13	18x 0.500		✓
x 9.5	x 0.375		✓
x 6.4	x 0.250		✓
406x 16	16x 0.625	✓	✓
x 13	x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 6.4	x 0.250	✓	✓
356x 13	14x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 6.4	x 0.250	✓	✓
324x 13	12.75x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 6.4	x 0.250	✓	✓
273x 13	10.75x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 7.9	x 0.313	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
245x 9.5	9.625x 0.375	✓	✓
x 6.4	x 0.250	✓	✓
219x 13	8.625x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
178x 13	7x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
168x 13	6.625x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 6.4	x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
141x 13	5.563x 0.500	✓	✓
x 9.5	x 0.375	✓	✓
x 6.4	x 0.250	✓	✓
127x 9.5	5x 0.375	✓	✓
x 6.4	x 0.250	✓	✓
89x 6.4	3.5x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
76x 6.4	3x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
73x 6.4	2.875x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
64x 6.4	2.5x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
60x 6.4	2.376x 0.250	✓	✓
x 4.8	x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
48x 4.8	1.9x 0.188	✓	✓
x 3.2	x 0.125	✓	✓
42x 3.2	1.66x 0.125		✓

ROLLED STRUCTURAL SHAPES

General

The majority of rolled shapes available in Canada are produced either to ASTM A992, ASTM A572 grade 50, or CSA Standard G40.21-350W. These grades have similar, but not identical, specified minimum values of yield. For more information on steel grades, tolerances, and mill practice, see *Grades, Types, Strength Levels* and *Standard Mill Practice* in Part 6.

The tables of properties and dimensions on the following pages include most of the rolled shapes used in construction. See *Principal Sources of Structural Sections* in Part 6 for information regarding Canadian and non-Canadian sections.

Special shapes, such as rolled Tees, Zees, bulb angles, car-building and shipbuilding channels are produced by some mills. These shapes are generally rolled only at irregular intervals and usually by special arrangement. Their use should, therefore, be avoided unless the quantity of any one size can warrant a rolling. Properties and dimensions of these shapes may be obtained from the appropriate mill catalogs.

Properties and Dimensions

The basic metric dimensions used to compute properties of the rolled steel shapes were originally taken from CSA Standard G312.3-M92 "Metric Dimensions for Structural Steel Shapes and Hollow Structural Sections". General requirements for rolled shapes are specified in CSA Standard G40.20/21, which refers mostly to ASTM A6 for the designation and dimensions of rolled shapes.

Section properties for hot-rolled shapes (except angles) are calculated using the smallest theoretical web-to-flange fillet radius, while dimensions for detailing are adjusted for the largest theoretical fillet radius. Due to differences in fillet radii among steel producers, actual properties may vary slightly from the tabulated values.

Most W and HP shapes are produced in the U.S. W-shapes available in Canada have essentially parallel flanges. HP shapes are essentially square (equal flange width and overall depth) with parallel flange surfaces, and with flanges and web of equal thickness. S-shapes and standard channels (C-shapes) have tapered flanges with the inside face sloping at approximately 16⅔% (2 in 12). The tabulated thickness is the mean thickness. All C-shapes listed in the tables are produced in Canada, except for sections denoted with an asterisk (*), although no information is given regarding availability. S-shapes are not available from any Canadian producer.

M and MC-shapes are essentially shapes that cannot be classified as W, HP, S or C-shapes. They are not rolled in Canada and are usually only produced by a single mill. Availability should be checked before specifying their use. These shapes may be produced with parallel flanges or with tapered flanges of various slopes. Dimensions and properties provided in this Handbook should be suitable for general use, in spite of possible variations in actual dimensions.

Availability of W-Shapes

Currently, structural steel is widely available and as such makes an excellent choice as a structural material. While there are thousands of sections listed at any one time, the availability of a specific section in a particular region of the country for a specific project and time frame may result in the fabricator requesting a substitution. Some sections are almost always available due to a constant demand for them. It is important to remember that the least-cost solution is not always the least-weight alternative.

W-shapes are not produced by Canadian mills. Their availability is indicated in this Handbook by means of yellow shading. The highlighted sections are the commonly used sizes which are generally readily available.

Angles

Properties and dimensions are provided for hot-rolled equal-leg and unequal-leg angles. The tables include properties and dimensions for single angles and for two equal-leg angles back-to-back, two unequal-leg angles with short legs back-to-back, and two unequal-leg angles with long legs back-to-back. Section properties of hot-rolled angles are based on flat rectangular legs, excluding the fillet and roundings.

All angles listed in the tables are produced in Canada, except for sections denoted with an asterisk (*), although no information is given regarding availability.

The properties of hot-rolled L254 angles produced by Arcelor-Mittal may be up to 3% less than the tabulated values due to the presence of a rounded heel. In general, the properties of angles produced by cold-forming may be up to 7% less than the properties of hot-rolled angles of similar size due to the rounded heel. Designers encountering cold-formed angles should consult the manufacturer's catalog for the exact dimensions and properties. Cold-formed members are generally designed according to CSA Standard S136.

The tables of properties and dimensions for single angles include both equal-leg and unequal-leg angles. Since equal-leg angles are the more commonly available of the two types, their properties about axis Y-Y (which are identical to those about axis X-X) have been omitted to help identify them more readily.

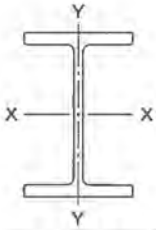
For the definition of torsional properties x_o, y_o, $\bar{r}_o$ and Ω given in the tables, see CSA S16-14 Clause 13.3.2. The y-axis of symmetry of equal-leg angles as defined in this Clause corresponds to X'-X' in the tables.

Tees Cut from W-Shapes

Properties and dimensions of Tees are based on W-shapes assuming a depth of the Tee equal to one-half the depth of the corresponding W-shape. Tees are not rolled and are usually fabricated from W-shapes by splitting the web using either rotary shears or flame cutting, and subsequently straightening to meet published tolerances.

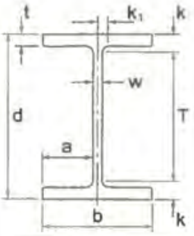
For the definition of torsional properties y_o, $\bar{r}_o$ and Ω, see CSA S16-14 Clause 13.3.2.

W SHAPES
W1100 - W1000



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm^2	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^3\ mm^4$	$10^9\ mm^6$
W1100												
x499	4.89	63 500	12 900	23 100	451	26 600	500	2 470	88.7	3 870	31 100	144 000
x433	4.24	55 100	11 300	20 300	452	23 200	434	2 160	88.7	3 360	21 200	124 000
x390	3.83	49 700	10 100	18 300	450	20 800	385	1 920	88.0	2 990	15 600	109 000
x343	3.36	43 600	8 670	15 900	446	18 100	331	1 660	87.1	2 570	10 300	92 900
W1000												
x976	9.56	124 300	23 500	42 400	435	50 300	1 190	5 540	97.7	8 840	244 000	307 000
x883	8.66	112 500	21 000	38 400	432	45 300	1 050	4 950	96.6	7 870	185 000	268 000
x748	7.34	95 300	17 300	32 400	426	37 900	851	4 080	94.5	6 460	116 000	212 000
x642	6.29	81 800	14 500	27 700	421	32 100	703	3 410	92.7	5 380	73 800	172 000
x591	5.79	75 300	13 300	25 600	421	29 500	640	3 130	92.2	4 920	59 000	155 000
x554	5.43	70 600	12 300	23 900	418	27 500	591	2 900	91.5	4 550	48 300	142 000
x539	5.29	68 700	12 000	23 400	418	26 800	576	2 830	91.6	4 440	45 300	138 000
x483	4.74	61 500	10 700	20 900	417	23 900	507	2 510	90.8	3 920	33 100	120 000
x443	4.34	56 400	9 670	19 100	414	21 800	455	2 260	89.8	3 530	25 400	107 000
x412	4.04	52 500	9 100	18 100	416	20 500	434	2 160	90.9	3 350	21 400	102 000
x371	3.64	47 300	8 140	16 300	415	18 400	386	1 930	90.3	2 980	15 900	89 600
x321	3.15	40 800	6 960	14 100	413	15 800	331	1 660	90.0	2 550	10 300	76 100
x296	2.91	37 700	6 200	12 600	405	14 300	290	1 450	87.6	2 240	7 640	66 000
W1000												
x584	5.73	74 400	12 500	23 600	409	28 000	334	2 130	67.0	3 470	71 500	82 200
x494	4.84	62 900	10 300	19 800	404	23 400	268	1 740	65.3	2 820	44 000	64 700
x486	4.77	61 900	10 200	19 700	406	23 200	266	1 730	65.5	2 790	42 900	64 100
x438	4.28	55 600	9 090	17 700	404	20 700	234	1 530	64.8	2 460	31 800	55 700
x415	4.07	52 800	8 530	16 700	402	19 600	217	1 430	64.1	2 300	27 000	51 500
x393	3.85	50 100	8 080	15 900	402	18 500	205	1 350	64.0	2 170	23 300	48 400
x350	3.43	44 600	7 230	14 300	403	16 600	185	1 220	64.4	1 940	17 200	43 200
x314	3.08	40 000	6 440	12 900	401	14 900	162	1 080	63.7	1 710	12 600	37 700
x272	2.67	34 600	5 540	11 200	400	12 800	140	933	63.5	1 470	8 350	32 200
x249	2.44	31 700	4 810	9 820	390	11 300	118	783	60.9	1 240	5 820	26 700
x222	2.18	28 200	4 080	8 410	380	9 800	95.4	636	58.1	1 020	3 900	21 500

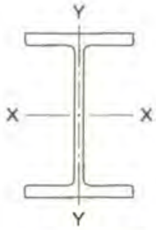


W SHAPES
W1100 - W1000

DIMENSIONS AND SURFACE AREAS

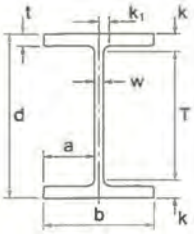
Nominal Mass	Theo-retical Mass	Depth	Flange Width	Flange Thick-ness	Web Thick-ness	Distances					Surface Area (m^2) per metre of length		Imperial Designation
		d	b	t	w	a	T	k	k_1	d - 2t	Total	Minus Top of Top Flange	
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm			
499	498.6	1 118	405	45.0	26.0	190	965	77	43	1 028	3.80	3.40	W44x335
433	432.7	1 108	402	40.0	22.0	190	965	72	41	1 028	3.78	3.38	W44x290
390	390.2	1 100	400	36.0	20.0	190	965	68	40	1 028	3.76	3.36	W44x262
343	342.6	1 090	400	31.0	18.0	191	965	63	39	1 028	3.74	3.34	W44x230
976	974.5	1 108	428	89.9	50.0	189	865	121	55	928	3.83	3.40	W40x655
883	883.4	1 092	424	82.0	45.5	189	865	114	53	928	3.79	3.37	W40x593
748	748.5	1 068	417	70.0	39.0	189	865	102	50	928	3.73	3.31	W40x503
642	641.9	1 048	412	60.0	34.0	189	865	92	47	928	3.68	3.26	W40x431
591	590.9	1 040	409	55.9	31.0	189	865	87	46	928	3.65	3.25	W40x397
554	554.1	1 032	408	52.0	29.5	189	865	84	45	928	3.64	3.23	W40x372
539	539.4	1 030	407	51.1	28.4	189	865	83	44	928	3.63	3.22	W40x362
483	482.9	1 020	404	46.0	25.4	189	865	78	43	928	3.61	3.20	W40x324
443	442.5	1 012	402	41.9	23.6	189	865	73	42	928	3.58	3.18	W40x297
412	412.2	1 008	402	40.0	21.1	190	865	72	41	928	3.58	3.18	W40x277
371	371.2	1 000	400	36.1	19.0	191	865	68	40	928	3.56	3.16	W40x249
321	320.9	990	400	31.0	16.5	192	865	63	38	928	3.55	3.15	W40x215
296	296.4	982	400	27.1	16.5	192	865	59	38	928	3.53	3.13	W40x199
584	583.8	1 056	314	64.0	36.0	139	865	96	48	928	3.30	2.98	W40x392
494	493.9	1 036	309	54.0	31.0	139	865	86	46	928	3.25	2.94	W40x331
486	486.2	1 036	308	54.1	30.0	139	865	86	45	928	3.24	2.94	W40x327
438	436.7	1 026	305	49.0	26.9	139	865	81	43	928	3.22	2.91	W40x294
415	415.0	1 020	304	46.0	26.0	139	865	78	43	928	3.20	2.90	W40x278
393	392.7	1 016	303	43.9	24.4	139	865	75	42	928	3.20	2.89	W40x264
350	349.4	1 008	302	40.0	21.1	140	865	72	41	928	3.18	2.88	W40x235
314	314.3	1 000	300	35.9	19.1	140	865	67	40	928	3.16	2.86	W40x211
272	272.3	990	300	31.0	16.5	142	865	63	38	928	3.15	2.85	W40x183
249	248.7	980	300	26.0	16.5	142	865	58	38	928	3.13	2.83	W40x167
222	222.0	970	300	21.1	16.0	142	865	53	38	928	3.11	2.81	W40x149

W SHAPES
W920 - W840



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	10^9 mm^6
W920												
x1377	13.5	175 400	30 300	55 500	416	67 600	2 060	8 720	108	14 200	596 000	493 000
x1269	12.4	161 700	29 000	53 000	423	63 900	1 900	8 240	108	13 100	514 000	454 000
x1194	11.7	152 200	26 900	49 800	421	59 800	1 750	7 660	107	12 200	435 000	413 000
x1077	10.6	137 200	23 800	44 800	416	53 400	1 530	6 770	106	10 700	326 000	353 000
x970	9.52	123 700	21 000	40 300	412	47 700	1 340	6 000	104	9 490	243 000	304 000
x787	7.71	100 400	16 500	32 600	405	38 000	1 030	4 730	102	7 420	134 000	227 000
x725	7.10	92 400	14 900	29 900	402	34 700	932	4 290	100	6 730	106 000	202 000
x656	6.43	83 700	13 400	27 100	400	31 300	830	3 850	99.7	6 020	79 500	178 000
x588	5.76	75 000	11 800	24 200	397	27 800	728	3 410	98.6	5 310	58 100	154 000
x537	5.25	68 500	10 700	22 100	395	25 300	656	3 080	98.0	4 790	44 500	137 000
x491	4.80	62 600	9 660	20 200	394	23 000	590	2 800	97.3	4 340	34 400	122 000
x449	4.40	57 600	8 750	18 500	391	20 900	540	2 550	97.2	3 950	26 300	111 000
x420	4.11	53 500	8 130	17 300	390	19 500	501	2 370	96.8	3 670	21 500	102 000
x390	3.81	49 700	7 420	15 800	387	17 900	453	2 160	95.7	3 330	16 900	91 500
x368	3.58	46 800	6 920	14 900	386	16 800	421	2 010	95.1	3 100	14 100	84 700
x344	3.37	43 900	6 450	13 900	384	15 700	390	1 870	94.5	2 880	11 600	78 100
W920												
x381	3.74	48 600	6 960	14 600	379	17 000	219	1 410	67.2	2 240	21 800	45 100
x345	3.38	44 000	6 250	13 300	377	15 300	195	1 270	66.7	2 000	16 400	39 800
x313	3.06	39 900	5 480	11 800	371	13 600	170	1 100	65.4	1 750	11 500	34 300
x289	2.83	36 800	5 040	10 900	370	12 500	156	1 020	65.3	1 600	9 160	31 300
x271	2.66	34 600	4 710	10 200	369	11 800	145	946	64.8	1 490	7 630	28 900
x253	2.48	32 300	4 370	9 510	368	10 900	134	874	64.3	1 370	6 210	26 500
x238	2.33	30 300	4 060	8 870	366	10 200	123	806	63.7	1 270	5 100	24 300
x223	2.20	28 500	3 760	8 260	363	9 520	112	738	62.7	1 160	4 180	22 100
x201	1.97	25 600	3 250	7 190	356	8 340	94.4	621	60.7	982	2 880	18 400
W840												
x576	5.65	73 500	10 100	22 200	371	25 600	672	3 270	95.7	5 100	61 700	123 000
x527	5.18	67 200	9 150	20 300	369	23 300	607	2 970	95.0	4 620	47 800	110 000
x473	4.65	60 300	8 130	18 200	367	20 800	537	2 640	94.3	4 100	35 100	95 800
x433	4.25	55 200	7 360	16 600	365	18 900	484	2 390	93.5	3 710	27 000	85 500
x392	3.85	49 900	6 600	15 000	363	17 000	430	2 140	92.7	3 310	20 300	75 300
x359	3.53	45 700	5 920	13 600	359	15 400	389	1 930	92.1	2 980	15 100	67 400
x329	3.24	41 900	5 360	12 400	357	14 000	349	1 740	91.1	2 690	11 600	60 000
x299	2.94	38 100	4 800	11 200	355	12 700	312	1 560	90.4	2 410	8 660	53 200
W840												
x251	2.46	31 900	3 860	9 000	348	10 300	129	884	63.6	1 380	7 350	22 100
x226	2.22	28 800	3 400	7 990	343	9 160	114	774	62.8	1 210	5 140	19 300
x210	2.07	26 800	3 110	7 340	340	8 430	103	700	61.8	1 100	4 050	17 300
x193	1.90	24 700	2 780	6 630	336	7 620	90.3	618	60.5	971	3 050	15 100
x176	1.73	22 400	2 460	5 900	331	6 810	78.2	536	59.1	844	2 220	13 000



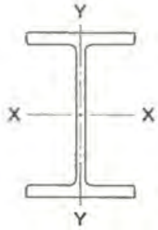
W SHAPES
W920 - W840

DIMENSIONS AND SURFACE AREAS

Nominal Mass	Theo-retical Mass	Depth d	Flange Width b	Flange Thick-ness t	Web Thick-ness w	Distances a	Distances T	Distances k	Distances k_1	Distances d - 2t	Surface Area (m^2) per metre of length: Total	Surface Area (m^2) per metre of length: Minus Top of Top Flange	Imperial Designation
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm			
1 377	1 376.7	1 093	473	115.1	76.7	198	800	147	68	863	3.92	3.45	W36x925
1 269	1 269.0	1 093	461	115.1	64.0	199	800	147	62	863	3.90	3.44	W36x853
1 194	1 194.4	1 081	457	109.0	60.5	198	800	141	60	863	3.87	3.41	W36x802
1 077	1 076.6	1 061	451	99.1	55.0	198	800	131	58	863	3.82	3.37	W36x723
970	970.7	1 043	446	89.9	50.0	198	800	121	55	863	3.77	3.32	W36x652
787	786.6	1 011	437	73.9	40.9	198	800	105	50	863	3.69	3.25	W36x529
725	724.5	999	434	68.1	38.1	198	800	100	49	863	3.66	3.22	W36x487
656	655.7	987	431	62.0	34.5	198	800	94	47	863	3.63	3.20	W36x441
588	587.2	975	427	55.9	31.0	198	800	87	46	863	3.60	3.17	W36x395
537	535.8	965	425	51.1	28.4	198	800	83	44	863	3.57	3.15	W36x361
491	489.3	957	422	47.0	25.9	198	800	79	43	863	3.55	3.13	W36x330
449	448.5	948	423	42.7	24.0	200	800	74	42	863	3.54	3.12	W36x302
420	419.2	943	422	39.9	22.5	200	800	71	41	863	3.53	3.11	W36x282
390	388.0	936	420	36.6	21.3	199	800	68	41	863	3.51	3.09	W36x262
368	365.5	931	419	34.3	20.3	199	799	66	40	862	3.50	3.08	W36x247
344	343.2	927	418	32.0	19.3	199	800	64	40	863	3.49	3.07	W36x231
381	381.1	951	310	43.9	24.4	143	800	75	42	863	3.09	2.78	W36x256
345	344.8	943	308	39.9	22.1	143	800	71	41	863	3.07	2.77	W36x232
313	312.4	932	309	34.5	21.1	144	800	66	41	863	3.06	2.75	W36x210
289	288.3	927	308	32.0	19.4	144	800	64	40	863	3.05	2.74	W36x194
271	271.4	923	307	30.0	18.4	144	800	62	39	863	3.04	2.73	W36x182
253	253.4	919	306	27.9	17.3	144	800	59	39	863	3.03	2.72	W36x170
238	238.0	915	305	25.9	16.5	144	800	57	38	863	3.02	2.71	W36x160
223	223.9	911	304	23.9	15.9	144	800	55	38	863	3.01	2.70	W36x150
201	201.0	903	304	20.1	15.2	144	800	52	38	863	2.99	2.69	W36x135
576	576.6	913	411	57.9	32.0	190	734	89	46	797	3.41	3.00	W33x387
527	528.2	903	409	53.1	29.5	190	734	85	45	797	3.38	2.97	W33x354
473	473.8	893	406	48.0	26.4	190	734	80	43	797	3.36	2.95	W33x318
433	433.8	885	404	43.9	24.4	190	734	75	42	797	3.34	2.93	W33x291
392	392.2	877	401	39.9	22.1	189	734	71	41	797	3.31	2.91	W33x263
359	359.9	868	403	35.6	21.1	191	734	67	41	797	3.31	2.90	W33x241
329	330.0	862	401	32.4	19.7	191	734	64	40	797	3.29	2.89	W33x221
299	299.9	855	400	29.2	18.2	191	734	61	39	797	3.27	2.87	W33x201
251	250.6	859	292	31.0	17.0	138	734	63	39	797	2.85	2.56	W33x169
226	226.6	851	294	26.8	16.1	139	734	58	38	797	2.85	2.55	W33x152
210	210.8	846	293	24.4	15.4	139	734	56	38	797	2.83	2.54	W33x141
193	193.5	840	292	21.7	14.7	139	734	53	37	797	2.82	2.53	W33x130
176	176.0	835	292	18.8	14.0	139	734	50	37	797	2.81	2.52	W33x118

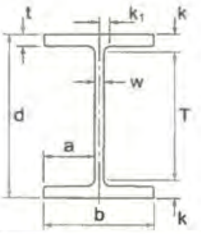
Sections highlighted in yellow are commonly used sizes and are generally readily available.

W SHAPES
W760 - W690



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm^2	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^3\ mm^4$	$10^9\ mm^6$
W760												
x582	5.72	74 200	8 620	20 400	341	23 800	644	3 250	93.2	5 080	72 200	98 300
x531	5.21	67 600	7 770	18 600	339	21 600	578	2 940	92.4	4 580	55 600	87 000
x484	4.76	61 700	6 990	17 000	336	19 500	517	2 650	91.4	4 120	42 800	76 800
x434	4.26	55 300	6 190	15 200	334	17 400	455	2 350	90.7	3 650	31 300	66 800
x389	3.82	49 500	5 450	13 600	332	15 500	399	2 070	89.8	3 210	22 500	57 800
x350	3.43	44 500	4 870	12 200	330	13 900	355	1 860	89.1	2 870	16 800	50 800
x314	3.09	40 000	4 290	10 900	327	12 300	316	1 640	88.7	2 540	11 800	44 700
x284	2.79	36 200	3 830	9 820	325	11 100	280	1 470	87.9	2 260	8 750	39 300
x257	2.53	32 800	3 430	8 880	323	9 970	250	1 310	87.2	2 020	6 510	34 800
W760												
x220	2.16	28 100	2 780	7 140	315	8 190	94.4	710	58.0	1 110	6 050	13 200
x196	1.93	25 100	2 400	6 240	309	7 170	81.7	610	57.1	959	4 040	11 300
x185	1.81	23 500	2 230	5 820	308	6 690	75.1	563	56.5	884	3 330	10 300
x173	1.70	22 100	2 060	5 400	305	6 210	68.7	515	55.7	810	2 690	9 420
x161	1.57	20 500	1 860	4 900	302	5 660	60.7	457	54.5	720	2 070	8 280
x147	1.44	18 800	1 660	4 410	298	5 100	52.9	399	53.1	631	1 560	7 160
x134	1.31	17 000	1 500	4 010	297	4 630	47.7	361	53.0	568	1 180	6 430
W690												
x802	7.86	102 200	10 600	25 700	322	30 900	875	4 520	92.6	7 140	203 000	119 000
x548	5.38	69 800	6 730	17 400	310	20 400	543	2 920	88.1	4 570	70 700	68 200
x500	4.91	63 700	6 060	15 900	308	18 500	487	2 640	87.4	4 110	54 600	60 300
x457	4.49	58 200	5 470	14 500	306	16 800	439	2 390	86.7	3 720	42 300	53 600
x419	4.11	53 300	4 950	13 300	305	15 300	395	2 170	86.0	3 370	33 000	47 700
x384	3.77	48 900	4 490	12 200	303	14 000	357	1 970	85.3	3 050	25 700	42 600
x350	3.44	44 600	4 030	11 100	300	12 600	319	1 770	84.4	2 740	19 500	37 600
x323	3.18	41 100	3 710	10 300	300	11 700	294	1 640	84.4	2 530	15 700	34 400
x289	2.83	36 800	3 260	9 140	298	10 300	256	1 440	83.4	2 220	11 200	29 600
x265	2.61	33 700	2 920	8 270	294	9 330	231	1 290	82.7	1 990	8 340	26 400
x240	2.36	30 600	2 630	7 490	292	8 430	206	1 160	82.0	1 790	6 270	23 400
x217	2.15	27 700	2 360	6 790	291	7 610	185	1 040	81.5	1 610	4 720	20 800
W690												
x192	1.88	24 400	1 980	5 640	285	6 460	76.4	602	56.0	941	4 610	8 680
x170	1.67	21 600	1 700	4 900	280	5 620	66.2	517	55.3	809	3 040	7 410
x152	1.49	19 400	1 510	4 380	279	5 000	57.8	455	54.6	710	2 200	6 420
x140	1.37	17 900	1 360	3 980	276	4 550	51.7	407	53.9	636	1 670	5 720
x125	1.23	16 000	1 180	3 500	272	4 010	44.1	349	52.5	546	1 170	4 830



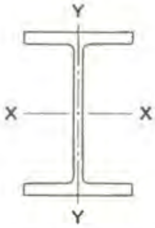
W SHAPES
W760 - W690

DIMENSIONS AND SURFACE AREAS

Nominal Mass	Theo-retical Mass	Depth	Flange Width	Flange Thick-ness	Web Thick-ness	Distances					Surface Area (m^2) per metre of length		Imperial Designation
		d	b	t	w	a	T	k	k_1	d - 2t	Total	Minus Top of Top Flange	
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm			
582	582.9	843	396	62.0	34.5	181	656	94	47	719	3.20	2.81	W30x391
531	531.6	833	393	56.9	31.5	181	656	88	46	719	3.18	2.78	W30x357
484	485.3	823	390	52.1	29.0	181	656	84	45	719	3.15	2.76	W30x326
434	434.4	813	387	47.0	25.9	181	656	79	43	719	3.12	2.74	W30x292
389	389.2	803	385	41.9	23.6	181	656	73	42	719	3.10	2.71	W30x261
350	350.3	795	382	38.1	21.1	180	656	70	41	719	3.08	2.69	W30x235
314	315.3	786	384	33.4	19.7	182	656	65	40	719	3.07	2.68	W30x211
284	284.8	779	382	30.1	18.0	182	656	62	39	719	3.05	2.67	W30x191
257	258.5	773	381	27.1	16.6	182	656	59	38	719	3.04	2.66	W30x173
220	220.2	779	266	30.0	16.5	125	656	62	38	719	2.59	2.32	W30x148
196	196.8	770	268	25.4	15.6	126	656	57	38	719	2.58	2.31	W30x132
185	184.8	766	267	23.6	14.9	126	656	55	37	719	2.57	2.30	W30x124
173	173.6	762	267	21.6	14.4	126	656	53	37	719	2.56	2.30	W30x116
161	160.4	758	266	19.3	13.8	126	656	51	37	719	2.55	2.29	W30x108
147	147.1	753	265	17.0	13.2	126	656	49	37	719	2.54	2.27	W30x99
134	133.2	750	264	15.5	11.9	126	656	47	36	719	2.53	2.27	W30x90
802	801.4	826	387	89.9	50.0	169	583	121	55	646	3.10	2.71	W27x539
548	548.6	772	372	63.0	35.1	168	583	95	48	646	2.96	2.59	W27x368
500	500.5	762	369	57.9	32.0	169	583	89	46	646	2.94	2.57	W27x336
457	458.2	752	367	53.1	29.5	169	583	85	45	646	2.91	2.55	W27x307
419	419.1	744	364	49.0	26.9	169	583	81	43	646	2.89	2.53	W27x281
384	384.7	736	362	45.0	24.9	169	583	77	42	646	2.87	2.51	W27x258
350	351.0	728	360	40.9	23.1	168	583	72	42	646	2.85	2.49	W27x235
323	324.4	722	359	38.1	21.1	169	583	70	41	646	2.84	2.48	W27x217
289	289.1	714	356	34.0	19.0	169	583	66	40	646	2.81	2.46	W27x194
265	265.7	706	358	30.2	18.4	170	583	62	39	646	2.81	2.45	W27x178
240	241.1	701	356	27.4	16.8	170	583	59	38	646	2.79	2.44	W27x161
217	218.9	695	355	24.8	15.4	170	582	56	38	645	2.78	2.42	W27x146
192	191.4	702	254	27.9	15.5	119	583	59	38	646	2.39	2.14	W27x129
170	169.9	693	256	23.6	14.5	121	583	55	37	646	2.38	2.13	W27x114
152	152.1	688	254	21.1	13.1	120	583	53	37	646	2.37	2.11	W27x102
140	139.8	684	254	18.9	12.4	121	583	50	36	646	2.36	2.11	W27x94
125	125.5	678	253	16.3	11.7	121	582	48	36	645	2.34	2.09	W27x84

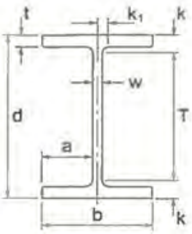
Sections highlighted in yellow are commonly used sizes and are generally readily available.

W SHAPES
W610 - W530



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^3$	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^3$	$10^3 mm^4$	$10^9 mm^6$
W610												
x551	5.40	70 200	5 570	15 700	282	18 600	484	2 790	83.0	4 380	83 800	49 900
x498	4.89	63 500	4 950	14 200	279	16 700	426	2 480	81.9	3 890	63 200	43 100
x455	4.45	57 900	4 440	12 900	277	15 100	381	2 240	81.1	3 500	48 800	37 900
x415	4.07	52 900	4 000	11 800	275	13 700	343	2 030	80.5	3 160	37 700	33 600
x372	3.65	47 400	3 530	10 600	273	12 200	302	1 800	79.8	2 800	27 700	29 100
x341	3.34	43 400	3 180	9 630	271	11 100	271	1 630	79.0	2 520	21 300	25 800
x307	3.01	39 100	2 840	8 690	269	9 930	240	1 450	78.2	2 240	15 900	22 500
x285	2.80	36 100	2 610	8 060	268	9 170	221	1 340	77.9	2 070	12 800	20 500
x262	2.56	33 300	2 360	7 360	266	8 350	198	1 210	77.2	1 870	9 900	18 300
x241	2.37	30 800	2 150	6 780	264	7 670	184	1 120	77.4	1 730	7 700	16 800
x217	2.14	27 700	1 910	6 070	262	6 850	163	995	76.7	1 530	5 600	14 700
x195	1.92	24 800	1 680	5 400	260	6 070	142	871	75.6	1 340	3 970	12 700
x174	1.71	22 200	1 470	4 780	257	5 360	124	761	74.7	1 170	2 800	10 900
x155	1.52	19 700	1 290	4 220	256	4 730	108	666	73.9	1 020	1 950	9 450
W610												
x153	1.51	19 600	1 250	4 020	253	4 600	50.0	437	50.5	682	2 950	4 470
x140	1.37	17 900	1 120	3 630	250	4 150	45.1	392	50.3	613	2 180	3 990
x125	1.23	15 900	985	3 220	249	3 670	39.3	343	49.7	535	1 540	3 450
x113	1.11	14 500	875	2 880	246	3 290	34.3	300	48.7	469	1 120	2 990
x101	0.997	13 000	764	2 530	243	2 900	29.5	259	47.7	404	781	2 550
W610												
x92	0.905	11 700	646	2 140	234	2 510	14.4	161	35.0	258	710	1 250
x82	0.803	10 500	560	1 870	232	2 200	12.1	136	34.0	218	488	1 040
W530												
x409	4.01	52 200	3 170	10 300	247	12 100	325	1 990	79.1	3 100	41 300	25 300
x369	3.61	47 000	2 810	9 310	245	10 800	287	1 770	78.3	2 750	30 800	21 900
x332	3.25	42 300	2 480	8 350	242	9 660	254	1 580	77.6	2 440	22 600	19 000
x300	2.94	38 200	2 210	7 550	241	8 670	225	1 410	76.7	2 180	17 000	16 600
x272	2.66	34 600	1 970	6 820	239	7 790	200	1 260	76.1	1 950	12 800	14 600
x248	2.42	31 500	1 770	6 220	238	7 060	180	1 140	75.7	1 760	9 770	13 000
x219	2.15	27 900	1 510	5 390	233	6 110	157	986	75.0	1 520	6 420	11 000
x196	1.93	25 000	1 340	4 840	231	5 460	139	877	74.4	1 350	4 700	9 640
x182	1.78	23 200	1 240	4 480	231	5 040	127	808	74.2	1 240	3 740	8 820
x165	1.62	21 100	1 110	4 060	230	4 550	114	726	73.4	1 110	2 830	7 790
x150	1.48	19 200	1 010	3 710	229	4 150	103	659	73.2	1 010	2 160	7 030
W530												
x138	1.36	17 600	861	3 140	221	3 610	38.7	362	46.9	569	2 500	2 670
x123	1.21	15 700	761	2 800	220	3 210	33.8	319	46.4	499	1 800	2 310
x109	1.07	13 900	667	2 480	219	2 830	29.5	280	46.1	437	1 260	2 000
x101	0.995	12 900	617	2 300	219	2 620	26.9	256	45.6	400	1 020	1 820
x92	0.907	11 800	552	2 070	217	2 360	23.8	228	44.9	355	762	1 590
x82	0.805	10 500	477	1 810	213	2 060	20.3	194	44.0	303	518	1 340
x72	0.706	9 180	401	1 530	209	1 760	16.2	156	42.0	245	338	1 060



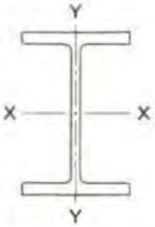
W SHAPES
W610 - W530

DIMENSIONS AND SURFACE AREAS

Nominal Mass (kg/m)	Theo-retical Mass (kg/m)	Depth d (mm)	Flange Width b (mm)	Flange Thick-ness t (mm)	Web Thick-ness w (mm)	Distances a (mm)	T (mm)	k (mm)	k_1 (mm)	d - 2t (mm)	Surface Area (m^2) per metre of length: Total	Minus Top of Top Flange	Imperial Designation
551	551.1	711	347	69.1	38.6	154	510	101	49	573	2.73	2.39	W24x370
498	498.2	699	343	63.0	35.1	154	510	95	48	573	2.70	2.36	W24x335
455	454.1	689	340	57.9	32.0	154	510	89	46	573	2.67	2.33	W24x306
415	415.5	679	338	53.1	29.5	154	510	85	45	573	2.65	2.31	W24x279
372	372.3	669	335	48.0	26.4	154	510	80	43	573	2.63	2.29	W24x250
341	340.4	661	333	43.9	24.4	154	510	75	42	573	2.61	2.27	W24x229
307	307.3	653	330	39.9	22.1	154	510	71	41	573	2.58	2.25	W24x207
285	285.3	647	329	37.1	20.6	154	510	69	40	573	2.57	2.24	W24x192
262	261.1	641	327	34.0	19.0	154	510	66	40	573	2.55	2.23	W24x176
241	241.7	635	329	31.0	17.9	156	510	63	39	573	2.55	2.22	W24x162
217	217.9	628	328	27.7	16.5	156	510	59	38	573	2.54	2.21	W24x146
195	195.6	622	327	24.4	15.4	156	510	56	38	573	2.52	2.19	W24x131
174	174.3	616	325	21.6	14.0	156	510	53	37	573	2.50	2.18	W24x117
155	154.9	611	324	19.0	12.7	156	510	51	36	573	2.49	2.17	W24x104
153	153.6	623	229	24.9	14.0	108	510	56	37	573	2.13	1.91	W24x103
140	140.1	617	230	22.2	13.1	108	510	54	37	573	2.13	1.90	W24x94
125	125.1	612	229	19.6	11.9	109	510	51	36	573	2.12	1.89	W24x84
113	113.4	608	228	17.3	11.2	108	510	49	36	573	2.11	1.88	W24x76
101	101.7	603	228	14.9	10.5	109	510	46	35	573	2.10	1.87	W24x68
92	92.3	603	179	15.0	10.9	84	528	38	26	573	1.90	1.72	W24x62
82	81.9	599	178	12.8	10.0	84	528	35	26	573	1.89	1.71	W24x55
409	408.6	613	327	55.6	31.0	148	439	87	46	502	2.47	2.15	W21x275
369	367.9	603	324	50.5	27.9	148	439	82	44	502	2.45	2.12	W21x248
332	331.2	593	322	45.5	25.4	148	439	77	43	502	2.42	2.10	W21x223
300	299.5	585	319	41.4	23.1	148	439	73	42	502	2.40	2.08	W21x201
272	271.3	577	317	37.6	21.1	148	439	69	41	502	2.38	2.06	W21x182
248	246.6	571	315	34.5	19.0	148	439	66	40	502	2.36	2.05	W21x166
219	218.9	560	318	29.2	18.3	150	439	61	39	502	2.36	2.04	W21x147
196	196.5	554	316	26.3	16.5	150	438	58	38	501	2.34	2.02	W21x132
182	181.7	551	315	24.4	15.2	150	439	56	38	502	2.33	2.02	W21x122
165	165.3	546	313	22.2	14.0	150	439	54	37	502	2.32	2.00	W21x111
150	150.6	543	312	20.3	12.7	150	439	52	36	502	2.31	2.00	W21x101
138	138.3	549	214	23.6	14.7	100	461	44	26	502	1.92	1.71	W21x93
123	123.2	544	212	21.2	13.1	99	461	42	26	502	1.91	1.70	W21x83
109	109.0	539	211	18.8	11.6	100	460	39	25	501	1.90	1.69	W21x73
101	101.4	537	210	17.4	10.9	100	461	38	24	502	1.89	1.68	W21x68
92	92.5	533	209	15.6	10.2	99	461	36	24	502	1.88	1.67	W21x62
82	82.1	528	209	13.3	9.5	100	460	34	24	501	1.87	1.66	W21x55
72	72.0	524	207	10.9	9.0	99	461	31	24	502	1.86	1.65	W21x48

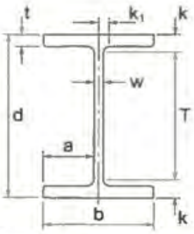
Sections highlighted in yellow are commonly used sizes and are generally readily available.

W SHAPES
W530 - W410



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^3$	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^3$	$10^3 mm^4$	$10^9 mm^6$
W530												
x85	0.830	10 800	485	1 810	212	2 100	12.6	152	34.2	242	737	849
x74	0.733	9 480	411	1 550	208	1 810	10.4	125	33.1	200	480	692
x66	0.644	8 390	351	1 340	205	1 560	8.57	104	32.0	166	320	565
W460												
x464	4.55	59 100	2 900	10 200	222	12 400	331	2 170	74.9	3 400	73 100	20 500
x421	4.14	53 700	2 570	9 250	219	11 100	293	1 940	73.9	3 030	55 700	17 700
x384	3.77	49 000	2 290	8 420	217	10 000	261	1 750	73.1	2 730	42 700	15 500
x349	3.42	44 400	2 040	7 640	214	9 010	233	1 570	72.3	2 440	32 800	13 500
x315	3.08	40 100	1 800	6 850	212	8 020	204	1 390	71.4	2 160	24 300	11 600
x286	2.80	36 400	1 610	6 230	210	7 240	183	1 260	70.9	1 940	18 600	10 200
x260	2.55	33 100	1 440	5 650	208	6 530	163	1 130	70.1	1 740	14 100	8 950
x235	2.30	29 900	1 270	5 080	206	5 840	145	1 010	69.5	1 550	10 500	7 790
x213	2.09	27 100	1 140	4 620	205	5 270	129	909	69.1	1 400	7 970	6 890
x193	1.90	24 700	1 020	4 190	204	4 750	115	816	68.5	1 250	6 030	6 060
x177	1.74	22 600	910	3 780	201	4 280	105	735	68.2	1 130	4 400	5 440
x158	1.55	20 100	796	3 350	199	3 770	91.4	643	67.4	989	3 110	4 670
x144	1.42	18 400	726	3 080	199	3 450	83.6	591	67.4	906	2 440	4 230
x128	1.26	16 300	637	2 730	197	3 050	73.3	520	67.0	796	1 710	3 670
x113	1.11	14 400	556	2 400	196	2 670	63.3	452	66.3	691	1 180	3 150
W460												
x106	1.04	13 400	488	2 080	190	2 390	25.1	259	43.2	405	1 460	1 260
x97	0.947	12 300	445	1 910	190	2 180	22.8	237	43.1	368	1 130	1 140
x89	0.875	11 400	409	1 770	190	2 010	20.9	218	42.9	339	905	1 040
x82	0.803	10 500	370	1 610	188	1 830	18.6	195	42.2	303	690	918
x74	0.727	9 480	332	1 460	188	1 650	16.6	175	41.9	271	516	813
W460												
x68	0.672	8 710	297	1 290	184	1 490	9.40	122	32.8	192	508	463
x60	0.584	7 610	255	1 120	183	1 280	7.96	104	32.4	163	334	388
x52	0.510	6 650	212	942	179	1 090	6.34	83.4	30.9	131	209	306
W410												
x149	1.46	19 000	618	2 870	180	3 250	77.7	586	63.9	900	3 210	3 200
x132	1.30	16 900	538	2 530	179	2 850	67.4	512	63.3	785	2 250	2 730
x114	1.12	14 600	461	2 200	178	2 460	57.2	439	62.7	671	1 480	2 300
x100	0.977	12 700	398	1 920	177	2 130	49.5	381	62.5	581	993	1 960
W410												
x85	0.833	10 800	315	1 510	171	1 720	18.0	199	40.8	310	924	717
x74	0.735	9 480	275	1 330	170	1 510	15.6	173	40.4	269	636	614
x67	0.662	8 580	245	1 200	169	1 360	13.8	154	40.1	239	468	540
x60	0.583	7 610	216	1 060	169	1 190	12.0	135	39.9	209	327	468
x54	0.524	6 840	186	923	165	1 050	10.1	114	38.5	177	225	388
W410												
x46	0.453	5 880	156	772	163	884	5.14	73.4	29.5	115	192	197
x39	0.384	4 950	126	634	159	730	4.04	57.6	28.4	90.6	110	154



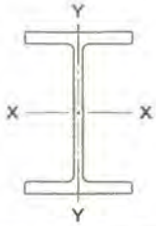
W SHAPES
W530 - W410

DIMENSIONS AND SURFACE AREAS

Nominal Mass	Theoretical Mass	Depth d	Flange Width b	Flange Thickness t	Web Thickness w	Distances a	Distances T	Distances k	Distances k_1	Distances d - 2t	Surface Area (m^2) per metre of length: Total	Surface Area (m^2) per metre of length: Minus Top of Top Flange	Imperial Designation
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm			
85	84.7	535	166	16.5	10.3	78	461	37	24	502	1.71	1.55	W21x57
74	74.7	529	166	13.6	9.7	78	461	34	24	502	1.70	1.54	W21x50
66	65.7	525	165	11.4	8.9	78	461	32	23	502	1.69	1.53	W21x44
464	464.0	567	305	69.6	38.6	133	385	91	39	428	2.28	1.97	W18x311
421	421.8	555	302	63.5	35.6	133	385	85	38	428	2.25	1.94	W18x283
384	384.1	545	299	58.4	32.5	133	385	80	36	428	2.22	1.92	W18x258
349	348.9	535	296	53.6	29.5	133	385	75	35	428	2.20	1.90	W18x234
315	314.2	525	293	48.5	26.9	133	385	70	33	428	2.17	1.88	W18x211
286	285.6	517	291	44.4	24.4	133	385	66	32	428	2.15	1.86	W18x192
260	259.9	509	289	40.4	22.6	133	385	62	32	428	2.13	1.84	W18x175
235	234.8	501	287	36.6	20.6	133	384	58	31	428	2.11	1.82	W18x158
213	212.7	495	285	33.5	18.5	133	384	55	30	428	2.09	1.81	W18x143
193	193.3	489	283	30.5	17.0	133	384	52	29	428	2.08	1.79	W18x130
177	177.3	482	286	26.9	16.6	135	385	49	29	428	2.07	1.79	W18x119
158	157.7	476	284	23.9	15.0	135	385	46	28	428	2.06	1.77	W18x106
144	144.5	472	283	22.1	13.6	135	384	44	27	428	2.05	1.77	W18x97
128	128.4	467	282	19.6	12.2	135	384	41	26	428	2.04	1.76	W18x86
113	113.0	463	280	17.3	10.8	135	385	39	26	428	2.02	1.74	W18x76
106	105.7	469	194	20.6	12.6	91	391	39	23	428	1.69	1.49	W18x71
97	96.5	466	193	19.0	11.4	91	391	38	23	428	1.68	1.49	W18x65
89	89.3	463	192	17.7	10.5	91	391	36	22	428	1.67	1.48	W18x60
82	81.9	460	191	16.0	9.9	91	391	35	22	428	1.66	1.47	W18x55
74	74.2	457	190	14.5	9.0	91	391	33	22	428	1.66	1.47	W18x50
68	68.5	459	154	15.4	9.1	72	391	34	22	428	1.52	1.36	W18x46
60	59.5	455	153	13.3	8.0	73	391	32	21	428	1.51	1.35	W18x40
52	52.0	450	152	10.8	7.6	72	391	29	21	428	1.49	1.34	W18x35
149	149.3	431	265	25.0	14.9	125	337	47	28	381	1.89	1.63	W16x100
132	132.1	425	263	22.2	13.3	125	337	44	27	381	1.88	1.61	W16x89
114	114.5	420	261	19.3	11.6	125	338	41	26	381	1.86	1.60	W16x77
100	99.6	415	260	16.9	10.0	125	338	39	25	381	1.85	1.59	W16x67
85	85.0	417	181	18.2	10.9	85	340	39	24	381	1.54	1.36	W16x57
74	74.9	413	180	16.0	9.7	85	340	37	24	381	1.53	1.35	W16x50
67	67.5	410	179	14.4	8.8	85	340	35	23	381	1.52	1.34	W16x45
60	59.5	407	178	12.8	7.7	85	340	33	23	381	1.51	1.33	W16x40
54	53.4	403	177	10.9	7.5	85	340	31	23	381	1.50	1.32	W16x36
46	46.2	403	140	11.2	7.0	67	344	30	21	381	1.35	1.21	W16x31
39	39.2	399	140	8.8	6.4	67	344	27	20	381	1.35	1.21	W16x26

Sections highlighted in yellow are commonly used sizes and are generally readily available.

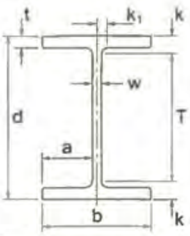
W SHAPES
W360



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^3$	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^3$	$10^3 mm^4$	$10^9 mm^6$
W360												
x1299	12.7	165 000	7 550	25 200	214	33 200	2 540	10 700	124	16 700	944 000	135 000
x1202	11.8	153 000	6 640	22 900	208	30 000	2 290	9 710	122	15 200	762 000	116 000
x1086	10.7	139 000	5 960	20 900	207	27 200	1 960	8 650	119	13 400	605 000	96 700
x990	9.72	126 000	5 190	18 900	203	24 300	1 730	7 740	117	12 000	469 000	82 000
x900	8.85	115 000	4 500	17 000	198	21 600	1 530	6 940	116	10 700	364 000	69 200
x818	8.03	105 000	3 920	15 300	194	19 300	1 360	6 200	114	9 560	278 000	58 900
x744	7.30	94 800	3 420	13 700	190	17 200	1 200	5 550	112	8 550	214 000	50 200
x677	6.65	86 500	2 990	12 400	186	15 300	1 070	4 990	111	7 680	164 000	43 100
x634	6.22	80 600	2 740	11 600	184	14 200	983	4 630	110	7 120	138 000	38 700
x592	5.81	75 500	2 500	10 800	182	13 100	902	4 280	109	6 570	114 000	34 800
x551	5.40	70 300	2 260	9 940	180	12 100	825	3 950	108	6 050	92 500	31 000
x509	5.00	65 200	2 050	9 170	178	11 000	754	3 630	108	5 550	73 900	27 700
x463	4.54	59 000	1 800	8 280	175	9 880	670	3 250	107	4 980	56 500	23 900
x421	4.13	53 700	1 600	7 510	172	8 880	601	2 940	106	4 490	43 400	20 800
x382	3.75	48 800	1 410	6 790	170	7 960	536	2 640	105	4 030	32 800	18 200
x347	3.40	44 200	1 250	6 140	168	7 140	481	2 380	104	3 630	24 800	15 900
x314	3.07	40 000	1 100	5 530	166	6 370	426	2 120	103	3 240	18 500	13 800
x287	2.82	36 600	997	5 070	165	5 810	388	1 940	103	2 960	14 500	12 300
x262	2.58	33 400	894	4 620	163	5 260	350	1 760	102	2 680	11 000	11 000
x237	2.32	30 100	788	4 150	162	4 690	310	1 570	102	2 390	8 180	9 500
x216	2.12	27 500	711	3 790	161	4 260	283	1 430	101	2 180	6 320	8 520
W360												
x196	1.93	25 000	636	3 420	159	3 840	229	1 220	95.6	1 860	5 130	6 830
x179	1.76	22 800	574	3 120	159	3 480	207	1 110	95.2	1 680	3 910	6 120
x162	1.59	20 600	515	2 830	158	3 140	186	1 000	94.9	1 520	2 940	5 430
x147	1.45	18 800	463	2 570	157	2 840	167	904	94.3	1 370	2 230	4 840
x134	1.31	17 100	415	2 330	156	2 560	151	817	94.0	1 240	1 680	4 310
W360												
x122	1.19	15 500	365	2 010	154	2 270	61.5	478	63.0	732	2 110	1 790
x110	1.08	14 100	331	1 840	154	2 060	55.7	435	63.0	664	1 600	1 610
x101	0.992	12 900	301	1 690	153	1 880	50.6	397	62.7	605	1 250	1 450
x91	0.890	11 500	267	1 510	152	1 680	44.8	353	62.3	538	914	1 270
W360												
x79	0.777	10 100	226	1 280	150	1 430	24.2	236	48.9	362	811	687
x72	0.701	9 100	201	1 150	149	1 280	21.4	210	48.5	322	601	600
x64	0.626	8 130	178	1 030	148	1 140	18.8	186	48.1	284	436	524

When subject to tension, bolted connections are preferred for these sections.



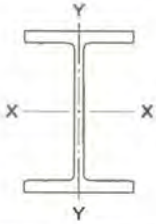
W SHAPES
W360

DIMENSIONS AND SURFACE AREAS

Nominal Mass	Theo-retical Mass	Depth	Flange Width	Flange Thick-ness	Web Thick-ness	Distances					Surface Area (m^2) per metre of length		Imperial Designation
		d	b	t	w	a	T	k	k_1	d - 2t	Total	Minus Top of Top Flange	
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm			
1 299	1 299.0	600	476	140.0	100.0	188	257	172	80	320	2.90	2.43	W14x873
1 202	1 201.5	580	471	130.0	95.0	188	257	162	78	320	2.85	2.38	W14x808
1 086	1 087.8	569	454	125.0	78.0	188	256	157	69	319	2.80	2.34	W14x730
990	991.0	550	448	115.0	71.9	188	257	147	66	320	2.75	2.30	W14x665
900	902.1	531	442	106.0	65.9	188	256	138	63	319	2.70	2.26	W14x605
818	819.0	514	437	97.0	60.5	188	257	129	60	320	2.66	2.22	W14x550
744	744.2	498	432	88.9	55.6	188	257	120	58	320	2.61	2.18	W14x500
677	677.8	483	428	81.5	51.2	188	257	113	56	320	2.58	2.15	W14x455
634	634.3	474	424	77.1	47.6	188	257	109	54	320	2.55	2.12	W14x426
592	592.6	465	421	72.3	45.0	188	257	104	53	320	2.52	2.10	W14x398
551	550.6	455	418	67.6	42.0	188	257	99	51	320	2.50	2.08	W14x370
509	509.4	446	416	62.7	39.1	188	258	94	50	321	2.48	2.06	W14x342
463	462.8	435	412	57.4	35.8	188	257	89	48	320	2.45	2.03	W14x311
421	421.6	425	409	52.6	32.8	188	257	84	46	320	2.42	2.01	W14x283
382	382.3	416	406	48.0	29.8	188	257	80	45	320	2.40	1.99	W14x257
347	346.9	407	404	43.7	27.2	188	257	75	44	320	2.38	1.97	W14x233
314	313.3	399	401	39.6	24.9	188	257	71	42	320	2.35	1.95	W14x211
287	287.5	393	399	36.6	22.6	188	257	68	41	320	2.34	1.94	W14x193
262	262.7	387	398	33.3	21.1	188	257	65	41	320	2.32	1.93	W14x176
237	236.2	380	395	30.2	18.9	188	257	62	39	320	2.30	1.91	W14x159
216	216.3	375	394	27.7	17.3	188	257	59	39	320	2.29	1.90	W14x145
196	196.5	372	374	26.2	16.4	179	257	58	38	320	2.21	1.83	W14x132
179	179.2	368	373	23.9	15.0	179	257	55	38	320	2.20	1.83	W14x120
162	161.9	364	371	21.8	13.3	179	257	53	37	320	2.19	1.81	W14x109
147	147.5	360	370	19.8	12.3	179	257	51	36	320	2.18	1.81	W14x99
134	133.9	356	369	18.0	11.2	179	257	50	36	320	2.17	1.80	W14x90
122	121.7	363	257	21.7	13.0	122	276	44	27	320	1.73	1.47	W14x82
110	110.2	360	256	19.9	11.4	122	277	42	26	320	1.72	1.47	W14x74
101	101.2	357	255	18.3	10.5	122	277	40	26	320	1.71	1.46	W14x68
91	90.8	353	254	16.4	9.5	122	277	38	25	320	1.70	1.45	W14x61
79	79.2	354	205	16.8	9.4	98	277	39	25	320	1.51	1.30	W14x53
72	71.5	350	204	15.1	8.6	98	276	37	25	320	1.50	1.29	W14x48
64	63.9	347	203	13.5	7.7	98	276	35	24	320	1.49	1.29	W14x43

Sections highlighted in yellow are commonly used sizes and are generally readily available.

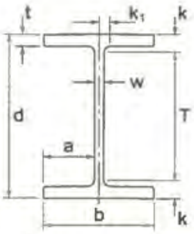
W SHAPES
W360 - W310



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm^2	$10^6\,mm^4$	$10^3\,mm^3$	mm	$10^3\,mm^3$	$10^6\,mm^4$	$10^3\,mm^3$	mm	$10^3\,mm^3$	$10^3\,mm^4$	$10^9\,mm^6$
W360												
x57	0.555	7 230	160	896	149	1 010	11.1	129	39.3	199	333	331
x51	0.496	6 450	141	796	148	893	9.68	113	38.8	174	237	285
x45	0.441	5 710	122	691	146	778	8.18	95.7	37.8	148	159	239
W360												
x39	0.383	4 960	102	580	143	662	3.75	58.6	27.4	91.6	150	110
x33	0.321	4 190	82.6	473	141	541	2.91	45.8	26.4	71.8	85.3	84.3
W310												
x500	4.91	63 700	1 690	7 910	163	9 880	494	2 910	88.0	4 490	101 000	15 300
x454	4.45	57 800	1 480	7 130	160	8 820	436	2 600	86.8	4 000	77 200	13 100
x415	4.07	52 800	1 300	6 450	157	7 900	391	2 340	86.0	3 610	59 500	11 300
x375	3.68	47 800	1 130	5 770	154	7 000	344	2 080	84.8	3 210	44 900	9 570
x342	3.37	43 700	1 010	5 260	152	6 330	310	1 890	84.2	2 910	34 900	8 420
x313	3.07	39 900	896	4 790	150	5 720	277	1 700	83.3	2 620	27 000	7 350
x283	2.77	36 000	787	4 310	148	5 100	246	1 530	82.6	2 340	20 300	6 330
x253	2.48	32 300	682	3 830	146	4 490	215	1 350	81.6	2 060	14 800	5 370
x226	2.22	28 800	596	3 420	144	3 970	189	1 190	81.0	1 830	10 800	4 620
x202	1.99	25 700	520	3 050	142	3 510	166	1 050	80.2	1 610	7 730	3 960
x179	1.75	22 800	445	2 670	140	3 050	144	919	79.5	1 400	5 370	3 340
x158	1.54	20 100	386	2 360	139	2 670	125	805	78.9	1 220	3 770	2 840
x143	1.40	18 200	348	2 150	138	2 420	113	729	78.6	1 110	2 860	2 540
x129	1.27	16 500	308	1 940	137	2 160	100	652	78.0	991	2 130	2 220
x118	1.15	15 000	275	1 750	136	1 950	90.2	588	77.6	893	1 600	1 970
x107	1.05	13 600	248	1 590	135	1 760	81.2	531	77.2	806	1 210	1 760
x97	0.949	12 300	222	1 440	134	1 590	72.9	478	76.9	725	909	1 560
W310												
x86	0.847	11 000	198	1 280	134	1 420	44.5	351	63.6	533	874	961
x79	0.773	10 100	177	1 150	133	1 280	39.9	314	63.0	478	655	847
W310												
x74	0.726	9 480	164	1 060	132	1 180	23.4	229	49.9	350	718	505
x67	0.650	8 520	144	942	131	1 050	20.7	203	49.5	310	522	439
x60	0.580	7 610	128	842	130	933	18.3	180	49.3	275	378	384
W310												
x52	0.513	6 650	118	747	133	837	10.3	123	39.2	189	308	237
x45	0.438	5 670	99.2	634	132	708	8.55	103	38.8	158	191	195
x39	0.380	4 940	85.1	549	131	610	7.27	88.1	38.4	135	126	164
W310												
x33	0.321	4 180	65.0	415	125	480	1.92	37.6	21.4	59.6	122	43.8
x28	0.278	3 590	54.3	351	123	407	1.58	31.0	20.9	49.2	75.7	35.6
x24	0.234	3 040	42.7	280	119	328	1.16	22.9	19.5	36.7	42.5	25.7
x21	0.207	2 680	37.0	244	117	287	0.983	19.5	19.1	31.2	29.4	21.7

When subject to tension, bolted connections are preferred for these sections.



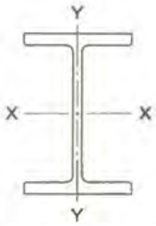
W SHAPES
W360 - W310

DIMENSIONS AND SURFACE AREAS

Nominal Mass	Theo-retical Mass	Depth	Flange Width	Flange Thick-ness	Web Thick-ness	Distances					Surface Area (m^2) per metre of length		Imperial Designation
		d	b	t	w	a	T	k	k_1	d - 2t	Total	Minus Top of Top Flange	
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm			
57	56.6	358	172	13.1	7.9	82	295	31	21	332	1.39	1.22	W14x38
51	50.6	355	171	11.6	7.2	82	295	30	20	332	1.38	1.21	W14x34
45	45.0	352	171	9.8	6.9	82	296	28	20	332	1.37	1.20	W14x30
39	39.1	353	128	10.7	6.5	61	295	29	20	332	1.21	1.08	W14x26
33	32.7	349	127	8.5	5.8	61	296	27	20	332	1.19	1.07	W14x22
500	500.4	427	340	75.1	45.1	147	233	97	43	277	2.12	1.78	W12x336
454	454.0	415	336	68.7	41.3	147	234	91	41	278	2.09	1.76	W12x305
415	415.1	403	334	62.7	38.9	148	234	85	40	278	2.06	1.73	W12x279
375	374.8	391	330	57.2	35.4	147	233	79	38	277	2.03	1.70	W12x252
342	343.2	382	328	52.6	32.6	148	233	74	37	277	2.01	1.68	W12x230
313	313.3	374	325	48.3	30.0	148	234	70	35	277	1.99	1.66	W12x210
283	282.9	365	322	44.1	26.9	148	233	66	34	277	1.96	1.64	W12x190
253	252.9	356	319	39.6	24.4	147	233	61	33	277	1.94	1.62	W12x170
226	226.7	348	317	35.6	22.1	147	233	57	31	277	1.92	1.60	W12x152
202	202.6	341	315	31.8	20.1	147	234	54	30	277	1.90	1.59	W12x136
179	178.7	333	313	28.1	18.0	148	233	50	29	277	1.88	1.57	W12x120
158	157.4	327	310	25.1	15.5	147	233	47	28	277	1.86	1.55	W12x106
143	143.1	323	309	22.9	14.0	148	234	45	27	277	1.85	1.55	W12x96
129	129.6	318	308	20.6	13.1	147	233	42	27	277	1.84	1.53	W12x87
118	117.5	314	307	18.7	11.9	148	233	41	26	277	1.83	1.53	W12x79
107	106.9	311	306	17.0	10.9	148	233	39	26	277	1.82	1.52	W12x72
97	96.8	308	305	15.4	9.9	148	234	37	25	277	1.82	1.51	W12x65
86	86.3	310	254	16.3	9.1	122	234	38	25	277	1.62	1.36	W12x58
79	78.9	306	254	14.6	8.8	123	234	36	24	277	1.61	1.36	W12x53
74	74.0	310	205	16.3	9.4	98	234	38	25	277	1.42	1.22	W12x50
67	66.3	306	204	14.6	8.5	98	234	36	24	277	1.41	1.21	W12x45
60	59.1	303	203	13.1	7.5	98	234	35	24	277	1.40	1.20	W12x40
52	52.3	317	167	13.2	7.6	80	256	31	20	291	1.29	1.12	W12x35
45	44.6	313	166	11.2	6.6	80	256	29	19	291	1.28	1.11	W12x30
39	38.7	310	165	9.7	5.8	80	256	27	19	291	1.27	1.10	W12x26
33	32.8	313	102	10.8	6.6	48	264	24	15	291	1.02	0.919	W12x22
28	28.4	309	102	8.9	6.0	48	264	22	15	291	1.01	0.912	W12x19
24	23.8	305	101	6.7	5.6	48	265	20	15	292	1.00	0.902	W12x16
21	21.1	303	101	5.7	5.1	48	265	19	15	292	1.00	0.899	W12x14

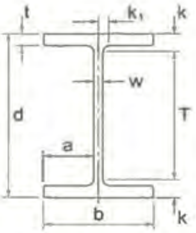
Sections highlighted in yellow are commonly used sizes and are generally readily available.

W SHAPES
W250 - W200



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^3$	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^3$	$10^3 mm^4$	$10^9 mm^6$
W250												
x167	1.64	21 200	300	2 080	119	2 430	98.8	746	68.1	1 140	6 310	1 630
x149	1.46	19 000	259	1 840	117	2 130	86.2	656	67.4	1 000	4 510	1 390
x131	1.29	16 700	221	1 610	115	1 850	74.5	571	66.8	870	3 120	1 160
x115	1.13	14 600	189	1 410	114	1 600	64.1	495	66.2	753	2 130	976
x101	0.992	12 900	164	1 240	113	1 400	55.5	432	65.6	656	1 490	829
x89	0.878	11 400	143	1 100	112	1 230	48.4	378	65.1	574	1 040	713
x80	0.786	10 200	126	982	111	1 090	43.1	338	65.0	513	757	623
x73	0.715	9 290	113	891	110	985	38.8	306	64.6	463	575	553
W250												
x67	0.658	8 580	104	806	110	901	22.2	218	51.0	332	625	324
x58	0.571	7 420	87.3	693	108	770	18.8	186	50.4	283	409	268
x49	0.481	6 260	70.6	572	106	633	15.1	150	49.2	228	241	211
W250												
x45	0.440	5 700	71.1	534	111	602	7.03	95.1	35.1	146	261	113
x39	0.379	4 910	60.1	459	110	513	5.94	80.8	34.7	124	169	93.4
x33	0.321	4 190	48.9	379	108	424	4.73	64.7	33.7	99.5	98.5	73.2
W250												
x28	0.279	3 630	40.0	307	105	353	1.78	34.8	22.1	54.7	96.7	27.7
x25	0.249	3 220	34.2	266	103	307	1.49	29.2	21.5	46.2	65.2	23.0
x22	0.219	2 850	28.9	227	101	263	1.23	24.0	20.7	38.1	43.4	18.7
x18	0.175	2 280	22.4	179	99.3	207	0.913	18.1	20.0	28.6	22.4	13.8
W200												
x100	0.976	12 700	113	989	94.5	1 150	36.6	349	53.8	533	2 090	386
x86	0.850	11 000	94.7	853	92.6	981	31.4	300	53.3	458	1 390	318
x71	0.701	9 100	76.6	709	91.7	803	25.4	246	52.8	375	817	250
x59	0.582	7 550	61.1	582	89.9	653	20.4	199	52.0	303	463	196
x52	0.512	6 650	52.7	512	89.0	569	17.8	175	51.8	266	323	167
x46	0.451	5 890	45.4	448	88.1	495	15.3	151	51.2	229	220	141
W200												
x42	0.409	5 320	40.9	399	87.7	445	9.00	108	41.2	165	222	84.0
x36	0.352	4 570	34.4	342	86.7	379	7.64	92.6	40.9	141	145	69.6
W200												
x31	0.308	3 970	31.4	299	88.6	335	4.10	61.1	32.0	93.8	119	40.9
x27	0.261	3 390	25.8	249	87.3	279	3.30	49.6	31.2	76.1	71.3	32.5
W200												
x22	0.220	2 860	20.0	194	83.6	222	1.42	27.8	22.3	43.7	56.6	13.9
x19	0.191	2 480	16.6	163	81.7	187	1.15	22.6	21.6	35.6	36.2	11.1
x15	0.147	1 910	12.7	127	81.8	145	0.869	17.4	21.4	27.1	17.6	8.24



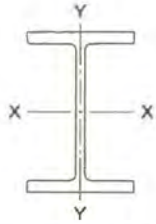
W SHAPES
W250 - W200

DIMENSIONS AND SURFACE AREAS

Nominal Mass	Theo-retical Mass	Depth d	Flange Width b	Flange Thick-ness t	Web Thick-ness w	Distances a	T	k	k_1	d - 2t	Surface Area (m^2) per metre of length Total	Minus Top of Top Flange	Imperial Designation
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm			
167	167.4	289	265	31.8	19.2	123	184	52	29	225	1.60	1.33	W10x112
149	148.9	282	263	28.4	17.3	123	184	49	28	225	1.58	1.32	W10x100
131	131.1	275	261	25.1	15.4	123	184	46	27	225	1.56	1.30	W10x88
115	114.8	269	259	22.1	13.5	123	184	43	26	225	1.55	1.29	W10x77
101	101.2	264	257	19.6	11.9	123	184	40	25	225	1.53	1.28	W10x68
89	89.6	260	256	17.3	10.7	123	184	38	24	225	1.52	1.27	W10x60
80	80.1	256	255	15.6	9.4	123	184	36	24	225	1.51	1.26	W10x54
73	72.9	253	254	14.2	8.6	123	184	35	23	225	1.50	1.25	W10x49
67	67.1	257	204	15.7	8.9	98	185	36	23	226	1.31	1.11	W10x45
58	58.2	252	203	13.5	8.0	98	184	34	23	225	1.30	1.10	W10x39
49	49.0	247	202	11.0	7.4	97	184	32	23	225	1.29	1.09	W10x33
45	44.9	266	148	13.0	7.6	70	209	29	18	240	1.11	0.961	W10x30
39	38.7	262	147	11.2	6.6	70	209	27	17	240	1.10	0.952	W10x26
33	32.7	258	146	9.1	6.1	70	211	24	16	240	1.09	0.942	W10x22
28	28.5	260	102	10.0	6.4	48	213	24	15	240	0.915	0.813	W10x19
25	25.3	257	102	8.4	6.1	48	213	22	15	240	0.910	0.808	W10x17
22	22.4	254	102	6.9	5.8	48	213	20	15	240	0.904	0.802	W10x15
18	17.9	251	101	5.3	4.8	48	213	19	14	240	0.896	0.795	W10x12
100	99.5	229	210	23.7	14.5	98	148	40	22	182	1.27	1.06	W8x67
86	86.7	222	209	20.6	13.0	98	147	37	22	181	1.25	1.04	W8x58
71	71.5	216	206	17.4	10.2	98	148	34	20	181	1.24	1.03	W8x48
59	59.3	210	205	14.2	9.1	98	148	31	20	182	1.22	1.02	W8x40
52	52.2	206	204	12.6	7.9	98	147	29	19	181	1.21	1.01	W8x35
46	46.0	203	203	11.0	7.2	98	148	28	19	181	1.20	1.00	W8x31
42	41.7	205	166	11.8	7.2	79	152	26	17	181	1.06	0.894	W8x28
36	35.9	201	165	10.2	6.2	79	152	25	16	181	1.05	0.885	W8x24
31	31.4	210	134	10.2	6.4	64	166	22	14	190	0.943	0.809	W8x21
27	26.6	207	133	8.4	5.8	64	167	20	13	190	0.934	0.801	W8x18
22	22.4	206	102	8.0	6.2	48	166	20	13	190	0.808	0.706	W8x15
19	19.4	203	102	6.5	5.8	48	165	19	14	190	0.802	0.700	W8x13
15	15.0	200	100	5.2	4.3	48	165	18	13	190	0.791	0.691	W8x10

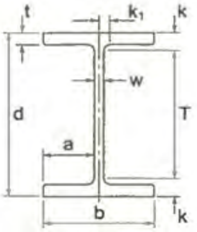
Sections highlighted in yellow are commonly used sizes and are generally readily available.

W SHAPES
W150 - W100



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm^2	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^3\ mm^4$	$10^9\ mm^6$
W150												
x37	0.364	4 740	22.2	274	68.5	310	7.07	91.8	38.7	140	192	40.0
x30	0.292	3 790	17.1	218	67.3	244	5.56	72.6	38.3	111	100	30.3
x22	0.219	2 860	12.0	159	65.1	176	3.87	50.9	36.9	77.5	41.5	20.4
W150												
x24	0.235	3 060	13.4	168	66.3	191	1.83	35.8	24.5	55.2	92.3	10.2
x18	0.176	2 290	9.15	120	63.3	136	1.26	24.7	23.5	38.2	36.9	6.70
x14	0.133	1 730	6.85	91.3	63.0	102	0.918	18.4	23.0	28.3	16.8	4.79
x13	0.124	1 630	6.13	82.8	61.7	93.0	0.818	16.4	22.5	25.3	13.6	4.19
W130												
x28	0.275	3 590	10.9	167	55.3	190	3.81	59.6	32.7	90.7	127	13.8
x24	0.231	3 040	8.79	138	54.1	156	3.11	49.0	32.2	74.5	76.2	10.8
W100												
x19	0.190	2 470	4.76	89.8	43.9	103	1.61	31.2	25.5	47.9	62.9	3.79



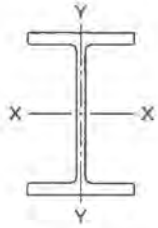
W SHAPES
W150 - W100

DIMENSIONS AND SURFACE AREAS

Nominal Mass	Theoretical Mass	Depth d	Flange Width b	Flange Thickness t	Web Thickness w	Distances a	Distances T	Distances k	Distances k_1	Distances d - 2t	Surface Area (m^2) per metre of length: Total	Surface Area (m^2) per metre of length: Minus Top of Top Flange	Imperial Designation
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm			
37	37.1	162	154	11.6	8.1	73	114	24	15	139	0.924	0.770	W6x25
30	29.8	157	153	9.3	6.6	73	113	22	14	138	0.913	0.760	W6x20
22	22.3	152	152	6.6	5.8	73	114	19	14	139	0.900	0.748	W6x15
24	24.0	160	102	10.3	6.6	48	116	22	14	139	0.715	0.613	W6x16
18	17.9	153	102	7.1	5.8	48	115	19	13	139	0.702	0.600	W6x12
14	13.6	150	100	5.5	4.3	48	115	17	12	139	0.691	0.591	W6x9
13	12.6	148	100	4.9	4.3	48	115	17	12	138	0.687	0.587	W6x8.5
28	28.1	131	128	10.9	6.9	61	86	23	14	109	0.760	0.632	W5x19
24	23.6	127	127	9.1	6.1	60	85	21	13	109	0.750	0.623	W5x16
19	19.4	106	103	8.8	7.1	48	65	21	14	88	0.610	0.507	W4x13

Sections highlighted in yellow are commonly used sizes and are generally readily available.

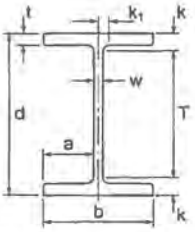
HP SHAPES



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	10^9 mm^6
HP460												
x304	2.98	38 700	1 440	6 200	193	7 060	465	2 020	110	3 130	12 200	22 000
x269	2.64	34 300	1 250	5 490	191	6 210	405	1 770	109	2 730	8 600	18 900
x234	2.30	29 800	1 080	4 780	190	5 370	345	1 520	108	2 340	5 770	15 900
x202	1.98	25 700	919	4 130	189	4 610	292	1 300	107	1 990	3 820	13 300
HP410												
x272	2.67	34 700	1 040	4 950	173	5 680	337	1 630	98.5	2 530	11 100	12 800
x242	2.37	30 800	907	4 390	172	5 000	293	1 430	97.5	2 210	7 850	11 000
x211	2.07	26 900	775	3 820	170	4 320	248	1 220	96.2	1 890	5 330	9 140
x181	1.78	23 000	658	3 290	169	3 690	209	1 040	95.1	1 600	3 490	7 570
x151	1.48	19 200	541	2 750	168	3 060	170	849	93.9	1 310	2 120	6 070
x131	1.28	16 700	462	2 370	167	2 630	144	724	93.1	1 110	1 430	5 080
HP360												
x174	1.70	22 200	508	2 820	152	3 180	184	973	91.1	1 490	3 310	5 330
x152	1.49	19 400	439	2 460	150	2 760	159	845	90.5	1 290	2 240	4 540
x132	1.29	16 800	375	2 140	149	2 380	135	724	89.6	1 110	1 490	3 800
x108	1.06	13 800	303	1 750	148	1 940	108	585	88.6	891	830	3 000
HP310												
x132	1.29	16 900	287	1 830	131	2 070	93.7	599	74.8	922	2 050	2 050
x125	1.22	15 900	270	1 730	130	1 960	88.2	566	74.6	870	1 760	1 910
x110	1.08	14 100	237	1 540	130	1 730	77.1	497	74.0	763	1 240	1 650
x94	0.915	11 900	196	1 300	129	1 450	63.9	415	73.3	635	762	1 340
x79	0.768	10 000	163	1 090	128	1 210	52.6	344	72.6	525	459	1 090
HP250												
x85	0.837	10 800	123	968	106	1 090	42.3	325	62.3	500	829	606
x62	0.614	8 000	87.5	711	105	792	30.0	234	61.3	358	339	415
HP200												
x54	0.525	6 840	49.8	488	85.5	551	16.7	162	49.6	249	319	155

Note: These sections are not available from Canadian mills.

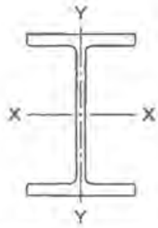


HP SHAPES

DIMENSIONS AND SURFACE AREAS

Nominal Mass (kg/m)	Theo-retical Mass (kg/m)	Depth d (mm)	Flange Width b (mm)	Flange Thick-ness t (mm)	Web Thick-ness w (mm)	Distances a (mm)	Distances T (mm)	Distances k (mm)	Distances k_1 (mm)	Distances d-2t (mm)	Surface Area (m^2) per metre of length: Total	Surface Area (m^2) per metre of length: Minus Top of Top Flange	Imperial Designation
304	303.9	464	460	28.6	28.6	216	344	60	44	407	2.71	2.25	HP18x204
269	269.3	457	457	25.4	25.4	216	343	57	43	406	2.69	2.23	HP18x181
234	234.2	451	454	22.1	22.1	216	344	54	41	407	2.67	2.22	HP18x157
202	202.3	445	451	19.1	19.1	216	344	51	40	407	2.66	2.20	HP18x135
272	272.7	419	413	28.6	28.6	192	299	60	44	362	2.43	2.02	HP16x183
242	241.8	413	410	25.4	25.4	192	299	57	43	362	2.42	2.01	HP16x162
211	210.6	406	406	22.2	22.2	192	299	54	41	362	2.39	1.99	HP16x141
181	181.2	400	403	19.1	19.1	192	299	51	40	362	2.37	1.97	HP16x121
151	151.1	394	400	15.9	15.9	192	299	47	38	362	2.36	1.96	HP16x101
131	130.6	389	398	13.7	13.7	192	299	45	37	362	2.34	1.94	HP16x88
174	173.9	361	378	20.4	20.4	179	257	52	40	320	2.19	1.82	HP14x117
152	152.2	356	376	17.9	17.9	179	257	49	39	320	2.18	1.80	HP14x102
132	132.0	351	373	15.6	15.6	179	257	47	38	320	2.16	1.79	HP14x89
108	108.1	346	370	12.8	12.8	179	257	44	36	320	2.15	1.78	HP14x73
132	131.3	314	313	18.3	18.3	147	234	40	29	277	1.84	1.53	HP12x89
125	124.6	312	312	17.4	17.4	147	234	39	29	277	1.84	1.53	HP12x84
110	110.4	308	310	15.5	15.4	147	234	37	28	277	1.83	1.52	HP12x74
94	93.3	303	308	13.1	13.1	147	234	35	27	277	1.81	1.50	HP12x63
79	78.3	299	306	11.0	11.0	148	234	33	26	277	1.80	1.49	HP12x53
85	85.3	254	260	14.4	14.4	123	184	35	26	225	1.52	1.26	HP10x57
62	62.6	246	256	10.7	10.5	123	184	31	24	225	1.50	1.24	HP10x42
54	53.5	204	207	11.3	11.3	98	148	28	21	181	1.21	1.01	HP8x36

M SHAPES

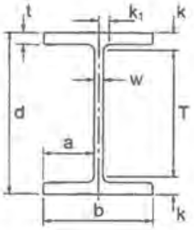


PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	10^9 mm^6
M318												
x18.5	0.179	2 361	37.0	233	126	269	0.830	17.5	18.9	27.3	20.5	20.2
x17.3	0.168	2 213	33.4	211	124	246	0.636	14.3	17.1	22.6	17.2	15.4
M310												
x17.6	0.173	2 240	30.1	197	116	235	0.453	11.6	14.2	18.8	20.8	10.1
x16.1	0.159	2 050	27.7	182	116	216	0.421	10.8	14.3	17.4	16.3	9.39
x14.9	0.147	1 900	25.8	170	116	201	0.440	10.6	15.2	16.9	12.1	9.85
M250												
x13.4	0.132	1 710	16.2	127	97.2	151	0.274	8.05	12.7	13.0	13.1	4.24
x11.9	0.118	1 520	14.4	113	96.9	134	0.242	7.12	12.6	11.4	9.31	3.73
x11.2	0.110	1 430	13.6	108	97.6	126	0.231	6.80	12.7	10.8	7.76	3.57
M200												
x9.7	0.094 3	1 240	7.61	74.9	78.8	87.9	0.149	5.22	11.0	8.36	7.60	1.46
x9.2	0.090 7	1 170	7.30	71.9	78.7	84.4	0.147	5.07	11.2	8.10	6.48	1.45
M150												
x6.6	0.064 2	832	2.99	39.3	59.8	45.7	0.074 7	3.18	9.47	5.05	4.10	0.407
x5.5+	0.054 3	703	2.48	33.0	59.3	38.3	0.073 1	2.87	10.2	4.52	2.21	0.394
M130												
x28.1+	0.276	3 580	10.1	158	53.0	182	3.620	57.1	31.8	87.2	130	12.3
M100												
x8.9	0.087 6	1 150	2.00	41.2	41.9	45.6	0.624	12.9	23.4	19.5	7.63	1.35
x6.1+	0.062 8	775	1.48	29.0	42.6	32.7	0.133	4.66	12.8	7.18	6.12	0.317
M75												
x4.3+	0.045 3	550	0.618	16.3	32.4	18.2	0.102	3.58	13.2	5.45	3.24	0.135

Note: These sections are not available from Canadian mills.

\+ This section had no known producer at time of printing.

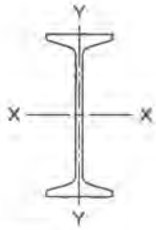


M SHAPES

DIMENSIONS AND SURFACE AREAS

Nominal Mass	Theo-retical Mass	Depth	Flange Width	Flange Thick-ness	Web Thick-ness	Distances				Surface Area (m^2) per metre of length		Imperial Designation
		d	b	t	w	a	T	k	k_1	Total	Minus Top of Top Flange	
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	mm			
18.5	18.3	318	95	5.8	3.9	46	290	14	10	1.01	0.913	M12.5x12.4
17.3	17.2	317	89	5.4	3.9	43	289	14	10	0.982	0.893	M12.5x11.6
17.6	17.6	305	78	5.7	4.5	37	277	14	10	0.913	0.835	M12x11.8
16.1	16.2	304	78	5.3	4.1	37	276	14	10	0.912	0.834	M12x10.8
14.9	15.0	304	83	4.6	3.8	40	278	13	10	0.932	0.849	M12x10.0
13.4	13.4	254	68	5.2	4.0	32	226	14	10	0.772	0.704	M10x9.0
11.9	12.0	253	68	4.6	3.6	32	225	14	10	0.771	0.703	M10x8.0
11.2	11.2	253	68	4.4	3.3	32	227	13	9	0.771	0.703	M10x7.5
9.7	9.6	203	57	4.8	3.4	27	175	14	10	0.627	0.570	M8x6.5
9.2	9.2	203	58	4.5	3.3	27	177	13	9	0.631	0.573	M8x6.2
6.6	6.5	152	47	4.3	2.9	22	132	10	6	0.486	0.439	M6x4.4
5.5	5.5	150	51	3.3	2.5	24	132	9	6	0.499	0.448	M6x3.7
28.1	28.2	127	127	10.6	8.0	60	85	21	13	0.746	0.619	M5x18.9
8.9	8.9	97	97	4.1	3.3	47	71	13	9	0.575	0.478	M4x6.0
6.1	6.4	102	57	4.3	2.9	27	74	14	10	0.426	0.369	M4x4.08
4.3	4.6	76	57	3.3	2.3	27	50	13	10	0.375	0.318	M3x2.9

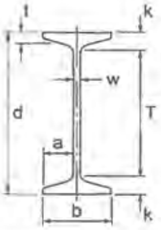
S SHAPES
S610 - S200



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm^2	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^3\ mm^4$	$10^9\ mm^6$
S610												
x180	1.76	23 000	1 310	4 220	239	5 020	33.9	332	38.5	592	5 330	2 990
x158	1.55	20 100	1 220	3 940	247	4 580	31.6	316	39.7	545	4 210	2 790
S610												
x149	1.46	18 900	996	3 270	229	3 930	19.7	214	32.2	393	3 150	1 700
x134	1.32	17 100	939	3 080	234	3 650	18.6	205	32.9	367	2 520	1 600
x119	1.17	15 200	879	2 880	241	3 360	17.5	197	34.0	342	2 030	1 510
S510												
x143	1.40	18 200	700	2 710	196	3 250	20.7	226	33.7	410	3 490	1 260
x128	1.26	16 300	658	2 550	200	3 010	19.2	214	34.2	378	2 770	1 160
S510												
x112	1.09	14 200	532	2 090	194	2 500	12.3	152	29.4	274	1 900	731
x98.2	0.964	12 500	497	1 950	199	2 290	11.5	145	30.3	253	1 480	684
S460												
x104	1.03	13 300	387	1 690	170	2 050	10.1	127	27.5	238	1 740	487
x81.4	0.800	10 400	335	1 470	180	1 710	8.62	113	28.8	199	983	416
S380												
x74	0.731	9 480	203	1 060	146	1 270	6.49	90.8	26.1	164	884	217
x64	0.627	8 130	187	980	151	1 140	6.01	85.9	27.2	149	641	200
S310												
x74	0.729	9 480	127	833	116	1 000	6.48	93.3	26.2	169	1 160	135
x60.7	0.595	7 740	113	743	121	868	5.56	83.7	26.8	145	721	116
S310												
x52	0.512	6 650	95.8	628	120	736	4.10	63.5	24.8	112	447	86.9
x47	0.465	6 030	91.0	597	123	689	3.88	61.2	25.4	105	372	82.3
S250												
x52	0.513	6 650	61.5	484	96.1	583	3.51	55.8	23.0	103	539	51.2
x38	0.370	4 810	51.4	405	103	465	2.80	47.5	24.1	81.3	250	40.9
S200												
x34	0.336	4 370	27.0	266	78.6	316	1.79	33.8	20.2	60.4	229	16.5
x27	0.269	3 480	24.0	236	82.9	271	1.56	30.7	21.2	52.4	138	14.4

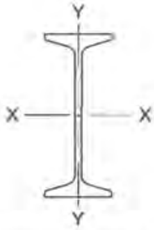
Note: These sections are not available from Canadian mills.



DIMENSIONS AND SURFACE AREAS

Nominal Mass	Theo-retical Mass	Depth	Flange Width	Mean Flange Thickness	Web Thickness	Distances			Surface Area (m²) per metre of length		Imperial Designation
		d	b	t	w	a	T	k	Total	Minus Top of Top Flange	
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm			
180	180.0	622	204	27.7	20.3	92	516	53	2.02	1.82	S24x121
158	157.8	622	200	27.7	15.7	92	516	53	2.01	1.81	S24x106
149	148.7	610	184	22.1	18.9	83	518	46	1.92	1.73	S24x100
134	134.4	610	181	22.1	15.9	83	518	46	1.91	1.73	S24x90
119	119.1	610	178	22.1	12.7	83	518	46	1.91	1.73	S24x80
143	142.9	516	183	23.4	20.3	81	420	48	1.72	1.54	S20x96
128	128.6	516	179	23.4	16.8	81	422	47	1.71	1.54	S20x86
112	111.4	508	162	20.2	16.1	73	420	44	1.63	1.47	S20x75
98.2	98.3	508	159	20.2	12.8	73	420	44	1.63	1.47	S20x66
104	104.7	457	159	17.6	18.1	70	383	37	1.51	1.35	S18x70
81.4	81.5	457	152	17.6	11.7	70	383	37	1.50	1.35	S18x54.7
74	74.6	381	143	15.8	14.0	65	313	34	1.31	1.16	S15x50
64	64.0	381	140	15.8	10.4	65	313	34	1.30	1.16	S15x42.9
74	74.4	305	139	16.7	17.4	61	235	35	1.13	0.992	S12x50
60.7	60.6	305	133	16.7	11.7	61	235	35	1.12	0.986	S12x40.8
52	52.2	305	129	13.8	10.9	59	245	30	1.10	0.975	S12x35
47	47.4	305	127	13.8	8.9	59	245	30	1.10	0.973	S12x31.8
52	52.3	254	126	12.5	15.1	55	200	27	0.982	0.856	S10x35
38	37.8	254	118	12.5	7.9	55	200	27	0.964	0.846	S10x25.4
34	34.3	203	106	10.8	11.2	47	155	24	0.808	0.702	S8x23
27	27.4	203	102	10.8	6.9	48	155	24	0.800	0.698	S8x18.4

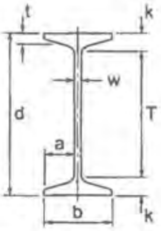
S SHAPES
S150 - S75



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm^2	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^3\ mm^4$	$10^9\ mm^6$
S150												
x26	0.251	3 270	10.9	143	57.8	173	0.969	21.3	17.2	38.9	152	4.95
x19	0.182	2 360	9.16	121	62.2	138	0.765	18.0	18.0	30.6	68.5	3.90
S130												
x15	0.145	1 880	5.11	80.5	52.0	92.7	0.501	13.2	16.3	22.3	47.0	1.76
S100												
x14.1	0.139	1 800	2.85	55.9	39.7	66.5	0.372	10.5	14.4	18.4	50.1	0.832
x11	0.113	1 450	2.56	50.2	41.8	57.9	0.320	9.40	14.8	15.9	30.4	0.715
S75												
x11	0.110	1 430	1.22	32.0	29.2	38.7	0.246	7.68	13.1	13.6	38.1	0.296
x8	0.083	1 080	1.04	27.4	31.2	31.8	0.187	6.34	13.2	10.6	18.2	0.225

Note: These sections are not available from Canadian mills.

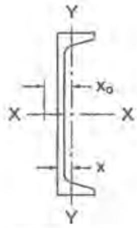


S SHAPES
S150 - S75

DIMENSIONS AND SURFACE AREAS

Nominal Mass	Theo-retical Mass	Depth	Flange Width	Mean Flange Thickness	Web Thickness	Distances			Surface Area (m^2) per metre of length		Imperial Designation
		d	b	t	w	a	T	k	Total	Minus Top of Top Flange	
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm			
26	25.6	152	91	9.1	11.8	40	112	20	0.644	0.553	S6x17.25
19	18.6	152	85	9.1	5.9	40	112	20	0.632	0.547	S6x12.5
15	14.8	127	76	8.3	5.4	35	89	19	0.547	0.471	S5x10
14.1	14.2	102	71	7.4	8.3	31	66	18	0.471	0.400	S4x9.5
11	11.5	102	68	7.4	4.9	32	66	18	0.466	0.398	S4x7.7
11	11.2	76	64	6.6	8.9	28	44	16	0.390	0.326	S3x7.5
8	8.4	76	59	6.6	4.3	27	44	16	0.379	0.320	S3x5.7

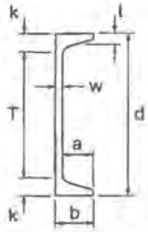
STANDARD CHANNELS (C SHAPES)



PROPERTIES

Designation	Dead Load	Area	Axis X-X			Axis Y-Y				Shear Centre	Torsional Constant	Warping Constant
			I_x	S_x	r_x	I_y	S_y	r_y	x	x_o	J	C_w
	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	10^6 mm^4	10^3 mm^3	mm	mm	mm	10^3 mm^4	10^9 mm^6
C380												
x74*	0.730	9 480	168	881	133	4.60	62.4	22.0	20.3	34.9	1 100	131
x60*	0.583	7 610	145	760	138	3.84	55.5	22.5	19.8	39.1	603	109
x50*	0.495	6 430	131	687	143	3.39	51.4	23.0	20.0	42.6	421	95.2
C310												
x45	0.438	5 690	67.3	442	109	2.12	33.6	19.3	17.0	32.4	360	39.9
x37	0.363	4 740	59.9	393	113	1.85	30.9	19.8	17.1	35.9	222	34.6
x31	0.301	3 930	53.5	351	117	1.59	28.1	20.1	17.6	39.3	152	29.3
C250												
x45	0.437	5 690	42.8	337	86.9	1.60	26.8	16.8	16.3	25.3	508	20.5
x37	0.365	4 740	37.9	299	89.4	1.40	24.3	17.1	15.7	28.1	289	18.2
x30	0.291	3 790	32.7	257	93.0	1.16	21.5	17.5	15.4	31.3	153	15.0
x23	0.221	2 900	27.8	219	98.2	0.920	18.8	17.9	15.9	35.7	86.4	11.7
C230												
x30*	0.292	3 790	25.5	222	81.9	1.01	19.3	16.3	14.8	27.7	179	10.5
x22	0.219	2 850	21.3	186	86.6	0.805	16.8	16.8	15.0	32.3	86.6	8.33
x20	0.195	2 540	19.8	173	88.6	0.715	15.6	16.8	15.2	33.7	69.5	7.35
C200												
x28	0.274	3 550	18.2	180	71.6	0.825	16.6	15.2	14.4	25.2	182	6.67
x21	0.200	2 610	14.9	147	75.8	0.627	13.9	15.5	14.0	29.1	77.0	5.04
x17	0.167	2 180	13.5	133	78.7	0.543	12.8	15.8	14.5	32.0	53.8	4.34
C180												
x22*	0.214	2 790	11.3	127	63.7	0.568	12.8	14.3	13.5	24.6	110	3.47
x18	0.178	2 320	10.0	113	65.9	0.476	11.4	14.3	13.2	26.5	66.8	2.90
x15	0.142	1 850	8.86	99.6	69.3	0.404	10.3	14.8	13.8	30.3	41.4	2.46
C150												
x19	0.188	2 470	7.11	93.6	53.9	0.425	10.3	13.2	12.9	22.3	98.9	1.84
x16	0.152	1 990	6.21	81.8	56.1	0.351	9.13	13.3	12.6	24.6	53.4	1.53
x12	0.118	1 550	5.36	70.5	59.1	0.278	7.93	13.5	12.9	27.7	30.6	1.21
C130												
x13	0.130	1 700	3.66	57.6	46.5	0.252	7.20	12.2	12.0	22.3	45.0	0.746
x10	0.097	1 270	3.09	48.6	49.5	0.195	6.14	12.4	12.3	26.1	22.5	0.579
C100												
x11	0.106	1 370	1.91	37.4	37.3	0.174	5.52	11.3	11.5	20.9	34.1	0.320
x9	0.088	1 190	1.68	33.0	38.3	0.146	4.73	11.3	11.1	22.2	20.5	0.281
x8	0.079	1 030	1.61	31.6	39.7	0.132	4.65	11.4	11.6	24.2	16.6	0.246
x7	0.069	852	1.53	30.0	41.4	0.122	4.45	11.7	12.6	27.3	13.3	0.233
C75												
x9	0.087	1 130	0.847	22.3	27.4	0.123	4.31	10.5	11.5	19.4	29.7	0.118
x7	0.072	948	0.749	19.7	28.3	0.095 9	3.67	10.1	10.9	20.3	17.5	0.093 4
x6	0.059	781	0.670	17.6	29.6	0.077 2	3.21	10.1	11.0	22.3	10.9	0.076 8
x5	0.054	665	0.651	17.1	30.4	0.073 7	3.13	10.2	11.4	24.0	9.49	0.074 7

* Not available from Canadian mills

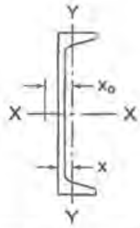


STANDARD CHANNELS (C SHAPES)

DIMENSIONS AND SURFACE AREAS

Nominal Mass	Theoretical Mass	Depth	Flange Width	Flange Thickness	Web Thickness	Distances			Surface Area (m²) per metre of length		Imperial Designation
		d	b	t	w	a	T	k	Total	Minus Top of Top Flange	
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm			
74	74.4	381	94	16.5	18.2	76	309	36	1.10	1.01	C15x50
60	59.4	381	89	16.5	13.2	76	309	36	1.09	1.00	C15x40
50	50.5	381	86	16.5	10.2	76	309	36	1.09	1.00	C15x33.9
45	44.7	305	80	12.7	13.0	67	246	29	0.904	0.824	C12x30
37	37.0	305	77	12.7	9.8	67	246	29	0.898	0.821	C12x25
31	30.7	305	74	12.7	7.2	67	246	29	0.892	0.818	C12x20.7
45	44.5	254	76	11.1	17.1	59	200	27	0.778	0.702	C10x30
37	37.3	254	73	11.1	13.4	60	200	27	0.773	0.700	C10x25
30	29.6	254	69	11.1	9.6	59	200	27	0.765	0.696	C10x20
23	22.6	254	65	11.1	6.1	59	200	27	0.756	0.691	C10x15.3
30	29.8	229	67	10.5	11.4	56	182	23	0.703	0.636	C9x20
22	22.3	229	63	10.5	7.2	56	182	23	0.696	0.633	C9x15
20	19.8	229	61	10.5	5.9	55	182	23	0.690	0.629	C9x13.4
28	27.9	203	64	9.9	12.4	52	159	22	0.637	0.573	C8x18.75
21	20.4	203	59	9.9	7.7	51	159	22	0.627	0.568	C8x13.75
17	17.0	203	57	9.9	5.6	51	159	22	0.623	0.566	C8x11.5
22	21.9	178	58	9.3	10.6	47	136	21	0.567	0.509	C7x14.75
18	18.2	178	55	9.3	8.0	47	136	21	0.560	0.505	C7x12.25
15	14.5	178	53	9.3	5.3	48	136	21	0.557	0.504	C7x9.8
19	19.2	152	54	8.7	11.1	43	113	20	0.498	0.444	C6x13
16	15.5	152	51	8.7	8.0	43	113	20	0.492	0.441	C6x10.5
12	12.0	152	48	8.7	5.1	43	113	20	0.486	0.438	C6x8.2
13	13.3	127	47	8.1	8.3	39	90	18	0.425	0.378	C5x9
10	9.9	127	44	8.1	4.8	39	90	19	0.420	0.376	C5x6.7
11	10.8	102	43	7.5	8.2	35	67	17	0.360	0.317	C4x7.25
9	9.0	102	42	6.9	6.3	36	68	17	0.359	0.317	C4x6.25
8	8.0	102	40	7.5	4.7	35	67	17	0.355	0.315	C4x5.4
7	7.0	102	40	7.5	3.2	37	67	17	0.358	0.318	C4x4.5
9	8.8	76	40	6.9	9.0	31	43	16	0.294	0.254	C3x6
7	7.3	76	37	6.9	6.6	30	43	16	0.287	0.250	C3x5
6	6.0	76	35	6.9	4.3	31	43	16	0.283	0.248	C3x4.1
5	5.5	76	35	6.9	3.4	32	43	16	0.285	0.250	C3x3.5

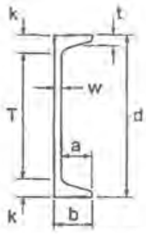
MISCELLANEOUS CHANNELS
MC460 - MC200



PROPERTIES

Designation	Dead Load	Area	Axis X-X			Axis Y-Y				Shear Centre	Torsional Constant	Warping Constant
			I_x	S_x	r_x	I_y	S_y	r_y	x	x_o	J	C_w
	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	10^6 mm^4	10^3 mm^3	mm	mm	mm	10^3 mm^4	10^9 mm^6
MC460												
x86*	0.848	11 000	282	1 230	160	7.36	86.4	25.8	21.9	39.6	1 170	290
x77.2*	0.758	9 870	261	1 140	163	6.81	82.8	26.3	21.8	42.1	842	264
x68.2*	0.669	8 710	241	1 050	166	6.18	77.1	26.7	21.9	45.2	605	243
x63.5*	0.623	8 130	231	1 010	169	5.94	76.3	27.1	22.2	46.8	513	227
MC330												
x74*	0.730	9 480	131	792	117	6.81	78.0	26.8	24.7	45.4	1 240	149
x60*	0.582	7 610	113	685	122	5.63	69.0	27.3	24.4	50.6	643	123
x52*	0.511	6 640	105	635	126	5.13	65.8	27.8	24.9	54.1	472	110
x47.3*	0.464	6 030	99.3	602	128	4.69	61.1	27.9	25.4	57.2	393	103
MC310												
x74*	0.731	9 480	112	738	109	7.26	92.7	27.7	26.7	45.5	1 340	111
x67*	0.656	8 500	105	688	111	6.55	86.5	27.7	26.3	47.9	966	101
x60*	0.585	7 610	97.8	641	113	5.92	81.4	27.9	26.3	50.5	708	90.9
x52*	0.510	6 620	90.3	592	117	5.26	76.1	28.2	26.8	54.2	514	80.6
x46*	0.453	5 890	84.4	554	120	4.69	71.6	28.2	27.6	57.2	418	71.5
MC310												
x21.3*	0.210	2 700	32.0	210	108	0.413	9.28	12.3	9.52	20.6	51.0	8.94
MC310												
x15.8*	0.154	2 000	23.0	151	107	0.157	5.04	8.88	6.81	14.0	24.8	3.11
MC250												
x61.2*	0.601	7 810	65.7	518	91.8	6.56	79.6	29.0	27.6	49.7	942	72.7
x50*	0.490	6 370	57.9	456	95.3	5.43	70.9	29.2	27.5	54.4	500	60.0
x42.4*	0.416	5 400	52.6	414	98.7	4.66	64.9	29.4	28.2	58.9	329	51.5
MC250												
x37*	0.365	4 740	45.8	360	98.2	3.02	48.9	25.2	24.2	49.9	264	33.1
x33*	0.321	4 160	42.7	336	101	2.67	45.2	25.3	25.1	53.4	213	29.6
MC250												
x12.5*	0.122	1 590	13.3	104	91.6	0.136	4.41	9.28	7.21	15.6	17.2	1.87
x9.7*	0.096	1 240	9.35	73.6	86.4	0.051 2	2.20	6.40	4.71	8.64	7.80	0.624
MC230												
x37.8*	0.369	4 820	36.6	319	87.3	3.06	48.0	25.3	24.3	49.0	286	27.3
x35.6*	0.347	4 530	35.2	308	88.4	2.88	46.0	25.3	24.4	50.4	246	25.8
MC200												
x33.9*	0.330	4 320	26.2	258	78.3	2.81	44.7	25.6	25.2	51.3	234	19.6
x31.8*	0.310	4 050	25.4	250	79.4	2.66	43.4	25.7	25.7	53.2	203	18.6

* Not available from Canadian mills

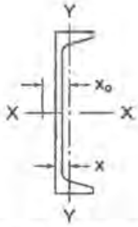


MISCELLANEOUS CHANNELS
MC460 - MC200

DIMENSIONS AND SURFACE AREAS

Nominal Mass	Theo-retical Mass	Depth d	Flange Width b	Flange Thick-ness t	Web Thick-ness w	Distances a	Distances T	Distances k	Surface Area (m²) per metre of length: Total	Surface Area (m²) per metre of length: Minus Top of Top Flange	Imperial Designation
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm			
86	86.5	457	107	15.9	17.8	89	385	36	1.31	1.20	MC18x58
77.2	77.2	457	104	15.9	15.2	89	385	36	1.30	1.20	MC18x51,9
68.2	68.2	457	102	15.9	12.7	89	385	36	1.30	1.19	MC18x45.8
63.5	63.6	457	100	15.9	11.4	89	385	36	1.29	1.19	MC18x42.7
74	74.5	330	112	15.5	20.0	92	258	36	1.07	0.956	MC13x50
60	59.3	330	106	15.5	14.2	92	258	36	1.06	0.950	MC13x40
52	52.1	330	103	15.5	11.4	92	258	36	1.05	0.946	MC13x35
47.3	47.3	330	102	15.5	9.5	93	258	36	1.05	0.947	MC13x31.8
74	74.5	305	105	17.8	21.2	84	237	34	0.988	0.883	MC12x50
67	66.9	305	102	17.8	18.0	84	237	34	0.982	0.880	MC12x45
60	59.7	305	99	17.8	15.0	84	237	34	0.976	0.877	MC12x40
52	52.0	305	96	17.8	11.8	84	237	34	0.970	0.874	MC12x35
46	46.2	305	93	17.8	9.4	84	237	34	0.963	0.870	MC12x31
21.3	21.4	305	54	8.0	6.4	48	265	20	0.813	0.759	MC12x14.3
15.8	15.7	305	38	7.8	4.8	33	267	19	0.752	0.714	MC12x10.6
61.2	61.3	254	110	14.6	20.2	90	188	33	0.908	0.798	MC10x41.1
50	50.0	254	104	14.6	14.6	89	188	33	0.895	0.791	MC10x33.6
42.4	42.4	254	100	14.6	10.8	89	188	33	0.886	0.786	MC10x28.5
37	37.2	254	86	14.6	9.7	76	188	33	0.833	0.747	MC10x25
33	32.7	254	84	14.6	7.4	77	188	33	0.829	0.745	MC10x22
12.5	12.4	254	38	7.1	4.3	34	218	18	0.651	0.613	MC10x8.4
9.7	9.8	254	28	5.1	3.9	24	226	14	0.612	0.584	MC10x6.5
37.8	37.7	229	88	14.0	11.4	77	167	31	0.787	0.699	MC9x25.4
35.6	35.4	229	87	14.0	10.2	77	167	31	0.786	0.699	MC9x23.9
33.9	33.6	203	88	13.3	10.8	77	143	30	0.736	0.648	MC8x22.8
31.8	31.6	203	87	13.3	9.5	78	143	30	0.735	0.648	MC8x21.4

MISCELLANEOUS CHANNELS
MC200 - MC75

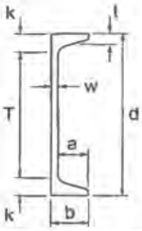


PROPERTIES

Designation	Dead Load	Area	Axis X-X			Axis Y-Y				Shear Centre	Torsional Constant	Warping Constant
			I_x	S_x	r_x	I_y	S_y	r_y	x	x_o	J	C_w
	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	10^6 mm^4	10^3 mm^3	mm	mm	mm	10^3 mm^4	10^9 mm^6
MC200												
x29.8*	0.290	3 790	22.4	221	77.1	1.77	32.2	21.7	20.9	41.9	182	12.4
x27.8*	0.271	3 550	21.6	213	78.2	1.66	30.7	21.7	21.1	43.3	157	11.7
MC200												
x12.6*	0.122	1 610	9.52	93.8	77.4	0.247	6.79	12.5	10.7	24.2	23.7	2.11
MC180												
x33.8*	0.330	4 300	19.7	221	67.8	2.92	45.0	26.1	26.1	51.5	257	15.4
x28.4*	0.277	3 620	17.9	201	70.5	2.43	40.4	26.0	26.9	55.9	168	13.0
MC150												
x26.8*	0.261	3 410	12.2	160	60.0	2.36	39.3	26.4	28.0	57.2	156	8.96
x22.8*	0.222	2 900	10.4	137	60.2	1.99	32.4	26.3	26.5	55.7	92.0	7.84
MC150												
x24.3*	0.238	3 090	10.8	142	59.1	1.56	29.7	22.5	23.5	47.1	141	5.90
x22.5*	0.218	2 860	10.2	135	60.1	1.39	27.4	22.1	23.5	48.2	117	5.31
MC150												
x17.9*	0.175	2 280	7.75	102	58.3	0.769	17.0	18.4	17.9	36.1	64.5	2.97
MC150												
x10.4+	0.103	1 341	4.72	62.1	59.4	0.251	7.10	13.7	12.6	27.7	19.1	1.08
x9.7+	0.096	1 250	4.53	59.7	60.3	0.234	6.88	13.7	12.9	28.6	17.0	0.999
MC100												
x20.5*	0.204	2 594	3.78	74.1	37.7	0.882	20.7	18.2	21.3	37.7	164	1.35
MC75												
x10.6*	0.104	1 348	1.12	29.6	28.9	0.277	8.55	14.3	16.5	31.0	37.5	0.243

* Not available from Canadian mills

\+ This section had no known producer at time of printing.

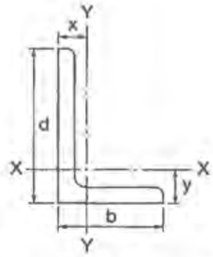


MISCELLANEOUS CHANNELS
MC200 - MC75

DIMENSIONS AND SURFACE AREAS

Nominal Mass	Theoretical Mass	Depth d	Flange Width b	Flange Thickness t	Web Thickness w	Distances a	Distances T	Distances k	Surface Area (m²) per metre of length: Total	Surface Area (m²) per metre of length: Minus Top of Top Flange	Imperial Designation
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm			
29.8	29.6	203	76	12.7	10.2	66	147	28	0.690	0.614	MC8x20
27.8	27.7	203	75	12.7	9.0	66	147	28	0.688	0.613	MC8x18.7
12.6	12.5	203	47	7.9	4.5	43	165	19	0.585	0.538	MC8x8.5
33.8	33.7	178	91	12.7	12.8	78	122	28	0.694	0.603	MC7x22.7
28.4	28.2	178	87	12.7	8.9	78	122	28	0.686	0.599	MC7x19.1
26.8	26.6	152	88	12.1	9.6	78	98	27	0.637	0.549	MC6x18
22.8	22.6	152	88	9.8	8.6	79	108	22	0.639	0.551	MC6x15.3
24.3	24.2	152	76	12.1	9.5	67	98	27	0.589	0.513	MC6x16.3
22.5	22.3	152	74	12.1	8.0	66	98	27	0.584	0.510	MC6x15.1
17.9	17.9	152	63	9.5	7.9	55	110	21	0.540	0.477	MC6x12
10.4	10.5	152	48	7.4	4.5	44	114	19	0.487	0.439	MC6x7.0
9.7	9.8	152	47	7.4	3.9	43	114	19	0.484	0.437	MC6x6.5
20.5	20.8	102	64	13.0	13.0	51	52	25	0.434	0.370	MC4x13.8
10.6	10.6	76	49	8.9	7.9	41	34	21	0.332	0.283	MC3x7.1

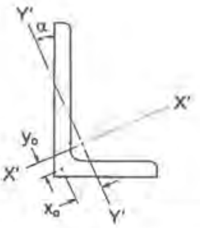
ANGLES
L254 - L178



PROPERTIES ABOUT GEOMETRIC AXES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	y	I_y	S_y	r_y	x	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^3 mm^4$	$10^9 mm^6$
L254x254												
x32*	1.17	15 100	90.5	506	77.3	75.2					5 100	24.1
x29*	1.06	13 700	82.9	460	77.7	74.0					3 740	17.9
x25*	0.944	12 300	74.9	414	78.2	72.9					2 640	12.8
x22*	0.830	10 800	66.7	366	78.6	71.7					1 770	8.71
x19*	0.719	9 310	58.4	318	79.1	70.6					1 140	5.65
L203x203												
x29*	0.831	10 800	40.7	287	61.4	61.2					2 940	8.73
x25*	0.744	9 680	36.9	258	61.8	60.1					2 080	6.27
x22*	0.656	8 500	33.0	229	62.2	58.9					1 400	4.30
x19*	0.566	7 360	28.9	199	62.7	57.8					885	2.76
x16*	0.477	6 200	24.7	169	63.1	56.6					523	1.66
x14*	0.431	5 600	22.5	153	63.3	56.0					382	1.22
x13*	0.385	5 000	20.2	137	63.6	55.5					269	0.865
L203x152												
x25*	0.644	8 390	33.5	247	63.3	67.4	16.0	145	43.7	41.9	1 800	4.37
x22*	0.569	7 420	30.0	219	63.7	66.2	14.4	129	44.1	40.7	1 210	3.00
x19*	0.491	6 410	26.2	190	64.1	65.1	12.7	113	44.5	39.6	768	1.93
x16*	0.415	5 390	22.5	162	64.6	64.0	10.9	95.9	44.9	38.5	454	1.16
x14*	0.375	4 880	20.4	146	64.8	63.4	9.94	87.1	45.2	37.9	332	0.857
x13*	0.335	4 350	18.4	131	65.0	62.8	8.96	78.1	45.4	37.3	234	0.609
x11*	0.294	3 830	16.3	115	65.3	62.2	7.94	68.9	45.6	36.7	157	0.412
L203x102												
x25*	0.547	7 100	29.0	230	63.8	77.2	4.90	65.1	26.3	26.7	1 530	3.46
x22*	0.483	6 280	25.9	204	64.3	76.0	4.43	57.9	26.6	25.5	1 030	2.38
x19*	0.418	5 450	22.8	178	64.7	74.8	3.93	50.6	26.9	24.3	654	1.53
x16*	0.354	4 590	19.5	151	65.2	73.6	3.41	43.3	27.3	23.1	387	0.921
x14*	0.320	4 150	17.8	137	65.4	73.0	3.13	39.4	27.4	22.5	283	0.680
x13*	0.286	3 710	16.0	123	65.7	72.4	2.84	35.4	27.6	21.9	200	0.482
x11*	0.251	3 260	14.2	108	65.9	71.8	2.53	31.4	27.9	21.3	134	0.327
L178x102												
x19*	0.382	4 960	15.8	138	56.4	63.7	3.80	49.9	27.7	25.7	597	1.06
x16*	0.323	4 180	13.6	118	56.8	62.6	3.31	42.7	28.1	24.6	354	0.642
x13	0.261	3 390	11.1	95.6	57.3	61.4	2.75	35.0	28.5	23.4	183	0.338
x11*	0.230	2 980	9.88	84.3	57.5	60.8	2.45	31.0	28.7	22.8	123	0.229
x9.5	0.198	2 570	8.60	73.0	57.8	60.2	2.15	26.9	28.9	22.2	78.0	0.147

* Not available from Canadian mills

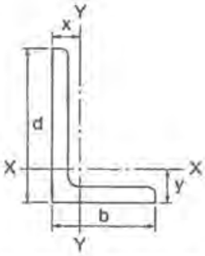


DIMENSIONS AND PROPERTIES ABOUT PRINCIPAL AXES

Mass	d	b	t	Axis X'-X'		Axis Y'-Y'		$\bar{r}_o$	Ω	tan α
				r_x	y_o	r_y	x_o			
kg/m	mm	mm	mm	mm	mm	mm	mm	mm		
119	254	254	31.8	97.4	0.00	49.7	83.8	138	0.630	1.00
108	254	254	28.6	98.0	0.00	49.8	84.4	139	0.629	1.00
96.2	254	254	25.4	98.6	0.00	49.9	85.1	140	0.628	1.00
84.6	254	254	22.2	99.3	0.00	50.1	85.7	140	0.627	1.00
73.1	254	254	19.1	99.9	0.00	50.3	86.3	141	0.627	1.00
84.7	203	203	28.6	77.3	0.00	39.6	66.3	109	0.631	1.00
75.9	203	203	25.4	77.9	0.00	39.7	67.0	110	0.630	1.00
67.0	203	203	22.2	78.5	0.00	39.8	67.6	111	0.629	1.00
57.9	203	203	19.0	79.1	0.00	40.0	68.2	112	0.628	1.00
48.7	203	203	15.9	79.7	0.00	40.1	68.8	113	0.627	1.00
44.0	203	203	14.3	80.0	0.00	40.2	69.2	113	0.626	1.00
39.3	203	203	12.7	80.3	0.00	40.3	69.5	114	0.626	1.00
65.5	203	152	25.4	69.7	34.2	32.4	51.7	98.8	0.606	0.541
57.9	203	152	22.2	70.3	34.2	32.5	52.4	99.6	0.605	0.545
50.1	203	152	19.0	70.9	34.2	32.6	53.1	100	0.604	0.549
42.2	203	152	15.9	71.5	34.3	32.8	53.8	101	0.603	0.553
38.1	203	152	14.3	71.8	34.3	32.9	54.1	102	0.603	0.554
34.1	203	152	12.7	72.1	34.3	33.0	54.5	102	0.603	0.556
29.9	203	152	11.1	72.4	34.3	33.1	54.8	103	0.603	0.558
55.4	203	102	25.4	65.6	59.2	21.6	29.2	95.5	0.523	0.249
49.3	203	102	22.2	66.1	59.3	21.6	30.0	96.2	0.523	0.255
42.5	203	102	19.0	66.6	59.5	21.7	30.8	96.9	0.523	0.260
36.0	203	102	15.9	67.2	59.6	21.9	31.5	97.7	0.523	0.265
32.4	203	102	14.3	67.4	59.7	22.0	31.9	98.0	0.524	0.267
29.0	203	102	12.7	67.7	59.8	22.1	32.2	98.4	0.524	0.269
25.6	203	102	11.1	68.0	59.8	22.2	32.6	98.8	0.524	0.272
38.8	178	102	19.0	58.9	46.5	21.9	32.2	84.6	0.552	0.326
32.7	178	102	15.9	59.4	46.6	22.1	33.0	85.3	0.552	0.331
26.5	178	102	12.7	60.0	46.7	22.2	33.7	86.1	0.552	0.336
23.4	178	102	11.1	60.3	46.8	22.3	34.1	86.5	0.552	0.339
20.2	178	102	9.53	60.6	46.8	22.4	34.4	86.9	0.553	0.341

See *Rolled Structural Shapes* for further information on the properties of angles.

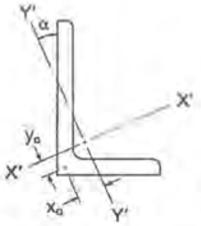
ANGLES
L152 - L127



PROPERTIES ABOUT GEOMETRIC AXES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	y	I_y	S_y	r_y	x	J	C_w
	kN/m	mm^2	$10^6\,mm^4$	$10^3\,mm^3$	mm	mm	$10^6\,mm^4$	$10^3\,mm^3$	mm	mm	$10^3\,mm^4$	$10^9\,mm^6$
L152x152												
x25*	0.545	7 100	14.6	140	45.5	47.2					1 520	2.46
x22*	0.482	6 280	13.2	124	45.9	46.1					1 030	1.70
x19	0.417	5 450	11.6	108	46.3	45.0					652	1.10
x16	0.353	4 590	9.99	92.3	46.7	43.9					386	0.668
x14*	0.319	4 150	9.12	83.8	46.9	43.3					282	0.494
x13	0.285	3 710	8.22	75.2	47.1	42.7					199	0.352
x11*	0.250	3 270	7.29	66.4	47.4	42.1					134	0.239
x9.5	0.216	2 810	6.36	57.5	47.6	41.5					85.0	0.153
x7.9*	0.181	2 360	5.38	48.4	47.8	41.0					49.4	0.090 2
L152x102												
x22*	0.396	5 150	11.5	117	47.2	53.7	4.10	55.9	28.2	28.7	845	1.08
x19	0.344	4 480	10.1	102	47.6	52.5	3.65	48.9	28.6	27.5	537	0.702
x16	0.291	3 780	8.73	86.8	48.0	51.4	3.17	41.9	28.9	26.4	319	0.427
x14*	0.264	3 430	7.98	78.8	48.2	50.8	2.91	38.2	29.1	25.8	234	0.316
x13	0.236	3 060	7.20	70.7	48.5	50.2	2.64	34.4	29.3	25.2	165	0.226
x11*	0.208	2 700	6.39	62.4	48.7	49.6	2.35	30.4	29.6	24.6	111	0.153
x9.5	0.179	2 330	5.58	54.2	48.9	49.1	2.06	26.5	29.8	24.1	70.5	0.098 8
x7.9	0.150	1 950	4.72	45.6	49.2	48.5	1.76	22.4	30.0	23.5	41.1	0.058 2
L152x89												
x13	0.223	2 900	6.86	69.1	48.6	52.7	1.77	26.1	24.7	21.2	156	0.208
x9.5	0.170	2 210	5.32	52.9	49.1	51.6	1.39	20.2	25.1	20.0	66.8	0.091 1
x7.9	0.142	1 850	4.50	44.6	49.3	51.0	1.19	17.1	25.3	19.4	38.9	0.053 6
L127x127												
x22*	0.396	5 150	7.39	84.7	37.9	39.8					845	0.946
x19	0.344	4 480	6.54	74.0	38.3	38.7					537	0.618
x16	0.291	3 780	5.66	63.3	38.7	37.6					319	0.377
x13	0.236	3 070	4.68	51.7	39.1	36.4					165	0.200
x11*	0.208	2 700	4.17	45.7	39.3	35.8					111	0.136
x9.5	0.179	2 330	3.64	39.7	39.5	35.3					70.5	0.087 8
x7.9	0.150	1 960	3.09	33.5	39.8	34.7					41.1	0.051 8
L127x89												
x19*	0.288	3 750	5.78	69.9	39.3	44.3	2.31	36.2	24.8	25.3	450	0.404
x16*	0.245	3 170	5.01	59.8	39.7	43.2	2.01	31.1	25.2	24.2	268	0.248
x13*	0.199	2 580	4.16	48.9	40.1	42.1	1.68	25.6	25.6	23.0	139	0.132
x9.5	0.151	1 970	3.24	37.6	40.6	40.9	1.33	19.8	26.0	21.9	59.5	0.058 2
x7.9	0.127	1 650	2.75	31.7	40.8	40.3	1.13	16.7	26.2	21.3	34.7	0.034 4
x6.4	0.102	1 330	2.24	25.7	41.0	39.7	0.928	13.6	26.4	20.7	17.9	0.018 0

* Not available from Canadian mills

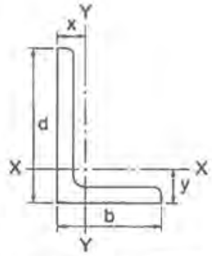


DIMENSIONS AND PROPERTIES ABOUT PRINCIPAL AXES

Mass	d	b	t	Axis X'-X'		Axis Y'-Y'		$\bar{r}_o$	Ω	tan α
				r_x	y_o	r_y	x_o			
kg/m	mm	mm	mm	mm	mm	mm	mm	mm		
55.7	152	152	25.4	57.1	0.00	29.6	48.8	80.8	0.634	1.00
49.3	152	152	22.2	57.7	0.00	29.6	49.5	81.6	0.632	1.00
42.7	152	152	19.0	58.3	0.00	29.7	50.2	82.5	0.630	1.00
36.0	152	152	15.9	58.9	0.00	29.8	50.8	83.3	0.628	1.00
32.6	152	152	14.3	59.2	0.00	29.9	51.1	83.7	0.628	1.00
29.2	152	152	12.7	59.5	0.00	30.0	51.4	84.2	0.627	1.00
25.6	152	152	11.1	59.8	0.00	30.1	51.7	84.6	0.627	1.00
22.2	152	152	9.53	60.1	0.00	30.2	52.0	85.1	0.626	1.00
18.5	152	152	7.94	60.5	0.00	30.3	52.3	85.5	0.626	1.00
40.3	152	102	22.2	50.5	32.2	21.9	32.9	71.7	0.588	0.427
35.0	152	102	19.0	51.0	32.3	21.9	33.6	72.5	0.586	0.434
29.6	152	102	15.9	51.6	32.3	22.0	34.4	73.3	0.585	0.440
26.8	152	102	14.3	51.8	32.4	22.1	34.7	73.7	0.585	0.443
24.0	152	102	12.7	52.1	32.4	22.2	35.1	74.1	0.585	0.446
21.2	152	102	11.1	52.4	32.4	22.3	35.5	74.5	0.584	0.449
18.2	152	102	9.53	52.7	32.4	22.4	35.8	74.9	0.584	0.451
15.3	152	102	7.94	53.0	32.5	22.5	36.1	75.3	0.584	0.454
22.7	152	88.9	12.7	51.0	39.0	19.3	29.2	73.1	0.556	0.345
17.3	152	88.9	9.53	51.6	39.1	19.5	29.9	73.9	0.557	0.351
14.5	152	88.9	7.94	51.9	39.1	19.6	30.2	74.3	0.557	0.354
40.5	127	127	22.2	47.5	0.00	24.7	40.6	67.2	0.635	1.00
35.1	127	127	19.0	48.1	0.00	24.8	41.3	68.1	0.632	1.00
29.8	127	127	15.9	48.7	0.00	24.8	41.9	68.9	0.630	1.00
24.1	127	127	12.7	49.3	0.00	25.0	42.5	69.8	0.628	1.00
21.3	127	127	11.1	49.6	0.00	25.0	42.8	70.2	0.627	1.00
18.3	127	127	9.53	49.9	0.00	25.1	43.2	70.6	0.627	1.00
15.3	127	127	7.94	50.3	0.00	25.2	43.5	71.1	0.626	1.00
29.3	127	88.9	19.0	42.4	24.9	19.0	29.0	60.2	0.597	0.464
24.9	127	88.9	15.9	43.0	25.0	19.1	29.7	61.0	0.594	0.472
20.2	127	88.9	12.7	43.5	25.0	19.2	30.5	61.8	0.593	0.479
15.4	127	88.9	9.53	44.1	25.0	19.3	31.2	62.6	0.592	0.486
12.9	127	88.9	7.94	44.4	25.1	19.4	31.5	63.0	0.592	0.489
10.4	127	88.9	6.35	44.7	25.1	19.6	31.8	63.4	0.592	0.492

See *Rolled Structural Shapes* for further information on the properties of angles.

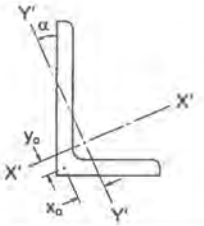
ANGLES
L127 - L89



PROPERTIES ABOUT GEOMETRIC AXES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	y	I_y	S_y	r_y	x	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^3 mm^4$	$10^9 mm^6$
L127x76												
x13	0.186	2 420	3.93	47.7	40.3	44.5	1.07	18.8	21.1	19.1	130	0.119
x11*	0.164	2 140	3.51	42.2	40.6	43.9	0.963	16.7	21.3	18.5	87.6	0.081 5
x9.5	0.142	1 850	3.07	36.7	40.8	43.3	0.849	14.6	21.5	17.9	55.9	0.052 7
x7.9	0.119	1 550	2.61	30.9	41.0	42.7	0.727	12.3	21.7	17.3	32.6	0.031 1
x6.4	0.096 2	1 250	2.13	25.0	41.2	42.1	0.598	10.1	21.9	16.7	16.8	0.016 3
L102x102												
x19	0.271	3 510	3.23	46.3	30.3	32.4					423	0.302
x16*	0.230	2 970	2.81	39.8	30.7	31.3					252	0.186
x13	0.187	2 420	2.34	32.6	31.1	30.2					131	0.099 6
x11	0.165	2 140	2.09	28.9	31.3	29.6					87.9	0.068 2
x9.5	0.143	1 850	1.84	25.2	31.5	29.0					56.1	0.044 2
x7.9	0.120	1 550	1.57	21.3	31.7	28.4					32.7	0.026 2
x6.4	0.096 6	1 250	1.28	17.3	31.9	27.9					16.9	0.013 7
L102x89												
x13*	0.174	2 260	2.24	32.0	31.5	31.9	1.58	24.9	26.4	25.4	122	0.081 8
x9.5	0.133	1 720	1.76	24.7	31.9	30.8	1.24	19.2	26.8	24.2	52.3	0.036 4
x7.9	0.112	1 450	1.50	20.9	32.1	30.2	1.06	16.3	27.1	23.6	30.5	0.021 6
x6.4	0.090 2	1 170	1.23	16.9	32.3	29.6	0.872	13.2	27.3	23.1	15.8	0.011 3
L102x76												
x16*	0.199	2 570	2.54	38.0	31.4	35.0	1.20	22.2	21.6	22.1	217	0.128
x13	0.162	2 100	2.12	31.2	31.8	33.9	1.01	18.3	21.9	21.0	113	0.069 2
x9.5	0.124	1 600	1.67	24.1	32.2	32.7	0.800	14.2	22.3	19.8	48.7	0.030 9
x7.9	0.104	1 350	1.42	20.4	32.4	32.1	0.686	12.0	22.5	19.2	28.4	0.018 3
x6.4	0.084 0	1 090	1.17	16.5	32.7	31.6	0.565	9.81	22.7	18.7	14.7	0.009 63
L89x89												
x13	0.161	2 100	1.51	24.4	26.9	26.9					113	0.064 0
x11*	0.142	1 850	1.36	21.7	27.1	26.3					76.0	0.044 0
x9.5	0.123	1 600	1.19	18.9	27.3	25.7					48.5	0.028 6
x7.9	0.104	1 350	1.02	16.0	27.5	25.2					28.3	0.017 0
x6.4	0.083 8	1 090	0.837	13.0	27.7	24.6					14.6	0.008 96

* Not available from Canadian mills

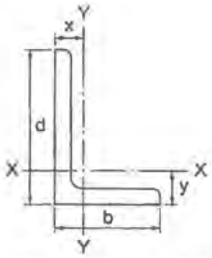


DIMENSIONS AND PROPERTIES ABOUT PRINCIPAL AXES

Mass	d	b	t	Axis X'-X'		Axis Y'-Y'		$\bar{r}_o$	Ω	tan α
				r_x	y_o	r_y	x_o			
kg/m	mm	mm	mm	mm	mm	mm	mm	mm		
19.0	127	76.2	12.7	42.4	31.6	16.5	24.8	60.7	0.562	0.357
16.7	127	76.2	11.1	42.7	31.7	16.5	25.1	61.1	0.562	0.361
14.5	127	76.2	9.53	43.0	31.7	16.6	25.5	61.5	0.562	0.364
12.1	127	76.2	7.94	43.3	31.7	16.7	25.9	61.9	0.562	0.368
9.8	127	76.2	6.35	43.6	31.8	16.8	26.2	62.3	0.562	0.371
27.5	102	102	19.0	38.0	0.00	19.8	32.4	53.7	0.637	1.00
23.4	102	102	15.9	38.5	0.00	19.9	33.0	54.5	0.633	1.00
19.0	102	102	12.7	39.1	0.00	19.9	33.7	55.3	0.630	1.00
16.8	102	102	11.1	39.4	0.00	20.0	34.0	55.8	0.629	1.00
14.6	102	102	9.53	39.7	0.00	20.1	34.3	56.2	0.628	1.00
12.2	102	102	7.94	40.1	0.00	20.2	34.6	56.6	0.627	1.00
9.8	102	102	6.35	40.4	0.00	20.3	34.9	57.1	0.626	1.00
17.6	102	88.9	12.7	36.8	9.16	18.4	30.5	52.0	0.625	0.744
13.5	102	88.9	9.53	37.4	9.15	18.5	31.2	52.8	0.622	0.749
11.4	102	88.9	7.94	37.7	9.15	18.6	31.5	53.3	0.621	0.751
9.2	102	88.9	6.35	38.0	9.15	18.7	31.8	53.7	0.621	0.753
20.2	102	76.2	15.9	34.5	17.3	16.2	25.2	48.8	0.609	0.529
16.4	102	76.2	12.7	35.0	17.3	16.2	25.9	49.6	0.606	0.538
12.6	102	76.2	9.53	35.6	17.3	16.4	26.6	50.4	0.604	0.547
10.7	102	76.2	7.94	35.9	17.3	16.5	27.0	50.9	0.603	0.550
8.6	102	76.2	6.35	36.2	17.3	16.6	27.3	51.3	0.603	0.554
16.5	88.9	88.9	12.7	33.8	0.00	17.3	29.0	47.8	0.632	1.00
14.6	88.9	88.9	11.1	34.1	0.00	17.4	29.3	48.2	0.630	1.00
12.6	88.9	88.9	9.53	34.4	0.00	17.4	29.7	48.7	0.629	1.00
10.7	88.9	88.9	7.94	34.7	0.00	17.5	30.0	49.1	0.627	1.00
8.6	88.9	88.9	6.35	35.0	0.00	17.6	30.3	49.5	0.627	1.00

See *Rolled Structural Shapes* for further information on the properties of angles.

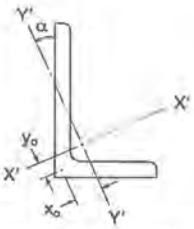
ANGLES
L89 - L76



PROPERTIES ABOUT GEOMETRIC AXES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	y	I_y	S_y	r_y	x	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^3 mm^4$	$10^9 mm^6$
L89x76												
x13	0.149	1 940	1.44	23.8	27.3	28.6	0.969	18.0	22.4	22.2	104	0.051 4
x11*	0.132	1 710	1.29	21.2	27.5	28.0	0.871	16.0	22.6	21.7	70.2	0.035 4
x9.5	0.114	1 480	1.13	18.5	27.7	27.4	0.769	14.0	22.8	21.1	44.9	0.023 1
x7.9	0.096 1	1 250	0.970	15.6	27.9	26.9	0.659	11.8	23.0	20.5	26.2	0.013 8
x6.4	0.077 6	1 010	0.796	12.7	28.1	26.3	0.543	9.65	23.2	19.9	13.5	0.007 25
L89x64												
x13	0.137	1 770	1.35	23.1	27.6	30.6	0.568	12.5	17.9	17.9	95.4	0.042 6
x9.5	0.105	1 360	1.07	17.9	28.0	29.5	0.454	9.71	18.3	16.8	41.2	0.019 2
x7.9	0.088 3	1 150	0.912	15.2	28.2	28.9	0.391	8.26	18.5	16.2	24.1	0.011 5
x6.4	0.071 4	929	0.749	12.4	28.4	28.3	0.323	6.75	18.7	15.6	12.5	0.006 04
L76x76												
x13	0.137	1 770	0.923	17.6	22.8	23.7					95.4	0.038 8
x11*	0.121	1 570	0.830	15.6	23.0	23.1					64.4	0.026 8
x9.5	0.105	1 360	0.733	13.7	23.2	22.5					41.2	0.017 5
x7.9	0.088 3	1 150	0.629	11.6	23.4	22.0					24.1	0.010 5
x6.4	0.071 4	929	0.518	9.45	23.6	21.4					12.5	0.005 54
x4.8	0.054 1	703	0.400	7.22	23.9	20.8					5.31	0.002 41
L76x64												
x13*	0.124	1 610	0.867	17.1	23.2	25.4	0.542	12.2	18.3	19.1	86.7	0.030 0
x11*	0.110	1 430	0.780	15.2	23.4	24.8	0.489	10.9	18.5	18.5	58.6	0.020 8
x9.5	0.095 5	1 240	0.690	13.3	23.6	24.3	0.434	9.52	18.7	17.9	37.6	0.013 6
x7.9	0.080 5	1 050	0.592	11.3	23.8	23.7	0.374	8.10	18.9	17.4	22.0	0.008 17
x6.4*	0.065 2	845	0.488	9.20	24.0	23.1	0.309	6.62	19.1	16.8	11.4	0.004 33
x4.8*	0.049 4	643	0.377	7.04	24.2	22.6	0.240	5.08	19.3	16.2	4.85	0.001 89
L76x51												
x13	0.112	1 450	0.800	16.4	23.5	27.5	0.280	7.77	13.9	14.8	78.0	0.024 4
x9.5	0.086 2	1 120	0.638	12.8	23.9	26.4	0.226	6.09	14.2	13.7	33.9	0.011 1
x7.9	0.072 8	942	0.548	10.9	24.1	25.8	0.196	5.20	14.4	13.1	19.9	0.006 67
x6.4	0.059 0	768	0.453	8.88	24.3	25.2	0.163	4.26	14.6	12.5	10.3	0.003 54
x4.8	0.044 8	582	0.350	6.79	24.5	24.6	0.128	3.28	14.8	11.9	4.39	0.001 55

* Not available from Canadian mills



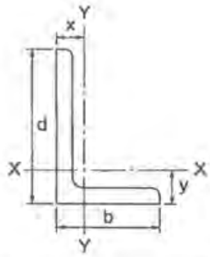
ANGLES
L89 - L76

DIMENSIONS AND PROPERTIES ABOUT PRINCIPAL AXES

Mass	d	b	t	Axis X'-X'		Axis Y'-Y'		$\bar{r}_o$	Ω	tan α
				r_x	y_o	r_y	x_o			
kg/m	mm	mm	mm	mm	mm	mm	mm	mm		
15.1	88.9	76.2	12.7	31.5	8.86	15.8	25.8	44.6	0.625	0.714
13.5	88.9	76.2	11.1	31.8	8.85	15.8	26.2	45.0	0.623	0.718
11.7	88.9	76.2	9.53	32.1	8.85	15.9	26.5	45.4	0.622	0.721
9.8	88.9	76.2	7.94	32.4	8.84	15.9	26.8	45.9	0.621	0.724
8.0	88.9	76.2	6.35	32.7	8.84	16.0	27.1	46.3	0.620	0.727
13.9	88.9	63.5	12.7	29.9	16.8	13.6	21.0	42.4	0.600	0.486
10.7	88.9	63.5	9.53	30.5	16.8	13.6	21.7	43.2	0.597	0.496
9.0	88.9	63.5	7.94	30.8	16.8	13.7	22.1	43.7	0.596	0.501
7.3	88.9	63.5	6.35	31.1	16.8	13.8	22.4	44.1	0.596	0.506
14.0	76.2	76.2	12.7	28.6	0.00	14.8	24.5	40.5	0.634	1.00
12.4	76.2	76.2	11.1	28.9	0.00	14.9	24.8	40.9	0.632	1.00
10.7	76.2	76.2	9.53	29.2	0.00	14.9	25.1	41.3	0.630	1.00
9.1	76.2	76.2	7.94	29.5	0.00	15.0	25.5	41.8	0.628	1.00
7.3	76.2	76.2	6.35	29.8	0.00	15.0	25.8	42.2	0.627	1.00
5.5	76.2	76.2	4.76	30.2	0.00	15.1	26.1	42.6	0.626	1.00
12.6	76.2	63.5	12.7	26.4	8.81	13.2	21.1	37.4	0.625	0.667
11.3	76.2	63.5	11.1	26.7	8.80	13.2	21.5	37.8	0.622	0.672
9.8	76.2	63.5	9.53	27.0	8.79	13.3	21.8	38.2	0.620	0.676
8.3	76.2	63.5	7.94	27.3	8.79	13.3	22.2	38.6	0.619	0.680
6.7	76.2	63.5	6.35	27.6	8.79	13.4	22.5	39.1	0.618	0.684
5.1	76.2	63.5	4.76	27.9	8.79	13.5	22.8	39.5	0.617	0.688
11.5	76.2	50.8	12.7	25.0	16.3	10.9	15.9	35.5	0.589	0.414
8.8	76.2	50.8	9.53	25.5	16.4	10.9	16.7	36.3	0.585	0.428
7.4	76.2	50.8	7.94	25.8	16.4	11.0	17.1	36.7	0.584	0.435
6.1	76.2	50.8	6.35	26.1	16.4	11.0	17.5	37.1	0.583	0.440
4.6	76.2	50.8	4.76	26.4	16.4	11.1	17.8	37.5	0.583	0.446

See *Rolled Structural Shapes* for further information on the properties of angles.

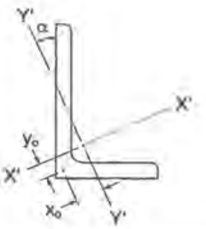
ANGLES
L64 - L38



PROPERTIES ABOUT GEOMETRIC AXES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	y	I_y	S_y	r_y	x	J	C_w
	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	mm	10^6 mm^4	10^3 mm^3	mm	mm	10^3 mm^4	10^9 mm^6
L64x64												
x13	0.112	1 450	0.511	11.9	18.8	20.5					78.0	0.021 2
x9.5	0.086 2	1 120	0.410	9.28	19.1	19.4					33.9	0.009 74
x7.9	0.072 8	942	0.353	7.90	19.3	18.8					19.9	0.005 87
x6.4	0.059 0	768	0.293	6.46	19.5	18.2					10.3	0.003 12
x4.8	0.044 8	581	0.227	4.96	19.8	17.6					4.39	0.001 37
L64x51												
x9.5	0.076 9	1 000	0.380	8.96	19.5	21.1	0.214	5.94	14.6	14.8	30.2	0.007 22
x7.9	0.065 0	845	0.328	7.64	19.7	20.6	0.186	5.08	14.8	14.2	17.7	0.004 36
x6.4	0.052 8	684	0.272	6.25	19.9	20.0	0.155	4.17	15.0	13.6	9.21	0.002 33
x4.8	0.040 1	522	0.212	4.80	20.1	19.4	0.121	3.21	15.2	13.1	3.94	0.001 02
L64x38												
x6.4*	0.046 6	605	0.246	5.96	20.2	22.2	0.067 1	2.35	10.5	9.52	8.13	0.001 86
x4.8*	0.035 5	461	0.192	4.58	20.4	21.6	0.053 0	1.82	10.7	8.94	3.48	0.000 821
L51x51												
x9.5	0.067 5	877	0.199	5.76	15.1	16.2					26.6	0.004 69
x7.9	0.057 2	742	0.173	4.92	15.3	15.6					15.6	0.002 86
x6.4	0.046 6	605	0.145	4.04	15.5	15.0					8.13	0.001 54
x4.8	0.035 5	461	0.113	3.12	15.7	14.5					3.48	0.000 680
x3.2	0.024 1	312	0.079 2	2.14	15.9	13.9					1.05	0.000 213
L51x38												
x6.4	0.040 4	525	0.131	3.87	15.8	16.9	0.063 0	2.28	11.0	10.5	7.05	0.001 07
x4.8	0.030 8	401	0.103	2.99	16.0	16.3	0.049 9	1.77	11.2	9.93	3.02	0.000 477
x3.2	0.021 0	272	0.072 1	2.06	16.3	15.7	0.035 3	1.23	11.4	9.35	0.919	0.000 150
L44x44												
x6.4	0.040 4	525	0.094 9	3.06	13.4	13.4					7.05	0.001 00
x4.8	0.030 9	401	0.074 8	2.36	13.7	12.9					3.03	0.000 448
x3.2	0.021 0	272	0.052 5	1.63	13.9	12.3					0.920	0.000 141
L38x38												
x6.4	0.034 1	444	0.057 7	2.20	11.4	11.8					5.96	0.000 606
x4.8	0.026 2	340	0.045 8	1.71	11.6	11.3					2.57	0.000 273
x4.0*	0.022 1	286	0.039 3	1.45	11.7	11.0					1.51	0.000 164
x3.2	0.017 9	232	0.032 4	1.18	11.8	10.7					0.783	0.000 087

* Not available from Canadian mills

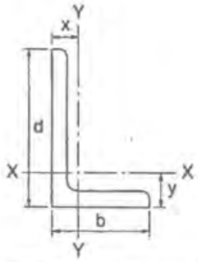


DIMENSIONS AND PROPERTIES ABOUT PRINCIPAL AXES

Mass	d	b	t	Axis X'-X'		Axis Y'-Y'		$\bar{r}_o$	Ω	tan α
				r_x	y_o	r_y	x_o			
kg/m	mm	mm	mm	mm	mm	mm	mm	mm		
11.4	63.5	63.5	12.7	23.5	0.00	12.4	20.0	33.2	0.639	1.00
8.7	63.5	63.5	9.53	24.1	0.00	12.4	20.6	34.0	0.632	1.00
7.4	63.5	63.5	7.94	24.4	0.00	12.4	21.0	34.4	0.630	1.00
6.1	63.5	63.5	6.35	24.7	0.00	12.5	21.3	34.9	0.628	1.00
4.6	63.5	63.5	4.76	25.0	0.00	12.6	21.6	35.3	0.627	1.00
7.9	63.5	50.8	9.53	21.9	8.70	10.7	17.1	31.0	0.618	0.614
6.7	63.5	50.8	7.94	22.2	8.70	10.7	17.4	31.4	0.616	0.620
5.4	63.5	50.8	6.35	22.5	8.70	10.8	17.8	31.9	0.614	0.626
4.2	63.5	50.8	4.76	22.8	8.70	10.9	18.1	32.3	0.612	0.631
4.8	63.5	38.1	6.35	21.2	15.8	8.23	12.4	30.3	0.562	0.357
3.6	63.5	38.1	4.76	21.5	15.8	8.31	12.8	30.7	0.562	0.364
7.0	50.8	50.8	9.53	18.9	0.00	9.89	16.1	26.7	0.637	1.00
5.8	50.8	50.8	7.94	19.2	0.00	9.90	16.4	27.1	0.633	1.00
4.7	50.8	50.8	6.35	19.5	0.00	9.93	16.8	27.6	0.630	1.00
3.6	50.8	50.8	4.76	19.8	0.00	10.0	17.1	28.0	0.628	1.00
2.4	50.8	50.8	3.18	20.1	0.00	10.1	17.4	28.4	0.626	1.00
4.2	50.8	38.1	6.35	17.5	8.53	8.12	13.0	24.7	0.606	0.543
3.1	50.8	38.1	4.76	17.8	8.53	8.18	13.3	25.1	0.604	0.551
2.1	50.8	38.1	3.18	18.0	8.54	8.27	13.7	25.6	0.603	0.558
4.1	44.5	44.5	6.35	16.9	0.00	8.68	14.5	23.9	0.632	1.00
3.1	44.5	44.5	4.76	17.2	0.00	8.73	14.8	24.4	0.629	1.00
2.1	44.5	44.5	3.18	17.5	0.00	8.82	15.2	24.8	0.627	1.00
3.4	38.1	38.1	6.35	14.3	0.00	7.42	12.2	20.2	0.634	1.00
2.7	38.1	38.1	4.76	14.6	0.00	7.45	12.6	20.7	0.630	1.00
2.2	38.1	38.1	3.97	14.8	0.00	7.48	12.7	20.9	0.628	1.00
1.8	38.1	38.1	3.18	14.9	0.00	7.52	12.9	21.1	0.627	1.00

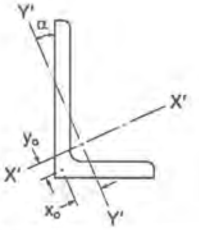
See *Rolled Structural Shapes* for further information on the properties of angles.

ANGLES
L32 - L19



PROPERTIES ABOUT GEOMETRIC AXES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warping Constant
			I_x	S_x	r_x	y	I_y	S_y	r_y	x	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^3 mm^4$	$10^9 mm^6$
L32x32												
x6.4	0.028 0	363	0.032 1	1.49	9.40	10.2					4.89	0.000 334
x4.8	0.021 6	280	0.025 7	1.16	9.58	9.69					2.12	0.000 153
x3.2	0.014 8	192	0.018 4	0.812	9.79	9.12					0.648	0.000 049
L25x25												
x6.4	0.021 7	283	0.015 3	0.915	7.37	8.62					3.79	0.000 156
x4.8	0.016 9	219	0.012 5	0.719	7.54	8.07					1.66	0.000 073
x3.2	0.011 7	151	0.009 05	0.506	7.73	7.52					0.510	0.000 024
L19x19												
x3.2	0.008 57	111	0.003 64	0.276	5.72	5.93					0.375	0.000 010

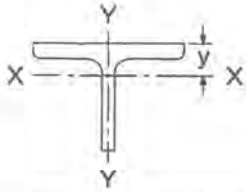


DIMENSIONS AND PROPERTIES ABOUT PRINCIPAL AXES

Mass	d	b	t	Axis X'-X'		Axis Y'-Y'		$\bar{r}_o$	Ω	tan α
				r_x	y_o	r_y	x_o			
kg/m	mm	mm	mm	mm	mm	mm	mm	mm		
2.8	31.8	31.8	6.35	11.8	0.00	6.19	10.0	16.6	0.639	1.00
2.2	31.8	31.8	4.76	12.0	0.00	6.20	10.3	17.0	0.632	1.00
1.5	31.8	31.8	3.18	12.4	0.00	6.25	10.7	17.5	0.628	1.00
2.2	25.4	25.4	6.35	9.17	0.00	4.98	7.70	13.0	0.647	1.00
1.8	25.4	25.4	4.76	9.45	0.00	4.94	8.05	13.4	0.637	1.00
1.2	25.4	25.4	3.18	9.74	0.00	4.97	8.38	13.8	0.630	1.00
0.9	19.1	19.1	3.18	7.18	0.00	3.72	6.14	10.2	0.634	1.00

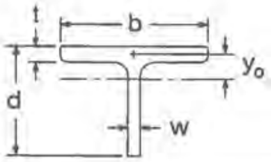
See *Rolled Structural Shapes* for further information on the properties of angles.

STRUCTURAL TEES
Cut from W Shapes
WT460 - WT345



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y			Torsional Constant	Warping Constant
			I_x	S_x	r_x	y	I_y	S_y	r_y	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^4$	$10^9 mm^6$
WT460											
x224.5	2.20	28 800	533	1 450	137	107	270	1 280	97.2	13 100	76.5
x210	2.06	26 800	497	1 360	136	106	250	1 190	96.8	10 700	62.4
x195	1.90	24 800	460	1 270	136	105	226	1 080	95.7	8 440	49.6
x184	1.79	23 400	434	1 200	136	105	211	1 010	95.1	7 020	41.6
x172	1.68	22 000	408	1 140	137	105	195	933	94.4	5 780	34.6
WT460											
x156.5	1.53	20 000	410	1 200	144	124	85.2	551	65.4	5 750	32.0
x144.5	1.41	18 400	376	1 100	143	122	78.2	508	65.2	4 570	24.9
x135.5	1.33	17 300	353	1 040	143	121	72.6	473	64.8	3 810	20.9
x126.5	1.24	16 200	329	969	143	121	66.8	437	64.3	3 100	17.1
x119	1.17	15 200	309	916	143	121	61.4	403	63.6	2 540	14.4
x111.5	1.10	14 200	292	874	143	122	56.1	369	62.7	2 080	12.4
x100.5	0.986	12 800	265	814	144	126	47.2	311	60.7	1 430	10.0
WT420											
x179.5	1.76	22 800	363	1 080	126	97.5	195	965	92.1	7 530	39.3
x164.5	1.62	21 000	333	997	126	96.9	174	870	91.1	5 780	30.4
x149.5	1.47	19 000	303	912	126	96.1	156	780	90.3	4 320	22.9
WT420											
x113	1.11	14 400	247	778	131	108	56.9	387	62.8	2 560	11.5
x105	1.03	13 400	230	733	131	109	51.3	350	61.8	2 020	9.57
x96.5	0.949	12 400	213	688	131	111	45.1	309	60.5	1 520	7.81
x88	0.863	11 200	196	646	132	114	39.1	268	59.1	1 100	6.35
WT380											
x157	1.55	20 000	254	828	112	86.2	158	822	88.7	5 900	26.0
x142	1.40	18 100	229	750	112	84.8	140	733	87.8	4 360	19.1
x128.5	1.27	16 400	207	684	112	83.9	125	657	87.1	3 250	14.3
WT380											
x98	0.965	12 600	175	613	118	99.0	40.9	305	57.1	2 020	7.63
x92.5	0.906	11 800	165	580	118	99.1	37.5	281	56.5	1 660	6.44
x86.5	0.851	11 000	156	554	119	100	34.4	257	55.7	1 340	5.54
x80.5	0.786	10 200	145	523	119	102	30.4	228	54.5	1 030	4.62
x73.5	0.722	9 400	134	493	120	104	26.4	200	53.1	778	3.83
WT345											
x132.5	1.30	16 800	172	624	101	77.2	116	646	82.7	4 160	15.5
x120	1.18	15 300	156	567	101	76.0	103	580	82.0	3 130	11.5
x108.5	1.07	13 800	140	514	100	74.7	92.6	522	81.5	2 350	8.57



STRUCTURAL TEES
Cut from W Shapes
WT460 - WT345

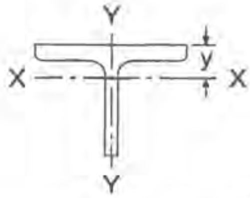
PROPERTIES AND DIMENSIONS

Nominal Mass	Theoretical Mass	Depth d	Flange Width b	Flange Thickness t	Stem Thickness w	β_x	y_o	$\bar{r}_o$	Ω
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	
224.5	224.3	474	423	42.7	24.0	310	86.0	188	0.792
210	209.7	472	422	39.9	22.5	310	86.0	188	0.791
195	194.0	468	420	36.6	21.3	309	87.1	188	0.785
184	182.8	466	419	34.3	20.3	309	87.8	188	0.782
172	171.7	464	418	32.0	19.3	309	88.6	188	0.778
156.5	156.2	466	309	34.5	21.1	334	107	190	0.686
144.5	144.2	464	308	32.0	19.4	334	106	190	0.688
135.5	135.8	462	307	30.0	18.4	333	106	190	0.685
126.5	126.8	460	306	27.9	17.3	333	107	189	0.683
119	119.0	458	305	25.9	16.5	333	108	190	0.678
111.5	112.0	456	304	23.9	15.9	333	110	191	0.669
100.5	100.6	452	304	20.1	15.2	335	116	195	0.645
179.5	180.0	434	403	35.6	21.1	282	79.7	175	0.793
164.5	165.0	431	401	32.4	19.7	282	80.7	175	0.788
149.5	150.0	428	400	29.2	18.2	282	81.5	175	0.783
113	113.4	426	294	26.8	16.1	305	95.0	173	0.700
105	105.4	423	293	24.4	15.4	305	96.9	174	0.691
96.5	96.8	420	292	21.7	14.7	305	99.9	176	0.677
88	88.0	418	292	18.8	14.0	307	104	179	0.659
157	157.6	393	384	33.4	19.7	250	69.5	159	0.809
142	142.5	390	382	30.1	18.0	250	69.8	159	0.807
128.5	129.3	387	381	27.1	16.6	249	70.4	159	0.803
98	98.4	385	268	25.4	15.6	275	86.3	157	0.698
92.5	92.4	383	267	23.6	14.9	275	87.3	157	0.693
86.5	86.8	381	267	21.6	14.4	275	89.3	159	0.683
80.5	80.2	379	266	19.3	13.8	276	92.2	160	0.669
73.5	73.6	377	265	17.0	13.2	277	95.7	162	0.652
132.5	132.8	353	358	30.2	18.4	222	62.1	144	0.815
120	120.6	351	356	27.4	16.8	222	62.3	144	0.813
108.5	109.5	348	355	24.8	15.4	221	62.3	143	0.812

Note: β_x is positive when the flange is in flexural compression, and negative otherwise.

See S16-14 Clauses 13.3.2 and 13.6 and the Commentary in Part 2 for further information on section properties.

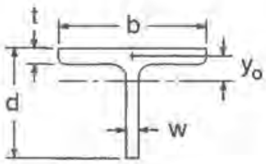
STRUCTURAL TEES
Cut from W Shapes
WT345 - WT265



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y			Torsional Constant	Warping Constant
			I_x	S_x	r_x	y	I_y	S_y	r_y	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^4$	$10^9 mm^6$
WT345											
x85	0.834	10 800	121	465	106	87.1	33.1	259	55.3	1 520	4.72
x76	0.746	9 700	107	415	105	85.8	28.9	227	54.6	1 100	3.38
x70	0.685	8 950	99.3	389	106	86.5	25.9	204	53.9	831	2.72
x62.5	0.616	8 000	89.9	359	106	88.3	22.0	174	52.5	584	2.10
WT305											
x120.5	1.19	15 400	123	491	89.2	68.6	92.1	560	77.3	3 840	11.8
x108.5	1.07	13 800	110	444	88.8	67.4	81.6	497	76.7	2 790	8.58
x97.5	0.959	12 400	99.4	408	89.3	67.4	71.2	435	75.6	1 980	6.23
x87	0.854	11 100	88.3	366	89.2	66.5	61.9	381	74.7	1 400	4.40
x77.5	0.760	9 850	78.9	329	89.4	66.1	53.9	333	73.9	975	3.10
WT305											
x70	0.688	8 950	77.8	334	93.3	76.2	22.6	196	50.3	1 090	2.58
x62.5	0.613	7 950	69.0	299	93.1	75.4	19.7	172	49.7	769	1.84
x56.5	0.556	7 250	63.2	278	93.5	76.3	17.1	150	48.7	559	1.43
x50.5	0.499	6 500	57.4	256	94.1	77.8	14.7	129	47.7	389	1.09
WT305											
x46	0.453	5 850	54.8	256	96.5	87.9	7.20	80.5	35.0	354	1.05
x41	0.402	5 250	48.7	231	96.6	89.1	6.04	67.9	34.0	243	0.785
WT265											
x109.5	1.07	14 000	85.0	388	78.1	60.8	78.4	493	75.0	3 200	8.74
x98	0.964	12 500	75.2	345	77.5	59.0	69.3	438	74.4	2 340	6.28
x91	0.891	11 600	69.3	317	77.3	57.9	63.6	404	74.1	1 860	4.94
x82.5	0.811	10 600	62.2	288	76.9	56.7	56.8	363	73.4	1 410	3.70
x75	0.739	9 600	56.5	261	76.7	55.5	51.4	330	73.2	1 080	2.79
WT265											
x69	0.679	8 800	60.2	293	82.6	69.6	19.3	181	46.8	1 250	2.50
x61.5	0.604	7 850	52.6	258	81.9	67.6	16.9	159	46.4	899	1.75
x54.5	0.535	6 950	46.2	227	81.5	66.1	14.8	140	46.1	630	1.20
x50.5	0.498	6 450	43.0	212	81.6	65.9	13.5	128	45.6	507	0.973
x46	0.454	5 900	39.3	196	81.7	66.0	11.9	114	44.9	380	0.754
x41	0.403	5 250	35.0	178	81.9	66.9	10.1	97.0	44.0	258	0.555
WT265											
x42.5	0.416	5 400	37.8	194	83.7	72.7	6.32	76.1	34.2	367	0.675
x37	0.367	4 740	33.7	177	84.1	74.7	5.21	62.7	33.1	239	0.516
x33	0.323	4 200	29.8	159	84.3	76.1	4.29	52.0	32.0	159	0.380

STRUCTURAL TEES
Cut from W Shapes
WT345 - WT265



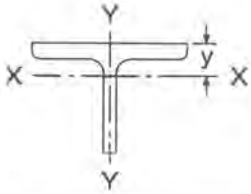
PROPERTIES AND DIMENSIONS

Nominal Mass (kg/m)	Theoretical Mass (kg/m)	Depth d (mm)	Flange Width b (mm)	Flange Thickness t (mm)	Stem Thickness w (mm)	β_x (mm)	y_o (mm)	$\bar{r}_o$ (mm)	Ω
85	85.0	347	256	23.6	14.5	245	75.3	141	0.715
76	76.0	344	254	21.1	13.1	244	75.3	140	0.713
70	69.9	342	254	18.9	12.4	244	77.1	141	0.703
62.5	62.8	339	253	16.3	11.7	244	80.2	143	0.685
120.5	120.9	318	329	31.0	17.9	195	53.1	129	0.832
108.5	108.9	314	328	27.7	16.5	194	53.5	129	0.828
97.5	97.8	311	327	24.4	15.4	193	55.2	129	0.818
87	87.1	308	325	21.6	14.0	193	55.7	129	0.813
77.5	77.5	306	324	19.0	12.7	193	56.6	129	0.808
70	70.1	309	230	22.2	13.1	216	65.1	124	0.726
62.5	62.5	306	229	19.6	11.9	215	65.6	124	0.721
56.5	56.7	304	228	17.3	11.2	216	67.7	125	0.708
50.5	50.9	302	228	14.9	10.5	216	70.3	127	0.692
46	46.2	302	179	15.0	10.9	227	80.4	130	0.620
41	41.0	300	178	12.8	10.0	228	82.7	132	0.606
109.5	109.5	280	318	29.2	18.3	163	46.2	118	0.846
98	98.3	277	316	26.3	16.5	163	45.9	117	0.846
91	90.9	276	315	24.4	15.2	163	45.7	116	0.846
82.5	82.7	273	313	22.2	14.0	162	45.6	116	0.845
75	75.4	272	312	20.3	12.7	162	45.4	115	0.845
69	69.2	275	214	23.6	14.7	189	57.8	111	0.730
61.5	61.6	272	212	21.2	13.1	188	57.0	110	0.731
54.5	54.6	270	211	18.8	11.6	188	56.7	109	0.732
50.5	50.8	269	210	17.4	10.9	188	57.2	110	0.728
46	46.3	267	209	15.6	10.2	188	58.2	110	0.719
41	41.1	264	209	13.3	9.5	187	60.2	111	0.704
42.5	42.4	268	166	16.5	10.3	197	64.4	111	0.663
37	37.4	265	166	13.6	9.7	197	67.9	113	0.639
33	32.9	263	165	11.4	8.9	198	70.4	114	0.621

Note: β_x is positive when the flange is in flexural compression, and negative otherwise.

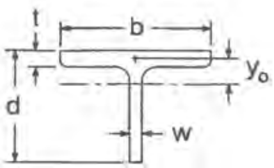
See S16-14 Clauses 13.3.2 and 13.6 and the Commentary in Part 2 for further information on section properties.

STRUCTURAL TEES
Cut from W Shapes
WT230 - WT205



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y			Torsional Constant	Warping Constant
			I_x	S_x	r_x	y	I_y	S_y	r_y	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^4$	$10^9 mm^6$
WT230											
x88.5	0.869	11 300	49.4	260	66.1	51.4	52.5	367	68.2	2 190	4.66
x79	0.773	10 000	43.5	231	65.8	50.1	45.7	322	67.4	1 550	3.25
x72	0.709	9 200	39.0	208	65.1	48.4	41.8	295	67.4	1 220	2.49
x64	0.630	8 150	34.5	185	64.9	47.2	36.7	260	66.9	855	1.74
x56.5	0.555	7 200	30.2	162	64.7	46.0	31.7	226	66.3	589	1.18
WT230											
x53	0.519	6 700	32.8	185	69.8	57.5	12.6	130	43.2	725	1.07
x48.5	0.473	6 150	29.4	166	69.1	55.8	11.4	118	43.1	562	0.802
x44.5	0.438	5 700	27.0	152	68.8	54.8	10.5	109	42.9	452	0.630
x41	0.402	5 250	24.8	141	68.9	54.8	9.31	97.5	42.2	344	0.493
x37	0.364	4 740	22.4	128	68.8	54.1	8.30	87.4	41.9	257	0.366
WT230											
x34	0.336	4 360	21.8	128	70.6	59.2	4.70	61.1	32.8	253	0.323
x30	0.292	3 800	18.8	111	70.4	58.3	3.98	52.0	32.4	167	0.213
x26	0.255	3 320	16.7	102	71.0	60.8	3.17	41.7	30.9	104	0.160
WT205											
x74.5	0.732	9 500	32.2	188	58.2	44.9	38.8	293	63.9	1 600	2.79
x66	0.648	8 450	28.0	165	57.7	43.3	33.7	256	63.3	1 120	1.92
x57	0.561	7 300	23.8	141	57.2	41.6	28.6	219	62.7	740	1.24
x50	0.489	6 350	20.3	121	56.6	39.8	24.8	191	62.5	495	0.810
WT205											
x42.5	0.417	5 400	20.4	127	61.3	49.3	9.02	99.6	40.8	461	0.536
x37	0.368	4 740	17.8	112	61.1	48.2	7.79	86.6	40.4	317	0.366
x33.5	0.331	4 290	15.8	100	60.7	47.3	6.90	77.0	40.1	233	0.265
x30	0.292	3 800	13.9	87.8	60.5	46.1	6.02	67.7	39.9	163	0.180
x27	0.262	3 420	12.8	83.3	61.4	48.1	5.05	57.0	38.5	112	0.139
WT205											
x23	0.227	2 940	11.5	76.3	62.4	51.5	2.57	36.7	29.5	95.6	0.099 0
x19.5	0.192	2 480	9.95	67.9	63.1	53.5	2.02	28.8	28.4	55.0	0.067 5



STRUCTURAL TEES
Cut from W Shapes
WT230 - WT205

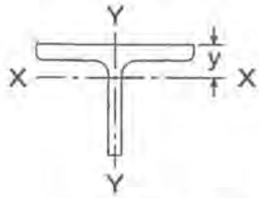
PROPERTIES AND DIMENSIONS

Nominal Mass (kg/m)	Theoretical Mass (kg/m)	Depth d (mm)	Flange Width b (mm)	Flange Thickness t (mm)	Stem Thickness w (mm)	β_x (mm)	y_o (mm)	$\bar{r}_o$ (mm)	Ω
88.5	88.6	241	286	26.9	16.6	135	38.0	102	0.862
79	78.8	238	284	23.9	15.0	135	38.1	102	0.859
72	72.3	236	283	22.1	13.6	134	37.3	101	0.863
64	64.3	234	282	19.6	12.2	134	37.4	100	0.861
56.5	56.6	232	280	17.3	10.8	134	37.4	99.9	0.860
53	52.9	235	194	20.6	12.6	158	47.2	94.6	0.752
48.5	48.3	233	193	19.0	11.4	158	46.3	93.7	0.756
44.5	44.7	232	192	17.7	10.5	157	45.9	93.2	0.757
41	41.0	230	191	16.0	9.9	157	46.8	93.4	0.749
37	37.1	229	190	14.5	9.0	157	46.8	93.2	0.747
34	34.3	230	154	15.4	9.1	165	51.5	93.4	0.696
30	29.8	228	153	13.3	8.0	165	51.7	93.2	0.692
26	26.0	225	152	10.8	7.6	165	55.4	95.2	0.661
74.5	74.7	216	265	25.0	14.9	118	32.4	92.3	0.877
66	66.1	213	263	22.2	13.3	117	32.2	91.5	0.876
57	57.2	210	261	19.3	11.6	116	31.9	90.6	0.876
50	49.8	208	260	16.9	10.0	116	31.4	90.0	0.878
42.5	42.5	209	181	18.2	10.9	139	40.2	83.9	0.770
37	37.5	207	180	16.0	9.7	139	40.2	83.6	0.768
33.5	33.7	205	179	14.4	8.8	138	40.1	83.0	0.767
30	29.8	204	178	12.8	7.7	138	39.7	82.6	0.769
27	26.7	202	177	10.9	7.5	138	42.6	84.0	0.743
23	23.1	202	140	11.2	7.0	146	45.9	82.9	0.694
19.5	19.6	200	140	8.8	6.4	147	49.1	84.9	0.665

Note: β_x is positive when the flange is in flexural compression, and negative otherwise.

See S16-14 Clauses 13.3.2 and 13.6 and the Commentary in Part 2 for further information on section properties.

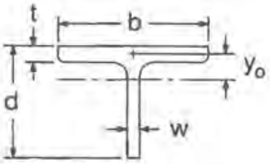
STRUCTURAL TEES
Cut from W Shapes
WT180



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y			Torsional Constant	Warping Constant
			I_x	S_x	r_x	y	I_y	S_y	r_y	J	C_w
	kN/m	mm^2	$10^6\ mm^4$	$10^3\ mm^3$	mm	mm	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^4$	$10^9\ mm^6$
WT180											
x543	5.34	69 500	308	1 570	66.7	88.2	981	4 320	119	299 000	1 410
x495	4.86	63 000	259	1 350	64.1	82.7	867	3 870	117	232 000	1 060
x450	4.43	57 500	219	1 160	61.7	77.5	767	3 470	115	180 000	791
x409	4.02	52 500	184	997	59.4	72.4	678	3 100	114	138 000	585
x372	3.65	47 400	156	864	57.4	67.9	600	2 780	112	106 000	434
x338.5	3.33	43 200	134	754	55.8	63.9	534	2 500	111	81 500	325
x317	3.11	40 300	119	677	54.3	61.0	491	2 320	110	68 300	266
x296	2.91	37 800	108	620	53.5	58.6	451	2 140	109	56 400	215
x275.5	2.70	35 200	95.9	557	52.3	55.8	412	1 970	108	45 900	172
x254.5	2.50	32 600	84.8	499	51.1	53.0	377	1 810	108	36 700	135
x231.5	2.27	29 500	73.8	439	50.0	50.1	335	1 630	107	28 100	100
x210.5	2.07	26 800	64.2	387	48.9	47.3	300	1 470	106	21 600	75.5
x191	1.87	24 400	55.4	338	47.7	44.5	268	1 320	105	16 300	56.0
x173.5	1.70	22 100	48.5	300	46.8	42.1	240	1 190	104	12 300	41.6
x157	1.54	20 000	42.6	266	46.2	39.9	213	1 060	103	9 210	30.3
x143.5	1.41	18 300	37.6	236	45.3	37.9	194	972	103	7 220	23.5
x131	1.29	16 700	33.9	215	45.0	36.4	175	880	102	5 500	17.6
x118.5	1.16	15 000	29.1	187	44.0	34.3	155	786	102	4 080	12.8
x108	1.06	13 800	26.2	169	43.6	32.9	141	717	101	3 150	9.79
WT180											
x98	0.964	12 500	24.0	157	43.8	32.7	114	611	95.6	2 560	7.17
x89.5	0.879	11 400	21.6	141	43.5	31.4	103	554	95.2	1 950	5.40
x81	0.794	10 300	18.8	124	42.7	29.8	92.8	500	94.9	1 470	4.00
x73.5	0.723	9 400	17.0	113	42.6	28.9	83.6	452	94.3	1 110	2.98
x67	0.657	8 550	15.2	101	42.2	27.8	75.4	409	94.0	839	2.22
WT180											
x61	0.597	7 750	17.3	118	47.2	35.5	30.7	239	62.9	1 050	1.51
x55	0.540	7 050	15.0	102	46.2	33.5	27.8	218	63.0	799	1.12
x50.5	0.497	6 450	13.7	93.7	46.1	32.7	25.3	199	62.6	626	0.863
x45.5	0.446	5 750	12.1	83.5	45.8	31.7	22.4	176	62.2	456	0.617
WT180											
x39.5	0.388	5 050	11.5	81.2	47.8	35.0	12.1	118	48.9	405	0.394
x36	0.350	4 550	10.3	73.1	47.5	34.2	10.7	105	48.5	300	0.286
x32	0.314	4 060	9.17	65.2	47.4	33.4	9.42	92.8	48.1	218	0.202

STRUCTURAL TEES
Cut from W Shapes
WT180



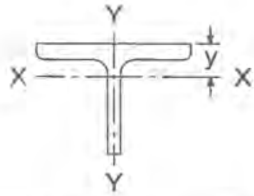
PROPERTIES AND DIMENSIONS

Nominal Mass	Theoretical Mass	Depth d	Flange Width b	Flange Thickness t	Stem Thickness w	β_x	y_o	$\bar{r}_o$	Ω
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	
543	544.2	285	454	125	78.0	61.7	25.7	139	0.966
495	495.5	275	448	115	71.9	58.4	25.2	136	0.966
450	451.3	266	442	106	65.9	55.5	24.5	133	0.966
409	409.5	257	437	97.0	60.5	52.3	23.9	131	0.966
372	372.1	249	432	88.9	55.6	49.6	23.5	128	0.967
338.5	339.1	242	428	81.5	51.2	47.3	23.1	127	0.967
317	317.1	237	424	77.1	47.6	45.4	22.4	125	0.968
296	296.5	233	421	72.3	45.0	44.8	22.4	124	0.967
275.5	275.5	228	418	67.6	42.0	42.8	22.0	122	0.968
254.5	254.7	223	416	62.7	39.1	40.4	21.6	121	0.968
231.5	231.5	218	412	57.4	35.8	39.3	21.4	120	0.968
210.5	210.9	213	409	52.6	32.8	37.3	21.0	118	0.969
191	191.2	208	406	48.0	29.8	35.1	20.5	117	0.969
173.5	173.6	204	404	43.7	27.2	33.7	20.2	116	0.970
157	156.8	200	401	39.6	24.9	32.7	20.1	115	0.969
143.5	143.9	197	399	36.6	22.6	31.5	19.6	114	0.970
131	131.4	194	398	33.3	21.1	30.5	19.8	113	0.970
118.5	118.1	190	395	30.2	18.9	28.6	19.2	112	0.971
108	108.2	188	394	27.7	17.3	28.2	19.0	112	0.971
98	98.3	186	374	26.2	16.4	37.2	19.6	107	0.966
89.5	89.6	184	373	23.9	15.0	36.7	19.5	106	0.966
81	81.0	182	371	21.8	13.3	36.3	18.9	106	0.968
73.5	73.7	180	370	19.8	12.3	35.8	19.0	105	0.967
67	67.0	178	369	18.0	11.2	35.0	18.8	105	0.968
61	60.9	182	257	21.7	13.0	87.4	24.6	82.4	0.911
55	55.1	180	256	19.9	11.4	86.4	23.6	81.6	0.916
50.5	50.6	179	255	18.3	10.5	86.8	23.6	81.3	0.916
45.5	45.4	177	254	16.4	9.5	86.3	23.5	80.8	0.915
39.5	39.6	177	205	16.8	9.4	102	26.6	73.4	0.868
36	35.7	175	204	15.1	8.6	102	26.7	73.0	0.866
32	32.0	174	203	13.5	7.7	102	26.6	72.6	0.865

Note: β_x is positive when the flange is in flexural compression, and negative otherwise.

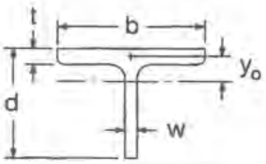
See S16-14 Clauses 13.3.2 and 13.6 and the Commentary in Part 2 for further information on section properties.

STRUCTURAL TEES
Cut from W Shapes
WT180 - WT155



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y			Torsional Constant	Warping Constant
			I_x	S_x	r_x	y	I_y	S_y	r_y	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^4$	$10^9 mm^6$
WT180											
x28.5	0.278	3 620	9.70	69.4	51.9	39.2	5.56	64.7	39.3	166	0.150
x25.5	0.248	3 220	8.73	62.8	52.0	39.0	4.84	56.6	38.7	118	0.107
x22.5	0.221	2 860	7.96	58.6	52.7	40.2	4.09	47.8	37.8	79.4	0.078 4
WT180											
x19.5	0.192	2 480	7.28	54.7	54.0	43.8	1.88	29.3	27.4	74.9	0.056 4
x16.5	0.161	2 100	6.19	47.6	54.5	44.8	1.45	22.9	26.4	42.6	0.035 7
WT155											
x250	2.46	31 800	79.7	513	50.0	58.7	247	1 450	88.0	50 100	130
x227	2.23	28 900	68.2	446	48.6	55.2	218	1 300	86.8	38 300	95.7
x207.5	2.04	26 400	59.6	397	47.4	52.2	195	1 170	85.9	29 500	71.9
x187.5	1.84	23 900	50.5	343	46.0	48.9	172	1 040	84.8	22 300	52.5
x171	1.68	21 800	43.8	302	44.8	46.1	155	946	84.2	17 300	40.0
x156.5	1.54	20 000	38.5	269	43.9	43.8	139	852	83.3	13 400	30.1
x141.5	1.39	18 000	33.1	233	42.8	41.1	123	764	82.6	10 100	22.1
x126.5	1.24	16 200	28.1	202	41.8	38.6	107	673	81.6	7 340	15.6
x113	1.11	14 400	24.3	176	41.0	36.4	94.6	597	81.0	5 350	11.1
x101	0.994	12 800	21.3	156	40.7	34.6	82.9	527	80.1	3 850	7.82
x89.5	0.877	11 400	18.2	135	39.9	32.5	71.9	459	79.4	2 680	5.30
x79	0.772	10 000	15.2	114	38.9	30.3	62.4	402	78.9	1 880	3.63
x71.5	0.702	9 100	13.5	101	38.4	28.9	56.3	365	78.6	1 430	2.72
x64.5	0.635	8 250	12.0	91.8	38.2	27.9	50.2	326	78.0	1 060	1.98
x59	0.576	7 500	10.7	82.0	37.8	26.8	45.1	294	77.6	798	1.46
x53.5	0.525	6 800	9.71	74.7	37.7	26.0	40.6	265	77.2	605	1.09
x48.5	0.475	6 150	8.59	66.6	37.3	25.0	36.4	239	76.9	454	0.804
WT155											
x43	0.423	5 500	7.93	61.5	38.0	26.1	22.3	175	63.6	436	0.559
x39.5	0.387	5 050	7.38	58.1	38.3	26.1	20.0	157	63.0	327	0.413
WT155											
x37	0.363	4 740	7.80	62.3	40.7	29.7	11.7	114	49.9	358	0.332
x33.5	0.325	4 260	6.88	55.3	40.3	28.8	10.3	101	49.5	260	0.236
x30	0.290	3 800	6.05	48.7	40.0	27.7	9.14	90.1	49.2	189	0.167
WT155											
x26	0.257	3 320	6.66	52.9	44.7	33.1	5.13	61.4	39.2	154	0.118
x22.5	0.219	2 840	5.64	45.2	44.5	32.2	4.27	51.5	38.7	95.5	0.072 3
x19.5	0.190	2 470	4.82	39.0	44.2	31.4	3.63	44.0	38.4	62.8	0.046 8



STRUCTURAL TEES
Cut from W Shapes
WT180 - WT155

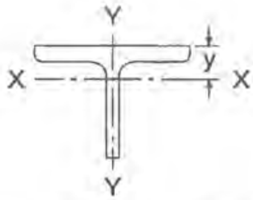
PROPERTIES AND DIMENSIONS

Nominal Mass	Theoretical Mass	Depth d	Flange Width b	Flange Thickness t	Stem Thickness w	β_x	y_o	$\bar{r}_o$	Ω
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	
28.5	28.3	179	172	13.1	7.9	116	32.6	72.8	0.799
25.5	25.3	178	171	11.6	7.2	116	33.2	72.9	0.793
22.5	22.5	176	171	9.8	6.9	117	35.3	73.8	0.771
19.5	19.6	177	128	10.7	6.5	126	38.5	71.8	0.712
16.5	16.4	175	127	8.5	5.8	127	40.6	72.9	0.690
250	250.4	214	340	75.1	45.1	56.3	21.1	103	0.958
227	227.1	208	336	68.7	41.3	54.9	20.8	102	0.958
207.5	207.7	202	334	62.7	38.9	52.7	20.8	100	0.957
187.5	187.5	196	330	57.2	35.4	50.7	20.3	98.6	0.958
171	171.6	191	328	52.6	32.6	48.4	19.8	97.4	0.959
156.5	156.6	187	325	48.3	30.0	47.7	19.6	96.2	0.958
141.5	141.6	183	322	44.1	26.9	46.7	19.1	95.0	0.960
126.5	126.4	178	319	39.6	24.4	44.9	18.8	93.6	0.960
113	113.4	174	317	35.6	22.1	43.5	18.6	92.6	0.960
101	101.4	171	315	31.8	20.1	43.4	18.7	91.8	0.959
89.5	89.4	167	313	28.1	18.0	41.9	18.5	90.8	0.959
79	78.7	164	310	25.1	15.5	41.3	17.8	89.7	0.961
71.5	71.6	162	309	22.9	14.0	40.7	17.5	89.2	0.962
64.5	64.8	159	308	20.6	13.1	39.2	17.6	88.6	0.960
59	58.7	157	307	18.7	11.9	38.5	17.4	88.1	0.961
53.5	53.5	156	306	17.0	10.9	39.0	17.5	87.7	0.960
48.5	48.4	154	305	15.4	9.9	38.2	17.3	87.2	0.961
43	43.2	155	254	16.3	9.1	61.5	18.0	76.3	0.944
39.5	39.4	153	254	14.6	8.8	60.9	18.8	76.1	0.939
37	37.0	155	205	16.3	9.4	80.5	21.6	67.9	0.899
33.5	33.2	153	204	14.6	8.5	79.9	21.5	67.4	0.898
30	29.6	152	203	13.1	7.5	80.1	21.2	66.9	0.900
26	26.2	159	167	13.2	7.6	98.0	26.5	65.1	0.835
22.5	22.3	157	166	11.2	6.6	97.7	26.6	64.7	0.831
19.5	19.4	155	165	9.7	5.8	97.0	26.5	64.3	0.830

Note: β_x is positive when the flange is in flexural compression, and negative otherwise.

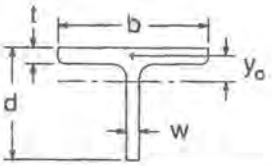
See S16-14 Clauses 13.3.2 and 13.6 and the Commentary in Part 2 for further information on section properties.

STRUCTURAL TEES
Cut from W Shapes
WT155 - WT100



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y			Torsional Constant	Warping Constant
			I_x	S_x	r_x	y	I_y	S_y	r_y	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^4$	$10^9 mm^6$
WT155											
x16.5	0.161	2 090	4.92	42.7	48.5	41.7	0.959	18.8	21.4	60.7	0.037 1
x14	0.139	1 800	4.27	37.8	48.6	42.1	0.790	15.5	20.9	37.8	0.025 7
x12	0.117	1 520	3.67	33.8	49.1	44.6	0.578	11.4	19.5	21.2	0.018 5
x10.5	0.104	1 340	3.25	30.3	49.1	45.0	0.491	9.73	19.1	14.6	0.013 6
WT125											
x83.5	0.821	10 600	12.1	106	33.7	30.8	49.4	373	68.0	3 140	4.58
x74.5	0.730	9 500	10.2	91.3	32.9	28.8	43.1	328	67.4	2 250	3.19
x65.5	0.643	8 350	8.73	78.7	32.3	27.0	37.2	285	66.7	1 560	2.15
x57.5	0.563	7 300	7.34	66.8	31.7	25.2	32.0	247	66.2	1 060	1.43
x50.5	0.496	6 450	6.17	57.0	31.0	23.6	27.7	216	65.6	741	0.973
x44.5	0.439	5 700	5.39	50.2	30.7	22.5	24.2	189	65.1	517	0.664
x40	0.393	5 100	4.61	43.2	30.1	21.2	21.6	169	65.0	377	0.477
x36.5	0.358	4 640	4.17	39.2	30.0	20.5	19.4	153	64.6	287	0.356
WT125											
x33.5	0.330	4 290	4.34	41.0	31.8	23.2	11.1	109	51.0	312	0.263
x29	0.286	3 710	3.68	35.5	31.5	22.2	9.42	92.8	50.4	204	0.167
x24.5	0.241	3 130	3.25	32.0	32.2	22.3	7.56	74.9	49.2	120	0.094 9
WT125											
x22.5	0.220	2 850	3.86	36.7	36.7	27.8	3.52	47.5	35.1	130	0.074 1
x19.5	0.190	2 460	3.26	31.3	36.4	26.7	2.97	40.4	34.7	84.1	0.046 7
x16.5	0.160	2 100	2.85	28.1	37.0	27.3	2.36	32.4	33.7	49.1	0.028 4
WT125											
x14	0.140	1 820	2.79	28.7	39.2	32.6	0.888	17.4	22.1	48.2	0.021 6
x12.5	0.124	1 610	2.56	26.8	39.8	33.6	0.746	14.6	21.5	32.5	0.016 6
x11	0.110	1 420	2.27	24.5	39.9	34.6	0.613	12.0	20.7	21.6	0.012 6
x9	0.087 7	1 140	1.83	20.0	40.1	34.8	0.457	9.04	20.0	11.2	0.006 8
WT100											
x50	0.488	6 350	4.61	50.6	27.0	23.9	18.3	174	53.7	1 040	0.949
x43	0.425	5 500	3.80	42.8	26.2	22.2	15.7	150	53.3	694	0.617
x35.5	0.350	4 550	2.86	32.5	25.1	19.8	12.7	123	52.8	407	0.349
x29.5	0.291	3 780	2.39	27.7	25.1	18.7	10.2	99.5	52.0	231	0.191
x26	0.256	3 320	2.00	23.4	24.5	17.5	8.92	87.4	51.8	161	0.130
x23	0.226	2 940	1.79	21.1	24.7	17.0	7.67	75.6	51.2	110	0.086 6
WT100											
x21	0.205	2 660	1.78	21.2	25.9	18.8	4.50	54.2	41.2	111	0.061 7
x18	0.176	2 280	1.48	17.7	25.4	17.7	3.82	46.3	40.9	72.5	0.038 9
WT100											
x15.5	0.154	1 980	1.63	19.4	28.5	21.1	2.05	30.6	32.0	59.4	0.025 0
x13.5	0.131	1 700	1.43	17.3	29.1	21.3	1.65	24.8	31.2	35.5	0.015 1



STRUCTURAL TEES
Cut from W Shapes
WT155 - WT100

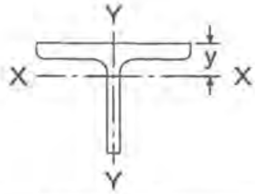
PROPERTIES AND DIMENSIONS

Nominal Mass (kg/m)	Theoretical Mass (kg/m)	Depth d (mm)	Flange Width b (mm)	Flange Thickness t (mm)	Stem Thickness w (mm)	β_x (mm)	y_o (mm)	$\bar{r}_o$ (mm)	Ω
16.5	16.4	157	102	10.8	6.6	114	36.3	64.3	0.681
14	14.2	155	102	8.9	6.0	113	37.6	64.9	0.664
12	11.9	153	101	6.7	5.6	115	41.3	67.1	0.621
10.5	10.6	152	101	5.7	5.1	115	42.2	67.5	0.609
83.5	83.8	145	265	31.8	19.2	34.3	14.9	77.4	0.963
74.5	74.5	141	263	28.4	17.3	32.5	14.6	76.4	0.963
65.5	65.6	138	261	25.1	15.4	31.9	14.5	75.6	0.963
57.5	57.4	135	259	22.1	13.5	31.0	14.2	74.7	0.964
50.5	50.6	132	257	19.6	11.9	29.8	13.8	73.9	0.965
44.5	44.8	130	256	17.3	10.7	29.5	13.9	73.4	0.964
40	40.1	128	255	15.6	9.4	28.4	13.4	72.9	0.966
36.5	36.5	127	254	14.2	8.6	28.7	13.4	72.5	0.966
33.5	33.6	129	204	15.7	8.9	52.9	15.4	62.0	0.939
29	29.1	126	203	13.5	8.0	51.6	15.5	61.4	0.937
24.5	24.6	124	202	11.0	7.4	52.3	16.8	61.1	0.925
22.5	22.5	133	148	13.0	7.6	78.6	21.3	55.1	0.851
19.5	19.3	131	147	11.2	6.6	78.1	21.1	54.5	0.850
16.5	16.4	129	146	9.1	6.1	78.3	22.7	54.9	0.829
14	14.2	130	102	10.0	6.4	90.0	27.6	52.9	0.727
12.5	12.7	129	102	8.4	6.1	90.7	29.4	53.9	0.702
11	11.2	127	102	6.9	5.8	90.6	31.1	54.7	0.676
9	8.9	126	101	5.3	4.8	91.3	32.1	55.1	0.660
50	49.8	115	210	23.7	14.5	28.3	12.1	61.3	0.961
43	43.4	111	209	20.6	13.0	25.9	11.9	60.6	0.961
35.5	35.7	108	206	17.4	10.2	25.4	11.1	59.5	0.965
29.5	29.7	105	205	14.2	9.1	24.7	11.6	58.9	0.961
26	26.1	103	204	12.6	7.9	23.6	11.2	58.4	0.963
23	23.0	102	203	11.0	7.2	24.2	11.5	58.0	0.961
21	20.9	103	166	11.8	7.2	41.8	12.9	50.3	0.935
18	18.0	101	165	10.2	6.2	41.0	12.6	49.7	0.936
15.5	15.7	105	134	10.2	6.4	57.1	16.0	45.8	0.877
13.5	13.3	104	133	8.4	5.8	57.9	17.1	45.9	0.862

Note: β_x is positive when the flange is in flexural compression, and negative otherwise.

See S16-14 Clauses 13.3.2 and 13.6 and the Commentary in Part 2 for further information on section properties.

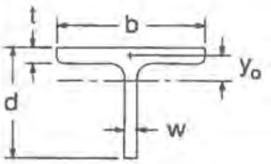
STRUCTURAL TEES
Cut from W Shapes
WT100 - WT50



PROPERTIES

Designation	Dead Load	Area	Axis X-X				Axis Y-Y			Torsional Constant	Warping Constant
			I_x	S_x	r_x	y	I_y	S_y	r_y	J	C_w
	kN/m	mm^2	$10^6 mm^4$	$10^3 mm^3$	mm	mm	$10^6 mm^4$	$10^3 mm^3$	mm	$10^3 mm^4$	$10^9 mm^6$
WT100											
x11	0.110	1 430	1.36	17.5	30.9	25.3	0.710	13.9	22.3	28.1	0.010 2
x9.5	0.095 6	1 240	1.22	16.0	31.3	26.1	0.577	11.3	21.6	18.0	0.007 2
x7.5	0.073 3	955	0.885	11.7	30.5	24.1	0.434	8.69	21.4	8.75	0.003 0
WT75											
x18.5	0.182	2 370	0.947	14.5	20.0	15.5	3.53	45.9	38.7	95.5	0.045 9
x15	0.146	1 900	0.725	11.3	19.6	14.2	2.78	36.3	38.3	49.9	0.023 2
x11	0.109	1 430	0.581	9.38	20.2	14.1	1.93	25.4	36.9	20.6	0.009 1
WT75											
x12	0.117	1 530	0.708	11.3	21.5	17.3	0.913	17.9	24.5	46.0	0.011 4
x9	0.087 9	1 140	0.544	9.16	21.8	17.1	0.629	12.3	23.5	18.3	0.004 7
x7	0.066 5	865	0.395	6.68	21.4	15.8	0.459	9.18	23.0	8.35	0.002 0
WT65											
x14	0.138	1 800	0.426	8.02	15.4	12.4	1.91	29.8	32.7	63.4	0.020 8
x12	0.116	1 520	0.350	6.74	15.3	11.6	1.55	24.5	32.2	38.0	0.012 0
WT50											
x9.5	0.095 1	1 240	0.221	5.28	13.4	11.2	0.803	15.6	25.5	31.2	0.006 3

STRUCTURAL TEES
Cut from W Shapes
WT100 - WT50



PROPERTIES AND DIMENSIONS

Nominal Mass	Theoretical Mass	Depth d	Flange Width b	Flange Thickness t	Stem Thickness w	β_x	y_o	$\bar{r}_o$	Ω
kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	
11	11.2	103	102	8.0	6.2	67.0	21.3	43.6	0.761
9.5	9.7	102	102	6.5	5.8	67.6	22.8	44.3	0.734
7.5	7.5	100	100	5.2	4.3	66.7	21.5	43.0	0.750
18.5	18.6	81.0	154	11.6	8.1	21.0	9.68	44.6	0.953
15	14.9	78.5	153	9.3	6.6	19.9	9.51	44.0	0.953
11	11.2	76.0	152	6.6	5.8	20.1	10.8	43.4	0.938
12	12.0	80.0	102	10.3	6.6	42.0	12.1	34.8	0.878
9	9.0	76.5	102	7.1	5.8	41.2	13.5	34.8	0.848
7	6.8	75.0	100	5.5	4.3	41.2	13.0	34.0	0.853
14	14.0	65.5	128	10.9	6.9	13.8	6.96	36.8	0.964
12	11.8	63.5	127	9.1	6.1	13.1	7.07	36.3	0.962
9.5	9.7	53.0	103	8.8	7.1	12.1	6.80	29.6	0.947

Note: β_x is positive when the flange is in flexural compression, and negative otherwise.

See S16-14 Clauses 13.3.2 and 13.6 and the Commentary in Part 2 for further information on section properties.

HOLLOW STRUCTURAL SECTIONS

General

Manufacturers of Hollow Structural Sections (HSS) may produce HSS to meet the requirements of either CSA Standard G40.20/G40.21, ASTM Specification A500 or ASTM A1085. The availability of HSS to these standards or specifications varies across the different regions of Canada. Round sections produced in accordance with common pipe specifications may sometimes be used as structural members, but are not classified as HSS.

For information on steel grades, manufacturing tolerances and Class of HSS, see *Standard Mill Practice* in Part 6.

Availability

Since the sections listed in this Handbook are those best suited for structural applications, designers may wish to consult the catalogs of HSS producers supplying HSS to their region of the country for sections not listed herein.

When a particular Hollow Structural Section is listed under both CSA G40 and ASTM A500 steel grades in Part 6, choosing the most readily available grade for a project may depend on the project location. In Ontario, most HSS sizes are available in either G40 and A500 grades. In western Canada, square and rectangular sections are more readily available in G40, while round sections are mainly available in A500. In Atlantic Canada and in Quebec, A500 is the prevalent grade.

A number of sizes are identified with an asterisk (*), denoting imported sections which are produced by non-Canadian mills and may be subject to a cost premium.

HOLLOW STRUCTURAL SECTIONS PRODUCED TO CSA G40.20

General

Hollow Structural Sections (HSS) are produced in Canada to the requirements of the CSA G40.20 Standard to either Class C or Class H, from steel meeting the requirements of the CSA G40.21 material Standard. The common grade of steel used is G40.21-350W.

Manufacture

HSS produced to the CSA G40.20 Standard may be manufactured using either a seamless or a welding process. Seamless products are produced by piercing solid material to form a tube or by an extrusion-type process (but are uncommon). Welded products are manufactured from flat-rolled steel which is formed and joined by various welding processes into a tubular shape. The tubular shape is then either cold-formed or hot-formed to the final shape and, if cold-formed, may be subsequently stress-relieved. Class H sections are either hot-formed to final shape (uncommon today), or are cold-formed to final shape and then stress-relieved. Class C sections are generally more readily available than Class H sections, although Class H sections have greater resistance in axial compression. Outside dimensions for HSS are constant for all sizes in the same size range, with the inside dimensions changing with material thickness.

Properties and Dimensions

The tables of properties and dimensions on the following pages include square, rectangular and round HSS currently produced in Canada. The metric section sizes (e.g. HSS 127×76×6.4) include the outer dimensions (depth × width for rectangular sections) and wall thickness in millimetres.

Section properties given in the following tables for square and rectangular sections are based on an interior corner radius taken equal to the wall thickness, and on an exterior corner radius taken equal to twice the wall thickness.

HOLLOW STRUCTURAL SECTIONS PRODUCED TO ASTM A500

General

ASTM A500 grade C HSS may be the product of choice in some regions of Canada when CSA G40.21-350W HSS may not be available in the quantities and time frame envisaged for a specific project.

Manufacture

HSS manufactured to ASTM Standard A500 Grade C are not equivalent to HSS meeting the requirements of CSA G40.21 grade 350W. Unlike CSA Standard G40.20/G40.21, the ASTM A500 specification has no restriction for mass variation and has a tolerance of ± 10% on the wall thickness. If HSS produced to A500 are offered as a substitute, it would be prudent to assess the influence of the differences that arise from a possible difference in wall thickness and material strengths.

Properties and Dimensions

The tables of properties and dimensions on the following pages, prepared for HSS produced to ASTM A500 Grade C, include a quantity termed the "Design Wall Thickness". In accordance with to CSA S16-14 Clause 5.1.3, this Design Wall Thickness is taken as 90% of the nominal wall thickness. The nominal wall thickness is the thickness that has been published in previous tables as the "wall thickness" and, when rounded, forms the third term of the HSS section size.

With the exception of the Mass and the Dead Load, the values of Properties and Dimensions published in the following tables were computed based on the value of the "Design Wall Thickness".

Information on ASTM A500 Grade C

The following information is taken from ASTM A500-10a, "Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes". For complete information on HSS produced to ASTM A500 Grade C, please refer to the ASTM specification.

Mechanical Properties of ASTM A500 Grade C Steel *

HSS Shape	F_y (min) **	F_u (min)
Round HSS	317 MPa	427 MPa
Square and Rectangular HSS	345 MPa	427 MPa

* Clause 1.2 Note 1: Products manufactured to this specification may not be suitable for those applications such as dynamically loaded elements in welded structures, etc., where low-temperature notch-toughness properties may be important.

** Clause 15.3: The yield strength corresponding to an offset of 0.2% of the gage length or to a total extension under load of 0.5% of the gage length shall be determined.

HOLLOW STRUCTURAL SECTIONS PRODUCED TO ASTM A1085

General

ASTM A1085 was introduced in 2013. HSS produced to A1085 meet requirements comparable to those of CSA G40.20/21-350WT Category 1. The material is required to conform to a minimum average Charpy V-notch impact value of 34 Joules at 4°C, as represented by the test specimen. In addition, a minimum yield stress at 345 MPa and a maximum yield stress of 485 MPa apply.

Manufacture

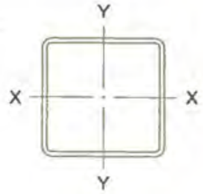
Square and rectangular A1085 HSS meet requirements for minimum and maximum corner radii as a function of wall thickness. See *Standard Mill Practice* in Part 6.

Purchasers of A1085 HSS may specify heat treatment as supplemental requirement S1, which also conforms to the stress-relieved requirement for Class H G40.20 HSS.

Properties and Dimensions

Wall thickness and mass tolerances for ASTM A1085 products are essentially the same as those specified for HSS in CSA G40.20. Section properties provided for CSA G40.20 HSS in Part 6, which are calculated from the nominal wall thickness, depth, width and diameter, may be used for design.

HOLLOW STRUCTURAL SECTIONS
CSA G40.20
Square

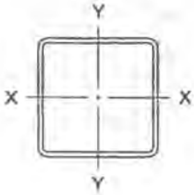


PROPERTIES AND DIMENSIONS

Section	Outside Dimen-sion	Wall Thick-ness	Mass	Dead Load	Area	I	S	r	Z	Torsional Constant J	Surface Area
mm x mm x mm	mm	mm	kg/m	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	m^2/ m
HSS 559x559											
x19*	558.8	19.05	316	3.10	40 200	1 930	6 900	219	8 070	3 050 000	2.17
HSS 508x508											
x22*	508.0	22.23	329	3.23	41 900	1 620	6 390	197	7 560	2 600 000	1.96
x19*	508.0	19.05	285	2.80	36 300	1 430	5 620	198	6 600	2 270 000	1.97
x16*	508.0	15.88	240	2.36	30 600	1 220	4 810	200	5 610	1 930 000	1.98
x13*	508.0	12.70	194	1.91	24 700	1 000	3 950	201	4 570	1 570 000	1.99
HSS 457x457											
x22*	457.2	22.23	294	2.88	37 400	1 160	5 070	176	6 030	1 870 000	1.75
x19*	457.2	19.05	255	2.50	32 500	1 020	4 470	178	5 280	1 640 000	1.76
x16*	457.2	15.88	215	2.11	27 400	878	3 840	179	4 490	1 390 000	1.77
x13*	457.2	12.70	174	1.71	22 200	723	3 160	181	3 670	1 130 000	1.79
HSS 406x406											
x22*	406.4	22.23	258	2.53	32 900	793	3 900	155	4 670	1 290 000	1.55
x19*	406.4	19.05	224	2.20	28 600	703	3 460	157	4 100	1 130 000	1.56
x16*	406.4	15.88	190	1.86	24 200	606	2 980	158	3 500	965 000	1.57
x13*	406.4	12.70	154	1.51	19 600	500	2 460	160	2 870	788 000	1.58
x9.5*	406.4	9.53	117	1.15	14 900	388	1 910	161	2 200	604 000	1.59
HSS 356x356											
x16*	355.6	15.88	164	1.61	20 900	396	2 230	138	2 640	637 000	1.37
x13*	355.6	12.70	133	1.31	17 000	329	1 850	139	2 170	522 000	1.38
x9.5*	355.6	9.53	102	0.998	13 000	256	1 440	141	1 670	401 000	1.39
x7.9*	355.6	7.94	85.4	0.838	10 900	218	1 220	141	1 410	338 000	1.40
HSS 305x305											
x16	304.8	15.88	139	1.36	17 700	242	1 590	117	1 890	392 000	1.16
x13	304.8	12.70	113	1.11	14 400	202	1 330	118	1 560	323 000	1.18
x9.5	304.8	9.53	86.5	0.849	11 000	158	1 040	120	1 210	250 000	1.19
x7.9	304.8	7.94	72.7	0.714	9 270	135	885	121	1 030	211 000	1.19
x6.4	304.8	6.35	58.7	0.576	7 480	110	723	121	833	171 000	1.20
HSS 254x254											
x16	254.0	15.88	114	1.11	14 500	134	1 050	96.1	1 270	220 000	0.961
x13	254.0	12.70	93.0	0.912	11 800	113	889	97.6	1 060	183 000	0.972
x9.5	254.0	9.53	71.3	0.700	9 090	89.3	703	99.1	825	142 000	0.983
x7.9	254.0	7.94	60.1	0.589	7 650	76.4	601	99.9	701	120 000	0.989
x6.4	254.0	6.35	48.6	0.476	6 190	62.7	494	101	571	97 800	0.994
x4.8	254.0	4.78	36.9	0.362	4 710	48.4	381	101	438	74 800	1.000
HSS 203x203											
x16	203.2	15.88	88.3	0.866	11 200	63.8	628	75.3	774	107 000	0.758
x13	203.2	12.70	72.7	0.713	9 260	54.7	538	76.9	651	90 200	0.769
x9.5	203.2	9.53	56.1	0.551	7 150	43.9	432	78.4	513	70 800	0.780
x7.9	203.2	7.94	47.4	0.465	6 040	37.8	372	79.2	438	60 300	0.786
x6.4	203.2	6.35	38.4	0.377	4 900	31.3	308	79.9	359	49 300	0.791
x4.8	203.2	4.78	29.3	0.288	3 730	24.3	239	80.7	276	37 800	0.796

* Imported section

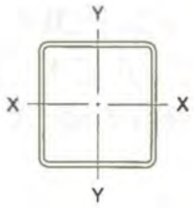
HOLLOW STRUCTURAL SECTIONS
CSA G40.20
Square



PROPERTIES AND DIMENSIONS

Section	Outside Dimension	Wall Thickness	Mass	Dead Load	Area	I	S	r	Z	Torsional Constant J	Surface Area
mm x mm x mm	mm	mm	kg/m	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	m^2/ m
HSS 178x178											
x16	177.8	15.88	75.6	0.742	9 640	40.6	457	64.9	571	69 300	0.657
x13	177.8	12.70	62.6	0.614	7 970	35.2	396	66.5	484	58 800	0.668
x9.5	177.8	9.53	48.5	0.476	6 180	28.6	322	68.0	385	46 500	0.678
x7.9	177.8	7.94	41.1	0.403	5 230	24.8	279	68.8	330	39 800	0.684
x6.4	177.8	6.35	33.4	0.327	4 250	20.6	231	69.6	271	32 600	0.689
x4.8	177.8	4.78	25.5	0.250	3 250	16.1	181	70.3	210	25 100	0.695
HSS 152x152											
x13	152.4	12.70	52.4	0.515	6 680	21.0	276	56.1	342	35 600	0.566
x9.5	152.4	9.53	40.9	0.401	5 210	17.3	227	57.6	275	28 500	0.577
x7.9	152.4	7.94	34.7	0.341	4 430	15.1	198	58.4	237	24 500	0.582
x6.4	152.4	6.35	28.3	0.278	3 610	12.6	166	59.2	196	20 200	0.588
x4.8	152.4	4.78	21.7	0.213	2 760	9.93	130	59.9	152	15 600	0.593
HSS 127x127											
x13	127.0	12.70	42.3	0.415	5 390	11.3	177	45.7	225	19 500	0.464
x9.5	127.0	9.53	33.3	0.327	4 240	9.48	149	47.3	183	15 900	0.475
x7.9	127.0	7.94	28.4	0.279	3 620	8.35	131	48.0	159	13 800	0.481
x6.4	127.0	6.35	23.2	0.228	2 960	7.05	111	48.8	132	11 400	0.486
x4.8	127.0	4.78	17.9	0.175	2 280	5.60	88.1	49.6	103	8 900	0.492
x3.2	127.0	3.18	12.2	0.119	1 550	3.92	61.8	50.3	71.5	6 120	0.497
HSS 102x102											
x13	101.6	12.70	32.2	0.316	4 100	5.10	100	35.3	131	9 070	0.363
x9.5	101.6	9.53	25.7	0.252	3 280	4.45	87.6	36.9	110	7 640	0.374
x7.9	101.6	7.94	22.1	0.217	2 810	3.99	78.5	37.7	96.7	6 710	0.379
x6.4	101.6	6.35	18.2	0.178	2 320	3.42	67.3	38.4	81.4	5 640	0.385
x4.8	101.6	4.78	14.1	0.138	1 790	2.75	54.2	39.2	64.3	4 440	0.390
x3.2	101.6	3.18	9.62	0.094	1 230	1.96	38.5	40.0	44.9	3 080	0.395
HSS 89x89											
x9.5	88.9	9.53	21.9	0.215	2 790	2.80	63.0	31.7	80.5	4 880	0.323
x7.9	88.9	7.94	18.9	0.186	2 410	2.54	57.1	32.5	71.3	4 330	0.328
x6.4	88.9	6.35	15.6	0.153	1 990	2.20	49.5	33.2	60.5	3 670	0.334
x4.8	88.9	4.78	12.2	0.119	1 550	1.79	40.3	34.0	48.2	2 920	0.339
HSS 76x76											
x9.5	76.2	9.53	18.1	0.178	2 310	1.61	42.4	26.5	55.5	2 870	0.272
x7.9	76.2	7.94	15.7	0.154	2 010	1.49	39.1	27.3	49.8	2 590	0.278
x6.4	76.2	6.35	13.1	0.129	1 670	1.31	34.5	28.0	42.8	2 230	0.283
x4.8	76.2	4.78	10.3	0.101	1 310	1.08	28.5	28.8	34.4	1 790	0.288
x3.2	76.2	3.18	7.09	0.070	903	0.790	20.7	29.6	24.5	1 260	0.294
HSS 64x64											
x6.4	63.5	6.35	10.6	0.104	1 350	0.703	22.2	22.8	28.1	1 220	0.232
x4.8	63.5	4.78	8.35	0.082	1 060	0.594	18.7	23.6	23.0	995	0.238
x3.2	63.5	3.18	5.82	0.057	741	0.441	13.9	24.4	16.6	715	0.243

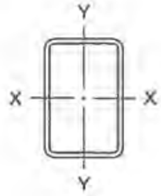
HOLLOW STRUCTURAL SECTIONS
CSA G40.20
Square



PROPERTIES AND DIMENSIONS

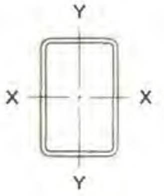
Section	Outside Dimension	Wall Thickness	Mass	Dead Load	Area	I	S	r	Z	Torsional Constant J	Surface Area
mm x mm x mm	mm	mm	kg/m	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	m^2/ m
HSS 51x51											
x6.4	50.8	6.35	8.05	0.079	1 030	0.319	12.6	17.6	16.4	567	0.181
x4.8	50.8	4.78	6.45	0.063	821	0.279	11.0	18.4	13.8	479	0.187
x3.2	50.8	3.18	4.55	0.045	580	0.214	8.42	19.2	10.2	353	0.192
HSS 38x38											
x4.8	38.1	4.78	4.54	0.045	578	0.101	5.30	13.2	6.95	180	0.136
x3.2	38.1	3.18	3.28	0.032	418	0.082 2	4.31	14.0	5.35	139	0.141

HOLLOW STRUCTURAL SECTIONS
CSA G40.20
Rectangular



DIMENSIONS

Section	Outside Dimensions		Wall Thickness	Mass	Dead Load	Area	Surface Area
	Depth	Width					
mm x mm x mm	mm	mm	mm	kg/m	kN/m	mm²	m²/ m
HSS 305x203							
x16	304.8	203.2	15.88	114	1.11	14 500	0.961
x13	304.8	203.2	12.70	93.0	0.912	11 800	0.972
x9.5	304.8	203.2	9.53	71.3	0.700	9 090	0.983
x7.9	304.8	203.2	7.94	60.1	0.589	7 650	0.989
x6.4	304.8	203.2	6.35	48.6	0.476	6 190	0.994
HSS 305x152							
x16	304.8	152.4	15.88	101	0.991	12 900	0.860
x13	304.8	152.4	12.70	82.8	0.813	10 600	0.871
x9.5	304.8	152.4	9.53	63.7	0.625	8 120	0.882
x7.9	304.8	152.4	7.94	53.7	0.527	6 850	0.887
x6.4	304.8	152.4	6.35	43.5	0.427	5 540	0.893
HSS 254x203							
x16	254.0	203.2	15.88	101	0.991	12 900	0.860
x13	254.0	203.2	12.70	82.8	0.813	10 600	0.871
x9.5	254.0	203.2	9.53	63.7	0.625	8 120	0.882
x7.9	254.0	203.2	7.94	53.7	0.527	6 850	0.887
x6.4	254.0	203.2	6.35	43.5	0.427	5 540	0.893
HSS 254x152							
x16	254.0	152.4	15.88	88.3	0.866	11 200	0.758
x13	254.0	152.4	12.70	72.7	0.713	9 260	0.769
x9.5	254.0	152.4	9.53	56.1	0.551	7 150	0.780
x7.9	254.0	152.4	7.94	47.4	0.465	6 040	0.786
x6.4	254.0	152.4	6.35	38.4	0.377	4 900	0.791
x4.8	254.0	152.4	4.78	29.3	0.288	3 730	0.796
HSS 203x152							
x16	203.2	152.4	15.88	75.6	0.742	9 640	0.657
x13	203.2	152.4	12.70	62.6	0.614	7 970	0.668
x9.5	203.2	152.4	9.53	48.5	0.476	6 180	0.678
x7.9	203.2	152.4	7.94	41.1	0.403	5 230	0.684
x6.4	203.2	152.4	6.35	33.4	0.327	4 250	0.689
x4.8	203.2	152.4	4.78	25.5	0.250	3 250	0.695
HSS 203x102							
x13	203.2	101.6	12.70	52.4	0.515	6 680	0.566
x9.5	203.2	101.6	9.53	40.9	0.401	5 210	0.577
x7.9	203.2	101.6	7.94	34.7	0.341	4 430	0.582
x6.4	203.2	101.6	6.35	28.3	0.278	3 610	0.588
x4.8	203.2	101.6	4.78	21.7	0.213	2 760	0.593
HSS 178x127							
x13	177.8	127.0	12.70	52.4	0.515	6 680	0.566
x9.5	177.8	127.0	9.53	40.9	0.401	5 210	0.577
x7.9	177.8	127.0	7.94	34.7	0.341	4 430	0.582
x6.4	177.8	127.0	6.35	28.3	0.278	3 610	0.588
x4.8	177.8	127.0	4.78	21.7	0.213	2 760	0.593

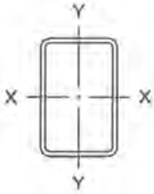


HOLLOW STRUCTURAL SECTIONS
CSA G40.20
Rectangular

PROPERTIES

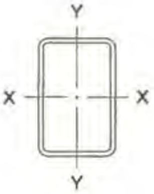
Axis X-X				Axis Y-Y				Torsional Constant	Section
I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	
10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	mm x mm x mm
									HSS 305x203
174	1 140	110	1 430	92.2	907	79.8	1 080	200 000	x16
147	964	111	1 190	78.2	769	81.2	897	167 000	x13
116	762	113	926	62.1	611	82.7	701	130 000	x9.5
99.3	652	114	786	53.2	524	83.4	596	110 000	x7.9
81.5	535	115	640	43.8	431	84.1	486	89 700	x6.4
									HSS 305x152
141	922	105	1 190	46.5	610	60.1	729	119 000	x16
119	784	106	999	40.0	524	61.5	613	100 000	x13
95.1	624	108	783	32.2	422	62.9	482	79 000	x9.5
81.6	535	109	666	27.7	364	63.7	411	67 400	x7.9
67.1	440	110	544	23.0	301	64.4	337	55 100	x6.4
									HSS 254x203
111	872	92.8	1 080	78.0	767	77.9	925	153 000	x16
94.0	740	94.4	903	66.4	654	79.3	774	127 000	x13
74.8	589	96.0	707	53.0	522	80.8	607	99 600	x9.5
64.2	505	96.8	602	45.5	448	81.6	517	84 600	x7.9
52.8	416	97.6	491	37.5	369	82.3	422	69 000	x6.4
									HSS 254x152
87.8	691	88.3	888	38.9	511	58.8	619	91 900	x16
75.2	592	90.1	747	33.6	442	60.3	522	77 700	x13
60.4	475	91.9	589	27.2	357	61.7	413	61 400	x9.5
52.0	409	92.8	502	23.5	309	62.4	353	52 400	x7.9
42.9	338	93.6	411	19.5	256	63.1	290	42 900	x6.4
33.3	262	94.5	317	15.2	200	63.8	224	33 000	x4.8
									HSS 203x152
49.6	488	71.7	623	31.4	412	57.1	509	65 900	x16
43.0	423	73.4	528	27.3	359	58.6	432	56 000	x13
34.8	343	75.1	420	22.3	292	60.0	344	44 400	x9.5
30.2	297	75.9	359	19.3	253	60.8	295	38 000	x7.9
25.0	246	76.7	295	16.1	211	61.5	243	31 200	x6.4
19.5	192	77.5	228	12.6	165	62.2	188	24 100	x4.8
									HSS 203x102
31.3	308	68.4	405	10.2	201	39.1	246	26 700	x13
25.8	254	70.3	326	8.57	169	40.5	199	21 700	x9.5
22.5	221	71.2	281	7.53	148	41.3	172	18 800	x7.9
18.8	185	72.2	232	6.35	125	42.0	143	15 600	x6.4
14.7	145	73.1	180	5.03	99.0	42.7	111	12 100	x4.8
									HSS 178x127
26.4	297	62.9	378	15.5	244	48.1	298	33 300	x13
21.7	244	64.6	303	12.8	202	49.6	240	26 800	x9.5
18.9	213	65.4	261	11.2	177	50.3	207	23 000	x7.9
15.8	178	66.2	216	9.40	148	51.1	171	19 000	x6.4
12.4	140	67.1	168	7.41	117	51.8	133	14 700	x4.8

HOLLOW STRUCTURAL SECTIONS
CSA G40.20
Rectangular



DIMENSIONS

Section	Outside Dimensions		Wall Thickness	Mass	Dead Load	Area	Surface Area
	Depth	Width					
mm x mm x mm	mm	mm	mm	kg/m	kN/m	mm^2	m^2/m
HSS 152x102							
x13	152.4	101.6	12.70	42.3	0.415	5 390	0.464
x9.5	152.4	101.6	9.53	33.3	0.327	4 240	0.475
x7.9	152.4	101.6	7.94	28.4	0.279	3 620	0.481
x6.4	152.4	101.6	6.35	23.2	0.228	2 960	0.486
x4.8	152.4	101.6	4.78	17.9	0.175	2 280	0.492
x3.2	152.4	101.6	3.18	12.2	0.119	1 550	0.497
HSS 152x76							
x13	152.4	76.2	12.70	37.3	0.365	4 750	0.414
x9.5	152.4	76.2	9.53	29.5	0.290	3 760	0.424
x7.9	152.4	76.2	7.94	25.2	0.248	3 220	0.430
x6.4	152.4	76.2	6.35	20.7	0.203	2 640	0.435
x4.8	152.4	76.2	4.78	16.0	0.157	2 040	0.441
x3.2	152.4	76.2	3.18	10.9	0.107	1 390	0.446
HSS 127x76							
x13	127.0	76.2	12.70	32.2	0.316	4 100	0.363
x9.5	127.0	76.2	9.53	25.7	0.252	3 280	0.374
x7.9	127.0	76.2	7.94	22.1	0.217	2 810	0.379
x6.4	127.0	76.2	6.35	18.2	0.178	2 320	0.385
x4.8	127.0	76.2	4.78	14.1	0.138	1 790	0.390
x3.2	127.0	76.2	3.18	9.62	0.094	1 230	0.395
HSS 102x76							
x9.5	101.6	76.2	9.53	21.9	0.215	2 790	0.323
x7.9	101.6	76.2	7.94	18.9	0.186	2 410	0.328
x6.4	101.6	76.2	6.35	15.6	0.153	1 990	0.334
x4.8	101.6	76.2	4.78	12.2	0.119	1 550	0.339
x3.2	101.6	76.2	3.18	8.35	0.082	1 060	0.345
HSS 102x51							
x9.5	101.6	50.8	9.53	18.1	0.178	2 310	0.272
x7.9	101.6	50.8	7.94	15.7	0.154	2 010	0.278
x6.4	101.6	50.8	6.35	13.1	0.129	1 670	0.283
x4.8	101.6	50.8	4.78	10.3	0.101	1 310	0.288
x3.2	101.6	50.8	3.18	7.09	0.070	903	0.294
HSS 89x64							
x6.4	88.9	63.5	6.35	13.1	0.129	1 670	0.283
x4.8	88.9	63.5	4.78	10.3	0.101	1 310	0.288
HSS 76x51							
x7.9	76.2	50.8	7.94	12.6	0.123	1 600	0.227
x6.4	76.2	50.8	6.35	10.6	0.104	1 350	0.232
x4.8	76.2	50.8	4.78	8.35	0.082	1 060	0.238
x3.2	76.2	50.8	3.18	5.82	0.057	741	0.243

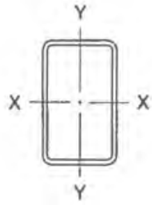


HOLLOW STRUCTURAL SECTIONS
CSA G40.20
Rectangular

PROPERTIES

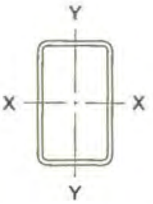
Axis X-X				Axis Y-Y				Torsional Constant	Section
I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	
10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	mm x mm x mm
									HSS 152x102
14.7	193	52.2	252	7.67	151	37.7	189	17 500	x13
12.4	162	54.0	206	6.51	128	39.2	155	14 400	x9.5
10.9	143	54.9	178	5.76	113	39.9	134	12 500	x7.9
9.19	121	55.7	148	4.88	96.2	40.6	112	10 400	x6.4
7.28	95.6	56.5	116	3.89	76.6	41.3	87.8	8 150	x4.8
5.10	66.9	57.4	80.2	2.74	53.9	42.1	60.8	5 620	x3.2
									HSS 152x76
11.5	152	49.3	207	3.71	97.3	28.0	124	9 960	x13
9.89	130	51.3	171	3.24	85.0	29.4	104	8 450	x9.5
8.78	115	52.3	149	2.91	76.3	30.1	91.1	7 440	x7.9
7.47	98.0	53.2	125	2.50	65.5	30.8	76.6	6 270	x6.4
5.96	78.2	54.1	98.1	2.02	52.9	31.5	60.5	4 950	x4.8
4.20	55.1	55.0	68.1	1.44	37.7	32.2	42.2	3 450	x3.2
									HSS 127x76
7.02	111	41.4	151	3.05	80.0	27.3	104	7 590	x13
6.13	96.5	43.3	126	2.70	70.8	28.7	87.8	6 500	x9.5
5.49	86.4	44.2	111	2.43	63.9	29.4	77.3	5 750	x7.9
4.70	74.1	45.1	93.4	2.10	55.2	30.1	65.3	4 860	x6.4
3.78	59.6	45.9	73.8	1.71	44.8	30.8	51.8	3 850	x4.8
2.69	42.3	46.8	51.5	1.22	32.1	31.6	36.3	2 690	x3.2
									HSS 102x76
3.42	67.4	35.0	87.9	2.16	56.6	27.8	71.6	4 630	x9.5
3.10	61.0	35.9	77.8	1.96	51.5	28.5	63.6	4 120	x7.9
2.69	52.9	36.7	66.0	1.71	44.8	29.3	54.0	3 500	x6.4
2.18	43.0	37.5	52.6	1.39	36.6	30.0	43.1	2 780	x4.8
1.57	30.8	38.4	37.0	1.01	26.4	30.7	30.4	1 950	x3.2
									HSS 102x51
2.39	47.1	32.2	65.6	0.762	30.0	18.2	39.2	2 070	x9.5
2.21	43.6	33.2	58.9	0.714	28.1	18.9	35.5	1 910	x7.9
1.95	38.5	34.2	50.7	0.640	25.2	19.6	30.8	1 670	x6.4
1.61	31.8	35.1	40.8	0.537	21.1	20.3	25.0	1 360	x4.8
1.17	23.1	36.1	29.0	0.397	15.6	21.0	17.9	976	x3.2
									HSS 89x64
1.65	37.1	31.4	47.2	0.968	30.5	24.1	37.3	2 080	x6.4
1.36	30.6	32.3	38.0	0.803	25.3	24.8	30.1	1 680	x4.8
									HSS 76x51
1.02	26.7	25.2	36.0	0.527	20.7	18.1	26.9	1 240	x7.9
0.919	24.1	26.1	31.5	0.479	18.9	18.9	23.6	1 100	x6.4
0.775	20.3	27.0	25.8	0.408	16.1	19.6	19.4	903	x4.8
0.575	15.1	27.8	18.6	0.306	12.0	20.3	14.0	652	x3.2

HOLLOW STRUCTURAL SECTIONS
CSA G40.20
Rectangular



DIMENSIONS

Section	Outside Dimensions		Wall Thickness	Mass	Dead Load	Area	Surface Area
	Depth	Width					
mm x mm x mm	mm	mm	mm	kg/m	kN/m	mm²	m²/ m
HSS 76x38							
x6.4	76.2	38.1	6.35	9.31	0.091	1 190	0.207
x4.8	76.2	38.1	4.78	7.40	0.073	942	0.212
x3.2	76.2	38.1	3.18	5.18	0.051	660	0.218
HSS 64x38							
x6.4	63.5	38.1	6.35	8.05	0.079	1 030	0.181
x4.8	63.5	38.1	4.78	6.45	0.063	821	0.187
x3.2	63.5	38.1	3.18	4.55	0.045	580	0.192
HSS 51x25							
x4.8	50.8	25.4	4.78	4.54	0.045	578	0.136
x3.2	50.8	25.4	3.18	3.28	0.032	418	0.141



HOLLOW STRUCTURAL SECTIONS
CSA G40.20
Rectangular

PROPERTIES

Axis X-X				Axis Y-Y				Torsional Constant	Section
I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	
10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	mm x mm x mm
									HSS 76x38
0.722	18.9	24.7	25.9	0.232	12.2	14.0	15.6	622	x6.4
0.620	16.3	25.6	21.4	0.203	10.6	14.7	13.0	529	x4.8
0.467	12.3	26.6	15.6	0.156	8.20	15.4	9.58	392	x3.2
									HSS 64x38
0.439	13.8	20.7	18.8	0.191	10.0	13.6	13.0	474	x6.4
0.384	12.1	21.6	15.8	0.169	8.87	14.3	11.0	407	x4.8
0.294	9.27	22.5	11.7	0.132	6.91	15.1	8.17	304	x3.2
									HSS 51x25
0.150	5.89	16.1	8.21	0.047 6	3.75	9.08	4.91	129	x4.8
0.122	4.81	17.1	6.34	0.040 0	3.15	9.78	3.85	104	x3.2

HOLLOW STRUCTURAL SECTIONS
CSA G40.20
Round

PROPERTIES AND DIMENSIONS

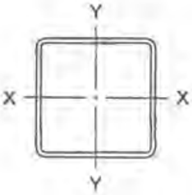
Section	Outside Dimension	Wall Thickness	Mass	Dead Load	Area	I	S	r	Z	Torsional Constant J	Surface Area
mm x mm x mm	mm	mm	kg/m	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	m^2/ m
HSS 406											
x16	406.4	15.88	153	1.50	19 500	372	1 830	138	2 420	744 000	1.28
x13	406.4	12.70	123	1.21	15 700	305	1 500	139	1 970	609 000	1.28
x9.5	406.4	9.53	93.3	0.915	11 900	234	1 150	140	1 500	468 000	1.28
x6.4	406.4	6.35	62.6	0.615	7 980	160	786	141	1 020	319 000	1.28
HSS 356											
x13	355.6	12.70	107	1.05	13 700	201	1 130	121	1 490	403 000	1.12
x9.5	355.6	9.53	81.3	0.798	10 400	155	873	122	1 140	310 000	1.12
x6.4	355.6	6.35	54.7	0.537	6 970	106	598	123	775	213 000	1.12
HSS 324											
x13	323.9	12.70	97.5	0.956	12 400	151	930	110	1 230	301 000	1.02
x9.5	323.9	9.53	73.9	0.725	9 410	116	719	111	942	233 000	1.02
x6.4	323.9	6.35	49.7	0.488	6 330	79.9	493	112	640	160 000	1.02
HSS 273											
x13	273.1	12.70	81.6	0.800	10 400	88.3	646	92.2	862	177 000	0.858
x9.5	273.1	9.53	61.9	0.608	7 890	68.6	502	93.2	662	137 000	0.858
x7.9	273.1	7.94	51.9	0.509	6 610	58.2	426	93.8	558	116 000	0.858
x6.4	273.1	6.35	41.8	0.410	5 320	47.4	347	94.3	452	94 700	0.858
x4.8	273.1	4.78	31.6	0.310	4 030	36.3	266	94.9	344	72 500	0.858
HSS 245											
x9.5	244.5	9.53	55.2	0.542	7 030	48.6	398	83.1	526	97 300	0.768
x6.4	244.5	6.35	37.3	0.366	4 750	33.7	276	84.2	360	67 400	0.768
HSS 219											
x13	219.1	12.70	64.6	0.634	8 230	44.0	402	73.1	542	88 000	0.688
x9.5	219.1	9.53	49.3	0.483	6 270	34.5	315	74.2	419	69 000	0.688
x6.4	219.1	6.35	33.3	0.327	4 240	24.0	219	75.3	288	48 100	0.688
x4.8	219.1	4.78	25.3	0.248	3 220	18.5	169	75.8	220	37 000	0.688
HSS 178											
x13	177.8	12.70	51.7	0.507	6 590	22.6	254	58.5	347	45 200	0.559
x9.5	177.8	9.53	39.5	0.388	5 040	17.9	201	59.6	270	35 800	0.559
HSS 168											
x13	168.3	12.70	48.7	0.478	6 210	18.9	225	55.2	308	37 800	0.529
x9.5	168.3	9.53	37.3	0.366	4 750	15.0	179	56.2	241	30 100	0.529
x6.4	168.3	6.35	25.4	0.249	3 230	10.6	126	57.3	167	21 200	0.529
x4.8	168.3	4.78	19.3	0.189	2 460	8.21	97.6	57.8	128	16 400	0.529
HSS 141											
x13	141.3	12.70	40.3	0.395	5 130	10.7	152	45.7	211	21 400	0.444
x9.5	141.3	9.53	31.0	0.304	3 950	8.61	122	46.7	166	17 200	0.444
x6.4	141.3	6.35	21.1	0.207	2 690	6.14	86.9	47.8	116	12 300	0.444

HOLLOW STRUCTURAL SECTIONS
CSA G40.20
Round

PROPERTIES AND DIMENSIONS

Section	Outside Dimension	Wall Thickness	Mass	Dead Load	Area	I	S	r	Z	Torsional Constant J	Surface Area
mm x mm x mm	mm	mm	kg/m	kN/m	mm^2	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^3\ mm^4$	m^2/m
HSS 127											
x9.5	127.0	9.53	27.6	0.271	3 520	6.11	96.2	41.7	132	12 200	0.399
x6.4	127.0	6.35	18.9	0.185	2 410	4.39	69.2	42.7	92.5	8 780	0.399
HSS 89											
x6.4	88.9	6.35	12.9	0.127	1 650	1.41	31.7	29.3	43.4	2 820	0.279
x4.8	88.9	4.78	9.92	0.097	1 260	1.12	25.2	29.8	33.9	2 240	0.279
x3.2	88.9	3.18	6.72	0.066	856	0.788	17.7	30.3	23.4	1 580	0.279
HSS 76											
x6.4	76.2	6.35	10.9	0.107	1 390	0.857	22.5	24.8	31.1	1 710	0.239
x4.8	76.2	4.78	8.42	0.083	1 070	0.687	18.0	25.3	24.4	1 370	0.239
x3.2	76.2	3.18	5.73	0.056	729	0.487	12.8	25.8	17.0	974	0.239
HSS 73											
x6.4	73.0	6.35	10.4	0.102	1 330	0.745	20.4	23.7	28.3	1 490	0.229
x4.8	73.0	4.78	8.04	0.079	1 020	0.599	16.4	24.2	22.3	1 200	0.229
x3.2	73.0	3.18	5.48	0.054	698	0.426	11.7	24.7	15.5	852	0.229
HSS 64											
x6.4	63.5	6.35	8.95	0.088	1 140	0.471	14.8	20.3	20.8	942	0.199
x4.8	63.5	4.78	6.92	0.068	882	0.383	12.0	20.8	16.5	765	0.199
x3.2	63.5	3.18	4.73	0.046	603	0.275	8.66	21.4	11.6	550	0.199
HSS 60											
x6.4	60.3	6.35	8.45	0.083	1 080	0.397	13.2	19.2	18.6	794	0.189
x4.8	60.3	4.78	6.54	0.064	834	0.324	10.7	19.7	14.8	647	0.189
x3.2	60.3	3.18	4.48	0.044	571	0.233	7.74	20.2	10.4	467	0.189
HSS 48											
x4.8	48.3	4.78	5.13	0.050	654	0.157	6.48	15.5	9.09	313	0.152
x3.2	48.3	3.18	3.54	0.035	451	0.115	4.77	16.0	6.48	231	0.152

HOLLOW STRUCTURAL SECTIONS
ASTM A500
Square

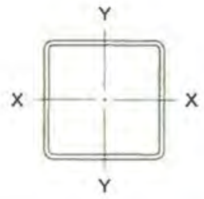


PROPERTIES AND DIMENSIONS

Section	Outside Dimension	Wall Thickness Nominal	Wall Thickness Design	Mass	Dead Load	Area	I	S	r	Z	Torsional Constant J	Surface Area
mm x mm x mm	mm	mm	mm	kg/m	kN/m	mm²	10⁶ mm⁴	10³ mm³	mm	10³ mm³	10³ mm⁴	m²/ m
HSS 406x406												
x16*	406.4	15.88	14.29	190	1.86	21 900	554	2 730	159	3 190	878 000	1.58
x13*	406.4	12.70	11.43	154	1.51	17 700	456	2 250	160	2 610	715 000	1.59
x9.5*	406.4	9.53	8.58	117	1.15	13 500	353	1 740	162	2 000	547 000	1.60
HSS 356x356												
x16*	355.6	15.88	14.29	164	1.61	19 000	363	2 040	138	2 410	580 000	1.37
x13*	355.6	12.70	11.43	133	1.31	15 400	301	1 690	140	1 970	474 000	1.38
x9.5*	355.6	9.53	8.58	102	0.998	11 700	233	1 310	141	1 520	363 000	1.39
x7.9*	355.6	7.94	7.15	85.4	0.838	9 830	198	1 110	142	1 280	306 000	1.40
HSS 305x305												
x16	304.8	15.88	14.29	139	1.36	16 100	222	1 460	118	1 730	358 000	1.17
x13	304.8	12.70	11.43	113	1.11	13 100	185	1 210	119	1 430	294 000	1.18
x9.5	304.8	9.53	8.58	86.5	0.849	9 980	144	948	120	1 100	227 000	1.19
x7.9	304.8	7.94	7.15	72.7	0.714	8 380	123	806	121	930	191 000	1.19
x6.4	304.8	6.35	5.72	58.7	0.576	6 760	100	657	122	755	155 000	1.20
HSS 254x254												
x16	254.0	15.88	14.29	114	1.11	13 200	124	973	96.8	1 170	202 000	0.967
x13	254.0	12.70	11.43	93.0	0.912	10 800	104	817	98.2	968	167 000	0.977
x9.5	254.0	9.53	8.58	71.3	0.700	8 230	81.7	643	99.6	752	129 000	0.987
x7.9	254.0	7.94	7.15	60.1	0.589	6 930	69.7	549	100	637	109 000	0.991
x6.4	254.0	6.35	5.72	48.6	0.476	5 600	57.1	449	101	518	88 700	0.996
x4.8	254.0	4.78	4.30	36.9	0.362	4 250	43.9	346	102	396	67 600	1.00
HSS 203x203												
x16	203.2	15.88	14.29	88.3	0.866	10 300	59.5	585	76.1	714	99 000	0.764
x13	203.2	12.70	11.43	72.7	0.713	8 430	50.6	498	77.5	598	82 700	0.774
x9.5	203.2	9.53	8.58	56.1	0.551	6 490	40.4	397	78.9	469	64 600	0.783
x7.9	203.2	7.94	7.15	47.4	0.465	5 480	34.6	341	79.5	399	54 900	0.788
x6.4	203.2	6.35	5.72	38.4	0.377	4 430	28.5	281	80.2	326	44 700	0.793
x4.8	203.2	4.78	4.30	29.3	0.288	3 370	22.1	217	80.9	250	34 300	0.798
HSS 178x178												
x16	177.8	15.88	14.29	75.6	0.742	8 820	38.1	428	65.7	529	64 300	0.662
x13	177.8	12.70	11.43	62.6	0.614	7 270	32.7	368	67.1	446	54 100	0.672
x9.5	177.8	9.53	8.58	48.5	0.476	5 620	26.3	296	68.5	352	42 600	0.682
x7.9	177.8	7.94	7.15	41.1	0.403	4 750	22.7	256	69.2	301	36 300	0.687
x6.4	177.8	6.35	5.72	33.4	0.327	3 850	18.8	212	69.9	247	29 700	0.692
x4.8	177.8	4.78	4.30	25.5	0.250	2 940	14.6	164	70.5	190	22 800	0.696
HSS 152x152												
x13	152.4	12.70	11.43	52.4	0.515	6 110	19.6	258	56.7	317	32 900	0.570
x9.5	152.4	9.53	8.58	40.9	0.401	4 750	16.0	210	58.1	252	26 200	0.580
x7.9	152.4	7.94	7.15	34.7	0.341	4 020	13.9	182	58.8	217	22 400	0.585
x6.4	152.4	6.35	5.72	28.3	0.278	3 270	11.6	152	59.5	178	18 400	0.590
x4.8	152.4	4.78	4.30	21.7	0.213	2 500	9.05	119	60.2	138	14 200	0.595

* Imported section

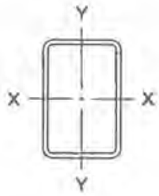
HOLLOW STRUCTURAL SECTIONS
ASTM A500
Square



PROPERTIES AND DIMENSIONS

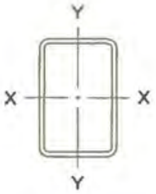
Section	Outside Dimension	Wall Thickness Nominal	Wall Thickness Design	Mass	Dead Load	Area	I	S	r	Z	Torsional Constant J	Surface Area
mm x mm x mm	mm	mm	mm	kg/m	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	m^2/ m
HSS 127x127												
x13	127.0	12.70	11.43	42.3	0.415	4 950	10.6	167	46.3	209	18 100	0.469
x9.5	127.0	9.53	8.58	33.3	0.327	3 870	8.82	139	47.7	169	14 600	0.479
x7.9	127.0	7.94	7.15	28.4	0.279	3 300	7.73	122	48.4	146	12 600	0.483
x6.4	127.0	6.35	5.72	23.2	0.228	2 690	6.49	102	49.1	121	10 400	0.488
x4.8	127.0	4.78	4.30	17.9	0.175	2 060	5.11	80.6	49.8	94.2	8 090	0.493
x3.2	127.0	3.18	2.86	12.2	0.119	1 400	3.57	56.2	50.5	64.8	5 540	0.498
HSS 102x102												
x13	101.6	12.70	11.43	32.2	0.316	3 790	4.88	96.1	35.9	124	8 570	0.367
x9.5	101.6	9.53	8.58	25.7	0.252	3 000	4.19	82.4	37.3	102	7 100	0.377
x7.9	101.6	7.94	7.15	22.1	0.217	2 570	3.72	73.2	38.0	89.3	6 190	0.382
x6.4	101.6	6.35	5.72	18.2	0.178	2 110	3.17	62.3	38.7	74.8	5 170	0.387
x4.8	101.6	4.78	4.30	14.1	0.138	1 630	2.53	49.7	39.4	58.7	4 050	0.392
x3.2	101.6	3.18	2.86	9.62	0.094	1 110	1.78	35.1	40.1	40.8	2 800	0.397
HSS 89x89												
x9.5	88.9	9.53	8.58	21.9	0.215	2 570	2.65	59.6	32.1	75.2	4 570	0.326
x7.9	88.9	7.94	7.15	18.9	0.186	2 210	2.38	53.5	32.8	66.2	4 020	0.331
x6.4	88.9	6.35	5.72	15.6	0.153	1 820	2.05	46.0	33.5	55.8	3 380	0.336
x4.8	88.9	4.78	4.30	12.2	0.119	1 410	1.65	37.1	34.2	44.1	2 660	0.341
HSS 76x76												
x9.5	76.2	9.53	8.58	18.1	0.178	2 130	1.55	40.6	26.9	52.2	2 710	0.275
x7.9	76.2	7.94	7.15	15.7	0.154	1 840	1.41	37.0	27.6	46.5	2 420	0.280
x6.4	76.2	6.35	5.72	13.1	0.129	1 530	1.23	32.2	28.4	39.6	2 060	0.285
x4.8	76.2	4.78	4.30	10.3	0.101	1 190	1.00	26.3	29.0	31.6	1 640	0.290
x3.2	76.2	3.18	2.86	7.09	0.070	818	0.724	19.0	29.7	22.3	1 150	0.295
HSS 64x64												
x6.4	63.5	6.35	5.72	10.6	0.104	1 240	0.664	20.9	23.2	26.2	1 130	0.234
x4.8	63.5	4.78	4.30	8.35	0.082	971	0.552	17.4	23.9	21.2	917	0.239
x3.2	63.5	3.18	2.86	5.82	0.057	673	0.406	12.8	24.6	15.1	652	0.244
HSS 51x51												
x6.4	50.8	6.35	5.72	8.05	0.079	947	0.305	12.0	18.0	15.5	536	0.184
x4.8	50.8	4.78	4.30	6.45	0.063	752	0.262	10.3	18.7	12.8	445	0.188
x3.2	50.8	3.18	2.86	4.55	0.045	527	0.198	7.79	19.4	9.35	323	0.193
HSS 38x38												
x4.8	38.1	4.78	4.30	4.54	0.045	534	0.096 7	5.08	13.5	6.54	170	0.138
x3.2	38.1	3.18	2.86	3.28	0.032	382	0.076 8	4.03	14.2	4.95	129	0.143

HOLLOW STRUCTURAL SECTIONS
ASTM A500
Rectangular



DIMENSIONS

Section	Outside Dimensions		Wall Thickness		Mass	Dead Load	Area	Surface Area
	Depth	Width	Nominal	Design				
mm x mm x mm	mm	mm	mm	mm	kg/m	kN/m	mm²	m²/ m
HSS 305x203								
x16	304.8	203.2	15.88	14.29	114	1.11	13 200	0.967
x13	304.8	203.2	12.70	11.43	93.0	0.912	10 800	0.977
x9.5	304.8	203.2	9.53	8.58	71.3	0.700	8 230	0.987
x7.9	304.8	203.2	7.94	7.15	60.1	0.589	6 930	0.991
x6.4	304.8	203.2	6.35	5.72	48.6	0.476	5 600	0.996
HSS 305x152								
x16	304.8	152.4	15.88	14.29	101	0.991	11 700	0.865
x13	304.8	152.4	12.70	11.43	82.8	0.813	9 590	0.875
x9.5	304.8	152.4	9.53	8.58	63.7	0.625	7 360	0.885
x7.9	304.8	152.4	7.94	7.15	53.7	0.527	6 200	0.890
x6.4	304.8	152.4	6.35	5.72	43.5	0.427	5 020	0.895
HSS 254x203								
x16	254.0	203.2	15.88	14.29	101	0.991	11 700	0.865
x13	254.0	203.2	12.70	11.43	82.8	0.813	9 590	0.875
x9.5	254.0	203.2	9.53	8.58	63.7	0.625	7 360	0.885
x7.9	254.0	203.2	7.94	7.15	53.7	0.527	6 200	0.890
x6.4	254.0	203.2	6.35	5.72	43.5	0.427	5 020	0.895
HSS 254x152								
x16	254.0	152.4	15.88	14.29	88.3	0.866	10 300	0.764
x13	254.0	152.4	12.70	11.43	72.7	0.713	8 430	0.774
x9.5	254.0	152.4	9.53	8.58	56.1	0.551	6 490	0.783
x7.9	254.0	152.4	7.94	7.15	47.4	0.465	5 480	0.788
x6.4	254.0	152.4	6.35	5.72	38.4	0.377	4 430	0.793
x4.8	254.0	152.4	4.78	4.30	29.3	0.288	3 370	0.798
HSS 203x152								
x16	203.2	152.4	15.88	14.29	75.6	0.742	8 820	0.662
x13	203.2	152.4	12.70	11.43	62.6	0.614	7 270	0.672
x9.5	203.2	152.4	9.53	8.58	48.5	0.476	5 620	0.682
x7.9	203.2	152.4	7.94	7.15	41.1	0.403	4 750	0.687
x6.4	203.2	152.4	6.35	5.72	33.4	0.327	3 850	0.692
x4.8	203.2	152.4	4.78	4.30	25.5	0.250	2 940	0.696
HSS 203x102								
x13	203.2	101.6	12.70	11.43	52.4	0.515	6 110	0.570
x9.5	203.2	101.6	9.53	8.58	40.9	0.401	4 750	0.580
x7.9	203.2	101.6	7.94	7.15	34.7	0.341	4 020	0.585
x6.4	203.2	101.6	6.35	5.72	28.3	0.278	3 270	0.590
x4.8	203.2	101.6	4.78	4.30	21.7	0.213	2 500	0.595
HSS 178x127								
x13	177.8	127.0	12.70	11.43	52.4	0.515	6 110	0.570
x9.5	177.8	127.0	9.53	8.58	40.9	0.401	4 750	0.580
x7.9	177.8	127.0	7.94	7.15	34.7	0.341	4 020	0.585
x6.4	177.8	127.0	6.35	5.72	28.3	0.278	3 270	0.590
x4.8	177.8	127.0	4.78	4.30	21.7	0.213	2 500	0.595

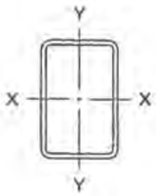


HOLLOW STRUCTURAL SECTIONS
ASTM A500
Rectangular

PROPERTIES

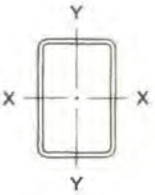
Axis X-X				Axis Y-Y				Torsional Constant	Section
I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	
10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	mm x mm x mm
									HSS 305x203
161	1 060	111	1 310	85.4	841	80.5	989	184 000	x16
135	886	112	1 090	72.0	709	81.8	821	152 000	x13
106	697	114	843	56.9	560	83.1	638	118 000	x9.5
90.6	594	114	714	48.6	478	83.8	542	100 000	x7.9
74.1	486	115	581	39.9	392	84.4	441	81 400	x6.4
									HSS 305x152
130	856	105	1 100	43.4	569	60.8	672	110 000	x16
110	722	107	915	37.0	485	62.1	562	92 000	x13
87.1	572	109	714	29.6	388	63.4	440	72 100	x9.5
74.5	489	110	606	25.4	333	64.0	375	61 300	x7.9
61.1	401	110	494	21.0	275	64.6	306	50 100	x6.4
									HSS 254x203
103	809	93.6	994	72.4	713	78.6	852	140 000	x16
86.7	682	95.1	827	61.3	603	79.9	709	117 000	x13
68.5	540	96.5	645	48.6	478	81.3	554	90 700	x9.5
58.6	462	97.2	548	41.6	410	81.9	470	76 900	x7.9
48.1	379	97.9	446	34.2	337	82.6	384	62 600	x6.4
									HSS 254x152
81.8	644	89.2	820	36.4	478	59.6	572	85 100	x16
69.6	548	90.8	686	31.2	410	60.8	480	71 400	x13
55.4	436	92.4	538	25.0	329	62.1	378	56 100	x9.5
47.5	374	93.2	458	21.6	283	62.8	322	47 700	x7.9
39.1	308	94.0	374	17.8	234	63.4	264	39 000	x6.4
30.3	238	94.7	287	13.8	182	64.1	203	29 900	x4.8
									HSS 203x152
46.5	457	72.6	577	29.5	387	57.8	472	61 100	x16
39.9	393	74.1	486	25.4	334	59.1	398	51 600	x13
32.1	316	75.6	384	20.5	270	60.5	315	40 600	x9.5
27.7	272	76.3	328	17.7	233	61.1	269	34 700	x7.9
22.9	225	77.1	269	14.7	193	61.8	221	28 400	x6.4
17.8	175	77.8	207	11.4	150	62.4	170	21 800	x4.8
									HSS 203x102
29.2	288	69.2	375	9.63	190	39.7	228	24 800	x13
23.8	235	70.9	299	7.97	157	41.0	183	20 000	x9.5
20.7	204	71.7	257	6.96	137	41.6	158	17 200	x7.9
17.2	169	72.5	211	5.84	115	42.2	131	14 200	x6.4
13.4	132	73.3	164	4.60	90.5	42.9	101	11 000	x4.8
									HSS 178x127
24.7	278	63.6	350	14.5	228	48.7	276	30 800	x13
20.1	226	65.1	279	11.9	187	50.0	221	24 600	x9.5
17.4	196	65.8	239	10.3	163	50.7	190	21 100	x7.9
14.5	163	66.6	197	8.63	136	51.4	156	17 300	x6.4
11.3	127	67.3	152	6.76	106	52.0	121	13 400	x4.8

HOLLOW STRUCTURAL SECTIONS
ASTM A500
Rectangular



DIMENSIONS

Section	Outside Dimensions		Wall Thickness		Mass	Dead Load	Area	Surface Area
	Depth	Width	Nominal	Design				
mm x mm x mm	mm	mm	mm	mm	kg/m	kN/m	mm²	m²/ m
HSS 152x102								
x13	152.4	101.6	12.70	11.43	42.3	0.415	4 950	0.469
x9.5	152.4	101.6	9.53	8.58	33.3	0.327	3 870	0.479
x7.9	152.4	101.6	7.94	7.15	28.4	0.279	3 300	0.483
x6.4	152.4	101.6	6.35	5.72	23.2	0.228	2 690	0.488
x4.8	152.4	101.6	4.78	4.30	17.9	0.175	2 060	0.493
x3.2	152.4	101.6	3.18	2.86	12.2	0.119	1 400	0.498
HSS 152x76								
x13	152.4	76.2	12.70	11.43	37.3	0.365	4 370	0.418
x9.5	152.4	76.2	9.53	8.58	29.5	0.290	3 440	0.428
x7.9	152.4	76.2	7.94	7.15	25.2	0.248	2 930	0.433
x6.4	152.4	76.2	6.35	5.72	20.7	0.203	2 400	0.438
x4.8	152.4	76.2	4.78	4.30	16.0	0.157	1 840	0.442
x3.2	152.4	76.2	3.18	2.86	10.9	0.107	1 250	0.447
HSS 127x76								
x13	127.0	76.2	12.70	11.43	32.2	0.316	3 790	0.367
x9.5	127.0	76.2	9.53	8.58	25.7	0.252	3 000	0.377
x7.9	127.0	76.2	7.94	7.15	22.1	0.217	2 570	0.382
x6.4	127.0	76.2	6.35	5.72	18.2	0.178	2 110	0.387
x4.8	127.0	76.2	4.78	4.30	14.1	0.138	1 630	0.392
x3.2	127.0	76.2	3.18	2.86	9.62	0.094	1 110	0.397
HSS 102x76								
x9.5	101.6	76.2	9.53	8.58	21.9	0.215	2 570	0.326
x7.9	101.6	76.2	7.94	7.15	18.9	0.186	2 210	0.331
x6.4	101.6	76.2	6.35	5.72	15.6	0.153	1 820	0.336
x4.8	101.6	76.2	4.78	4.30	12.2	0.119	1 410	0.341
x3.2	101.6	76.2	3.18	2.86	8.35	0.082	963	0.346
HSS 102x51								
x9.5	101.6	50.8	9.53	8.58	18.1	0.178	2 130	0.275
x7.9	101.6	50.8	7.94	7.15	15.7	0.154	1 840	0.280
x6.4	101.6	50.8	6.35	5.72	13.1	0.129	1 530	0.285
x4.8	101.6	50.8	4.78	4.30	10.3	0.101	1 190	0.290
x3.2	101.6	50.8	3.18	2.86	7.09	0.070	818	0.295
HSS 89x64								
x6.4	88.9	63.5	6.35	5.72	13.1	0.129	1 530	0.285
x4.8	88.9	63.5	4.78	4.30	10.3	0.101	1 190	0.290
HSS 76x51								
x7.9	76.2	50.8	7.94	7.15	12.6	0.123	1 480	0.229
x6.4	76.2	50.8	6.35	5.72	10.6	0.104	1 240	0.234
x4.8	76.2	50.8	4.78	4.30	8.35	0.082	971	0.239
x3.2	76.2	50.8	3.18	2.86	5.82	0.057	673	0.244

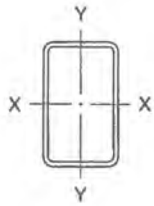


HOLLOW STRUCTURAL SECTIONS
ASTM A500
Rectangular

PROPERTIES

Axis X-X				Axis Y-Y				Torsional Constant	Section
I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	
10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	mm x mm x mm
									HSS 152x102
13.9	182	52.9	235	7.26	143	38.3	176	16 400	x13
11.5	151	54.5	190	6.08	120	39.6	143	13 300	x9.5
10.1	132	55.3	164	5.34	105	40.3	124	11 500	x7.9
8.45	111	56.0	136	4.50	88.6	40.9	103	9 530	x6.4
6.65	87.3	56.8	106	3.56	70.1	41.6	80.0	7 410	x4.8
4.63	60.8	57.5	72.6	2.49	49.1	42.2	55.1	5 090	x3.2
									HSS 152x76
11.0	144	50.1	194	3.55	93.2	28.5	117	9 430	x13
9.25	121	51.9	158	3.05	80.0	29.8	96.5	7 860	x9.5
8.15	107	52.7	137	2.71	71.2	30.4	84.1	6 880	x7.9
6.89	90.4	53.6	114	2.31	60.7	31.0	70.3	5 760	x6.4
5.45	71.6	54.4	89.4	1.85	48.6	31.7	55.2	4 520	x4.8
3.82	50.1	55.2	61.8	1.31	34.4	32.3	38.3	3 130	x3.2
									HSS 127x76
6.72	106	42.1	142	2.93	77.0	27.8	98.2	7 220	x13
5.76	90.7	43.8	117	2.55	66.9	29.1	81.7	6 070	x9.5
5.12	80.6	44.6	103	2.28	59.8	29.8	71.6	5 320	x7.9
4.35	68.5	45.4	85.8	1.95	51.2	30.4	60.1	4 470	x6.4
3.47	54.6	46.2	67.4	1.57	41.2	31.1	47.3	3 510	x4.8
2.45	38.5	47.0	46.8	1.11	29.3	31.7	33.0	2 440	x3.2
									HSS 102x76
3.24	63.8	35.5	82.1	2.05	53.7	28.2	67.0	4 340	x9.5
2.91	57.2	36.3	72.2	1.84	48.4	28.9	59.0	3 820	x7.9
2.50	49.1	37.0	60.8	1.59	41.7	29.6	49.8	3 220	x6.4
2.01	39.6	37.8	48.1	1.29	33.7	30.2	39.5	2 550	x4.8
1.43	28.1	38.5	33.6	0.919	24.1	30.9	27.6	1 770	x3.2
									HSS 102x51
2.29	45.2	32.8	61.8	0.736	29.0	18.6	37.1	1 980	x9.5
2.09	41.2	33.7	55.0	0.681	26.8	19.2	33.3	1 800	x7.9
1.83	36.0	34.6	46.9	0.602	23.7	19.9	28.6	1 550	x6.4
1.49	29.4	35.4	37.5	0.499	19.6	20.5	23.0	1 250	x4.8
1.08	21.2	36.3	26.4	0.365	14.4	21.1	16.3	890	x3.2
									HSS 89x64
1.54	34.7	31.8	43.7	0.907	28.6	24.4	34.5	1 930	x6.4
1.26	28.3	32.5	34.9	0.744	23.4	25.0	27.6	1 540	x4.8
									HSS 76x51
0.974	25.6	25.7	33.9	0.506	19.9	18.5	25.4	1 170	x7.9
0.867	22.8	26.5	29.4	0.454	17.9	19.1	22.0	1 030	x6.4
0.721	18.9	27.2	23.8	0.380	15.0	19.8	17.9	834	x4.8
0.528	13.9	28.0	17.0	0.281	11.1	20.5	12.8	596	x3.2

HOLLOW STRUCTURAL SECTIONS
ASTM A500
Rectangular



DIMENSIONS

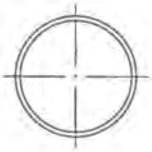
Section	Outside Dimensions		Wall Thickness		Mass	Dead Load	Area	Surface Area
	Depth	Width	Nominal	Design				
mm x mm x mm	mm	mm	mm	mm	kg/m	kN/m	mm^2	m^2/ m
HSS 76x38								
x6.4	76.2	38.1	6.35	5.72	9.31	0.091	1 090	0.209
x4.8	76.2	38.1	4.78	4.30	7.40	0.073	861	0.214
x3.2	76.2	38.1	3.18	2.86	5.18	0.051	600	0.219
HSS 64x38								
x6.4	63.5	38.1	6.35	5.72	8.05	0.079	947	0.184
x4.8	63.5	38.1	4.78	4.30	6.45	0.063	752	0.188
x3.2	63.5	38.1	3.18	2.86	4.55	0.045	527	0.193
HSS 51x25								
x4.8	50.8	25.4	4.78	4.30	4.54	0.045	534	0.138
x3.2	50.8	25.4	3.18	2.86	3.28	0.032	382	0.143

HOLLOW STRUCTURAL SECTIONS
ASTM A500
Rectangular

PROPERTIES

Axis X-X				Axis Y-Y				Torsional Constant	Section
I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	
10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	mm x mm x mm
									HSS 76x38
0.686	18.0	25.1	24.2	0.222	11.7	14.3	14.6	590	x6.4
0.579	15.2	25.9	19.8	0.191	10.0	14.9	12.1	492	x4.8
0.430	11.3	26.8	14.3	0.145	7.59	15.5	8.79	360	x3.2
									HSS 64x38
0.420	13.2	21.1	17.8	0.183	9.63	13.9	12.3	452	x6.4
0.361	11.4	21.9	14.7	0.159	8.37	14.6	10.2	380	x4.8
0.272	8.56	22.7	10.7	0.122	6.40	15.2	7.51	279	x3.2
									HSS 51x25
0.144	5.65	16.4	7.74	0.046 0	3.63	9.29	4.65	124	x4.8
0.114	4.50	17.3	5.86	0.037 6	2.96	9.93	3.57	97.1	x3.2

HOLLOW STRUCTURAL SECTIONS
ASTM A500
Round



PROPERTIES AND DIMENSIONS

Section	Outside Dimen-sion	Wall Thickness		Mass	Dead Load	Area	I	S	r	Z	Torsional Constant	Surface Area
		Nom-inal	Design								J	
mm x mm x mm	mm	mm	mm	kg/m	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	m^2/ m
HSS 508												
x13*	508.0	12.70	11.43	155	1.52	17 800	550	2 160	176	2 820	1 100 000	1.60
x9.5*	508.0	9.53	8.58	117	1.15	13 500	420	1 650	177	2 140	840 000	1.60
x6.4*	508.0	6.35	5.72	78.6	0.771	9 030	285	1 120	178	1 440	569 000	1.60
HSS 457												
x13*	457.2	12.70	11.43	139	1.37	16 000	398	1 740	158	2 270	796 000	1.44
x9.5*	457.2	9.53	8.58	105	1.03	12 100	304	1 330	159	1 730	609 000	1.44
x6.4*	457.2	6.35	5.72	70.6	0.693	8 110	207	904	160	1 170	413 000	1.44
HSS 406												
x16	406.4	15.88	14.29	153	1.50	17 600	339	1 670	139	2 200	678 000	1.28
x13	406.4	12.70	11.43	123	1.21	14 200	277	1 360	140	1 780	554 000	1.28
x9.5	406.4	9.53	8.58	93.3	0.915	10 700	212	1 040	141	1 360	424 000	1.28
x6.4	406.4	6.35	5.72	62.6	0.615	7 200	145	711	142	918	289 000	1.28
HSS 356												
x13	355.6	12.70	11.43	107	1.05	12 400	183	1 030	122	1 350	366 000	1.12
x9.5	355.6	9.53	8.58	81.3	0.798	9 350	141	792	123	1 030	282 000	1.12
x6.4	355.6	6.35	5.72	54.7	0.537	6 290	96.2	541	124	700	192 000	1.12
HSS 324												
x13	323.9	12.70	11.43	97.5	0.956	11 200	137	847	111	1 120	274 000	1.02
x9.5	323.9	9.53	8.58	73.9	0.725	8 500	106	653	112	853	211 000	1.02
x6.4	323.9	6.35	5.72	49.7	0.488	5 720	72.4	447	113	579	145 000	1.02
HSS 273												
x13	273.1	12.70	11.43	81.6	0.800	9 400	80.6	590	92.6	783	161 000	0.858
x9.5	273.1	9.53	8.58	61.9	0.608	7 130	62.4	457	93.6	601	125 000	0.858
x7.9	273.1	7.94	7.15	51.9	0.509	5 970	52.9	387	94.1	506	106 000	0.858
x6.4	273.1	6.35	5.72	41.8	0.410	4 800	43.0	315	94.6	409	85 900	0.858
x4.8	273.1	4.78	4.30	31.6	0.310	3 630	32.8	240	95.0	311	65 600	0.858
HSS 245												
x9.5	244.5	9.53	8.58	55.2	0.542	6 360	44.3	362	83.5	478	88 600	0.768
x6.4	244.5	6.35	5.72	37.3	0.366	4 290	30.6	250	84.4	326	61 200	0.768
HSS 219												
x13	219.1	12.70	11.43	64.6	0.634	7 460	40.3	368	73.5	493	80 600	0.688
x9.5	219.1	9.53	8.58	49.3	0.483	5 670	31.5	287	74.5	380	63 000	0.688
x6.4	219.1	6.35	5.72	33.3	0.327	3 830	21.8	199	75.5	260	43 700	0.688
x4.8	219.1	4.78	4.30	25.3	0.248	2 900	16.7	153	76.0	198	33 500	0.688
HSS 178												
x13	177.8	12.70	11.43	51.7	0.507	5 970	20.8	234	59.0	317	41 500	0.559
x9.5	177.8	9.53	8.58	39.5	0.388	4 560	16.4	184	59.9	246	32 700	0.559

* Imported section

HOLLOW STRUCTURAL SECTIONS
ASTM A500
Round

PROPERTIES AND DIMENSIONS

Section	Outside Dimension	Wall Thickness Nominal	Wall Thickness Design	Mass	Dead Load	Area	I	S	r	Z	Torsional Constant J	Surface Area
mm x mm x mm	mm	mm	mm	kg/m	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	m^2/ m
HSS 168												
x13	168.3	12.70	11.43	48.7	0.478	5 630	17.4	207	55.6	282	34 800	0.529
x9.5	168.3	9.53	8.58	37.3	0.366	4 310	13.8	164	56.6	219	27 500	0.529
x6.4	168.3	6.35	5.72	25.4	0.249	2 920	9.66	115	57.5	151	19 300	0.529
x4.8	168.3	4.78	4.30	19.3	0.189	2 220	7.45	88.6	58.0	116	14 900	0.529
HSS 141												
x13	141.3	12.70	11.43	40.3	0.395	4 660	9.91	140	46.1	193	19 800	0.444
x9.5	141.3	9.53	8.58	31.0	0.304	3 580	7.91	112	47.0	151	15 800	0.444
x6.4	141.3	6.35	5.72	21.1	0.207	2 440	5.61	79.4	48.0	105	11 200	0.444
HSS 127												
x9.5	127.0	9.53	8.58	27.6	0.271	3 190	5.62	88.6	42.0	121	11 200	0.399
x6.4	127.0	6.35	5.72	18.9	0.185	2 180	4.02	63.2	42.9	84.2	8 030	0.399
HSS 89												
x6.4	88.9	6.35	5.72	12.9	0.127	1 490	1.30	29.2	29.5	39.6	2 600	0.279
x4.8	88.9	4.78	4.30	9.92	0.097	1 140	1.03	23.1	29.9	30.8	2 050	0.279
x3.2	88.9	3.18	2.86	6.72	0.066	773	0.716	16.1	30.4	21.2	1 430	0.279
HSS 76												
x6.4	76.2	6.35	5.72	10.9	0.107	1 270	0.792	20.8	25.0	28.5	1 580	0.239
x4.8	76.2	4.78	4.30	8.42	0.083	971	0.630	16.5	25.5	22.3	1 260	0.239
x3.2	76.2	3.18	2.86	5.73	0.056	659	0.444	11.6	25.9	15.4	887	0.239
HSS 73												
x6.4	73.0	6.35	5.72	10.4	0.102	1 210	0.689	18.9	23.9	26.0	1 380	0.229
x4.8	73.0	4.78	4.30	8.04	0.079	928	0.550	15.1	24.3	20.3	1 100	0.229
x3.2	73.0	3.18	2.86	5.48	0.054	630	0.388	10.6	24.8	14.1	776	0.229
HSS 64												
x6.4	63.5	6.35	5.72	8.95	0.088	1 040	0.438	13.8	20.5	19.2	875	0.199
x4.8	63.5	4.78	4.30	6.92	0.068	800	0.352	11.1	21.0	15.1	704	0.199
x3.2	63.5	3.18	2.86	4.73	0.046	545	0.251	7.91	21.5	10.5	502	0.199
HSS 60												
x6.4	60.3	6.35	5.72	8.45	0.083	981	0.369	12.2	19.4	17.1	738	0.189
x4.8	60.3	4.78	4.30	6.54	0.064	756	0.298	9.89	19.9	13.5	597	0.189
x3.2	60.3	3.18	2.86	4.48	0.044	516	0.213	7.08	20.3	9.44	427	0.189
HSS 48												
x4.8	48.3	4.78	4.30	5.13	0.050	594	0.145	6.01	15.6	8.35	290	0.152
x3.2	48.3	3.18	2.86	3.54	0.035	408	0.106	4.38	16.1	5.91	212	0.152
HSS 42												
x3.2	42.2	3.18	2.86	3.06	0.030	353	0.069	3.26	13.9	4.43	137	0.133

NOTES

PIPE

General

Tables of properties and dimensions for steel pipe provided on the following pages are based on ASTM A53 "Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless". Although not a normal structural quality steel, pipe produced in accordance with the ASTM A53 Standard is available in two grades with the following mechanical properties:

Grade A: $F_y = 205$ MPa, $F_u = 330$ MPa

Grade B: $F_y = 240$ MPa, $F_u = 415$ MPa

and in three types:

F: Furnace-butt-welded, continuous welded Grade A

E: Electric-resistance-welded, Grades A and B

S: Seamless, Grades A and B

Ordering Information

When ordering pipe according to ASTM A53, the size may be specified using either the NPS (nominal pipe size) designator or DN (diameter nominal) designator. The wall thickness of pipe is expressed in terms of "standard wall" (STD), "extra strong" (XS), "double extra strong" (XXS), and in terms of "schedule numbers" (Sch). STD is the same as Sch 40 for all sizes up to and including 273.0 mm outside diameter; XS is the same as Sch 80 for all sizes up to and including 219.1 mm outside diameter; and XXS is the next heavier pipe to the Sch 160 pipe for all sizes up to and including 168.3 mm outside diameter. See ASTM A53 for further information.

Tolerances and Section Properties

Permissible tolerances for pipe are ± 1% on the outside diameter and ± 10% on the mass. The under-tolerance on the wall thickness is 12.5%.

Tabulated section properties (Area, I, S, r, Z and J) are based on a design wall thickness taken equal to 90% of the nominal thickness.

PIPE

ASTM A53

PROPERTIES AND DIMENSIONS

DN Designator	NPS Designator	Weight Class*	Mass	Dead Load	Outside Diameter	Nominal Wall Thickness	Design Wall Thickness
			kg/m	kN/m	mm	mm	mm
300	12	XXS	187	1.83	323.8	25.40	22.86
		XS	97.4	0.956	323.8	12.70	11.43
		STD	73.8	0.724	323.8	9.52	8.57
250	10	XXS	155	1.52	273.0	25.40	22.86
		XS	81.5	0.800	273.0	12.70	11.43
		STD	60.3	0.591	273.0	9.27	8.34
200	8	XXS	108	1.06	219.1	22.22	20.00
		XS	64.6	0.634	219.1	12.70	11.43
		STD	42.6	0.417	219.1	8.18	7.36
150	6	XXS	79.2	0.777	168.3	21.95	19.76
		XS	42.6	0.418	168.3	10.97	9.87
		STD	28.3	0.277	168.3	7.11	6.40
125	5	XXS	57.4	0.563	141.3	19.05	17.15
		XS	30.9	0.304	141.3	9.52	8.57
		STD	21.8	0.214	141.3	6.55	5.90
100	4	XXS	41.0	0.403	114.3	17.12	15.41
		XS	22.3	0.219	114.3	8.56	7.70
		STD	16.1	0.158	114.3	6.02	5.42
90	3½	XS	18.6	0.183	101.6	8.08	7.27
		STD	13.6	0.133	101.6	5.74	5.17
80	3	XXS	27.7	0.272	88.9	15.24	13.72
		XS	15.3	0.150	88.9	7.62	6.86
		STD	11.3	0.111	88.9	5.49	4.94
65	2½	XXS	20.4	0.200	73.0	14.02	12.62
		XS	11.4	0.112	73.0	7.01	6.31
		STD	8.63	0.084 7	73.0	5.16	4.64
50	2	XXS	13.4	0.132	60.3	11.07	9.96
		XS	7.48	0.073 4	60.3	5.54	4.99
		STD	5.44	0.053 4	60.3	3.91	3.52
40	1½	XXS	9.56	0.093 8	48.3	10.16	9.14
		XS	5.41	0.053 1	48.3	5.08	4.57
		STD	4.05	0.039 7	48.3	3.68	3.31
32	1¼	XXS	7.77	0.076 2	42.2	9.70	8.73
		XS	4.47	0.043 9	42.2	4.85	4.37
		STD	3.39	0.033 3	42.2	3.56	3.20
25	1	XXS	5.45	0.053 5	33.4	9.09	8.18
		XS	3.24	0.031 8	33.4	4.55	4.10
		STD	2.50	0.024 5	33.4	3.38	3.04
20	¾	XXS	3.64	0.035 7	26.7	7.82	7.04
		XS	2.20	0.021 6	26.7	3.91	3.52
		STD	1.69	0.016 6	26.7	2.87	2.58
15	½	XXS	2.55	0.025 0	21.3	7.47	6.72
		XS	1.62	0.015 9	21.3	3.73	3.36
		STD	1.27	0.012 5	21.3	2.77	2.49

* Weight Class: Standard Weight - STD, Extra Strong - XS, Double Extra Strong - XXS

Area	I	S	r	Z	J	Surface Area
mm^2	10^6 mm^4	10^3 mm^3	mm	10^3 mm^3	10^3 mm^4	m^2/m
21 600	246	1 520	107	2 070	492 000	1.02
11 200	137	846	111	1 120	274 000	1.02
8 490	105	652	111	852	211 000	1.02
18 000	142	1 040	88.8	1 430	283 000	0.858
9 390	80.5	590	92.6	783	161 000	0.858
6 930	60.8	445	93.6	584	122 000	0.858
12 500	62.6	572	70.7	795	125 000	0.688
7 460	40.3	368	73.5	493	80 600	0.688
4 900	27.5	251	74.9	330	54 900	0.688
9 220	25.9	308	53.0	439	51 800	0.529
4 910	15.5	184	56.1	248	30 900	0.529
3 260	10.7	127	57.3	168	21 400	0.529
6 690	13.1	186	44.3	266	26 300	0.444
3 570	7.90	112	47.0	151	15 800	0.444
2 510	5.76	81.6	47.9	108	11 500	0.444
4 790	5.99	105	35.4	152	12 000	0.359
2 580	3.68	64.4	37.8	87.7	7 360	0.359
1 850	2.75	48.2	38.5	64.3	5 510	0.359
2 150	2.41	47.5	33.4	64.8	4 820	0.319
1 570	1.83	35.9	34.1	48.1	3 650	0.319
3 240	2.37	53.2	27.0	78.4	4 730	0.279
1 770	1.50	33.7	29.1	46.3	3 000	0.279
1 300	1.15	25.9	29.7	34.9	2 300	0.279
2 390	1.14	31.2	21.8	46.7	2 280	0.229
1 320	0.742	20.3	23.7	28.1	1 480	0.229
996	0.585	16.0	24.2	21.7	1 170	0.229
1 580	0.518	17.2	18.1	25.6	1 040	0.189
867	0.334	11.1	19.6	15.3	669	0.189
628	0.254	8.42	20.1	11.4	508	0.189
1 120	0.227	9.41	14.2	14.3	455	0.152
628	0.152	6.28	15.5	8.77	303	0.152
468	0.119	4.93	15.9	6.71	238	0.152
918	0.137	6.51	12.2	10.0	275	0.133
519	0.094 1	4.46	13.5	6.28	188	0.133
392	0.075 0	3.56	13.8	4.88	150	0.133
648	0.056 9	3.41	9.37	5.39	114	0.105
377	0.041 3	2.47	10.5	3.54	82.6	0.105
290	0.033 7	2.02	10.8	2.81	67.5	0.105
435	0.023 7	1.78	7.38	2.84	47.4	0.083 9
256	0.017 6	1.32	8.29	1.91	35.2	0.083 9
196	0.014 4	1.08	8.58	1.51	28.8	0.083 9
308	0.009 92	0.931	5.68	1.53	19.8	0.066 9
189	0.007 89	0.740	6.45	1.09	15.8	0.066 9
147	0.006 62	0.622	6.71	0.886	13.2	0.066 9

Note: Section properties are based on a design wall thickness taken equal to 90% of the nominal thickness.

NOTES

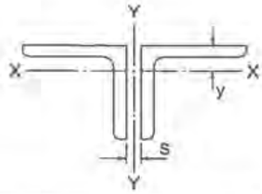
BUILT-UP SECTIONS

Built-up sections may be fabricated from plates and shapes in various configurations to produce efficient and economical structural sections. Generally, the components are joined by welding, although bolting may also be used for some combinations. Frequently used built-up sections include double angles back-to-back, double channels back-to-back or toe-to-toe, and a channel or C shape in combination with a W shape.

Tables of properties and dimensions on the following pages include: equal-leg angles, unequal-leg angles with long legs back-to-back and with short legs back-to-back, double channels, and built-up shapes consisting of W shapes and channels (C shapes). For information on β_x, the monosymmetry constant (or asymmetry parameter) for singly-symmetric beams, see CSA S16-14 Clause 13.6(e).

Many other combinations of built-up members are possible. The information on built-up sections concludes with diagrams and formulas for computing the properties of some possible combinations.

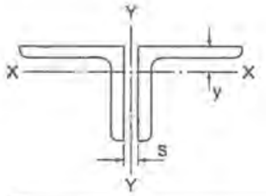
TWO ANGLES EQUAL LEGS
Back-to-Back



PROPERTIES OF SECTIONS

Designation	Mass of 2 Angles	Dead Load	Area of 2 Angles	Axis X-X				Radii of Gyration about Axis Y-Y					
				I	S	r	y	Back-to-back spacing, s, millimetres					
	kg/m	kN/m	mm²	10^6 mm⁴	10^3 mm³	mm	mm	0	8	10	12	16	20
L254x254													
x32	238	2.33	30 200	181	1010	77.3	75.2	108	111	111	112	114	115
x29	216	2.11	27 400	166	921	77.7	74.0	107	110	111	112	113	114
x25	192	1.89	24 600	150	827	78.2	72.9	107	110	110	111	112	114
x22	169	1.66	21 600	133	731	78.6	71.7	106	109	110	111	112	113
x19	146	1.44	18 600	117	636	79.1	70.6	106	109	109	110	111	113
L203x203													
x29	169	1.66	21 600	81.4	574	61.4	61.2	86.7	89.6	90.3	91.0	92.5	94.0
x25	152	1.49	19 400	73.8	517	61.8	60.1	86.2	89.0	89.7	90.5	91.9	93.4
x22	134	1.31	17 000	66.0	458	62.2	58.9	85.7	88.5	89.2	89.9	91.4	92.8
x19	116	1.13	14 700	57.7	398	62.7	57.8	85.2	88.0	88.7	89.4	90.8	92.3
x16	97.4	0.955	12 400	49.4	337	63.1	56.6	84.8	87.5	88.2	88.9	90.3	91.8
x14	88.0	0.862	11 200	44.9	306	63.3	56.0	84.6	87.3	88.0	88.7	90.1	91.5
x13	78.6	0.769	10 000	40.4	274	63.6	55.5	84.4	87.0	87.7	88.4	89.8	91.2
L152x152													
x25	111	1.09	14 200	29.3	279	45.5	47.2	65.6	68.5	69.3	70.0	71.5	73.1
x22	98.6	0.963	12 600	26.3	249	45.9	46.1	65.0	67.9	68.7	69.4	70.9	72.5
x19	85.4	0.834	10 900	23.2	217	46.3	45.0	64.5	67.4	68.1	68.8	70.3	71.9
x16	72.0	0.705	9 180	20.0	185	46.7	43.9	64.1	66.9	67.6	68.3	69.8	71.3
x14	65.2	0.638	8 300	18.2	168	46.9	43.3	63.8	66.6	67.3	68.0	69.5	71.0
x13	58.4	0.570	7 420	16.4	150	47.1	42.7	63.6	66.3	67.1	67.8	69.2	70.7
x11	51.2	0.501	6 540	14.6	133	47.4	42.1	63.4	66.1	66.8	67.5	69.0	70.4
x9.5	44.4	0.432	5 620	12.7	115	47.6	41.5	63.2	65.9	66.6	67.3	68.7	70.1
x7.9	37.0	0.362	4 720	10.8	96.8	47.8	41.0	63.0	65.6	66.3	67.0	68.4	69.9
L127x127													
x22	81.0	0.792	10 300	14.8	169	37.9	39.8	55.0	57.9	58.7	59.4	61.0	62.6
x19	70.2	0.687	8 960	13.1	148	38.3	38.7	54.4	57.3	58.1	58.8	60.4	61.9
x16	59.6	0.583	7 560	11.3	127	38.7	37.6	53.9	56.8	57.5	58.3	59.8	61.3
x13	48.2	0.472	6 140	9.37	103	39.1	36.4	53.4	56.2	57.0	57.7	59.2	60.7
x11	42.6	0.415	5 400	8.33	91.4	39.3	35.8	53.2	56.0	56.7	57.4	58.9	60.4
x9.5	36.6	0.359	4 660	7.28	79.4	39.5	35.3	53.0	55.7	56.4	57.2	58.6	60.1
x7.9	30.6	0.301	3 920	6.18	66.9	39.8	34.7	52.8	55.5	56.2	56.9	58.3	59.8
L102x102													
x19	55.0	0.541	7 020	6.45	92.7	30.3	32.4	44.3	47.3	48.1	48.9	50.5	52.1
x16	46.8	0.460	5 940	5.62	79.5	30.7	31.3	43.8	46.7	47.5	48.3	49.8	51.4
x13	38.0	0.374	4 840	4.69	65.3	31.1	30.2	43.3	46.2	46.9	47.7	49.2	50.8
x11	33.6	0.330	4 280	4.19	57.8	31.3	29.6	43.0	45.9	46.6	47.4	48.9	50.4
x9.5	29.2	0.285	3 700	3.68	50.4	31.5	29.0	42.8	45.6	46.4	47.1	48.6	50.1
x7.9	24.4	0.240	3 100	3.13	42.6	31.7	28.4	42.6	45.4	46.1	46.8	48.3	49.8
x6.4	19.6	0.193	2 500	2.56	34.5	31.9	27.9	42.4	45.1	45.8	46.5	48.0	49.5
L89x89													
x13	33.0	0.323	4 200	3.03	48.8	26.9	26.9	38.0	40.9	41.7	42.4	44.0	45.6
x11	29.2	0.285	3 700	2.71	43.3	27.1	26.3	37.7	40.6	41.4	42.1	43.7	45.3
x9.5	25.2	0.247	3 200	2.39	37.8	27.3	25.7	37.5	40.3	41.1	41.8	43.4	45.0
x7.9	21.4	0.208	2 700	2.04	32.0	27.5	25.2	37.3	40.1	40.8	41.6	43.1	44.6
x6.4	17.2	0.168	2 180	1.67	26.0	27.7	24.6	37.0	39.8	40.5	41.3	42.8	44.3

See *Rolled Structural Shapes* for further information on the properties of angles.



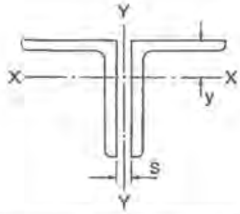
TWO ANGLES EQUAL LEGS
Back-to-Back

PROPERTIES OF SECTIONS

Designation	Mass of 2 Angles	Dead Load	Area of 2 Angles	Axis X-X				Radii of Gyration about Axis Y-Y					
				I	S	r	y	Back-to-back spacing, s, millimetres					
	kg/m	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	mm	0	8	10	12	16	20
L76x76													
x13	28.0	0.273	3 540	1.85	35.1	22.8	23.7	32.9	35.9	36.6	37.4	39.0	40.7
x11	24.8	0.241	3 140	1.66	31.2	23.0	23.1	32.6	35.5	36.3	37.1	38.7	40.3
x9.5	21.4	0.210	2 720	1.47	27.3	23.2	22.5	32.3	35.3	36.0	36.8	38.4	40.0
x7.9	18.2	0.177	2 300	1.26	23.2	23.4	22.0	32.1	35.0	35.7	36.5	38.0	39.6
x6.4	14.6	0.143	1 860	1.04	18.9	23.6	21.4	31.9	34.7	35.4	36.2	37.7	39.3
x4.8	11.0	0.108	1 410	0.800	14.4	23.9	20.8	31.7	34.4	35.2	35.9	37.4	39.0
L64x64													
x13	22.8	0.223	2 900	1.02	23.7	18.8	20.5	27.8	30.8	31.6	32.4	34.1	35.8
x9.5	17.4	0.172	2 240	0.819	18.6	19.1	19.4	27.2	30.2	31.0	31.8	33.4	35.0
x7.9	14.8	0.146	1 880	0.707	15.8	19.3	18.8	27.0	29.9	30.7	31.4	33.0	34.7
x6.4	12.2	0.118	1 540	0.585	12.9	19.5	18.2	26.7	29.6	30.3	31.1	32.7	34.3
x4.8	9.2	0.090	1 160	0.455	9.92	19.8	17.6	26.5	29.3	30.1	30.8	32.4	34.0
L51x51													
x9.5	14.0	0.135	1 750	0.399	11.5	15.1	16.2	22.1	25.2	26.0	26.8	28.5	30.2
x7.9	11.6	0.114	1 480	0.347	9.84	15.3	15.6	21.8	24.8	25.6	26.4	28.1	29.8
x6.4	9.4	0.093	1 210	0.289	8.09	15.5	15.0	21.6	24.5	25.3	26.1	27.7	29.4
x4.8	7.2	0.071	922	0.227	6.24	15.7	14.5	21.3	24.2	25.0	25.8	27.4	29.1
x3.2	4.8	0.048	624	0.158	4.29	15.9	13.9	21.1	23.9	24.7	25.5	27.1	28.7
L44x44													
x6.4	8.2	0.081	1 050	0.190	6.11	13.4	13.4	19.0	22.0	22.8	23.6	25.3	27.0
x4.8	6.2	0.062	802	0.150	4.73	13.7	12.9	18.8	21.7	22.5	23.3	24.9	26.6
x3.2	4.2	0.042	544	0.105	3.26	13.9	12.3	18.5	21.4	22.2	23.0	24.6	26.3
L38x38													
x6.4	6.8	0.068	888	0.115	4.39	11.4	11.8	16.4	19.5	20.3	21.2	22.9	24.6
x4.8	5.4	0.052	680	0.091 5	3.41	11.6	11.3	16.2	19.2	20.0	20.8	22.5	24.2
x4.0	4.4	0.044	572	0.078 6	2.90	11.7	11.0	16.1	19.0	19.8	20.6	22.3	24.0
x3.2	3.6	0.036	464	0.064 8	2.37	11.8	10.7	15.9	18.9	19.6	20.5	22.1	23.8
L32x32													
x6.4	5.6	0.056	726	0.064 2	2.98	9.40	10.2	13.9	17.1	17.9	18.8	20.5	22.3
x4.8	4.4	0.043	560	0.051 4	2.33	9.58	9.69	13.6	16.7	17.5	18.4	20.1	21.9
x3.2	3.0	0.030	384	0.036 8	1.62	9.79	9.12	13.4	16.4	17.2	18.0	19.7	21.5
L25x25													
x6.4	4.4	0.043	566	0.030 7	1.83	7.37	8.62	11.3	14.6	15.5	16.4	18.2	20.0
x4.8	3.6	0.034	438	0.024 9	1.44	7.54	8.07	11.0	14.2	15.1	16.0	17.8	19.6
x3.2	2.4	0.023	302	0.018 1	1.01	7.73	7.52	10.8	13.9	14.7	15.6	17.3	19.1
L19x19													
x3.2	1.8	0.017	222	0.007 3	0.55	5.72	5.93	8.2	11.5	12.3	13.2	15.1	16.9

See *Rolled Structural Shapes* for further information on the properties of angles.

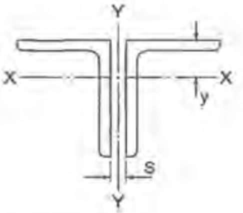
TWO ANGLES UNEQUAL LEGS
Long Legs Back-to-Back



PROPERTIES OF SECTIONS

Designation	Mass of 2 Angles	Dead Load	Area of 2 Angles	Axis X-X				Radii of Gyration about Axis Y-Y					
				I	S	r	y	Back-to-back spacing, s, millimetres					
	kg/m	kN/m	mm²	10^6 mm⁴	10^3 mm³	mm	mm	0	8	10	12	16	20
L203x152													
x25	131	1.29	16 800	67.0	494	63.3	67.4	60.6	63.4	64.1	64.9	66.3	67.9
x22	116	1.14	14 800	59.9	438	63.7	66.2	60.1	62.8	63.6	64.3	65.7	67.2
x19	100	0.983	12 800	52.5	381	64.1	65.1	59.6	62.3	63.0	63.7	65.2	66.6
x16	84.4	0.830	10 800	44.9	323	64.6	64.0	59.1	61.8	62.5	63.2	64.6	66.1
x14	76.2	0.750	9 760	40.9	293	64.8	63.4	58.9	61.6	62.3	63.0	64.4	65.8
x13	68.2	0.669	8 700	36.8	262	65.0	62.8	58.7	61.4	62.0	62.7	64.1	65.5
x11	59.8	0.588	7 660	32.5	231	65.3	62.2	58.5	61.1	61.8	62.5	63.9	65.3
L203x102													
x25	111	1.09	14 200	57.9	460	63.8	77.2	37.4	40.4	41.2	41.9	43.5	45.1
x22	98.6	0.967	12 600	51.9	408	64.3	76.0	36.8	39.7	40.4	41.2	42.8	44.3
x19	85.0	0.837	10 900	45.5	355	64.7	74.8	36.3	39.0	39.8	40.5	42.0	43.6
x16	72.0	0.708	9 180	39.1	302	65.2	73.6	35.8	38.5	39.2	39.9	41.4	42.9
x14	64.8	0.640	8 300	35.6	274	65.4	73.0	35.5	38.2	38.9	39.6	41.1	42.6
x13	58.0	0.572	7 420	32.0	245	65.7	72.4	35.3	37.9	38.6	39.3	40.7	42.2
x11	51.2	0.502	6 520	28.3	216	65.9	71.8	35.1	37.6	38.3	39.0	40.4	41.9
L178x102													
x19	77.6	0.764	9 920	31.5	276	56.4	63.7	37.8	40.6	41.4	42.1	43.6	45.2
x16	65.4	0.647	8 360	27.1	235	56.8	62.6	37.3	40.0	40.8	41.5	43.0	44.5
x13	53.0	0.523	6 780	22.3	191	57.3	61.4	36.8	39.5	40.2	40.9	42.4	43.9
x11	46.8	0.460	5 960	19.8	169	57.5	60.8	36.6	39.2	39.9	40.6	42.1	43.6
x9.5	40.4	0.397	5 140	17.2	146	57.8	60.2	36.4	39.0	39.7	40.4	41.8	43.3
L152x102													
x22	80.6	0.792	10 300	22.9	233	47.2	53.7	40.2	43.2	43.9	44.7	46.3	47.9
x19	70.0	0.687	8 960	20.2	203	47.6	52.5	39.7	42.5	43.3	44.0	45.6	47.2
x16	59.2	0.583	7 560	17.5	174	48.0	51.4	39.2	42.0	42.7	43.4	44.9	46.5
x14	53.6	0.528	6 860	16.0	158	48.2	50.8	38.9	41.7	42.4	43.1	44.6	46.2
x13	48.0	0.472	6 120	14.4	141	48.5	50.2	38.7	41.4	42.1	42.8	44.3	45.8
x11	42.4	0.415	5 400	12.8	125	48.7	49.6	38.5	41.1	41.9	42.6	44.0	45.5
x9.5	36.4	0.359	4 660	11.2	108	48.9	49.1	38.3	40.9	41.6	42.3	43.7	45.2
x7.9	30.6	0.301	3 900	9.44	91.2	49.2	48.5	38.1	40.7	41.3	42.0	43.5	44.9
L152x89													
x13	45.4	0.446	5 800	13.7	138	48.6	52.7	32.5	35.3	36.0	36.7	38.2	39.8
x9.5	34.6	0.339	4 420	10.6	106	49.1	51.6	32.1	34.7	35.4	36.2	37.6	39.1
x7.9	29.0	0.285	3 700	9.01	89.1	49.3	51.0	31.9	34.5	35.2	35.9	37.3	38.8
L127x89													
x19	58.6	0.576	7 500	11.6	140	39.3	44.3	35.4	38.4	39.2	39.9	41.5	43.1
x16	49.8	0.490	6 340	10.0	120	39.7	43.2	34.9	37.8	38.5	39.3	40.8	42.4
x13	40.4	0.397	5 160	8.31	97.9	40.1	42.1	34.4	37.2	37.9	38.7	40.2	41.8
x9.5	30.8	0.303	3 940	6.48	75.2	40.6	40.9	33.9	36.6	37.4	38.1	39.6	41.1
x7.9	25.8	0.254	3 300	5.50	63.5	40.8	40.3	33.7	36.4	37.1	37.8	39.3	40.8
x6.4	20.8	0.205	2 660	4.48	51.4	41.0	39.7	33.5	36.2	36.8	37.5	39.0	40.5

See *Rolled Structural Shapes* for further information on the properties of angles.



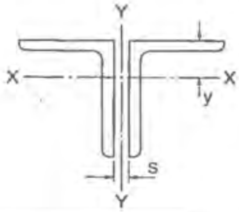
TWO ANGLES UNEQUAL LEGS
Long Legs Back-to-Back

PROPERTIES OF SECTIONS

Designation	Mass of 2 Angles	Dead Load	Area of 2 Angles	Axis X-X				Radii of Gyration about Axis Y-Y					
				I	S	r	y	Back-to-back spacing, s, millimetres					
	kg/m	kN/m	mm²	10^6 mm⁴	10^3 mm³	mm	mm	0	8	10	12	16	20
L127x76													
x13	38.0	0.372	4 840	7.87	95.3	40.3	44.5	28.4	31.2	32.0	32.7	34.3	35.9
x11	33.4	0.328	4 280	7.01	84.4	40.6	43.9	28.2	30.9	31.7	32.4	33.9	35.5
x9.5	29.0	0.284	3 700	6.14	73.3	40.8	43.3	27.9	30.6	31.4	32.1	33.6	35.2
x7.9	24.2	0.239	3 100	5.21	61.9	41.0	42.7	27.7	30.4	31.1	31.8	33.3	34.8
x6.4	19.6	0.192	2 500	4.25	50.1	41.2	42.1	27.5	30.1	30.8	31.5	33.0	34.5
L102x89													
x13	35.2	0.348	4 520	4.48	63.9	31.5	31.9	36.6	39.5	40.2	41.0	42.6	44.1
x9.5	27.0	0.266	3 440	3.52	49.4	31.9	30.8	36.1	38.9	39.7	40.4	41.9	43.5
x7.9	22.8	0.224	2 900	3.00	41.7	32.1	30.2	35.9	38.7	39.4	40.1	41.6	43.2
x6.4	18.4	0.180	2 340	2.45	33.9	32.3	29.6	35.7	38.4	39.1	39.9	41.3	42.9
L102x76													
x16	40.4	0.397	5 140	5.09	75.9	31.4	35.0	30.9	33.9	34.6	35.4	37.0	38.7
x13	32.8	0.324	4 200	4.25	62.4	31.8	33.9	30.3	33.2	34.0	34.8	36.3	37.9
x9.5	25.2	0.247	3 200	3.34	48.2	32.2	32.7	29.8	32.6	33.4	34.1	35.7	37.2
x7.9	21.4	0.208	2 700	2.85	40.7	32.4	32.1	29.6	32.4	33.1	33.8	35.4	36.9
x6.4	17.2	0.168	2 180	2.33	33.1	32.7	31.6	29.4	32.1	32.8	33.6	35.0	36.6
L89x76													
x13	30.2	0.298	3 880	2.87	47.7	27.3	28.6	31.5	34.5	35.2	36.0	37.6	39.2
x11	27.0	0.263	3 420	2.58	42.3	27.5	28.0	31.3	34.2	34.9	35.7	37.3	38.9
x9.5	23.4	0.228	2 960	2.27	36.9	27.7	27.4	31.0	33.9	34.6	35.4	36.9	38.5
x7.9	19.6	0.192	2 500	1.94	31.3	27.9	26.9	30.8	33.6	34.3	35.1	36.6	38.2
x6.4	16.0	0.155	2 020	1.59	25.4	28.1	26.3	30.6	33.3	34.1	34.8	36.3	37.9
L89x64													
x13	27.8	0.273	3 540	2.70	46.2	27.6	30.6	25.3	28.3	29.1	29.8	31.5	33.1
x9.5	21.4	0.210	2 720	2.13	35.9	28.0	29.5	24.8	27.6	28.4	29.2	30.8	32.4
x7.9	18.0	0.177	2 300	1.82	30.4	28.2	28.9	24.5	27.4	28.1	28.9	30.4	32.0
x6.4	14.6	0.143	1 860	1.50	24.7	28.4	28.3	24.3	27.1	27.8	28.6	30.1	31.7
L76x64													
x13	25.2	0.248	3 220	1.73	34.1	23.2	25.4	26.4	29.5	30.2	31.0	32.7	34.4
x11	22.6	0.220	2 860	1.56	30.4	23.4	24.8	26.2	29.1	29.9	30.7	32.3	34.0
x9.5	19.6	0.191	2 480	1.38	26.6	23.6	24.3	25.9	28.8	29.6	30.4	32.0	33.6
x7.9	16.6	0.161	2 100	1.18	22.6	23.8	23.7	25.7	28.5	29.3	30.0	31.6	33.3
x6.4	13.4	0.130	1 690	0.977	18.4	24.0	23.1	25.4	28.2	29.0	29.7	31.3	32.9
x4.8	10.2	0.099	1 290	0.755	14.1	24.2	22.6	25.2	28.0	28.7	29.4	31.0	32.6
L76x51													
x13	23.0	0.223	2 900	1.60	32.9	23.5	27.5	20.3	23.4	24.2	25.0	26.7	28.4
x9.5	17.6	0.172	2 240	1.28	25.6	23.9	26.4	19.7	22.7	23.5	24.3	25.9	27.6
x7.9	14.8	0.146	1 880	1.10	21.8	24.1	25.8	19.5	22.4	23.1	23.9	25.6	27.2
x6.4	12.2	0.118	1 540	0.905	17.8	24.3	25.2	19.2	22.1	22.8	23.6	25.2	26.8
x4.8	9.2	0.090	1 160	0.700	13.6	24.5	24.6	19.0	21.8	22.5	23.3	24.8	26.5

See *Rolled Structural Shapes* for further information on the properties of angles.

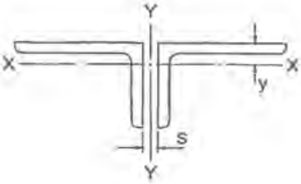
TWO ANGLES UNEQUAL LEGS
Long Legs Back-to-Back



PROPERTIES OF SECTIONS

Designation	Mass of 2 Angles	Dead Load	Area of 2 Angles	Axis X-X				Radii of Gyration about Axis Y-Y					
				I	S	r	y	Back-to-back spacing, s, millimetres					
	kg/m	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	mm	0	8	10	12	16	20
L64x51													
x9.5	15.8	0.154	2 000	0.760	17.9	19.5	21.1	20.8	23.8	24.6	25.4	27.1	28.8
x7.9	13.4	0.130	1 690	0.656	15.3	19.7	20.6	20.5	23.5	24.3	25.1	26.7	28.4
x6.4	10.8	0.106	1 370	0.544	12.5	19.9	20.0	20.3	23.2	23.9	24.7	26.3	28.0
x4.8	8.4	0.080	1 040	0.423	9.60	20.1	19.4	20.1	22.9	23.6	24.4	26.0	27.6
L64x38													
x6.4	9.6	0.093	1 210	0.492	11.9	20.2	22.2	14.2	17.1	17.9	18.8	20.4	22.2
x4.8	7.2	0.071	922	0.383	9.16	20.4	21.6	14.0	16.8	17.6	18.4	20.0	21.8
L51x38													
x6.4	8.4	0.081	1 050	0.263	7.74	15.8	16.9	15.2	18.2	19.0	19.8	21.5	23.3
x4.8	6.2	0.062	802	0.206	5.97	16.0	16.3	14.9	17.9	18.6	19.5	21.1	22.8
x3.2	4.2	0.042	544	0.144	4.11	16.3	15.7	14.7	17.5	18.3	19.1	20.7	22.4

See *Rolled Structural Shapes* for further information on the properties of angles.



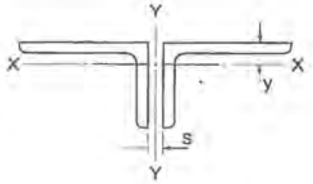
TWO ANGLES UNEQUAL LEGS
Short Legs Back-to-Back

PROPERTIES OF SECTIONS

Designation	Mass of 2 Angles	Dead Load	Area of 2 Angles	Axis X-X				Radii of Gyration about Axis Y-Y					
				I	S	r	y	Back-to-back spacing, s, millimetres					
	kg/m	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	mm	0	8	10	12	16	20
L203x152													
x25	131	1.29	16 800	32.0	291	43.7	41.9	92.4	95.4	96.1	96.9	98.4	99.9
x22	116	1.14	14 800	28.8	259	44.1	40.7	91.9	94.8	95.6	96.3	97.8	99.3
x19	100	0.983	12 800	25.3	225	44.5	39.6	91.4	94.3	95.0	95.7	97.2	98.7
x16	84.4	0.830	10 800	21.8	192	44.9	38.5	90.9	93.7	94.5	95.2	96.7	98.2
x14	76.2	0.750	9 760	19.9	174	45.2	37.9	90.6	93.5	94.2	94.9	96.4	97.9
x13	68.2	0.669	8 700	17.9	156	45.4	37.3	90.4	93.2	93.9	94.6	96.1	97.6
x11	59.8	0.588	7 660	15.9	138	45.6	36.7	90.1	92.9	93.7	94.4	95.8	97.3
L203x102													
x25	111	1.09	14 200	9.81	130	26.3	26.7	100	103	104	105	106	108
x22	98.6	0.967	12 600	8.87	116	26.6	25.5	99.5	103	103	104	106	107
x19	85.0	0.837	10 900	7.87	101	26.9	24.3	98.9	102	103	104	105	107
x16	72.0	0.708	9 180	6.83	86.6	27.3	23.1	98.3	101	102	103	104	106
x14	64.8	0.640	8 300	6.26	78.8	27.4	22.5	98.0	101	102	103	104	106
x13	58.0	0.572	7 420	5.67	70.9	27.6	21.9	97.8	101	102	102	104	105
x11	51.2	0.502	6 520	5.06	62.7	27.9	21.3	97.5	100	101	102	104	105
L178x102													
x19	77.6	0.764	9 920	7.61	99.7	27.7	25.7	85.1	88.1	88.9	89.7	91.2	92.8
x16	65.4	0.647	8 360	6.61	85.4	28.1	24.6	84.5	87.5	88.3	89.1	90.6	92.2
x13	53.0	0.523	6 780	5.50	69.9	28.5	23.4	84.0	86.9	87.7	88.5	90.0	91.5
x11	46.8	0.460	5 960	4.90	61.9	28.7	22.8	83.7	86.6	87.4	88.2	89.7	91.2
x9.5	40.4	0.397	5 140	4.30	53.9	28.9	22.2	83.4	86.4	87.1	87.9	89.4	90.9
L152x102													
x22	80.6	0.792	10 300	8.20	112	28.2	28.7	71.5	74.5	75.3	76.1	77.6	79.2
x19	70.0	0.687	8 960	7.29	97.9	28.6	27.5	70.9	73.9	74.7	75.4	77.0	78.6
x16	59.2	0.583	7 560	6.34	83.8	28.9	26.4	70.3	73.3	74.1	74.8	76.4	77.9
x14	53.6	0.528	6 860	5.82	76.4	29.1	25.8	70.1	73.0	73.8	74.5	76.1	77.6
x13	48.0	0.472	6 120	5.28	68.7	29.3	25.2	69.8	72.7	73.5	74.2	75.8	77.3
x11	42.4	0.415	5 400	4.71	60.9	29.6	24.6	69.5	72.4	73.2	73.9	75.5	77.0
x9.5	36.4	0.359	4 660	4.13	53.0	29.8	24.1	69.3	72.2	72.9	73.7	75.2	76.7
x7.9	30.6	0.301	3 900	3.51	44.7	30.0	23.5	69.0	71.9	72.6	73.4	74.9	76.4
L152x89													
x13	45.4	0.446	5 800	3.54	52.2	24.7	21.2	71.7	74.7	75.5	76.3	77.8	79.4
x9.5	34.6	0.339	4 420	2.78	40.4	25.1	20.0	71.2	74.2	74.9	75.7	77.2	78.7
x7.9	29.0	0.285	3 700	2.37	34.1	25.3	19.4	70.9	73.9	74.6	75.4	76.9	78.4
L127x89													
x19	58.6	0.576	7 500	4.61	72.5	24.8	25.3	59.2	62.3	63.1	63.9	65.5	67.1
x16	49.8	0.490	6 340	4.03	62.2	25.2	24.2	58.7	61.7	62.5	63.2	64.8	66.4
x13	40.4	0.397	5 160	3.37	51.2	25.6	23.0	58.1	61.1	61.9	62.6	64.2	65.7
x9.5	30.8	0.303	3 940	2.65	39.6	26.0	21.9	57.6	60.5	61.3	62.0	63.6	65.1
x7.9	25.8	0.254	3 300	2.26	33.5	26.2	21.3	57.4	60.3	61.0	61.7	63.3	64.8
x6.4	20.8	0.205	2 660	1.86	27.2	26.4	20.7	57.1	60.0	60.7	61.5	63.0	64.5

See *Rolled Structural Shapes* for further information on the properties of angles.

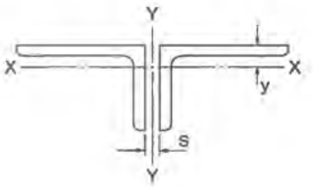
TWO ANGLES UNEQUAL LEGS
Short Legs Back-to-Back



PROPERTIES OF SECTIONS

Designation	Mass of 2 Angles	Dead Load	Area of 2 Angles	Axis X-X				Radii of Gyration about Axis Y-Y					
				I	S	r	y	Back-to-back spacing, s, millimetres					
	kg/m	kN/m	mm²	10⁶ mm⁴	10³ mm³	mm	mm	0	8	10	12	16	20
L127x76													
x13	38.0	0.372	4 840	2.15	37.6	21.1	19.1	60.0	63.0	63.8	64.6	66.2	67.8
x11	33.4	0.328	4 280	1.93	33.4	21.3	18.5	59.7	62.7	63.5	64.3	65.8	67.4
x9.5	29.0	0.284	3 700	1.70	29.1	21.5	17.9	59.5	62.4	63.2	64.0	65.5	67.1
x7.9	24.2	0.239	3 100	1.45	24.7	21.7	17.3	59.2	62.1	62.9	63.7	65.2	66.8
x6.4	19.6	0.192	2 500	1.20	20.1	21.9	16.7	58.9	61.9	62.6	63.4	64.9	66.4
L102x89													
x13	35.2	0.348	4 520	3.16	49.7	26.4	25.4	44.8	47.7	48.5	49.3	50.8	52.4
x9.5	27.0	0.266	3 440	2.49	38.5	26.8	24.2	44.3	47.2	47.9	48.7	50.2	51.8
x7.9	22.8	0.224	2 900	2.13	32.6	27.1	23.6	44.1	46.9	47.6	48.4	49.9	51.4
x6.4	18.4	0.180	2 340	1.74	26.5	27.3	23.1	43.8	46.6	47.4	48.1	49.6	51.1
L102x76													
x16	40.4	0.397	5 140	2.40	44.3	21.6	22.1	47.0	50.1	50.9	51.6	53.2	54.9
x13	32.8	0.324	4 200	2.02	36.6	21.9	21.0	46.5	49.4	50.2	51.0	52.6	54.2
x9.5	25.2	0.247	3 200	1.60	28.4	22.3	19.8	45.9	48.9	49.6	50.4	51.9	53.5
x7.9	21.4	0.208	2 700	1.37	24.1	22.5	19.2	45.7	48.6	49.3	50.1	51.6	53.2
x6.4	17.2	0.168	2 180	1.13	19.6	22.7	18.7	45.4	48.3	49.0	49.8	51.3	52.9
L89x76													
x13	30.2	0.298	3 880	1.94	35.9	22.4	22.2	39.5	42.5	43.2	44.0	45.6	47.2
x11	27.0	0.263	3 420	1.74	31.9	22.6	21.7	39.2	42.2	42.9	43.7	45.3	46.9
x9.5	23.4	0.228	2 960	1.54	27.9	22.8	21.1	39.0	41.9	42.6	43.4	45.0	46.6
x7.9	19.6	0.192	2 500	1.32	23.7	23.0	20.5	38.7	41.6	42.3	43.1	44.6	46.2
x6.4	16.0	0.155	2 020	1.09	19.3	23.2	19.9	38.5	41.3	42.1	42.8	44.3	45.9
L89x64													
x13	27.8	0.273	3 540	1.14	24.9	17.9	17.9	41.2	44.2	45.0	45.8	47.4	49.1
x9.5	21.4	0.210	2 720	0.908	19.4	18.3	16.8	40.6	43.6	44.4	45.2	46.8	48.4
x7.9	18.0	0.177	2 300	0.782	16.5	18.5	16.2	40.4	43.3	44.1	44.9	46.4	48.0
x6.4	14.6	0.143	1 860	0.647	13.5	18.7	15.6	40.1	43.0	43.8	44.5	46.1	47.7
L76x64													
x13	25.2	0.248	3 220	1.08	24.4	18.3	19.1	34.4	37.4	38.2	39.0	40.7	42.3
x11	22.6	0.220	2 860	0.978	21.7	18.5	18.5	34.1	37.1	37.9	38.7	40.3	42.0
x9.5	19.6	0.191	2 480	0.868	19.0	18.7	17.9	33.8	36.8	37.6	38.4	40.0	41.6
x7.9	16.6	0.161	2 100	0.748	16.2	18.9	17.4	33.6	36.5	37.3	38.1	39.6	41.3
x6.4	13.4	0.130	1 690	0.619	13.2	19.1	16.8	33.3	36.2	37.0	37.8	39.3	40.9
x4.8	10.2	0.099	1 290	0.480	10.2	19.3	16.2	33.1	36.0	36.7	37.5	39.0	40.6
L76x51													
x13	23.0	0.223	2 900	0.559	15.5	13.9	14.8	36.2	39.3	40.1	40.9	42.6	44.3
x9.5	17.6	0.172	2 240	0.452	12.2	14.2	13.7	35.6	38.6	39.4	40.2	41.9	43.5
x7.9	14.8	0.146	1 880	0.392	10.4	14.4	13.1	35.3	38.3	39.1	39.9	41.5	43.2
x6.4	12.2	0.118	1 540	0.326	8.52	14.6	12.5	35.0	38.0	38.8	39.6	41.2	42.8
x4.8	9.2	0.090	1 160	0.255	6.56	14.8	11.9	34.8	37.7	38.5	39.3	40.8	42.5

See *Rolled Structural Shapes* for further information on the properties of angles.



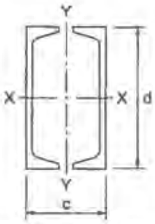
TWO ANGLES UNEQUAL LEGS
Short Legs Back-to-Back

PROPERTIES OF SECTIONS

Designation	Mass of 2 Angles	Dead Load	Area of 2 Angles	Axis X-X				Radii of Gyration about Axis Y-Y					
				I	S	r	y	Back-to-back spacing, s, millimetres					
	kg/m	kN/m	mm²	10^6 mm⁴	10^3 mm³	mm	mm	0	8	10	12	16	20
L64x51													
x9.5	15.8	0.154	2 000	0.428	11.9	14.6	14.8	28.7	31.8	32.6	33.4	35.0	36.7
x7.9	13.4	0.130	1 690	0.372	10.2	14.8	14.2	28.5	31.5	32.3	33.1	34.7	36.4
x6.4	10.8	0.106	1 370	0.310	8.34	15.0	13.6	28.2	31.2	32.0	32.7	34.3	36.0
x4.8	8.4	0.080	1 040	0.242	6.42	15.2	13.1	28.0	30.9	31.6	32.4	34.0	35.6
L64x38													
x6.4	9.6	0.093	1 210	0.134	4.69	10.5	9.53	30.0	33.1	33.9	34.7	36.3	38.0
x4.8	7.2	0.071	922	0.106	3.64	10.7	8.94	29.7	32.8	33.5	34.3	36.0	37.6
L51x38													
x6.4	8.4	0.081	1 050	0.126	4.57	11.0	10.5	23.1	26.2	27.0	27.8	29.5	31.2
x4.8	6.2	0.062	802	0.100	3.54	11.2	9.93	22.9	25.9	26.6	27.5	29.1	30.8
x3.2	4.2	0.042	544	0.071	2.46	11.4	9.35	22.6	25.5	26.3	27.1	28.7	30.4

See *Rolled Structural Shapes* for further information on the properties of angles.

TWO CHANNELS
Toe-to-Toe

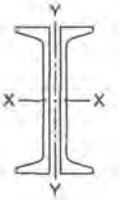


PROPERTIES OF SECTIONS

Channel Size	For Two Channels			Axis X-X			Axis Y-Y					
							Toe-to-Toe			c = d		
	Mass	Dead Load	Area	I_x	S_x	r_x	I_y	S_y	r_y	I_y	S_y	r_y
	kg/m	kN/m	mm²	10^6 mm⁴	10^3 mm³	mm	10^6 mm⁴	10^3 mm³	mm	10^6 mm⁴	10^3 mm³	mm
MC460												
x86*	172	1.70	22 000	564	2 460	160	174	1 630	88.9	956	4 180	208
x77.2*	154	1.52	19 700	522	2 280	163	147	1 410	86.3	855	3 740	208
x68.2*	136	1.34	17 400	482	2 100	166	124	1 210	84.4	754	3 300	208
x63.5*	127	1.25	16 300	462	2 020	169	110	1 100	82.4	701	3 070	208
C380												
x74*	148	1.46	19 000	336	1 760	133	112	1 190	76.9	558	2 930	172
x60*	120	1.17	15 200	290	1 520	138	80.2	901	72.8	449	2 360	172
x50*	100	0.990	12 900	262	1 370	143	62.8	730	69.9	381	2 000	172
C310												
x45	90	0.876	11 400	135	884	109	49.4	618	65.9	213	1 400	137
x37	74	0.727	9 480	120	786	113	37.6	488	63.1	177	1 160	137
x31	62	0.603	7 860	107	702	117	28.1	380	59.9	146	956	136
C250												
x45	90	0.873	11 400	85.6	674	86.9	43.6	574	62.0	142	1 120	112
x37	74	0.731	9 480	75.8	598	89.4	34.0	465	59.8	120	948	113
x30	60	0.582	7 580	65.4	514	93.0	24.0	348	56.4	96.4	759	113
x23	46	0.443	5 800	55.6	438	98.2	15.7	242	52.3	72.8	574	113
C230												
x30*	60	0.585	7 580	51.0	444	81.9	22.7	339	54.7	77.5	677	101
x22	44	0.437	5 700	42.6	372	86.6	14.7	233	50.9	57.8	505	101
x20	40	0.389	5 080	39.6	346	88.6	12.0	197	48.8	51.3	448	101
C200												
x28	56	0.548	7 100	36.4	360	71.6	19.1	299	51.9	55.6	548	88.4
x21	42	0.400	5 220	29.8	294	75.8	11.8	199	47.6	41.0	404	88.9
x17	34	0.334	4 360	27.0	266	78.7	8.93	157	45.3	33.9	334	88.4
C180												
x22*	44	0.429	5 580	22.6	254	63.7	12.2	210	46.7	32.9	369	76.8
x18	36	0.356	4 640	20.0	226	65.9	9.04	164	44.2	27.5	310	77.1
x15	30	0.284	3 700	17.7	199	69.3	6.48	122	41.9	21.7	244	76.6
C150												
x19	38	0.377	4 940	14.2	187	53.9	9.12	169	43.2	20.3	268	64.5
x16	32	0.305	3 980	12.4	164	56.1	6.53	128	40.6	16.6	218	64.8
x12	24	0.236	3 100	10.7	141	59.1	4.34	90.4	37.6	12.8	168	64.5
C130												
x13	26	0.261	3 400	7.32	115	46.5	4.65	99.0	37.1	9.49	149	52.9
x10	20	0.194	2 540	6.18	97.2	49.5	2.92	66.3	34.1	6.98	110	52.7
C100												
x11	22	0.211	2 740	3.82	74.8	37.3	3.07	71.4	33.5	4.63	90.7	41.1
x9	18	0.177	2 380	3.36	66.0	38.3	2.49	59.2	32.9	3.95	77.4	41.5
x8	16	0.157	2 060	3.22	63.2	39.7	1.91	47.8	30.6	3.44	67.4	41.0
C75												
x9	18	0.173	2 260	1.69	44.6	27.4	2.07	51.8	30.4	†	†	†
x7	14	0.144	1 900	1.50	39.4	28.3	1.46	39.6	28.0	1.56	41.1	28.9
x6	12	0.118	1 560	1.34	35.2	29.6	1.03	29.5	26.0	1.27	33.3	28.8

* Not available from Canadian mills

† The condition c = d cannot be met for this section.



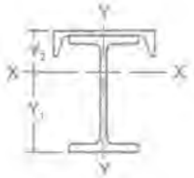
TWO CHANNELS
Back-to-Back

PROPERTIES OF SECTIONS

Channel Size	For Two Channels			Axis X-X			Radii of Gyration about Axis Y-Y					
	Mass	Dead Load	Area	I_x	S_x	r_x	Back-to-Back Channels, millimetres					
	kg/m	kN/m	mm^2	10^6 mm^4	10^3 mm^3	mm	0	8	10	12	16	20
MC460												
x86*	172	1.70	22 000	564	2 460	160	33.9	36.6	37.3	38.0	39.5	41.1
x77.2*	154	1.52	19 700	522	2 280	163	34.2	36.8	37.6	38.3	39.8	41.3
x68.2*	136	1.34	17 400	482	2 100	166	34.5	37.2	37.9	38.6	40.1	41.6
x63.5*	127	1.25	16 300	462	2 020	169	35.0	37.7	38.4	39.1	40.6	42.1
C380												
x74*	148	1.46	19 000	336	1 760	133	30.0	32.8	33.5	34.3	35.9	37.5
x60*	120	1.17	15 200	290	1 520	138	30.0	32.8	33.5	34.2	35.8	37.4
x50*	100	0.990	12 900	262	1 370	143	30.5	33.2	33.9	34.7	36.2	37.8
C310												
x45	90	0.876	11 400	135	884	109	25.7	28.5	29.3	30.0	31.6	33.2
x37	74	0.727	9 480	120	786	113	26.2	28.9	29.7	30.4	32.0	33.6
x31	62	0.603	7 860	107	702	117	26.8	29.5	30.3	31.0	32.6	34.2
C250												
x45	90	0.873	11 400	85.6	674	86.9	23.4	26.3	27.1	27.9	29.5	31.2
x37	74	0.731	9 480	75.8	598	89.4	23.3	26.1	26.9	27.7	29.3	30.9
x30	60	0.582	7 580	65.4	514	93.0	23.3	26.1	26.9	27.7	29.2	30.9
x23	46	0.443	5 800	55.6	438	98.2	23.9	26.8	27.5	28.3	29.9	31.5
C230												
x30*	60	0.585	7 580	51.0	444	81.9	22.0	24.9	25.7	26.4	28.0	29.7
x22	44	0.437	5 700	42.6	372	86.6	22.5	25.4	26.1	26.9	28.5	30.1
x20	40	0.389	5 080	39.6	346	88.6	22.7	25.5	26.3	27.1	28.7	30.3
C200												
x28	56	0.548	7 100	36.4	360	71.6	21.0	23.9	24.7	25.5	27.1	28.8
x21	42	0.400	5 220	29.8	294	75.8	20.9	23.8	24.5	25.3	26.9	28.6
x17	34	0.334	4 360	27.0	266	78.7	21.5	24.3	25.1	25.9	27.5	29.2
C180												
x22*	44	0.429	5 580	22.6	254	63.7	19.7	22.6	23.4	24.2	25.8	27.5
x18	36	0.356	4 640	20.0	226	65.9	19.5	22.4	23.2	24.0	25.6	27.3
x15	30	0.284	3 700	17.7	199	69.3	20.2	23.1	23.9	24.7	26.3	28.0
C150												
x19	38	0.377	4 940	14.2	187	53.9	18.4	21.4	22.2	23.0	24.7	26.4
x16	32	0.305	3 980	12.4	164	56.1	18.3	21.3	22.1	22.9	24.5	26.2
x12	24	0.236	3 100	10.7	141	59.1	18.6	21.6	22.4	23.2	24.9	26.6
C130												
x13	26	0.261	3 400	7.32	115	46.5	17.1	20.1	20.9	21.7	23.4	25.2
x10	20	0.194	2 540	6.18	97.2	49.5	17.5	20.5	21.3	22.1	23.8	25.5
C100												
x11	22	0.211	2 740	3.82	74.8	37.3	16.1	19.2	20.0	20.8	22.5	24.3
x9	18	0.177	2 380	3.36	66.0	38.3	15.8	18.8	19.7	20.5	22.2	23.9
x8	16	0.157	2 060	3.22	63.2	39.7	16.2	19.3	20.1	20.9	22.7	24.4
C75												
x9	18	0.173	2 260	1.69	44.6	27.4	15.5	18.7	19.5	20.4	22.1	23.9
x7	14	0.144	1 900	1.50	39.4	28.3	14.9	18.0	18.9	19.7	21.4	23.2
x6	12	0.118	1 560	1.34	35.2	29.6	14.9	18.1	18.9	19.8	21.5	23.3

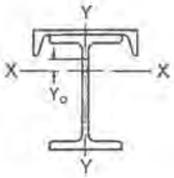
* Not available from Canadian mills

W SHAPES AND CHANNELS



PROPERTIES OF SECTIONS

Beam	Channel	Dead Load	Total Area	Axis X-X					
				I	$S_1 = I / Y_1$	$S_2 = I / Y_2$	r	Y_1	Y_2
		kN/m	mm^2	10^6 mm^4	10^3 mm^3	10^3 mm^3	mm	mm	mm
W920x289	MC460x63.5	3.46	44 900	6 410	11 800	16 300	378	545	393
	C380x50	3.33	43 200	6 180	11 600	15 200	378	531	406
x271	MC460x63.5	3.29	42 700	6 050	11 100	15 600	376	547	387
	C380x50	3.16	41 000	5 830	11 000	14 500	377	532	401
x253	MC460x63.5	3.11	40 400	5 680	10 300	14 900	375	549	381
	C380x50	2.98	38 700	5 460	10 200	13 800	376	534	395
x238	MC460x63.5	2.96	38 500	5 340	9 680	14 200	372	551	375
	C380x50	2.83	36 800	5 120	9 560	13 100	373	536	390
x223	MC460x63.5	2.83	36 700	5 020	9 070	13 600	370	554	369
	C380x50	2.69	35 000	4 810	8 950	12 500	371	537	384
W840x226	MC460x63.5	2.85	37 000	4 490	8 700	13 000	348	516	346
	C380x50	2.72	35 300	4 310	8 600	12 000	349	501	360
x210	MC460x63.5	2.69	34 900	4 170	8 040	12 300	346	519	339
	C380x50	2.56	33 200	4 000	7 950	11 300	347	503	353
x193	MC460x63.5	2.53	32 800	3 800	7 290	11 500	340	521	330
	C380x50	2.39	31 100	3 640	7 210	10 500	342	505	345
W760x196	MC460x63.5	2.56	33 200	3 260	6 840	10 700	313	476	305
	C380x50	2.42	31 500	3 120	6 760	9 790	315	462	319
x185	MC460x63.5	2.43	31 600	3 060	6 400	10 200	311	478	299
	C380x50	2.30	29 900	2 930	6 330	9 360	313	463	313
x173	MC460x63.5	2.32	30 200	2 870	5 980	9 790	308	480	293
	C380x50	2.19	28 500	2 750	5 920	8 940	311	465	308
x161	MC460x63.5	2.19	28 500	2 650	5 480	9 270	305	484	286
	C380x50	2.06	26 800	2 530	5 410	8 410	307	467	301
W690x170	C380x50	2.16	28 000	2 260	5 330	8 090	284	424	279
	C310x31	1.96	25 500	2 070	5 200	6 850	285	398	302
x152	C380x50	1.99	25 800	2 050	4 800	7 560	282	427	271
	C310x31	1.79	23 300	1 870	4 670	6 340	283	400	295
x140	C380x50	1.86	24 200	1 880	4 370	7 120	279	430	264
	C310x31	1.67	21 700	1 710	4 260	5 910	281	402	289
W610x125	C380x50	1.72	22 300	1 390	3 550	6 020	250	391	231
	C310x31	1.52	19 800	1 260	3 460	4 950	252	364	255
x113	C380x50	1.60	20 800	1 260	3 190	5 640	246	395	223
	C310x31	1.41	18 300	1 140	3 110	4 590	250	367	248
W530x101	C380x50	1.49	19 300	904	2 550	4 690	216	355	193
	C310x31	1.29	16 800	817	2 490	3 790	221	329	216
x92	C380x50	1.40	18 200	826	2 310	4 440	213	357	186
	C310x31	1.21	15 700	745	2 260	3 550	218	330	210
W460x74	C380x50	1.22	15 900	516	1 630	3 440	180	317	150
	C310x31	1.03	13 400	465	1 590	2 710	186	292	172
W410x54	C380x50	1.02	13 200	308	1 050	2 600	153	295	119
	C310x31	0.824	10 700	277	1 020	1 990	161	271	139
W360x45	C310x31	0.743	9 650	186	765	1 600	139	243	116
	C250x23	0.663	8 610	175	756	1 380	143	232	127
W310x39	C310x31	0.682	8 860	131	598	1 330	122	219	98.2
	C250x23	0.602	7 820	123	590	1 140	125	208	108
W250x33	C250x23	0.543	7 050	73.1	411	846	102	178	86.4
	C200x17	0.488	6 340	69.5	409	743	105	170	93.5
W200x27	C200x17	0.428	5 560	37.6	268	521	82.2	140	72.2



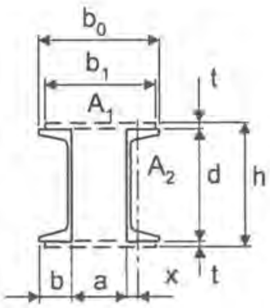
W SHAPES AND CHANNELS

PROPERTIES OF SECTIONS

Mass	Axis Y-Y			Shear Centre	Torsional Constant	Warping Constant	Monosymmetry Constant †
	I	S	r	Y_0	J	C_w	β_X
kg/m	10^6 mm^4	10^3 mm^3	mm	mm	10^3 mm^4	10^9 mm^6	mm
352.5	387	1 690	92.8	212	9 740	53 500	523
339.1	287	1 510	81.5	156	9 650	48 200	395
335.2	376	1 650	93.8	216	8 210	50 100	538
321.9	276	1 450	82.0	161	8 120	45 100	411
317.1	365	1 600	95.1	220	6 780	46 500	554
303.8	265	1 390	82.7	167	6 690	42 000	428
302.2	354	1 550	95.9	225	5 660	43 200	570
288.9	254	1 330	83.1	173	5 570	39 100	445
288.1	343	1 500	96.7	230	4 730	39 800	586
274.8	243	1 280	83.3	180	4 640	36 100	464
290.5	345	1 510	96.6	214	5 650	35 000	545
277.1	245	1 290	83.3	167	5 560	31 700	430
274.0	334	1 460	97.8	219	4 560	31 800	562
260.6	234	1 230	84.0	174	4 470	28 900	450
257.5	321	1 410	99.0	224	3 560	28 200	581
244.1	221	1 160	84.4	182	3 470	25 800	474
260.6	313	1 370	97.0	215	4 550	21 600	548
247.3	213	1 120	82.2	178	4 460	19 800	452
248.1	306	1 340	98.4	217	3 840	19 900	558
234.7	206	1 080	83.0	182	3 750	18 300	465
237.1	300	1 310	99.6	219	3 200	18 300	568
223.7	200	1 050	83.7	186	3 110	16 900	479
223.7	292	1 280	101	222	2 580	16 300	582
210.4	192	1 010	84.6	192	2 490	15 200	498
219.8	197	1 040	83.9	172	3 470	13 500	442
200.2	120	785	68.5	114	3 200	11 400	292
202.5	189	991	85.5	176	2 620	12 000	460
182.9	111	730	69.1	122	2 350	10 200	314
190.0	183	959	86.9	179	2 090	10 800	474
170.3	105	690	69.6	128	1 820	9 250	331
175.1	170	894	87.4	173	1 960	6 820	457
155.4	92.8	609	68.5	133	1 690	5 910	338
163.3	165	868	89.1	175	1 540	6 000	469
143.7	87.8	576	69.3	139	1 270	5 240	357
151.5	158	829	90.5	162	1 440	3 790	434
131.9	80.4	527	69.2	136	1 170	3 350	347
142.9	155	813	92.2	162	1 180	3 360	439
123.2	77.3	507	70.2	139	914	2 990	360
124.8	148	775	96.3	143	938	1 800	384
105.2	70.1	460	72.3	131	669	1 630	342
103.6	141	741	103	125	647	897	313
84.0	63.6	417	77.1	124	378	831	331
75.8	61.7	404	79.9	111	312	533	287
67.6	36.0	283	64.6	100	246	493	268
69.6	60.8	398	82.8	98.2	278	377	231
61.4	35.1	276	67.0	89.9	212	349	237
55.3	32.5	256	67.9	83.8	185	168	192
49.8	18.2	180	53.6	71.7	152	153	188
43.6	16.8	166	55.0	65.2	125	74.0	147

† β_X is positive when the larger flange is in flexural compression, and negative otherwise.

BUILT-UP SECTIONS



$h = d + 2t$

$b_0 = a + 2b$

$A_1 = b_1 t$

$A = 2(A_1 + A_2)$

$$I_{xx} = 2I_{xc} + \frac{b_1}{12}\left(h^3 - d^3\right)$$

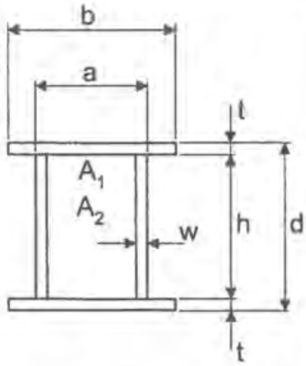
$$S_{xx} = 2I_{xx}/h \qquad r_{xx} = \sqrt{I_{xx}/A}$$

$$I_{yy} = 2I_{yc} + \frac{A_1}{6}b_1^2 + 2A_2(x + a/2)^2$$

$S_{yy} = 2I_{yy}/b_0$ if $b_1 < b_0$

$S_{yy} = 2I_{yy}/b_1$ if $b_1 \ge b_0$

$$r_{yy} = \sqrt{I_{yy}/A}$$



$h = d - 2t$

$A_1 = bt$

$A_2 = wh$

$A = 2(A_1 + A_2)$

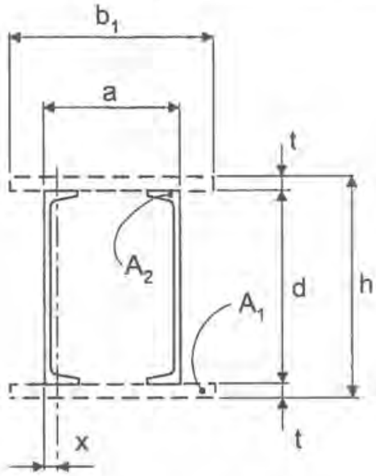
$c = a - 2w$

$$I_{xx} = \frac{1}{12}\left\{b(d^3 - h^3) + 2A_2h^2\right\} \qquad r_{xx} = \sqrt{I_{xx}/A}$$

$$S_{xx} = 2I_{xx}/d \qquad Z_{xx} = \frac{b}{4}\left(d^2 - h^2\right) + \frac{A_2h}{2}$$

$$I_{yy} = \frac{1}{12}\left\{2A_1b^2 + h(a^3 - c^3)\right\} \qquad r_{yy} = \sqrt{I_{yy}/A}$$

$$S_{yy} = 2I_{yy}/b \qquad Z_{yy} = \frac{h}{4}\left(a^2 - c^2\right) + \frac{A_1b}{2}$$



$h = d + 2t \qquad A_1 = b_1 t \qquad A = 2(A_1 + A_2)$

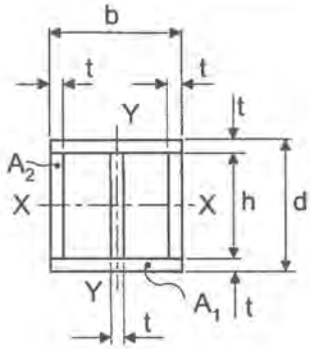
$$I_{xx} = 2I_{xc} + \frac{b_1}{12}(h^3 - d^3) \qquad S_{xx} = 2I_{xx}/h$$

$$I_{yy} = 2I_{yc} + \frac{A_1}{6}b_1^2 + 2A_2(a/2 - x)^2$$

$S_{yy} = 2I_{yy}/b_1$ if $a < b_1$

$S_{yy} = 2I_{yy}/a$ if $a \ge b_1$

$$r_{xx} = \sqrt{I_{xx}/A} \qquad r_{yy} = \sqrt{I_{yy}/A}$$



$h = d - 2t$

$A_1 = bt$

$A_2 = ht$

$A = 2A_1 + 3A_2$

$$I_{xx} = \frac{1}{12}\left\{3A_2h^2 + b(d^3 - h^3)\right\}$$

$$S_{xx} = 2I_{xx}/d \qquad Z_{xx} = \frac{3A_2h}{4} + A_1(d - t)$$

$$I_{yy} = \frac{1}{12}\left\{2A_1b^2 + A_2t^2 + h\left[b^3 - (b - 2t)^3\right]\right\}$$

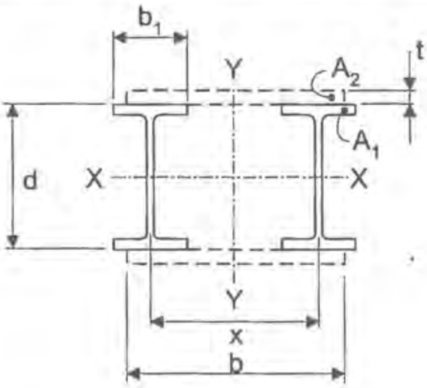
$$S_{yy} = 2I_{yy}/b \qquad Z_{yy} = \frac{A_1b}{2} + \frac{A_2t}{4} + A_2(b - t)$$

$$r_{xx} = \sqrt{I_{xx}/A} \qquad r_{yy} = \sqrt{I_{yy}/A}$$

Elements of the shape which are shown in dotted outline are optional and, if omitted, the variable defining their size should be set equal to zero.

All elements of the shape are assumed to be continuous along the length of the shape.

BUILT-UP SECTIONS



$A = 2\,(A_1 + A_2) \qquad A_2 = b\,t$

$I_{xx} = 2\,I_{xw} + \dfrac{1}{12} b\left[(d + 2t)^3 - d^3\right]$

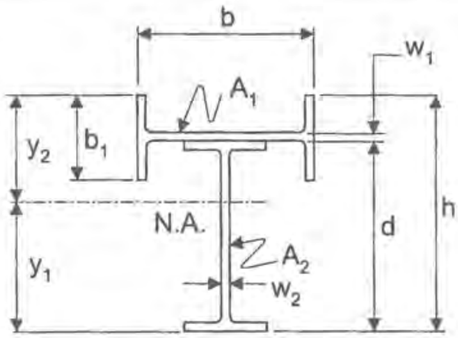
$S_{xx} = 2\,I_{xx} / (d + 2\,t)$

$I_{yy} = 2\,I_{yw} + \dfrac{1}{6} A_2 b^2 + \dfrac{1}{2} A_1 x^2$

For $(x + b_1) > b$: $\quad S_{yy} = 2\,I_{yy} / (x + b_1)$

For $(x + b_1) \le b$: $\quad S_{yy} = 2\,I_{yy} / b$

$r_{xx} = \sqrt{I_{xx} / A} \qquad r_{yy} = \sqrt{I_{yy} / A}$



$h = d + \dfrac{1}{2}(b_1 + w_1) \qquad A = A_1 + A_2$

$y_1 = \dfrac{A_1(d + w_1/2) + A_2 d/2}{A_1 + A_2} \qquad y_2 = h - y_1$

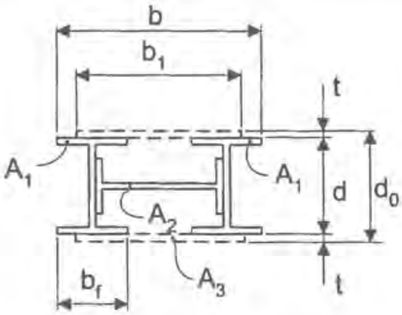
$I_{xx} = I_{y1} + I_{x2} + A_1\,(y_2 - b_1/2)^2 + A_2\,(y_1 - d/2)^2$

$S_{x1} = I_{xx} / y_1 \qquad S_{x2} = I_{xx} / y_2$

$I_{yy} = I_{x1} + I_{y2} \qquad S_{yy} = 2\,I_{yy} / b$

$^*I_{yT} = I_{x1} + I_{y2}/2 - (y_1 - d/2) w_2^3 / 12$

$r_{xx} = \sqrt{I_{xx} / A} \qquad r_{yy} = \sqrt{I_{yy} / A}$



$d_0 = d + 2\,t$

$A = 2(A_1 + A_3) + A_2 \qquad A_3 = b_1\,t$

$I_{xx} = 2\,I_{x1} + I_{y2} + \dfrac{b_1}{12}(d_0^3 - d^3)$

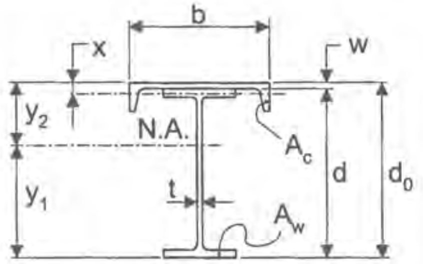
$S_{xx} = 2\,I_{xx} / d_0$

$I_{yy} = I_{x2} + 2\,I_{y1} + \dfrac{A_3}{6} b_1^2 + A_1(b - b_f)^2/2$

$S_{yy} = 2\,I_{yy} / b_1 \quad$ if $b < b_1$

$S_{yy} = 2\,I_{yy} / b \quad$ if $b \ge b_1$

$r_{xx} = \sqrt{I_{xx} / A} \qquad r_{yy} = \sqrt{I_{yy} / A}$



$A = A_c + A_w \qquad d_0 = d + w$

$y_1 = \dfrac{A_w d/2 + A_c(d_0 - x)}{A} \qquad y_2 = d_0 - y_1$

$I_{xx} = I_{xw} + I_{yc} + A_w\,(y_1 - d/2)^2 + A_c\,(y_2 - x)^2$

$I_{yy} = I_{yw} + I_{xc}$

$^*I_{yT} = I_{xc} + \dfrac{I_{yw}}{2} - (y_1 - d/2)\dfrac{t^3}{12}$

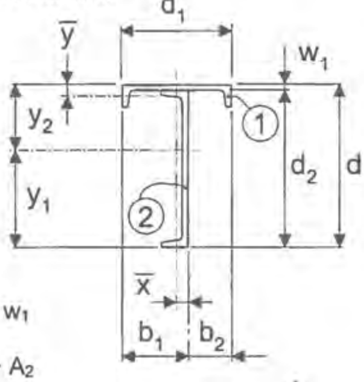
$S_{x1} = I_{xx} / y_1 \qquad S_{x2} = I_{xx} / y_2 \qquad S_{yy} = 2\,I_{yy} / b$

$r_{xx} = \sqrt{I_{xx} / A} \qquad r_{yy} = \sqrt{I_{yy} / A}$

$^*I_{yT}$ is the moment of inertia of the T-section above the neutral axis.

BUILT-UP SECTIONS

Note: Centres of gravity of both channels are on the same vertical line.



$d = d_2 + w_1$

$A = A_1 + A_2$

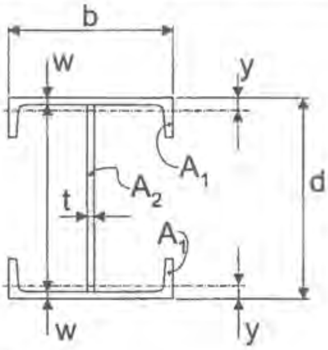
$b_1 = (d_1/2) + \bar{x}$ $\quad y_1 = \dfrac{A_1(d-\bar{y}) + \dfrac{A_2}{2}d_2}{A}$

$b_2 = d_1 - b_1$ $\quad y_2 = d - y_1$

$I_{xx} = I_{1y} + I_{2x} + A_1(y_2 - \bar{y})^2 + A_2(y_1 - \dfrac{d_2}{2})^2$

$S_{x1} = I_{xx}/y_1 \quad S_{x2} = I_{xx}/y_2 \quad r_{xx} = \sqrt{I_{xx}/A}$

$I_{yy} = I_{x1} + I_{y2} \quad S_y = 2\,I_{yy}/d_1 \quad r_{yy} = \sqrt{I_{yy}/A}$



$h = d - 2\,w$

$A = 2\,A_1 + A_2 \quad A_2 = h\,t$

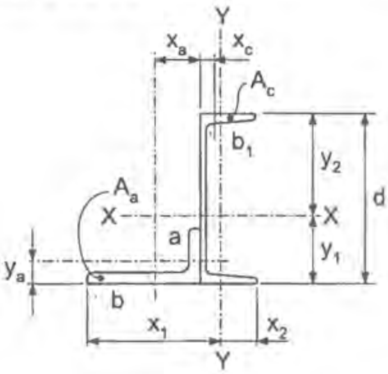
$I_{xx} = 2I_{yc} + \dfrac{1}{12}A_2h^2 + 2A_1(d/2 - y)^2$

$S_{xx} = 2\,I_{xx}/d$

$I_{yy} = 2\,I_{xc} + \dfrac{1}{12}A_2t^2 \quad S_{yy} = 2\,I_{yy}/b$

$r_{xx} = \sqrt{I_{xx}/A} \quad r_{yy} = \sqrt{I_{yy}/A}$

Note: a and b are the angle leg lengths, and b_1 is the width of the channel flange.



$A = A_a + A_c \quad y_1 = \dfrac{A_a y_a + A_c d/2}{A} \quad y_2 = d - y_1$

$x_1 = \dfrac{A_a(b - x_a) + A_c(b + x_c)}{A} \quad x_2 = b_1 + b - x_1$

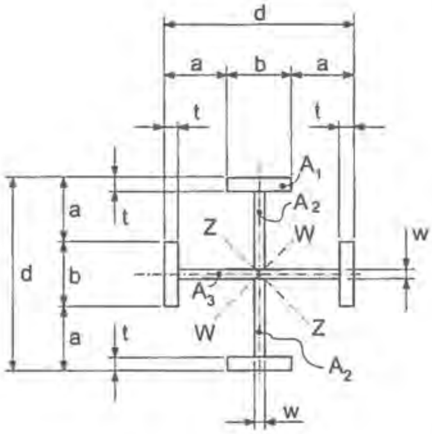
$I_{xx} = I_{ya} + I_{xc} + A_a(y_1 - y_a)^2 + A_c(\dfrac{d}{2} - y_1)^2$

$S_{x1} = I_{xx}/y_1 \quad S_{x2} = I_{xx}/y_2$

$I_{yy} = I_{xa} + I_{yc} + A_a(x_1 - b + x_a)^2 + A_c(b_1 - x_2 - x_c)^2$

$S_{y1} = I_{yy}/x_1 \quad S_{y2} = I_{yy}/x_2$

$r_{xx} = \sqrt{I_{xx}/A} \quad r_{yy} = \sqrt{I_{yy}/A}$



$A = 4A_1 + 2A_2 + A_3 \quad A_1 = b\,t$

$A_2 = (d - w - 2\,t)\,w/2 \quad A_3 = 2A_2 + w^2$

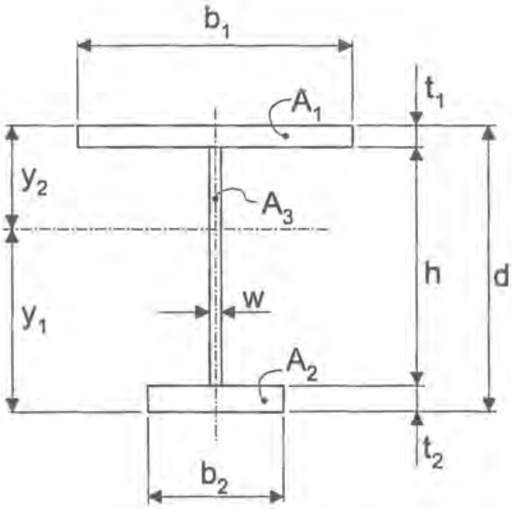
$I_x = I_y = \dfrac{1}{12}\left\{b(d^3 - E^3) + wE^3 + 2tb^3 + Ew^3 - w^4\right\}$

$E = d - 2\,t$

$S_x = S_y = 2\,I_x/d$

$r_x = r_y = \sqrt{I_x/A}$

BUILT-UP SECTIONS



$A_1 = b_1 t_1 \quad A_2 = b_2 t_2 \quad A_3 = w\,h$

$d = h + t_1 + t_2$

$A = A_1 + A_2 + A_3$

$$y_1 = \frac{A_1(d - t_1/2) + A_3(t_2 + h/2) + A_2 t_2/2}{A}$$

$y_2 = d - y_1$

$$J = \frac{1}{3}\left\{A_1 t_1^2 + A_3 w^2 + A_2 t_2^2\right\}$$

$$C_w = \frac{\left(d - \dfrac{t_1 + t_2}{2}\right)^2 b_1^3 t_1}{12\left[1 + (b_1/b_2)^3 (t_1/t_2)\right]}$$

$$I_{xx} = \frac{1}{12}\left[A_1 t_1^2 + A_2 t_2^2 + A_3 h^2\right] + A_1(y_2 - t_1/2)^2 + A_2(y_1 - t_2/2)^2 + A_3(y_1 - t_2 - h/2)^2$$

$S_{x1} = I_{xx}/y_1 \quad S_{x2} = I_{xx}/y_2$

$I_{yy} = \dfrac{1}{12}\left[A_1 b_1^2 + A_2 b_2^2 + A_3 w^2\right] \quad S_{yy} = 2\,I_{yy}/b_1$

$^{*}\,I_{yT} = \dfrac{1}{12}\left[A_1 b_1^2 + (y_2 - t_1) w^3\right] \quad r_{xx} = \sqrt{I_{xx}/A} \quad r_{yy} = \sqrt{I_{yy}/A}$

$A = A_1 + A_2 + A_S \quad h = d + t_1 + t_2$

$$y_1 = \frac{A_1(h - t_1/2) + A_S(t_2 + d/2) + A_2 t_2/2}{A}$$

$y_2 = h - y_1$

$$I_{xx} = I_{xS} + \frac{1}{12}(A_1 t_1^2 + A_2 t_2^2) + A_S(y_1 - t_2 - d/2)^2 + A_1(y_2 - t_1/2)^2 + A_2(y_1 - t_2/2)^2$$

$S_{x1} = I_{xx}/y_1 \quad S_{x2} = I_{xx}/y_2$

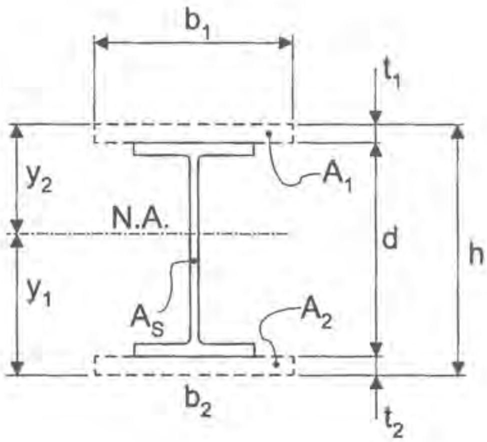
$r_{xx} = \sqrt{I_{xx}/A}$

$$I_{yy} = I_{yS} + \frac{1}{12}\left[A_1 b_1^2 + A_2 b_2^2\right]$$

$S_{yy} = 2\,I_{yy}/b_1 \quad \text{if } b_1 > b_2$

$S_{yy} = 2\,I_{yy}/b_2 \quad \text{if } b_1 \le b_2$

$r_{yy} = \sqrt{I_{yy}/A}$



*I_{yT} is the moment of inertia of the T-section above the neutral axis.

COLD-FORMED STEEL C- and Z-SECTIONS

General

While various proprietary cold-formed C- and Z-sections are available from Canadian roll formers, the sections listed on the following pages are representative of those included in CSA Standard G40.20/G40.21-13, and other products generally available. Coated sections refer to products that are typically supplied with a metallic coating such as zinc or aluminum-zinc alloy. Uncoated products do not have this coating. The metallic coating, if present, does not affect the calculated properties of the section. Both gross and effective section properties are presented in these tables. For coated sections the calculated values were based on an inside bend radius, R, taken as the greater of $R_1 = (2.381 - t/2)$ and $R_2 = 1.5t$, and for uncoated sections the inside bend radius was taken as $2t$. The effective section properties, factored shear and moment resistances were computed in accordance with the applicable sections of CSA Standard S136-12, *North American Specification for the Design of Cold-Formed Steel Structural Members*. For coated sections with a design base steel thickness less than or equal to 1.146 mm, F_y = 230 MPa and F_u = 310 MPa. For coated sections with a design base steel thickness greater than 1.146 mm, F_y = 345 MPa and F_u = 450 MPa. For all uncoated sections, F_y = 345 MPa and F_u = 450 MPa. Cold work of forming was not included. Distortional buckling calculations were based on $K_\varphi = 0$.

Material

For coated sections, steel meets the requirements of ASTM A653/A653M Grade 340 (Grade 50), F_y = 345 MPa, and for uncoated sections, steel meets the requirements of ASTM A1011/A1011M Grade 340 (Grade 50), F_y = 345 MPa.

Tables

Only some of the noteworthy terms are defined below. All others are self-explanatory.

- I_{xd} = effective deflection moment of inertia about X-X axis (10^6 mm^4) at $0.6F_y$
- S_{xe} = effective section modulus about X-X axis (10^3 mm^3)
- I_{ye} = effective moment of inertia about Y-Y axis assuming lips in tension (10^6 mm^4)
- S_{ye} = effective section modulus about Y-Y axis (10^3 mm^3)
- M_{rlb} = factored moment resistance based on local buckling about X-X axis (kN·m)
- L_{cr} = critical unbraced length of distortional buckling (mm)
- M_{rdb} = factored moment resist. based on distortional buckling about X-X axis (kN·m)
- V_r = factored shear resistance (kN)
- L_u = maximum unbraced length of compression flange beyond which appropriate values in the Table must be reduced for lateral-torsional buckling (mm)
- t = design base steel thickness (mm)
- x_o = distance from shear centre to centroid of gross area (mm)
- r_o = polar radius of gyration (mm)
- J = Saint-Venant torsion constant (10^3 mm^4)
- j = flexural-torsional buckling parameter (mm)
- C_w = torsional warping constant (10^9 mm^6)

The minimum base steel thickness is 95% of the design base steel thickness. The design base steel thickness was used to calculate values in the tables.

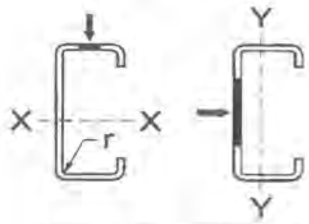
Minimum base steel thickness (mm)	Design base steel thickness (mm)
5.41	5.69
4.68	4.93
3.96	4.17
3.62	3.81
3.26	3.43
2.90	3.05
2.54	2.67
2.18	2.29
1.81	1.91
1.44	1.52

Minimum base steel thickness (mm)	Design base steel thickness (mm)
2.997	3.155
2.454	2.583
1.720	1.811
1.367	1.438
1.087	1.146

These tables have been prepared by Dr. R.M. Schuster, Professor Emeritus of Structural Engineering and Director of the Canadian Cold-Formed Steel Research Group at the University of Waterloo.

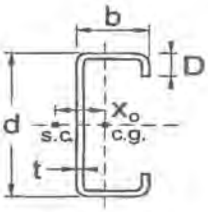
COLD-FORMED C-SECTIONS, COATED

Effective Properties



Designation	Mass	Gross Area	Effective Section Properties				$M_{r\ell b}$	L_{cr}	M_{rdb}	V_r	L_u
			X-X Axis		Y-Y Axis						
			I_{xd}	S_{xe}	I_{ye}	S_{ye}					
	kg/m	mm²	10^6 mm⁴	10^3 mm³	10^6 mm⁴	10^3 mm³	kN·m	mm	kN·m	kN	mm
1400S300-118	12.8	1 629	27.2	138	0.702	13.5	42.9	474	35.4	72.4	1 407
1400S300-97	10.6	1 345	21.7	104	0.562	11.3	32.4	527	26.5	39.4	1 418
1400S300-68	7.48	953	14.3	59.8	0.369	7.98	18.6	637	15.8	13.4	1 433
1400S250-118	12.2	1 549	24.7	129	0.443	10.1	40.1	426	33.8	72.4	1 173
1400S250-97	10.0	1 279	20.1	98.4	0.357	8.44	30.6	474	25.3	39.4	1 184
1400S250-68	7.12	907	13.5	58.1	0.237	6.04	18.1	573	15.0	13.4	1 199
1400S200-118	11.5	1 469	22.2	116	0.252	6.99	36.1	375	31.4	72.4	933
1400S200-97	9.53	1 213	18.1	91.4	0.206	5.94	28.4	417	23.6	39.4	945
1400S200-68	6.76	861	12.3	57.4	0.138	4.31	17.8	503	14.0	13.4	961
1400S162-118	10.9	1 388	19.8	103	0.133	4.33	32.0	297	27.5	72.4	714
1400S162-97	9.01	1 148	16.1	80.5	0.111	3.74	25.0	325	20.5	39.4	727
1400S162-68	6.40	815	10.9	51.4	0.076	2.78	15.9	386	12.0	13.4	745
1200S300-118	11.5	1 469	18.8	119	0.698	13.5	36.9	454	30.0	85.1	1 428
1200S300-97	9.53	1 213	15.4	95.5	0.559	11.3	29.7	506	22.6	46.3	1 437
1200S300-68	6.76	861	10.7	54.3	0.368	7.97	16.9	612	13.6	15.7	1 450
1200S250-118	10.9	1 388	17.0	107	0.441	10.0	33.3	408	28.7	85.1	1 195
1200S250-97	9.01	1 148	14.0	82.5	0.356	8.43	25.6	454	21.7	46.3	1 205
1200S250-68	6.40	815	9.53	49.2	0.236	6.03	15.3	550	13.1	15.7	1 219
1200S200-118	10.3	1 308	15.1	96.1	0.251	6.97	29.8	357	26.8	85.1	956
1200S200-97	8.50	1 082	12.5	76.3	0.205	5.93	23.7	398	20.4	46.3	967
1200S200-68	6.04	769	8.62	48.5	0.138	4.31	15.1	482	12.3	15.7	982
1200S162-118	9.64	1 228	13.4	84.7	0.132	4.32	26.3	278	23.6	85.1	736
1200S162-97	7.98	1 017	11.1	67.0	0.110	3.74	20.8	306	17.9	46.3	748
1200S162-68	5.68	723	7.60	43.3	0.076	2.78	13.5	368	10.6	15.7	765
1000S300-97	8.50	1 082	9.95	73.7	0.555	11.2	22.9	482	18.6	56.0	1 455
1000S300-68	6.04	769	6.92	45.9	0.366	7.95	14.3	585	11.3	19.0	1 467
1000S300-54	4.82	615	5.33	31.1	0.276	6.28	9.67	661	8.13	9.4	1 472
1000S250-97	7.98	1 017	9.09	69.0	0.353	8.41	21.4	433	17.9	56.0	1 226
1000S250-68	5.68	723	6.47	45.3	0.235	6.02	14.1	525	10.9	19.0	1 239
1000S250-54	4.54	578	5.08	30.8	0.177	4.78	9.56	595	7.89	9.4	1 245
1000S200-97	7.47	951	8.05	61.3	0.204	5.92	19.0	379	16.8	56.0	990
1000S200-68	5.32	677	5.66	39.6	0.137	4.30	12.3	460	10.4	19.0	1 004
1000S200-54	4.25	542	4.43	27.9	0.104	3.44	8.66	521	7.48	9.4	1 010
1000S162-97	6.95	885	7.06	53.6	0.110	3.73	16.6	289	14.9	56.0	771
1000S162-68	4.95	631	4.96	35.3	0.076	2.77	11.0	349	9.04	19.0	786
1000S162-54	3.96	505	3.87	25.7	0.058	2.24	7.99	395	6.48	9.4	794

Designation Example: 1400S300-97; where 1400 = 14 in. section depth; S = stud or joist C-section; 300 = 3 in. flange width; 97 = minimum base steel thickness in mils;

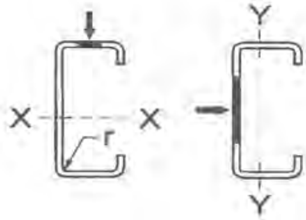


COLD-FORMED C-SECTIONS, COATED

Dimensions and Gross Properties

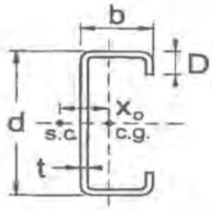
Depth	Flange Width	Stiff'r Depth	Thick-ness	Gross Section Properties										
				X-X Axis			Y-Y Axis			x_o	r_o	J	j	C_w
d	b	D	t	I_x	S_x	r_x	I_y	S_y	r_y					
mm	mm	mm	mm	10^6 mm^4	10^3 mm^3	mm	10^6 mm^4	10^3 mm^3	mm	mm	mm	10^3 mm^4	mm	10^9 mm^6
356	76	15.9	3.15	27.3	154	130	0.905	14.8	23.6	39.2	137	5.40	238	23.0
356	76	15.9	2.58	22.8	128	130	0.772	12.7	24.0	39.8	138	2.99	233	19.4
356	76	15.9	1.81	16.3	91.8	131	0.570	9.35	24.5	40.7	139	1.04	227	14.2
356	64	15.9	3.15	24.9	140	127	0.563	10.9	19.1	30.6	132	5.14	267	14.8
356	64	15.9	2.58	20.7	117	127	0.483	9.35	19.4	31.1	132	2.84	260	12.5
356	64	15.9	1.81	14.9	83.7	128	0.360	6.97	19.9	31.9	133	0.99	251	9.16
356	51	15.9	3.15	22.4	126	123	0.314	7.49	14.6	22.4	126	4.87	317	8.56
356	51	15.9	2.58	18.7	105	124	0.273	6.49	15.0	23.0	127	2.70	306	7.29
356	51	15.9	1.81	13.4	75.6	125	0.206	4.89	15.5	23.7	128	0.94	293	5.39
356	41	12.7	3.15	20.0	112	120	0.161	4.60	10.8	15.5	121	4.61	403	4.57
356	41	12.7	2.58	16.7	94.0	121	0.142	4.05	11.1	16.0	122	2.55	384	3.93
356	41	12.7	1.81	12.1	67.8	122	0.109	3.11	11.6	16.6	123	0.89	361	2.94
305	76	15.9	3.15	18.8	123	113	0.872	14.7	24.4	42.3	123	4.87	189	16.2
305	76	15.9	2.58	15.7	103	114	0.743	12.5	24.8	43.0	124	2.70	185	13.7
305	76	15.9	1.81	11.3	73.8	114	0.549	9.24	25.3	43.8	125	0.94	181	9.97
305	64	15.9	3.15	17.0	111	111	0.544	10.8	19.8	33.2	117	4.61	206	10.4
305	64	15.9	2.58	14.2	92.9	111	0.467	9.25	20.2	33.8	118	2.55	201	8.79
305	64	15.9	1.81	10.2	66.9	112	0.348	6.89	20.7	34.6	119	0.89	195	6.45
305	51	15.9	3.15	15.1	99.3	108	0.305	7.43	15.3	24.5	111	4.34	239	6.03
305	51	15.9	2.58	12.7	83.1	108	0.264	6.43	15.6	25.1	112	2.41	231	5.14
305	51	15.9	1.81	9.14	60.0	109	0.199	4.85	16.1	25.8	113	0.84	222	3.81
305	41	12.7	3.15	13.4	87.8	104	0.157	4.57	11.3	17.0	106	4.07	299	3.22
305	41	12.7	2.58	11.2	73.7	105	0.138	4.02	11.7	17.6	107	2.26	286	2.77
305	41	12.7	1.81	8.13	53.3	106	0.106	3.09	12.1	18.3	108	0.79	270	2.08
254	76	15.9	2.58	10.1	79.7	96.7	0.708	12.3	25.6	46.7	110	2.41	146	9.01
254	76	15.9	1.81	7.29	57.4	97.4	0.524	9.10	26.1	47.6	111	0.84	143	6.59
254	76	15.9	1.44	5.86	46.1	97.7	0.426	7.41	26.3	48.1	112	0.42	142	5.34
254	64	15.9	2.58	9.09	71.6	94.5	0.446	9.12	21.0	36.9	104	2.26	153	5.81
254	64	15.9	1.81	6.56	51.6	95.2	0.333	6.80	21.4	37.8	105	0.79	149	4.27
254	64	15.9	1.44	5.28	41.6	95.6	0.272	5.56	21.7	38.2	105	0.40	148	3.47
254	51	15.9	2.58	8.05	63.4	92.0	0.254	6.35	16.3	27.6	97	2.12	170	3.40
254	51	15.9	1.81	5.83	45.9	92.8	0.191	4.79	16.8	28.4	98	0.74	164	2.52
254	51	15.9	1.44	4.70	37.0	93.1	0.157	3.93	17.0	28.8	99	0.37	161	2.06
254	41	12.7	2.58	7.06	55.6	89.3	0.133	3.97	12.3	19.5	92	1.97	204	1.83
254	41	12.7	1.81	5.13	40.4	90.2	0.103	3.06	12.7	20.3	93	0.69	193	1.38
254	41	12.7	1.44	4.14	32.6	90.6	0.085	2.53	13.0	20.6	94	0.35	189	1.13

COLD-FORMED C-SECTIONS, COATED
Effective Properties



Designation	Mass	Gross Area	Effective Section Properties X-X Axis I_{xd}	Effective Section Properties X-X Axis S_{xe}	Effective Section Properties Y-Y Axis I_{ye}	Effective Section Properties Y-Y Axis S_{ye}	M_{rb}	L_{cr}	M_{rdb}	V_r	L_u
	kg/m	mm²	10^6 mm⁴	10^3 mm³	10^6 mm⁴	10^3 mm³	kN·m	mm	kN·m	kN	mm
800S300-97	7.47	951	5.88	54.1	0.549	11.2	16.8	456	14.4	61.9	1 473
800S300-68	5.32	677	4.10	35.1	0.363	7.93	10.9	552	8.90	24.0	1 481
800S300-54	4.25	542	3.19	25.1	0.274	6.27	7.80	625	6.45	11.9	1 486
800S250-97	6.95	885	5.32	50.4	0.350	8.37	15.7	408	13.9	61.9	1 248
800S250-68	4.95	631	3.80	33.7	0.233	6.00	10.5	496	8.62	24.0	1 257
800S250-54	3.96	505	2.98	25.0	0.176	4.77	7.75	562	6.28	11.9	1 263
800S200-97	6.44	820	4.66	45.9	0.202	5.89	14.3	357	13.0	61.9	1 015
800S200-68	4.59	585	3.39	32.6	0.136	4.29	10.1	434	8.20	24.0	1 026
800S200-54	3.68	468	2.74	24.5	0.103	3.43	7.62	492	5.99	11.9	1 031
800S162-97	5.92	754	4.04	39.8	0.108	3.72	12.4	270	11.5	61.9	796
800S162-68	4.23	539	2.94	27.3	0.075	2.77	8.46	329	7.24	24.0	808
800S162-54	3.39	432	2.32	20.1	0.058	2.24	6.25	373	5.26	11.9	815
600S300-97	4.59	585	2.11	23.7	0.359	7.89	7.35	514	6.50	30.4	1 495
600S300-68	3.68	468	1.64	18.1	0.272	6.24	5.63	582	4.75	16.0	1 498
600S300-54	2.95	376	1.37	15.5	0.218	5.05	3.20	656	2.71	8.0	1 840
600S250-97	4.23	539	1.97	24.9	0.243	6.10	5.16	461	4.76	24.9	1 570
600S250-68	3.39	432	1.59	18.9	0.185	4.86	3.92	523	3.54	15.7	1 572
600S250-54	2.72	347	1.27	15.0	0.141	3.86	3.11	590	2.62	8.0	1 574
600S200-97	3.87	493	1.71	22.4	0.142	4.36	4.64	404	4.38	24.9	1 291
600S200-68	3.10	395	1.38	18.1	0.109	3.50	3.75	458	3.33	15.7	1 294
600S200-54	2.49	318	1.12	14.3	0.083	2.79	2.96	517	2.49	8.0	1 298
600S162-97	3.51	447	1.47	19.3	0.078	2.81	3.99	305	3.77	24.9	1 023
600S162-68	2.82	359	1.19	15.6	0.061	2.28	3.23	346	2.94	15.7	1 028
600S162-54	2.26	288	0.96	12.7	0.046	1.83	2.62	392	2.19	8.0	1 032
362S250-97	3.37	430	0.62	13.0	0.232	5.99	2.69	407	2.64	16.6	1 629
362S250-68	2.71	345	0.50	9.9	0.179	4.79	2.04	461	2.01	13.5	1 622
362S250-54	2.18	278	0.41	7.8	0.137	3.81	1.61	520	1.51	9.9	1 618
362S200-97	3.01	384	0.53	11.4	0.135	4.28	2.37	356	2.24	16.6	1 357
362S200-68	2.42	309	0.43	9.3	0.105	3.45	1.93	404	1.82	13.5	1 352
362S200-54	1.95	248	0.35	7.3	0.081	2.76	1.52	456	1.41	9.9	1 350
362S162-97	2.65	338	0.44	9.7	0.075	2.76	2.00	268	1.89	16.6	1 076
362S162-68	2.14	272	0.36	7.9	0.059	2.25	1.63	305	1.54	13.5	1 073
362S162-54	1.72	219	0.30	6.4	0.045	1.81	1.33	345	1.23	9.9	1 073

Designation Example: 600S200-97; where 600 = 6 in. section depth; S = stud or joist C-section;
200 = 2 in. flange width; 97 = minimum base steel thickness in mils;



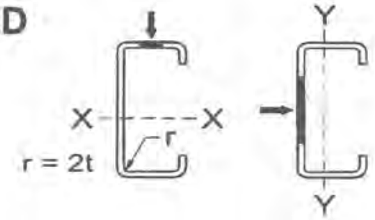
COLD-FORMED C-SECTIONS, COATED

Dimensions and Gross Properties

Depth	Flange Width	Stiff'r Depth	Thick-ness	Gross Section Properties												
				X-X Axis			Y-Y Axis									
d	b	D	t	I_x	S_x	r_x	I_y	S_y	r_y	x_o	r_o	J	j	C_w		
mm	mm	mm	mm	10^6 mm^4	10^3 mm^3	mm	10^6 mm^4	10^3 mm^3	mm	mm	mm	10^3 mm^4	mm	10^9 mm^6		
203	76	15.9	2.58	5.98	58.9	79.3	0.664	12.0	26.4	51.2	98	2.12	115	5.45		
203	76	15.9	1.81	4.32	42.5	79.9	0.491	8.90	26.9	52.2	99	0.74	114	4.00		
203	76	15.9	1.44	3.48	34.2	80.2	0.399	7.25	27.2	52.7	100	0.37	113	3.24		
203	64	15.9	2.58	5.32	52.4	77.6	0.420	8.94	21.8	40.8	90	1.97	115	3.52		
203	64	15.9	1.81	3.86	37.9	78.2	0.313	6.67	22.3	41.8	91	0.69	113	2.59		
203	64	15.9	1.44	3.11	30.6	78.5	0.256	5.45	22.5	42.2	92	0.35	112	2.11		
203	51	15.9	2.58	4.66	45.9	75.4	0.240	6.24	17.1	30.8	83	1.82	121	2.06		
203	51	15.9	1.81	3.39	33.4	76.1	0.181	4.71	17.6	31.7	84	0.64	117	1.53		
203	51	15.9	1.44	2.74	26.9	76.4	0.149	3.87	17.8	32.1	85	0.32	116	1.25		
203	41	12.7	2.58	4.04	39.8	73.2	0.127	3.91	13.0	22.0	78	1.68	139	1.10		
203	41	12.7	1.81	2.95	29.1	74.0	0.098	3.01	13.5	22.8	79	0.59	133	0.83		
203	41	12.7	1.44	2.39	23.5	74.4	0.081	2.50	13.7	23.2	79	0.30	130	0.68		
152	76	15.9	1.81	2.23	29.2	61.7	0.447	8.61	27.7	57.9	89	0.64	93	2.13		
152	76	15.9	1.44	1.80	23.6	62.0	0.364	7.01	27.9	58.4	90	0.32	93	1.73		
152	76	15.9	1.15	1.45	19.0	62.1	0.296	5.70	28.1	58.8	90	0.16	93	1.41		
152	64	15.9	1.81	1.97	25.8	60.4	0.286	6.47	23.1	46.8	80	0.59	87	1.38		
152	64	15.9	1.44	1.59	20.9	60.7	0.234	5.29	23.3	47.2	80	0.30	87	1.13		
152	64	15.9	1.15	1.28	16.9	60.9	0.191	4.32	23.5	47.6	81	0.15	86	0.92		
152	51	15.9	1.81	1.71	22.4	58.8	0.166	4.59	18.4	35.9	71	0.54	83	0.82		
152	51	15.9	1.44	1.38	18.1	59.1	0.137	3.77	18.6	36.4	72	0.27	83	0.67		
152	51	15.9	1.15	1.12	14.7	59.3	0.112	3.09	18.8	36.7	72	0.14	82	0.55		
152	41	12.7	1.81	1.47	19.3	57.3	0.091	2.94	14.2	26.2	65	0.49	87	0.44		
152	41	12.7	1.44	1.19	15.6	57.6	0.075	2.44	14.5	26.6	65	0.25	86	0.36		
152	41	12.7	1.15	0.97	12.7	57.8	0.062	2.01	14.6	27.0	65	0.13	85	0.29		
92	64	15.9	1.81	0.62	13.5	38.0	0.240	6.07	23.6	55.0	71	0.47	69	0.49		
92	64	15.9	1.44	0.50	10.9	38.2	0.197	4.97	23.9	55.5	71	0.24	70	0.40		
92	64	15.9	1.15	0.41	8.9	38.4	0.160	4.06	24.0	55.9	72	0.12	70	0.33		
92	51	15.9	1.81	0.53	11.4	37.0	0.140	4.33	19.1	43.1	60	0.42	60	0.29		
92	51	15.9	1.44	0.43	9.3	37.3	0.115	3.57	19.3	43.6	60	0.21	60	0.24		
92	51	15.9	1.15	0.35	7.6	37.4	0.094	2.92	19.5	43.9	61	0.11	60	0.20		
92	41	12.7	1.81	0.44	9.7	36.3	0.077	2.79	15.1	32.1	51	0.37	54	0.15		
92	41	12.7	1.44	0.36	7.9	36.5	0.064	2.32	15.4	32.6	51	0.19	54	0.12		
92	41	12.7	1.15	0.30	6.4	36.7	0.053	1.91	15.5	32.9	52	0.10	53	0.10		

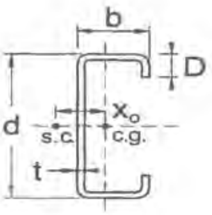
COLD-FORMED C-SECTIONS, UNCOATED

Effective Properties



Designation	Mass	Gross Area	Effective Section Properties X-X Axis I_{xe}	S_{xe}	Y-Y Axis I_{ye}	S_{ye}	M_{rb}	L_{cr}	M_{rdb}	V_r	L_u
	kg/m	mm²	10^6 mm⁴	10^3 mm³	10^6 mm⁴	10^3 mm³	kN·m	mm	kN·m	kN	mm
406S76-290M	13.9	1 771	37.2	169	0.787	15.9	52.3	659	43.3	57.2	1 489
406S76-254M	12.2	1 558	32.4	140	0.676	14.1	43.4	708	36.0	38.1	1 496
356S89-326M	15.0	1 906	33.5	183	1.34	23.2	56.9	684	47.0	94.3	1 763
356S89-290M	13.4	1 704	30.1	163	1.17	20.8	50.5	730	40.2	65.8	1 769
356S89-254M	11.8	1 499	26.6	140	1.00	18.4	43.4	785	33.5	43.8	1 774
356S89-218M	10.1	1 292	23.0	108	0.832	15.8	33.6	853	27.1	27.4	1 780
356S76-290M	12.7	1 616	27.2	144	0.783	15.9	44.8	636	37.8	65.8	1 512
356S76-254M	11.2	1 423	23.8	120	0.673	14.1	37.4	684	31.5	43.8	1 518
305S89-326M	13.6	1 732	23.1	148	1.33	23.1	45.8	658	39.5	109	1 786
305S89-290M	12.2	1 549	20.8	131	1.16	20.8	40.7	702	33.9	77.5	1 791
305S89-254M	10.7	1 364	18.4	112	0.996	18.3	34.9	755	28.4	51.5	1 796
305S89-218M	9.23	1 176	15.9	93.4	0.829	15.8	29.0	820	23.1	32.2	1 801
305S89-181M	7.74	986	13.1	71.2	0.663	13.1	22.1	904	18.0	18.5	1 806
305S76-290M	11.5	1 462	18.9	124	0.779	15.9	38.4	611	31.9	77.5	1 536
305S76-254M	10.1	1 287	16.7	107	0.670	14.0	33.3	658	26.8	51.5	1 541
305S76-218M	8.72	1 111	14.5	90.4	0.559	12.1	28.1	715	21.8	32.2	1 546
305S76-181M	7.31	932	12.3	67.8	0.449	10.1	21.0	789	17.0	18.5	1 552
254S89-326M	12.2	1 557	15.0	115	1.31	23.1	35.6	628	32.0	109	1 812
254S89-290M	10.9	1 394	13.5	102	1.15	20.7	31.7	670	27.6	86.2	1 815
254S89-254M	9.64	1 228	12.0	87.3	0.988	18.3	27.1	721	23.2	62.5	1 819
254S89-218M	8.32	1 060	10.4	72.4	0.824	15.7	22.5	783	18.9	39.0	1 822
254S89-181M	6.98	889	8.53	60.0	0.660	13.1	18.6	864	14.8	22.3	1 827
254S89-144M	5.62	716	6.76	43.0	0.501	10.4	13.4	973	10.9	11.3	1 831
254S76-290M	10.3	1 307	12.2	95.9	0.772	15.8	29.8	583	25.9	86.2	1 561
254S76-254M	9.04	1 152	10.8	83.1	0.665	14.0	25.8	628	21.9	62.5	1 565
254S76-218M	7.81	995	9.39	70.0	0.556	12.1	21.7	683	17.9	39.0	1 570
254S76-181M	6.55	835	7.94	56.6	0.447	10.1	17.6	754	14.0	22.3	1 574
254S76-144M	5.28	673	6.29	41.7	0.339	8.08	12.9	849	10.3	11.3	1 579
229S89-326M	11.5	1 470	11.7	99.4	1.31	23.0	30.9	611	28.3	109	1 827
229S89-290M	10.3	1 317	10.5	88.4	1.15	20.7	27.5	652	24.4	86.2	1 829
229S89-254M	9.11	1 160	9.35	75.7	0.983	18.2	23.5	702	20.6	66.0	1 831
229S89-218M	7.86	1 002	8.10	62.7	0.820	15.7	19.5	763	16.8	43.6	1 834
229S89-181M	6.60	841	6.67	52.0	0.658	13.1	16.1	841	13.2	25.0	1 838
203S76-290M	9.04	1 152	7.17	70.6	0.762	15.7	21.9	552	19.9	86.2	1 591
203S76-254M	7.98	1 016	6.38	61.3	0.657	13.9	19.0	594	16.9	66.0	1 593
203S76-218M	6.90	878	5.55	51.6	0.550	12.1	16.0	646	13.9	48.5	1 596
203S76-181M	5.79	738	4.70	41.6	0.443	10.1	12.9	713	11.0	28.3	1 599
203S76-144M	4.67	595	3.72	33.5	0.337	8.06	10.4	803	8.14	14.3	1 603

Designation Example: 356S89-254M; where 356 = section depth (mm); S = stud or joist C-section; 89 = flange width (mm); 254 = minimum base steel thickness x 100 (mm); M = metric designation

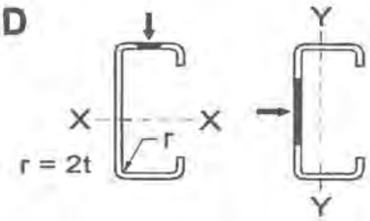


COLD-FORMED C-SECTIONS, UNCOATED
Dimensions and Gross Properties

Depth	Flange Width	Stiff'r Depth	Thick-ness	Gross Section Properties										
				X-X Axis			Y-Y Axis			x_o	r_o	J	j	C_w
d	b	D	t	I_x	S_x	r_x	I_y	S_y	r_y					
mm	mm	mm	mm	10^6 mm^4	10^3 mm^3	mm	10^6 mm^4	10^3 mm^3	mm	mm	mm	10^3 mm^4	mm	10^9 mm^6
406	76	24	3.05	38.1	188	147	1.07	17.6	24.5	40.8	154	5.49	281	36.7
406	76	24	2.67	33.7	166	147	0.956	15.8	24.8	41.2	155	3.69	277	32.7
356	89	25	3.43	33.5	188	133	1.71	25.2	30.0	53.7	146	7.47	213	44.8
356	89	25	3.05	30.1	169	133	1.56	23.0	30.2	54.1	147	5.28	211	40.5
356	89	25	2.67	26.6	150	133	1.39	20.6	30.5	54.6	147	3.55	209	36.0
356	89	25	2.29	23.1	130	134	1.22	18.0	30.7	55.0	148	2.25	206	31.4
356	76	24	3.05	27.5	154	130	1.03	17.4	25.3	43.7	140	5.01	227	27.2
356	76	24	2.67	24.3	137	131	0.926	15.6	25.5	44.1	140	3.37	223	24.2
305	89	25	3.43	23.1	152	116	1.64	24.9	30.8	57.6	133	6.79	174	31.8
305	89	25	3.05	20.8	136	116	1.49	22.7	31.0	58.0	133	4.80	173	28.8
305	89	25	2.67	18.4	121	116	1.34	20.3	31.3	58.5	134	3.23	171	25.6
305	89	25	2.29	16.0	105	117	1.17	17.8	31.6	58.9	134	2.05	170	22.3
305	89	25	1.91	13.5	88.4	117	0.997	15.2	31.8	59.4	135	1.19	169	18.9
305	76	24	3.05	18.9	124	114	0.991	17.2	26.0	47.1	126	4.53	181	19.3
305	76	24	2.67	16.7	110	114	0.890	15.4	26.3	47.5	126	3.05	178	17.2
305	76	24	2.29	14.5	95.3	114	0.782	13.6	26.5	48.0	127	1.93	176	15.0
305	76	24	1.91	12.3	80.4	115	0.668	11.6	26.8	48.4	127	1.13	174	12.8
254	89	25	3.43	15.0	118	98.1	1.55	24.5	31.6	62.2	120	6.10	143	21.3
254	89	25	3.05	13.5	106	98.4	1.41	22.3	31.8	62.6	121	4.32	142	19.3
254	89	25	2.67	12.0	94.2	98.7	1.27	19.9	32.1	63.1	121	2.91	141	17.2
254	89	25	2.29	10.4	81.8	99.0	1.11	17.5	32.3	63.6	122	1.85	140	15.0
254	89	25	1.91	8.77	69.1	99.3	0.944	14.9	32.6	64.1	123	1.08	139	12.7
254	89	25	1.52	7.11	56.0	99.6	0.772	12.2	32.8	64.5	123	0.55	139	10.4
254	76	24	3.05	12.2	95.9	96.5	0.941	16.9	26.8	51.1	112	4.05	143	12.9
254	76	24	2.67	10.8	85.1	96.9	0.845	15.2	27.1	51.6	113	2.73	142	11.5
254	76	24	2.29	9.40	74.0	97.2	0.743	13.4	27.3	52.0	114	1.73	140	10.1
254	76	24	1.91	7.94	62.5	97.5	0.635	11.4	27.6	52.5	114	1.01	139	8.56
254	76	24	1.52	6.44	50.7	97.8	0.521	9.37	27.8	52.9	115	0.52	138	6.98
229	89	25	3.43	11.7	102	89.2	1.50	24.2	32.0	64.8	115	5.76	130	17.0
229	89	25	3.05	10.5	92.2	89.5	1.37	22.0	32.2	65.3	115	4.08	129	15.4
229	89	25	2.67	9.35	81.8	89.8	1.22	19.7	32.5	65.7	116	2.75	129	13.7
229	89	25	2.29	8.13	71.1	90.1	1.07	17.3	32.7	66.2	116	1.75	128	12.0
229	89	25	1.91	6.86	60.0	90.3	0.913	14.7	33.0	66.7	117	1.02	128	10.2
203	76	24	3.05	7.17	70.6	78.9	0.878	16.5	27.6	56.0	101	3.57	114	7.99
203	76	24	2.67	6.38	62.8	79.2	0.788	14.9	27.9	56.5	101	2.41	114	7.14
203	76	24	2.29	5.55	54.7	79.5	0.693	13.1	28.1	56.9	102	1.53	113	6.25
203	76	24	1.91	4.70	46.3	79.8	0.593	11.2	28.3	57.4	102	0.89	112	5.32
203	76	24	1.52	3.82	37.6	80.1	0.486	9.17	28.6	57.9	103	0.46	112	4.35

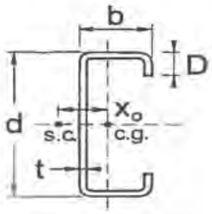
COLD-FORMED C-SECTIONS, UNCOATED

Effective Properties



Designation	Mass	Gross Area	Effective Section Properties				M_{rb}	L_{cr}	M_{rdb}	V_r	L_u
			X-X Axis		Y-Y Axis						
			I_{xd}	S_{xe}	I_{ye}	S_{ye}					
	kg/m	mm^2	$10^6\ mm^4$	$10^3\ mm^3$	$10^6\ mm^4$	$10^3\ mm^3$	kN·m	mm	kN·m	kN	mm
203S70-326M	9.83	1 252	7.57	74.5	0.719	15.8	23.1	512	21.9	104	1 496
203S70-290M	8.82	1 123	6.84	67.4	0.635	14.3	20.9	547	19.7	86.2	1 498
203S70-254M	7.78	991	6.09	59.9	0.548	12.7	18.6	589	16.8	66.0	1 500
203S70-218M	6.72	857	5.30	52.2	0.459	11.0	16.2	641	13.8	48.5	1 503
203S70-181M	5.65	720	4.49	41.9	0.370	9.25	13.0	708	10.9	28.3	1 506
203S70-144M	4.56	581	3.59	32.5	0.281	7.40	10.1	797	8.15	14.3	1 510
152S76-290M	7.83	997	3.66	48.0	0.743	15.6	14.9	513	14.1	67.7	1 636
152S76-254M	6.91	881	3.26	41.8	0.643	13.8	13.0	552	12.1	60.2	1 635
152S76-218M	5.98	762	2.84	35.2	0.541	12.0	10.9	601	9.99	48.5	1 634
152S76-181M	5.03	641	2.41	28.2	0.437	10.0	8.77	663	7.94	33.7	1 635
152S76-144M	4.06	518	1.91	22.7	0.334	8.03	7.05	747	5.94	19.4	1 637
152S70-326M	8.46	1 078	3.83	50.2	0.698	15.7	15.6	477	14.7	74.9	1 550
152S70-290M	7.60	968	3.47	45.5	0.619	14.2	14.1	509	13.4	67.7	1 549
152S70-254M	6.72	856	3.09	40.6	0.537	12.6	12.6	548	11.9	60.2	1 548
152S70-218M	5.81	741	2.70	35.4	0.451	11.0	11.0	597	9.88	48.5	1 548
152S70-181M	4.89	623	2.29	28.4	0.365	9.20	8.83	659	7.88	33.7	1 549
152S70-144M	3.95	503	1.83	22.0	0.279	7.37	6.83	742	5.92	19.4	1 550

Designation Example: 152S76-181M; where 152 = section depth (mm); S = stud or joist C-section; 76 = flange width (mm); 181 = minimum base steel thickness x 100 (mm); M = metric designation



COLD-FORMED C-SECTIONS, UNCOATED

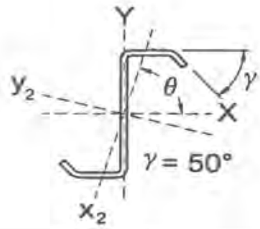
Dimensions and Gross Properties

Depth	Flange Width	Stiff'r Depth	Thick-ness	Gross Section Properties										
				X-X Axis			Y-Y Axis							
d	b	D	t	I_x	S_x	r_x	I_y	S_y	r_y	x_o	r_o	J	j	C_w
mm	mm	mm	mm	10^6 mm^4	10^3 mm^3	mm	10^6 mm^4	10^3 mm^3	mm	mm	mm	10^3 mm^4	mm	10^9 mm^6
203	70	25	3.43	7.57	74.5	77.8	0.800	16.4	25.3	51.1	96	4.91	113	7.53
203	70	25	3.05	6.84	67.4	78.1	0.732	15.0	25.5	51.6	97	3.48	112	6.84
203	70	25	2.67	6.09	59.9	78.4	0.658	13.5	25.8	52.0	98	2.35	111	6.12
203	70	25	2.29	5.30	52.2	78.7	0.579	11.9	26.0	52.5	98	1.49	110	5.36
203	70	25	1.91	4.49	44.2	79.0	0.496	10.2	26.2	53.0	99	0.87	110	4.57
203	70	25	1.52	3.65	35.9	79.3	0.407	8.37	26.5	53.4	99	0.45	109	3.74
152	76	24	3.05	3.66	48.0	60.6	0.795	16.0	28.2	62.1	91	3.09	94	4.43
152	76	24	2.67	3.26	42.8	60.8	0.714	14.4	28.5	62.6	92	2.09	94	3.97
152	76	24	2.29	2.84	37.3	61.1	0.629	12.7	28.7	63.1	92	1.33	94	3.48
152	76	24	1.91	2.41	31.7	61.3	0.538	10.8	29.0	63.6	93	0.78	94	2.97
152	76	24	1.52	1.96	25.8	61.6	0.441	8.89	29.2	64.1	94	0.40	94	2.43
152	70	25	3.43	3.83	50.2	59.6	0.724	15.9	25.9	56.9	86	4.23	90	4.21
152	70	25	3.05	3.47	45.5	59.8	0.663	14.5	26.2	57.4	87	3.00	90	3.83
152	70	25	2.67	3.09	40.6	60.1	0.596	13.1	26.4	57.9	88	2.03	90	3.44
152	70	25	2.29	2.70	35.4	60.4	0.525	11.5	26.6	58.4	88	1.29	90	3.02
152	70	25	1.91	2.29	30.1	60.6	0.450	9.89	26.9	58.9	89	0.75	89	2.58
152	70	25	1.52	1.87	24.5	60.9	0.369	8.13	27.1	59.3	89	0.39	89	2.11

COLD-FORMED Z-SECTIONS, UNCOATED

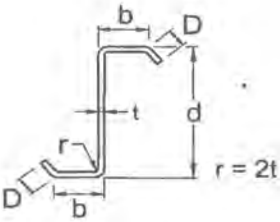
Effective Properties

F_y = 345 MPa



Designation	Mass	Gross Area	Effective Section Properties X-X Axis I_{xd}	Effective Section Properties X-X Axis S_{xe}	M_{rb}	L_{cr}	M_{rdb}	V_r	L_u
	kg/m	mm²	10^6 mm⁴	10^3 mm³	kN·m	mm	kN·m	kN	mm
356Z76-326M	14.5	1 844	30.8	170	52.9	520	42.5	94.3	1 337
356Z76-290M	12.9	1 646	27.5	147	45.8	554	35.9	65.8	1 339
356Z76-254M	11.4	1 445	24.0	122	37.8	595	29.6	43.8	1 342
356Z76-218M	9.76	1 244	20.5	95.3	29.6	646	23.6	27.4	1 344
356Z76-181M	8.17	1 040	16.9	72.8	22.6	711	18.1	15.7	1 347
305Z76-326M	13.1	1 670	21.2	142	44.0	499	35.8	109	1 371
305Z76-290M	11.7	1 491	19.1	126	39.1	532	30.4	77.5	1 373
305Z76-254M	10.3	1 310	16.9	108	33.6	571	25.1	51.5	1 375
305Z76-218M	8.85	1 127	14.6	89.2	27.7	621	20.2	32.2	1 377
305Z76-181M	7.41	943	12.3	67.1	20.8	684	15.5	18.5	1 379
254Z76-326M	11.7	1 496	13.7	110	34.2	476	29.0	109	1 408
254Z76-290M	10.5	1 336	12.3	98.0	30.4	507	24.7	86.2	1 409
254Z76-254M	9.22	1 174	10.9	84.0	26.1	545	20.6	62.5	1 410
254Z76-218M	7.94	1 011	9.49	69.1	21.5	592	16.6	39.0	1 412
254Z76-181M	6.65	847	8.01	56.1	17.4	653	12.8	22.3	1 413
229Z76-326M	11.1	1 409	10.7	95.4	29.6	463	25.6	109	1 429
229Z76-290M	9.88	1 258	9.61	84.9	26.4	494	21.9	86.2	1 429
229Z76-254M	8.69	1 107	8.52	72.8	22.6	531	18.3	66.0	1 430
229Z76-218M	7.48	953	7.40	59.8	18.6	577	14.8	43.6	1 431
229Z76-181M	6.27	798	6.25	48.4	15.0	636	11.4	25.0	1 432
203Z76-326M	10.4	1 322	8.08	81.4	25.3	450	22.3	104	1 452
203Z76-290M	9.27	1 181	7.29	72.5	22.5	479	19.1	86.2	1 451
203Z76-254M	8.15	1 039	6.47	62.2	19.3	515	16.0	66.0	1 451
203Z76-218M	7.03	895	5.62	51.0	15.8	560	12.9	48.5	1 451
203Z76-181M	5.89	750	4.75	41.2	12.8	617	10.1	28.3	1 451
203Z76-144M	4.73	603	3.74	32.5	10.1	695	7.40	14.3	1 452
152Z76-326M	9.01	1 147	4.14	55.7	17.3	418	15.8	74.9	1 507
152Z76-290M	8.06	1 026	3.74	49.7	15.4	446	13.6	67.7	1 503
152Z76-254M	7.09	903	3.32	42.6	13.2	479	11.5	60.2	1 500
152Z76-218M	6.12	779	2.89	34.8	10.8	521	9.40	48.5	1 498
152Z76-181M	5.13	653	2.45	28.1	8.70	574	7.30	33.7	1 496
152Z76-144M	4.13	526	1.93	22.1	6.90	646	5.40	19.4	1 495

Designation Example: 229Z76-290M; where 229 = section depth (mm); Z = Z-section;
76 = flange width (mm); 290 = minimum base steel thickness x 100 (mm); M = metric designation



COLD-FORMED Z-SECTIONS, UNCOATED

Dimensions and Gross Properties

Depth	Flange Width	Stiff'r Depth	Thick-ness	Gross Section Properties										
				X-X Axis		Y-Y Axis		I_{xy}	I_{x2}	I_{y2}	r_{min}	θ	J	C_w
d	b	D	t	S_x	r_x	S_y	r_y							
mm	mm	mm	mm	10^3 mm^3	mm	10^3 mm^3	mm	10^6 mm^4	10^6 mm^4	10^6 mm^4	mm	deg	10^3 mm^4	10^9 mm^6
356	76	24	3.43	178	131	21.7	32.5	5.41	1.00	32.7	23.3	80.0	7.23	47.2
356	76	24	3.05	160	131	19.5	32.7	4.85	0.903	29.3	23.4	80.0	5.10	42.5
356	76	24	2.67	141	132	17.2	32.8	4.29	0.801	25.8	23.5	80.0	3.43	37.7
356	76	24	2.29	122	132	14.9	33.0	3.71	0.696	22.3	23.7	80.0	2.17	32.7
356	76	24	1.91	102	132	12.6	33.1	3.12	0.588	18.8	23.8	80.0	1.26	27.6
305	76	24	3.43	143	114	21.7	34.2	4.61	0.937	22.9	23.7	77.6	6.55	33.6
305	76	24	3.05	129	115	19.5	34.3	4.14	0.845	20.5	23.8	77.6	4.62	30.3
305	76	24	2.67	114	115	17.2	34.5	3.65	0.750	18.1	23.9	77.5	3.11	26.8
305	76	24	2.29	98.1	115	14.9	34.6	3.16	0.653	15.7	24.1	77.5	1.96	23.3
305	76	24	1.91	82.5	116	12.6	34.8	2.66	0.552	13.2	24.2	77.5	1.14	19.7
254	76	24	3.43	111	97.2	21.7	36.1	3.81	0.858	15.2	24.0	74.0	5.86	22.4
254	76	24	3.05	99.9	97.5	19.5	36.3	3.42	0.775	13.7	24.1	74.0	4.14	20.2
254	76	24	2.67	88.3	97.7	17.2	36.4	3.02	0.688	12.1	24.2	74.0	2.78	17.9
254	76	24	2.29	76.4	98.0	14.9	36.6	2.62	0.599	10.5	24.3	74.0	1.76	15.6
254	76	24	1.91	64.3	98.2	12.6	36.7	2.20	0.507	8.80	24.5	74.0	1.02	13.2
229	76	24	3.43	96.4	88.4	21.7	37.2	3.41	0.811	12.2	24.0	71.5	5.52	17.7
229	76	24	3.05	86.6	88.7	19.5	37.4	3.07	0.732	10.9	24.1	71.5	3.90	16.0
229	76	24	2.67	76.5	88.9	17.2	37.5	2.71	0.651	9.66	24.3	71.5	2.62	14.2
229	76	24	2.29	66.3	89.2	14.9	37.7	2.35	0.567	8.36	24.4	71.5	1.66	12.3
229	76	24	1.91	55.8	89.4	12.6	37.8	1.97	0.480	7.04	24.5	71.5	0.966	10.4
203	76	24	3.43	82.2	79.5	21.7	38.4	3.02	0.756	9.55	23.9	68.3	5.18	13.7
203	76	24	3.05	73.9	79.7	19.5	38.6	2.71	0.683	8.58	24.1	68.4	3.66	12.3
203	76	24	2.67	65.4	80.0	17.2	38.7	2.39	0.608	7.59	24.2	68.4	2.46	10.9
203	76	24	2.29	56.7	80.2	14.9	38.9	2.07	0.529	6.58	24.3	68.4	1.56	9.51
203	76	24	1.91	47.7	80.4	12.6	39.0	1.75	0.448	5.54	24.5	68.4	0.907	8.04
203	76	24	1.52	38.6	80.6	10.2	39.1	1.41	0.364	4.48	24.6	68.4	0.467	6.53
152	76	24	3.43	56.1	61.0	21.7	41.3	2.22	0.610	5.62	23.1	58.8	4.50	7.20
152	76	24	3.05	50.5	61.2	19.5	41.4	1.99	0.552	5.05	23.2	58.8	3.18	6.51
152	76	24	2.67	44.8	61.5	17.2	41.5	1.76	0.492	4.48	23.3	58.9	2.14	5.79
152	76	24	2.29	38.9	61.7	14.9	41.7	1.53	0.430	3.89	23.5	58.9	1.36	5.04
152	76	24	1.91	32.8	61.9	12.6	41.8	1.29	0.364	3.28	23.6	58.9	0.790	4.27
152	76	24	1.52	26.6	62.1	10.2	41.9	1.04	0.297	2.65	23.8	58.9	0.407	3.47

BARS AND PLATES

Bars

The term "bars" means:

(a) Rounds, squares and hexagons of all sizes;

(b) Flats up to 150 mm in width and over 5 mm in thickness;
flats over 150 mm to 200 mm in width and over 6 mm in thickness.

Bar-size shapes include rolled flanged sections and angles under 75 mm in maximum dimension.

Plates

The term "plate" means flat hot-rolled steel, when ordered to thickness:

(a) Over 200 mm in width and 6 mm or over in thickness;

(b) Over 1200 mm in width and 4.5 mm or over in thickness.

Slabs, sheet bars, and skelp, although frequently falling within these size ranges, are not classified as plate. The table on the following page, Standard Product Classification for Flat Hot-Rolled Steel Products and Bars, summarizes the ranges for plate, bar, strip and sheet products.

Plates may be further defined as "Universal Mill Plates" or "Sheared Plates". Sheared plates are rolled on a mill with horizontal rolls only, producing a product with uneven edges which must be sheared (or, at the option of the producer, flame cut) to ordered dimensions.

Universal mill plates are rolled to the ordered width on a mill having side rollers to control the width. Slab or ingot on a universal mill plate are not cross-rolled, but are only elongated during the rolling process. The mill order must specify universal mill plate when it is required.

Extreme plate sizes produced by mills vary greatly with the size of various mills, and individual mills should be consulted for this information.

Various extras for thickness, width, length, cutting, quality, quantity (or quantity discounts), and for other special requirements are added to the base price of plates. Particulars of these extras should be obtained from the producing mills.

Sketch Plates

Sketch plates of special or unusual shape usually require flame cutting, for which flame cutting extras apply. Some mills can supply sketch plates of certain shapes by shearing to size.

Floor Plates

Floor plates in different styles, patterns, and extreme dimensions are produced by different mills. The nominal, or ordered, thickness is that of the flat plate exclusive of the raised pattern. Individual producers should be consulted for more details.

Bearing Plates

Rolled steel bearing plates are used for column bases, and other bearing plates. Depending on the thickness required by design, bearing plates may require additional thickness for machining to ensure proper bearing. According to CSA S16-14 Clause 25.4.1.3, column base plates up to and including 55 mm in thickness are rolled flat with surfaces sufficiently smooth to receive, without machining or flattening, the milled or machine-cut ends of column shafts. Bearing plates over 55 mm in thickness may be flattened by pressing or machining to achieve the required flatness tolerances.

Tables

The following Tables are included in this section:

Standard Product Classification of Flat Hot-Rolled Steel Products and Bars

Flat Metal Products – Plate

SI Wire Size – Wire Gauges Comparison

SI Thickness – Imperial Gauge Comparisons

STANDARD PRODUCT CLASSIFICATION

Flat Hot-Rolled Steel Products and Bars

Width, w (mm)	Thickness, t (mm)					
	$t > 6$	$6 \geq t > 5$	$5 \geq t > 4.5$	$4.5 \geq t > 1.2$	$1.2 \geq t > 0.9$	$0.9 \geq t > 0.65$
$w \leq 100$	BAR	BAR	STRIP	STRIP	STRIP	STRIP
$100 < w \leq 150$	BAR	BAR	STRIP	STRIP	STRIP	
$150 < w \leq 200$	BAR	STRIP	STRIP	STRIP		
$200 < w \leq 300$	PLATE	STRIP	STRIP	STRIP		
$300 < w \leq 1200$	PLATE	SHEET*	SHEET*	SHEET*		
$1200 < w$	PLATE	PLATE	PLATE	SHEET		

* For alloy steels, sheet begins at widths over 600 mm.

FLAT METAL PRODUCTS* – PLATE

If metric plate thicknesses are desired

Nominal Thickness,** mm		Mass†	Dead Load
First Preference	Second Preference	kg/m^2	kN/m^2
4.5		35.3	0.347
	4.8	37.7	0.370
5.0		39.3	0.385
	5.5	43.2	0.424
6.0		47.1	0.462
7.0		55.0	0.539
8.0		62.8	0.616
	9.0	70.7	0.693
10		78.5	0.770
	11	86.4	0.847
12		94.2	0.924
	14	110	1.08
16		126	1.23
	18	141	1.39
20		157	1.54
	22	173	1.69
25		196	1.93
	28	220	2.16
30		236	2.31
	32	251	2.46
35		275	2.70
	38	298	2.93
40		314	3.08
	45	353	3.47
50		393	3.85
	55	432	4.24
60		471	4.62
	70	550	5.39
80		628	6.16
	90	707	6.93
100		785	7.70
	110	864	8.47
120		942	9.24
	130	1020	10.0
140		1100	10.8
	150	1180	11.6
160		1260	12.3
180		1410	13.9
200		1570	15.4
250		1960	19.3
300		2360	23.1

* Sizes are those listed in CAN3-G312.1-75. Metric plate thickness preferences apply mostly to bridge structures.

** For coated structural sheet, the nominal thickness applies to the base metal. For metric thickness dimensions for zinc coated structural quality sheet steel, see Part 7, Structural Sheet Steel Products.

† Computed using steel density of 7 850 kg/m^3.

SI WIRE SIZE – WIRE GAUGES COMPARISON

SI Wire Size Preferred Diam.* (mm)	United States Steel Wire Gauge	American or Brown & Sharpe Wire Gauge	British Imperial or English Legal Standard Wire Gauge	Birming-ham or Stubs Iron Wire Gauge
25.0				
24.0				
23.0				
22.0				
21.0				
20.0				
19.0				
18.0				
17.0				
16.0				
15.0				
		6/0		
14.0				
		5/0		
13.0				
			7/0	5/0
12.5				
	7/0			
12.0				
11.8				
	6/0	4/0	6/0	4/0
11.2				
11.0				
	5/0		5/0	3/0
10.6				
		3/0	4/0	
10.0				
	4/0			2/0
9.5				
	3/0	2/0	3/0	
9.0				
			2/0	1/0
8.5				
	2/0	1/0	1/0	
8.0				
	1/0		1	1
7.5				
	1	1	2	2
7.0				
6.7				
	2	2		3
6.5				
			3	
6.3				
	3			4
6.0				

SI Wire Size Preferred Diam.* (mm)	United States Steel Wire Gauge	American or Brown & Sharpe Wire Gauge	British Imperial or English Legal Standard Wire Gauge	Birming-ham or Stubs Iron Wire Gauge
6.0				
	4	3	4	
5.6				
			5	5
5.3				
	5	4		6
5.0				
	6		6	
4.8				
		5		
4.6				
	7		7	7
4.4				
4.2				
	8	6	8	8
4.0				
3.8				
	9	7	9	9
3.6				
	10			10
3.4				
		8	10	
3.2				
	11			11
3.0				
		9	11	
2.8				
	12		12	12
2.6				
		10		13
2.4				
	13	11	13	
2.3				
2.2				
				14
2.1				
	14	12	14	
2.0				
1.90				
	15	13	15	15
1.80				
1.70				
		14	16	16
1.60				
	16			
1.50				

* From CAN3-G312.2-M76

SI THICKNESS – IMPERIAL GAUGE COMPARISONS†

SI Preferred Thickness		United States Standard Gauge*				Birmingham Sheet Gauge		
		Weight	Ga. No.	Approximate Thickness		Gauge Number	Thickness	
First mm	Second mm	Oz. per sq. ft.		Inches	mm		Inches	mm
	18							
						7/0	0.6666	16.932
16								
						6/0	0.6250	15.875
						5/0	0.5883	14.943
	14							
						4/0	0.5416	13.757
						3/0	0.5000	12.700
12								
						2/0	0.4452	11.308
	11							
						0	0.3964	10.069
10								
	9.0							
						1	0.3532	8.971
8.0								
						2	0.3147	7.993
						3	0.2804	7.122
7.0								
		160	3	0.2391	6.073	4	0.2500	6.350
6.0								
		150	4	0.2242	5.695	5	0.2225	5.652
	5.5							
		140	5	0.2092	5.314	6	0.1981	5.032
5.0								
		130	6	0.1943	4.935			
	4.8							
		120	7	0.1793	4.554			
4.5								

† Preferred thicknesses are as per CAN3-G312.1-75

* U.S. Standard Gauge is officially a weight gauge, in oz. per sq. ft. as tabulated. The Approx. thickness shown is the "Manufacturers' Standard" of the AISI based on a steel density of 501.81 lb. per $ft.^3$

CRANE RAILS

General

Crane rails are designated by their mass in pounds per yard, with bolt sizes, hole diameters, and washer sizes dimensioned in inches. The SI metric dimensions and properties for crane rails and their accessories given on the following pages are soft-converted from manufacturers' catalogs. For ordering information, refer to ASTM standards A1 and A759 for tee rails (60 lb/yd and over) and crane rails (104 to 175 lb/yd), respectively.

Rails listed in this handbook are the most popular sizes used for crane runways. For dimensions and properties not provided in the tables, consult the supplier.

Rails are typically supplied in lengths ranging from 9140 mm for the lighter rails up to 23 800 mm for the heavier sections. Consult the supplier for further information.

If bolted rail bar splices are to be used, the number of rail lengths required, plus one short length in each run, should be specified to permit staggering of the joints. Orders must clearly specify that "*These Rails Are Intended for Crane Service*".

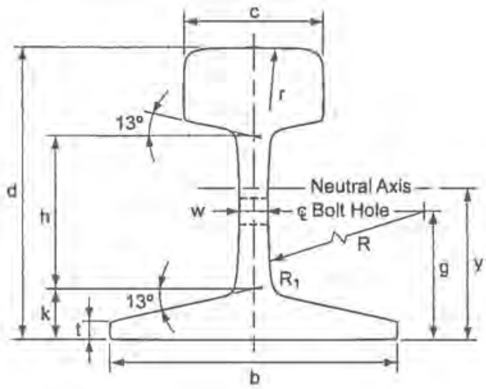
Most manufacturers will chamfer the top and sides of the rail head at the ends, unless specified otherwise by the purchaser. Chamfering permits mild deformations to occur and minimizes chipping of the running surfaces.

When selecting a rail for crane service, the characteristics of operation must be considered. Some common variables which affect service life are:

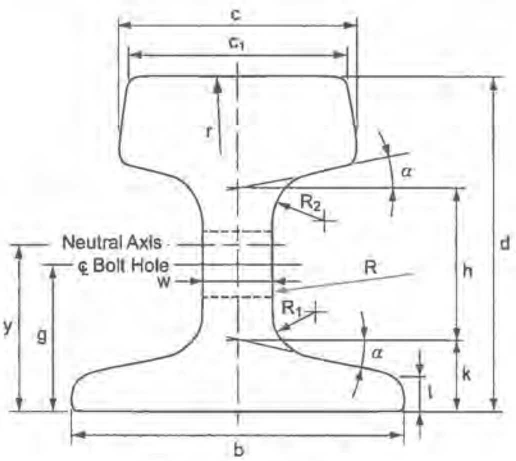
- Frequency of operation
- Crane carriage speed and impact – rate of loading and unloading
- Corrosion – acidic mill conditions
- Abrasion
- Alignment of crane and supporting members
- Crane operating procedures

Crane rails are joined together end-to-end by either mechanical fasteners or welding. When bolting is used, special joint bars are employed, as shown on the following pages. If welded, manual arc welding is usually used and joint bars are not required. Welding has the advantage of eliminating mechanical joints, thus reducing the problem of aligning the top of rails.

CRANE RAILS – PROPERTIES AND DIMENSIONS



30 to 104 lb/yd



135 to 175 lb/yd

Dimensions

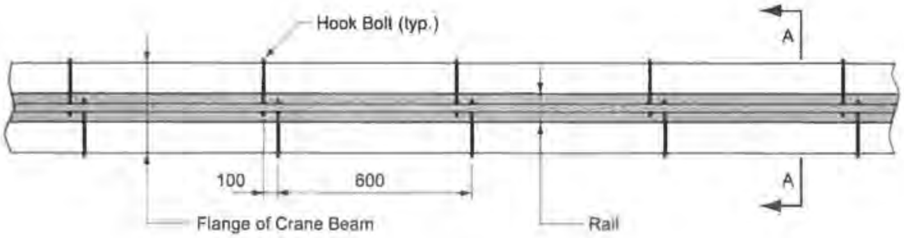
Rail type		Depth	Head		Base		Web		k	h	r	R	R_1	R_2	α
		d	c	c_1	b	t	w	Gauge g							
		mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	deg
ASCE	30	79	43	43	79	4.4	8.3	35	13	44	305	305	6.4	6.4	13
	40	89	48	48	89	5.6	9.9	39	16	47	305	305	6.4	6.4	13
	60	108	60	60	108	7.1	12	48	19	58	305	305	6.4	6.4	13
	80	127	64	64	127	7.5	14	56	22	67	305	305	6.4	6.4	13
	85	132	65	65	132	7.5	14	58	23	70	305	305	6.4	6.4	13
	100	146	70	70	146	7.9	14	64	25	53	305	305	6.4	6.4	13
ASTM A759	104	127	64	64	127	13	25	62	27	62	305	89	13	13	13
	135	146	87	76	132	12	32	63	27	71	356	305	19	19	13
	171	152	109	102	152	16	32	67	32	70	Flat	Vert.	19	22	12
	175	152	108	102	152	13	38	67	29	79	457	Vert.	29	51	12

Properties

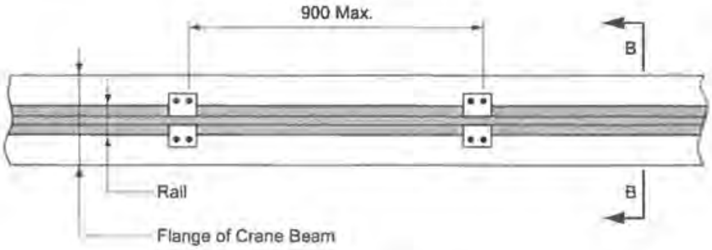
Rail type		Mass	Dead Load	Area	I_x	S_x Head	S_x Base	y
		kg/m	kN/m	mm²	10^6 mm⁴	10^3 mm³	10^3 mm³	mm
ASCE	30	14.9	0.146	1 940	1.71	41.8	—	—
	40	19.8	0.195	2 540	2.72	58.8	63.7	42.7
	60	29.8	0.292	3 830	6.08	109	117	52.1
	80	39.7	0.389	5 070	11.0	166	182	60.5
	85	42.2	0.413	5 370	12.5	182	200	62.7
	100	49.6	0.486	6 350	18.3	239	264	69.3
ASTM A759	104	51.6	0.506	6 650	12.4	175	221	56.1
	135	67.0	0.657	8 580	21.1	283	297	71.4
	171	84.8	0.832	10 800	30.6	401	400	76.5
	175	86.8	0.851	11 000	29.3	383	387	75.7

Rail Fasteners

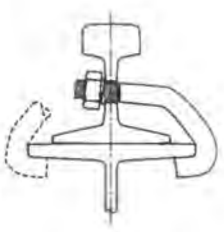
Hook bolts are primarily used when the flange of the crane beam is too narrow to permit the use of rail clamps. Hook bolts are used in groups of 2, located about 100 mm to 140 mm apart, at 600 mm centres, and may be adjusted plus or minus 12 mm. Suggested dimensions are shown in Section A-A. Rails require special preparation either in the fabricator's shop or by the crane rail supplier.



Suggested rail clamp dimensions are shown in Section B-B. For prefabricated rail clamps, reference should be made to manufacturers' catalogs of track accessories. Two types of clamps are available: the tight clamp and the floating clamp. Floating clamps are used when longitudinal and controlled transverse movement is required for thermal expansion and alignment. Rail clamps are fabricated from pressed or forged steel and usually have single or double bolts.

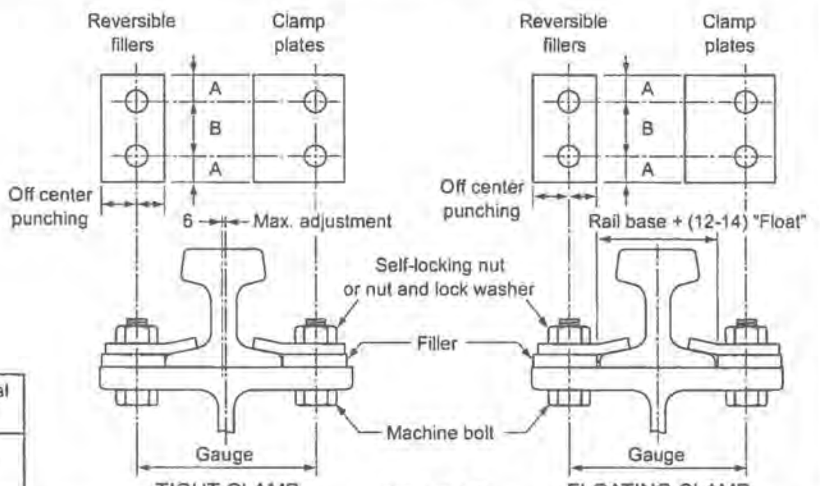


RAIL FASTENERS



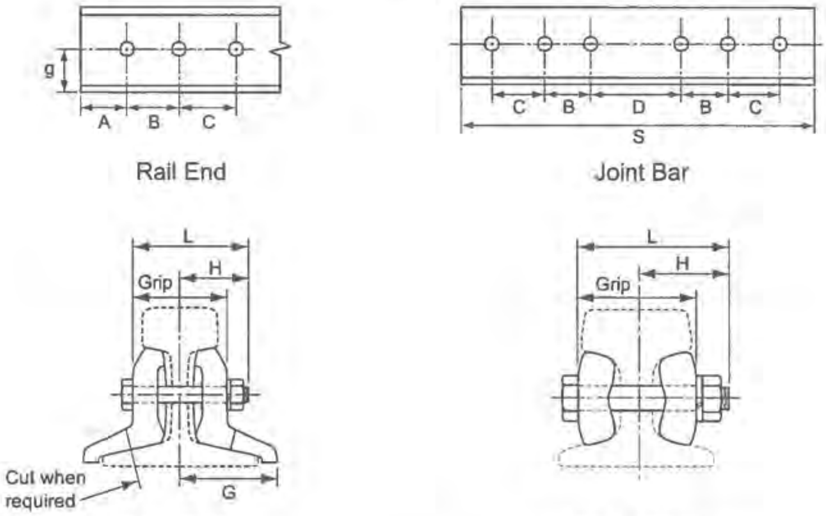
Section A-A

Distance	SI (mm)	Imperial (in.)
A	40	1½
B	80	3



Section B-B

RAIL SPLICES



Rail End — Joint Bar

40 to 104 lbs. — 135 to 175 lbs.

Rail Type	Rail					Joint Bar					
	g	Hole dia.	A	B	C	Hole dia.	D	B	C	S	G
	mm	inch.	mm	mm	mm	inch.	mm	mm	mm	mm	mm
40	39.5	*13/16	63.5	127	–	*13/16	125	127	–	508	55.6
60	48.2	*13/16	63.5	127	–	*13/16	125	127	–	610	68.3
85	57.5	*15/16	63.5	127	–	*15/16	125	127	–	610	84.9
104	61.9	1-1/16	102	127	152	1-1/16	202	127	152	864	88.9
135	62.7	1-3/16	102	127	152	1-3/16	202	127	152	864	–
171	66.7	1-3/16	102	127	152	1-3/16	202	127	152	864	–
175	67.5	1-3/16	102	127	152	1-3/16	202	127	152	864	–

* Special rail drilling and joint bar punching.

Rail Type	Bolt				Spring Washer		Mass of Ass'y	
	diam.	Grip	L	H	Hole dia.	Thk. & width	With Flg.	Without Flg.
	in.	mm	mm	mm	in.	in. in.	kg.	kg.
40	3/4	49.2	88.9	63.5	13/16	7/16 x 3/8	9.07	7.48
60	3/4	65.9	102	68.3	13/16	7/16 x 3/8	16.56	13.43
85	7/8	80.2	121	81.0	15/16	7/16 x 3/8	25.67	20.55
104	1	88.9	133	88.9	1-1/16	7/16 x 1/2	33.34	25.13
135	1-1/8	92.1	140	93.7	1-3/16	7/16 x 1/2	–	34.16
171	1-1/8	113	159	103	1-3/16	7/16 x 1/2	–	41.19
175	1-1/8	105	152	100	1-3/16	7/16 x 1/2	–	39.78

Splices

Rail drilling and joint bar punching as supplied for track work is not recommended for crane rails, since oversize holes may allow too much movement at the rail ends and result in failure. Tight joints which require special rail and joint bar drilling (see table on previous page) and squaring of the rail ends are recommended.

Light rails are not finished at the mill and are usually finished at the fabricator's shop or at the erection site. This may require reaming of holes for proper fit of bolts if dimensional tolerances are cumulative.

Joint bars are provided for crane service to match the rails ordered and may be ordered blank. Under no circumstances should these joint bars be used as welding straps. Manufacturer's catalogs should be consulted for joint bar specifications, dimensions and identification necessary to match the crane rail specified.

Joint bar bolts for crane service are readily identified from those used for track work, as they have straight shanks and are manufactured to ASTM A449 specification. Matching nuts are manufactured to ASTM A563 Grade B. The bolted assembly includes an alloy spring washer which is furnished to American Railway Engineering and Maintenance of Way Association (AREMA) specifications. Bolts and nuts manufactured to ASTM A325 may also be acceptable.

To prolong the life of the runway, bolts should be retightened within 30 days after installation and every 3 months thereafter.

FASTENERS

General

The information on fasteners provided herein is based on standards, specifications and publications of the:

Canadian Standards Association (CSA Group)

American National Standards Institute (ANSI)

American Society of Mechanical Engineers (ASME)

Industrial Fasteners Institute (IFI)

Research Council on Structural Connections (RCSC)

Additional fastener information can be obtained from the various manufacturers and from the Canadian Fasteners Institute (CFI).

Availability

The more commonly used fasteners for structural purposes in Canada have included the following:

⅝-inch ASTM A307 bolts for light steel framing such as girts, purlins, etc.

¾-inch ASTM A325 bolts for building structures

⅞-inch ASTM A325 bolts for bridge structures

While other diameters and types of bolts have been used on specific projects in Canada, larger sizes of ASTM A325 bolts, all sizes of ASTM A490 bolts, and all sizes of metric bolts (A325M and A490M) have not been in common use in Canada, and designers contemplating their use should first check for their availability.

Definitions

Body Length: Distance from the underside of the head bearing surface to either the last scratch of thread or the top of the extrusion angle, whichever is the closest to the head.

Bolt Length: Length from the underside of the head bearing surface to the extreme point.

Finished Fastener: Fastener made to close tolerances and having surfaces other than the threads and bearing surface finished to provide a general high-grade appearance.

Grip: Total thickness of the plies of a joint through which the bolt passes, exclusive of washers or direct-tension indicators.

Height of Bolt Head: Overall distance, measured parallel to the fastener axis, from the extreme top (excluding raised identification marks) to the bearing surface and including the thickness of the washer face where provided.

Natural Finish: As-processed finish, unplated or uncoated, of the bolt or nut.

Nominal Size: Designation used for the purpose of general identification.

Proof Load: Specified test load which a fastener must withstand without any indication of significant deformation or failure.

Thickness of Nut: Overall distance from the top of the nut to the bearing surface, measured parallel to the axis of the nut.

Thread Length of a Bolt: Distance from the extreme point to the last complete thread.

Transition Thread Length: Distance from the last complete thread to either the last scratch of thread or the top of the extrusion angle, whichever is the closest to the head.

Washer Face: Circular boss on the bearing surface of a bolt or nut.

Tables

The following tables are included in this section:

- Markings – ASTM High-Strength Bolts, Nuts and Assemblies
- High-Strength Bolts, Nuts and Assemblies – Dimensions
- High-Strength Bolts, Nuts and Assemblies – Acceptable ASTM A563 Nut Grade and Finish, and ASTM F436 Washer Type and Finish
- Bolt Lengths for Various Grips – ASTM A325 and A490 Bolts
- Weight of ASTM A325 Bolts, Nuts and Washers
- ASTM F436 Washer Dimensions
- ASTM A307 Hex Bolts and Heavy Hex Nuts – Dimensions
- High-Strength Bolts – Purchase Order Information
- Fasteners – Miscellaneous Detailing Data (Diagonal Distance for Staggered Fasteners, Bolt Length Tolerances, and Minimum Edge Distance for Bolt Holes)
- Usual Gauges – W, M, S, C shapes, and Angles
- Installation Clearances

Metric Fasteners

Archival material on metric-size bolts found in previous editions of the Handbook is provided in *Metric Fastener Data* at the end of this section.

Anchor Rods

See *Anchor Rods* in Part 4.

MARKINGS – ASTM HIGH-STRENGTH BOLTS, NUTS AND ASSEMBLIES[1]

Bolt Head[2]		
Designation / Grade	**Type 1**	**Type 3**
A325 Bolt[3]	 Three radial lines 120° apart are optional.	
F1852 Bolt Assembly[4]	 Three radial lines 120° apart are optional.	
A490 Bolt		
F2280 Bolt Assembly[4]		

Nut[2]			
A563 Nut	 Arcs indicate Grade C	 Arcs with "3" indicate Grade C3	 Grade D
	 Grade DH		 Grade DH3

Notes:

1. Adapted from the Specification for Structural Joints Using High-Strength Bolts, Research Council on Structural Connections (RCSC), 2014.
2. XYZ represents the manufacturer's indentification mark.
3. For A325 bolts threaded full length and their bolt head markings, see next page.
4. For F1852 and F2280 twist-off-type tension-control bolt assemblies, the letters "TC" are optional, in accordance with ASTM Standard F3125. These assemblies are also produced with a heavy-hex head that has similar markings.

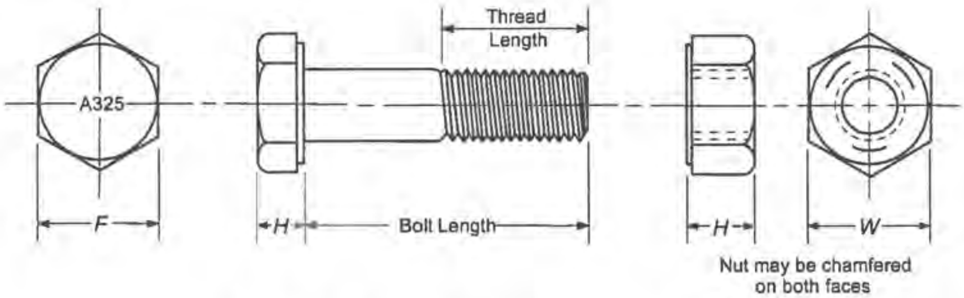
HIGH-STRENGTH BOLTS, NUTS AND ASSEMBLIES

Dimensions

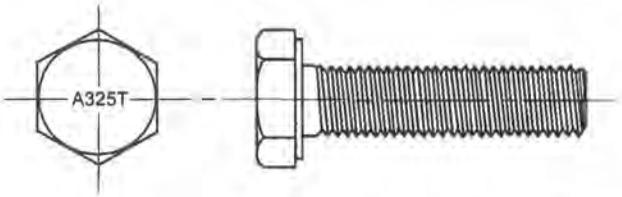
Imperial Dimensions					Nominal Bolt Size	Metric Dimensions (Soft)				
Bolt Dimensions* Heavy Hex Structural Bolts in.			Nut Dimensions* Heavy Hex Nuts in.			Bolt Dimensions* Heavy Hex Structural Bolts mm			Nut Dimensions* Heavy Hex Nuts mm	
Width across flats F	Height H	Thread length†	Width across flats W	Height H	D in.	Width across flats F	Height H	Thread length†	Width across flats W	Height H
7/8	5/16	1	7/8	31/64	1/2	22.2	7.9	25.4	22.2	12.3
1 1/16	25/64	1 1/4	1 1/16	39/64	5/8	27.0	9.9	31.8	27.0	15.5
1 1/4	15/32	1 3/8	1 1/4	47/64	3/4	31.8	11.9	34.9	31.8	18.7
1 7/16	35/64	1 1/2	1 7/16	55/64	7/8	36.5	13.9	38.1	36.5	21.8
1 5/8	39/64	1 3/4	1 5/8	63/64	1	41.3	15.5	44.5	41.3	25.0
1 13/16	11/16	2	1 13/16	1 7/64	1 1/8	46.0	17.5	50.8	46.0	28.2
2	25/32	2	2	1 7/32	1 1/4	50.8	19.8	50.8	50.8	31.0
2 3/16	27/32	2 1/4	2 3/16	1 11/32	1 3/8	55.6	21.4	57.2	55.6	34.1
2 3/8	15/16	2 1/4	2 3/8	1 15/32	1 1/2	60.3	23.8	57.2	60.3	37.3

* Dimensions according to ASME B18.2.6.

† Certain A325 bolts may be ordered threaded full length. See notes and figure below.



A325 Bolt with Standard Thread Length



A325 Bolt Threaded Full Length

Note: A325 bolts threaded full length are permitted under Supplementary Requirement S1 of ASTM A325. They are restricted to bolts with nominal lengths no greater than four times the nominal diameter.

Acceptable ASTM A563 Nut Grade and Finish and ASTM F436 Washer Type and Finish

ASTM Desig.	Bolt Type	Bolt Finish [d]	ASTM A563 Nut Grade and Finish [d]	ASTM F436 Washer Type and Finish [a, d]
A325	1	Plain (uncoated)	C, C3, D, DH [c] and DH3; plain	1; plain
		Galvanized	DH [c]; galvanized and lubricated	1; galvanized
		Zn/Al Inorganic, per ASTM F1136 Grade 3	DH [c]; Zn/Al Inorganic, per ASTM F1136 Grade 5	1; Zn/Al Inorganic, per ASTM F1136 Grade 3
	3	Plain	C3 and DH3; plain	3; plain
F1852	1	Plain (uncoated)	C, C3, DH [c] and DH3; plain	1; plain [b]
		Mechanically Galvanized	DH [c]; mechanically galvanized and lubricated	1; mechanically galvanized [b]
	3	Plain	C3 and DH3; plain	3; plain [b]
A490	1	Plain	DH [c] and DH3; plain	1; plain
		Zn/Al Inorganic, per ASTM F1136 Grade 3	DH [c]; Zn/Al Inorganic, per ASTM F1136 Grade 5	1; Zn/Al Inorganic, per ASTM F1136 Grade 3
	3	Plain	DH3; plain	3; plain
F2280	1	Plain	DH [c] and DH3; plain	1; plain [b]
	3	Plain	DH3; plain	3; plain [b]

[a] Applicable only if washer is required.

[b] Required in all cases under nut.

[c] The substitution of ASTM A194 grade 2H nuts in place of ASTM A563 grade DH nuts is permitted.

[d] "Galvanized" as used in this table refers to hot-dip galvanizing in accordance with ASTM F2329 or mechanical galvanizing in accordance with ASTM B695.

[e] "Zn/Al Inorganic" as used in this table refers to application of a Zn/Al Corrosion Protective Coating in accordance with ASTM F1136 which has met all the requirements of IFI-144.

Source: Specification for Structural Joints Using High-Strength Bolts, Research Council on Structural Connections (RCSC), 2014.

BOLT LENGTHS* FOR VARIOUS GRIPS**
ASTM A325 AND A490 BOLTS

Grip mm	Grip in.	Bolt diameter, in. ½	⅝	¾	⅞	1	1⅛	1¼	1⅜	1½
19	¾	1½								
21	13/16		1¾		2			2½		2¾
22	⅞			2		2¼	2½		2¾	
24	15/16	1¾								
25	1		2		2¼			2¾		3
27	1 1/16									
29	1⅛			2¼		2½	2¾		3	
30	1 3/16	2								
32	1¼		2¼		2½			3		3¼
33	1 5/16									
35	1⅜			2½		2¾	3		3¼	
37	1 7/16	2¼								
38	1½		2½		2¾			3¼		3½
40	1 9/16									
41	1⅝			2¾		3	3¼		3½	
43	1 11/16	2½								
44	1¾		2¾		3			3½		3¾
46	1 13/16									
48	1⅞			3		3¼	3½		3¾	
49	1 15/16	2¾								
51	2		3		3¼			3¾		4
52	2 1/16									
54	2⅛			3¼		3½	3¾		4	
56	2 3/16	3								
57	2¼		3¼		3½			4		4¼
59	2 5/16									
60	2⅜			3½		3¾	4		4¼	
62	2 7/16	3¼								
64	2½		3½		3¾			4¼		4½
65	2 9/16									
67	2⅝			3¾		4	4¼		4½	
68	2 11/16	3½								
70	2¾		3¾		4			4½		4¾
71	2 13/16									
73	2⅞			4		4¼	4½		4¾	
75	2 15/16	3¾								
76	3		4		4¼			4¾		5
78	3 1/16									

Grip mm	Grip in.	Bolt diameter, in. ½	⅝	¾	⅞	1	1⅛	1¼	1⅜	1½
76	3									
78	3 1/16		4		4¼			4¾		5
79	3⅛			4¼		4½	4¾		5	
81	3 3/16	4								
83	3¼		4¼		4½			5		5¼
84	3 5/16									
86	3⅜			4½		4¾	5		5¼	
87	3 7/16	4¼								
89	3½		4½		4¾			5¼		5½
90	3 9/16									
92	3⅝			4¾		5	5¼		5½	
94	3 11/16	4½								
95	3¾		4¾		5			5½		5¾
97	3 13/16									
98	3⅞			5		5¼	5½		5¾	
100	3 15/16	4¾								
102	4		5		5¼			5¾		6
103	4 1/16									
105	4⅛			5¼		5½	5¾		6	
106	4 3/16	5								
108	4¼		5¼		5½			6		6¼
110	4 5/16									
111	4⅜			5½		5¾	6		6¼	
113	4 7/16	5¼								
114	4½		5½		5¾			6¼		6½
116	4 9/16									
117	4⅝			5¾		6	6¼		6½	
119	4 11/16	5½								
121	4¾		5¾		6			6½		6¾
122	4 13/16									
124	4⅞			6		6¼	6½		6¾	
125	4 15/16	5¾								
127	5		6		6¼			6¾		7
129	5 1/16									
130	5⅛			6¼		6½	6¾		7	
132	5 3/16	6								
133	5¼		6¼		6½			7		7½
135	5 5/16									

* Bolt lengths must be specified in inches for ASTM A325 and A490 bolts.

** Grip is thickness of material to be connected exclusive of washers.

For each flat washer, add 4 mm (5/32 inch) to grip.

For each beveled washer, add 8 mm (5/16 inch) to grip.

For information on A325 bolts threaded full length, see *High-Strength Bolts, Nuts and Assemblies* .

WEIGHT OF ASTM A325 BOLTS, NUTS AND WASHERS

WEIGHT IN POUNDS PER 100 UNITS

HEAVY HEX STRUCTURAL BOLTS WITH HEAVY HEX NUTS (WITHOUT WASHERS)									
Length Under Head, Inches	Bolt Diameter, Inches								
	½	⅝	¾	⅞	1	1⅛	1¼	1⅜	1½
1	16.5	29.4	47.0						
1¼	17.8	31.1	49.6	74.4	104				
1½	19.2	33.1	52.2	78.0	109	148	197		
1¾	20.5	35.3	55.3	81.9	114	154	205	261	333
2	21.9	37.4	58.4	86.1	119	160	212	270	344
2¼	23.3	39.8	61.6	90.3	124	167	220	279	355
2½	24.7	41.7	64.7	94.6	130	174	229	290	366
2¾	26.1	43.9	67.8	98.8	135	181	237	300	379
3	27.4	46.1	70.9	103	141	188	246	310	391
3¼	28.8	48.2	74.0	107	146	195	255	321	403
3½	30.2	50.4	77.1	111	151	202	263	332	416
3¾	31.6	52.5	80.2	116	157	209	272	342	428
4	33.0	54.7	83.3	120	162	216	280	353	441
4¼	34.3	56.9	86.4	124	168	223	289	363	453
4½	35.7	59.0	89.5	128	173	230	298	374	465
4¾	37.1	61.2	92.7	133	179	237	306	384	478
5	38.5	63.3	95.8	137	184	244	315	395	490
5¼	39.9	65.5	98.9	141	190	251	324	405	503
5½	41.2	67.7	102	146	196	258	332	416	515
5¾	42.6	69.8	105	150	201	265	341	426	527
6	44.0	71.9	108	154	207	272	349	437	540
6¼		74.1	111	158	212	279	358	447	552
6½		76.3	114	163	218	286	367	458	565
6¾		78.5	118	167	223	293	375	468	577
7		80.6	121	171	229	300	384	479	589
7¼		82.8	124	175	234	307	392	489	602
7½		84.9	127	179	240	314	401	500	614
7¾		87.1	130	183	246	321	410	510	626
8		89.2	133	187	251	328	418	521	639
8¼				192	257	335	427	531	651
8½				196	262	342	435	542	664
8¾							444	552	676
9							453	563	689
Per inch additional	5.5	8.6	12.4	16.9	22.1	28.0	34.4	42.5	49.7

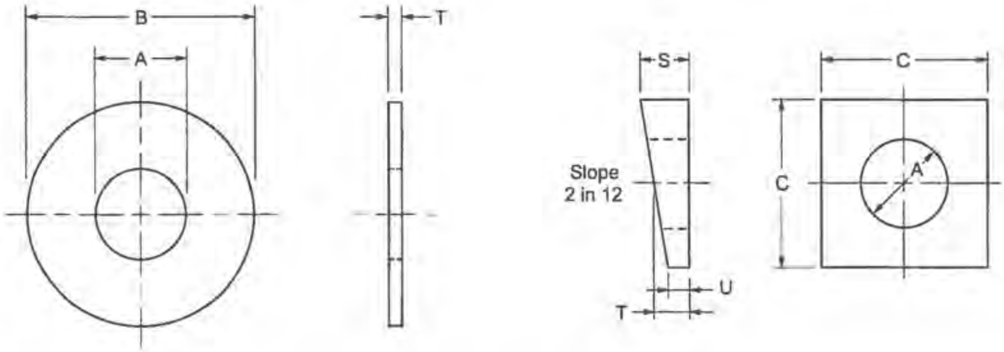
Plain round washers	2.1	3.6	4.8	7.0	9.4	11.3	13.8	16.8	20.0
Beveled square washers	23.1	22.4	21.0	20.2	19.2	34.0	31.6	31.2	32.9

ASTM F436 WASHER DIMENSIONS

PLAIN CIRCULAR WASHERS

Bolt Size	B		A		T	
	Outside Diameter mm		Hole Diameter mm		Thickness mm	
in.	Max	Min	Max	Min	Max	Min
½	27.8	26.2	14.3	13.5	4.5	2.5
⅝	34.2	32.5	18.3	17.5	4.5	3.1
¾	38.1	36.5	21.5	20.7	4.5	3.1
⅞	45.3	43.6	24.6	23.8	4.5	3.5
1	52.4	49.2	28.6	27.0	4.5	3.5
1⅛	58.8	55.5	31.8	30.2	4.5	3.5
1¼	65.1	61.9	36.5	34.9	4.5	3.5
1⅜	71.5	68.2	39.7	38.1	4.5	3.5
1½	77.8	74.6	42.9	41.3	4.5	3.5

Note: Minimum thickness 7.7 mm and maximum thickness 9.5 mm for extra thick washers
Metric dimensions have been soft-converted. For official dimensions, refer to ASTM F436.

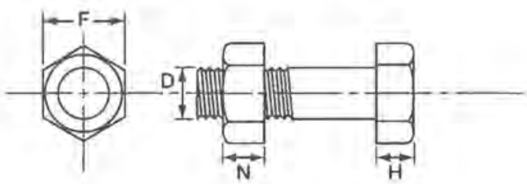


BEVELLED SQUARE WASHERS

Bolt Size	C		A		S	T	U
	Width mm		Hole Diameter mm		Thickness, mm		
in.	Max	Min	Max	Min	Thick Side	Mean Nom.	Thin Side
½	45.3	43.6	14.3	13.5	11.6	7.9	4.2
⅝	45.3	43.6	18.3	17.5	11.6	7.9	4.2
¾	45.3	43.6	21.5	20.6	11.6	7.9	4.2
⅞	45.3	43.6	24.6	23.8	11.6	7.9	4.2
1	46.1	42.8	30.2	28.6	11.6	7.9	4.2
1⅛	58.8	55.5	33.4	31.8	12.7	7.9	3.2
1¼	58.8	55.5	36.5	34.9	12.7	7.9	3.2
1⅜	58.8	55.5	39.7	38.1	12.7	7.9	3.2
1½	58.8	55.5	42.9	41.3	12.7	7.9	3.2

Note: Metric dimensions have been soft-converted. For official dimensions, refer to ASTM F436.

ASTM A307 HEX BOLTS AND HEAVY HEX NUTS



DIMENSIONS

Imperial Dimensions						Nominal Bolt Size	Metric Dimensions (Soft-Converted)					
Bolt Dimensions Hex Structural Bolts in.				Nut Dimensions Heavy Hex Nuts in.			Bolt Dimensions Hex Structural Bolts mm				Nut Dimensions Heavy Hex Nuts mm	
Width across flats	Height	Minimum Thread Length		Width across flats	Height	D	Width across flats	Height	Minimum Thread Length		Width across flats	Height
F	H	L ≤ 6 in.	L > 6 in.	F	N	in.	F	H	L ≤ 152	L > 152	F	N
¾	11/32	1 ¼	1 ½	⅞	31/64	½	19	9	32	38	22	12
15/16	27/64	1 ½	1 ¾	1 1/16	39/64	⅝	24	11	38	44	27	15
1 ⅛	½	1 ¾	2	1 ¼	47/64	¾	29	13	44	51	32	19
1 5/16	37/64	2	2 ¼	1 7/16	55/64	⅞	33	15	51	57	37	22
1 ½	43/64	2 ¼	2 ½	1 ⅝	63/64	1	38	17	57	64	41	25
1 11/16	¾	2 ½	2 ¾	1 13/16	1 7/64	1 ⅛	43	19	64	70	46	28
1 ⅞	27/32	2 ¾	3	2	1 7/32	1 ¼	48	21	70	76	51	31
2 1/16	29/32	3	3 ¼	2 3/16	1 11/32	1 ⅜	52	23	76	83	56	34
2 ¼	1	3 ¼	3 ½	2 ⅜	1 15/32	1 ½	57	25	83	89	60	37

Note: ASTM A307 bolts shall be Grade A hex bolts with heavy hex nuts as per ASTM A563, according to S16-14 Clause 13.12.1.2

Imperial dimensions for Hex Structural Bolts and Heavy Hex Nuts conform to ASME B18.2.1 and B18.2.2, respectively. Metric dimensions in millimetres have been soft-converted and rounded to the nearest millimetre.

The minimum thread lengths are in agreement with the requirements of ASME B18.2.1 In general, these requirements are as follows:

- Bolts 6 inches or less in length – twice diameter plus ¼-inch.
- Bolts longer than 6 inches – twice diameter plus ½-inch.
- Bolts too short for the above thread lengths shall be threaded as close to the head as practicable.

Note: A307 bolts and nuts are manufactured in imperial units only.

HIGH-STRENGTH BOLTS –
PURCHASE ORDER INFORMATION

ASTM F3125, a consolidation and replacement of six standards (A325, A325M, A490, A490M, F1852, and F2280) was published in January 2015. In this "umbrella" standard, the name of each bolt standard becomes a bolt grade (e.g. A490 becomes F3125 Grade A490). The traditional bolt type designations remain, i.e. Type 3 for weathering steel and Type 1 for bolts of other high-strength steel compositions. There are two bolt styles: F1852 and F2280 are referred to as Twist-off Style bolts, while the others are Heavy Hex Style bolts. All bolts manufactured after the publication date of F3125 must comply with the requirements of F3125. The bolt head markings, however, remain essentially unchanged, as shown in the table entitled *Markings – ASTM High-Strength Bolts, Nuts and Assemblies* above.

The design of bolted connections must comply with CSA S16-14, which specifies the bolt strength and resistances, and references the ASTM bolt standards prior to the consolidation. New purchase orders, however, may be placed in accordance with the ordering requirements in ASTM F3125 as summarized below:

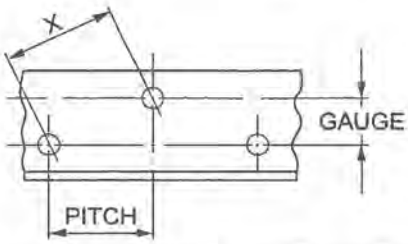
- ASTM designation
- Quantity: Number of bolts or assemblies, including washers, if required
- Size: Including nominal bolt diameter and bolt length, and thread pitch if other than standard
- Grade: A325, A325M, A490, A490M, F1852 or F2280
- Type: Type 1 or Type 3. When the Type is not specified, either Type 1 or Type 3 may be furnished at the supplier's option
- Style: Heavy Hex or Twist-Off Style

Additional ordering information may include, if required: coatings or finishes, test reports, details of other assembly components such as nuts and washers, rotational capacity testing, special observations or inspection requirements, and country of origin requirements. Heavy hex bolts may be ordered individually, packaged with nuts, packaged with nuts and washers, or as assemblies. See ASTM F3125 for further information.

A typical description: 1000 pieces ¾" × 3" ASTM F3125–15, Grade A325 heavy hex bolt, Type 1, each with one hardened ASTM F436 Type 1 washer and one A563 Grade DH heavy hex nut

FASTENERS - MISCELLANEOUS DETAILING DATA

Diagonal Distance for Staggered Fasteners



Pitch mm	Gauge, mm																	
	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100	105	110
5	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100	105	110
10	27	32	36	41	46	51	56	61	66	71	76	81	86	91	96	100	105	110
15	29	34	38	43	47	52	57	62	67	72	76	81	86	91	96	101	106	111
20	32	36	40	45	49	54	59	63	68	73	78	82	87	92	97	102	107	112
25	35	39	43	47	51	56	60	65	70	74	79	84	89	93	98	103	108	113
30	39	42	46	50	54	58	63	67	72	76	81	85	90	95	100	104	109	114
35	43	46	49	53	57	61	65	69	74	78	83	87	92	97	101	106	111	115
40	47	50	53	57	60	64	68	72	76	81	85	89	94	98	103	108	112	117
45	51	54	57	60	64	67	71	75	79	83	87	92	96	101	105	110	114	119
50	56	58	61	64	67	71	74	78	82	86	90	94	99	103	107	112	116	121
55	60	63	65	68	71	74	78	81	85	89	93	97	101	105	110	114	119	123
60	65	67	69	72	75	78	81	85	88	92	96	100	104	108	112	117	121	125
65	70	72	74	76	79	82	85	88	92	96	99	103	107	111	115	119	123	128
70	74	76	78	81	83	86	89	92	96	99	103	106	110	114	118	122	126	130
75	79	81	83	85	87	90	93	96	99	103	106	110	113	117	121	125	129	133
80	84	85	87	89	92	94	97	100	103	106	110	113	117	120	124	128	132	136
85	89	90	92	94	96	99	101	104	107	110	113	117	120	124	127	131	135	139
90	93	95	97	98	101	103	105	108	111	114	117	120	124	127	131	135	138	142

BOLT LENGTH TOLERANCES

Nominal Length mm	Nominal size, in.					
	⅝	¾	⅞	1	1⅛	1¼
Up to 25	+0.5	+0.5	---	---	---	---
	-0.8	-0.8	---	---	---	---
Over 25 to 64	+1.5	+1.5	+2.0	+2.0	+3.0	+3.0
	-2.0	-2.0	-2.5	-2.5	-3.0	-3.0
Over 64 to 102	+2.0	+2.0	+2.5	+2.5	+4.1	+4.1
	-2.5	-2.5	-3.6	-3.6	-4.1	-4.1
Over 102 to 152	+2.5	+2.5	+3.0	+3.0	+4.6	+4.6
	-2.5	-2.5	-4.1	-4.1	-4.6	-4.6
Over 152	+3.6	+3.6	+4.1	+4.1	+5.6	+5.6
	-4.6	-4.6	-5.1	-5.1	-5.6	-5.6

Note: Metric dimensions have been soft-converted.
Refer to ASME B18.2.1 for further information.

MINIMUM EDGE DISTANCE FOR BOLT HOLES

Bolt diameter in.	At sheared edge mm	At rolled or sawn edges, or edges cut by gas*, plasma, laser or water jet, mm
⅝	28	22
¾	32	25
⅞	38†	28
1	44†	32
1⅛	51	38
1¼	57	41
Over 1¼	1.75 × diameter	1.25 × diameter

* Gas-cut edges shall be smooth and free from notches. The edge distance in this column may be decreased by 3 mm when the hole is at a point where the calculated stress under factored loads is not more than 0.3 of the yield stress.

† At the ends of beam-framing angles, this distance may be 32 mm.

USUAL GAUGES

Usual Gauges for W, M, S and C Shapes, Millimetres

S SHAPES

Flange Width	g†
60 to 70	40 ø
75 to 90	45 ø
92 to 100	50 ø
102 to 120	60
130 to 145	80
150 to 185	100
190 to 200	130

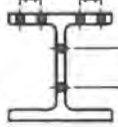
STANDARD CHANNELS

Flange Width	g†
35 to 40	22 ø
45	25 ø
47 to 50	30 ø
55	32
57 to 60	35
64 to 70	38
73 to 80	45
85 to 92	60
95 to 110	65

W AND M SHAPES

Flange Width	g†
100	60
130 to 145	80
150 to 180	100
190 and up	130

Extra gauges for W columns: 50, 70, 85 and 60, 80, 90

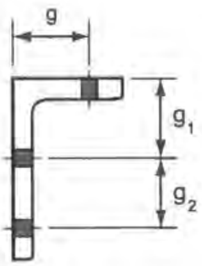


Depth of W or M column	g
150	60
200	100
250 to 350	130

ø Holes usually drilled due to size of punch die block

† Some of the gauge and flange width combinations may not meet edge distance requirements in S16-14 Table 6.

Usual Gauges for Angles, Millimetres



Notes:

$g_2 \geq 2.7$ bolt diameters

(See CSA S16-14 Clause 22.3.1)

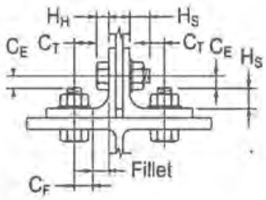
Leg	Gauge	
	g	g_1
203	115	75
178	100	65
152	90	60
127	75	50
102	65	
89	50	
76	45	
64	35	
51	29	
44	25	

Note: Bolt gauges shown do not necessarily comply with S16 installation clearances. Clearance and edge distance limitations should be verified for the selected bolt size.

INSTALLATION CLEARANCES, MILLIMETRES

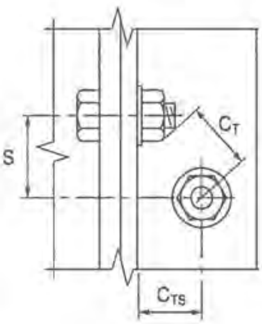
STRUCTURAL ASTM A325 and A490 Bolts

Aligned Bolts



D	B	H_H	H_S	C_T	C_E	C_F Circular	C_F Clipped
⅝	44.5	9.9	31.8	25.4	17.5	17.5	14.3
¾	57.2	11.9	34.9	31.8	19.1	19.1	17.5
⅞	63.5	13.9	38.1	34.9	22.2	22.2	20.6
1	66.7	15.5	41.3	36.5	23.8	25.4	22.2
1⅛	73.0	17.5	47.6	39.7	27.0	28.6	25.4
1¼	79.4	19.8	50.8	42.9	28.6	31.8	28.6
1⅜	82.6	21.4	54.0	44.5	31.8	34.9	31.8
1½	88.9	23.8	57.2	47.6	33.3	38.1	33.3

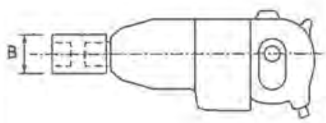
Staggered Bolts



C_{TS}	Stagger, S — Nominal Bolt Diameter, D: ⅝	¾	⅞	1	1⅛	1¼	1⅜	1½
26	41							
28	41							
30	40							
32	39	50						
34	38	49						
36	36	48	56					
38	34	47	55	60				
40	33	46	54	60	67			
42	32	45	53	59	66			
44	31	43	52	58	66	72		
46	30	41	51	57	65	72	77	
48	28	39	49	56	64	71	77	83
50	27	38	48	55	63	71	76	83
52	24	37	46	54	62	70	75	82
54	21	36	43	52	61	69	75	82
56	16	34	42	50	60	68	74	81
58		32	41	48	58	67	73	81
60		30	39	45	56	65	72	80
62		27	38	44	54	64	71	79
64		23	36	42	52	62	70	78
66		17	33	41	50	60	68	77
68			30	39	49	58	66	76
70			26	36	48	56	65	74
72			21	33	47	54	62	73
74				30	45	53	60	71
76				25	43	52	57	69
78					41	50	56	67
80					38	48	55	64
82					35	46	53	61
84					31	44	51	59
86					25	41	49	58
88						38	47	56
90						34	45	55

D = Nominal Bolt Diameter
B = Socket Diameter
H_H = Height of Head
H_S = Maximum Shank Extension*
C_T = Clearance for Tightening
C_E = Clearance for Entering
C_F = Clearance for Fillet*
S = Bolt Stagger
C_{TS} = Clearance for Tightening Staggered Bolts

* Based on the use of one ASTM F436 washer



METRIC FASTENER DATA

General

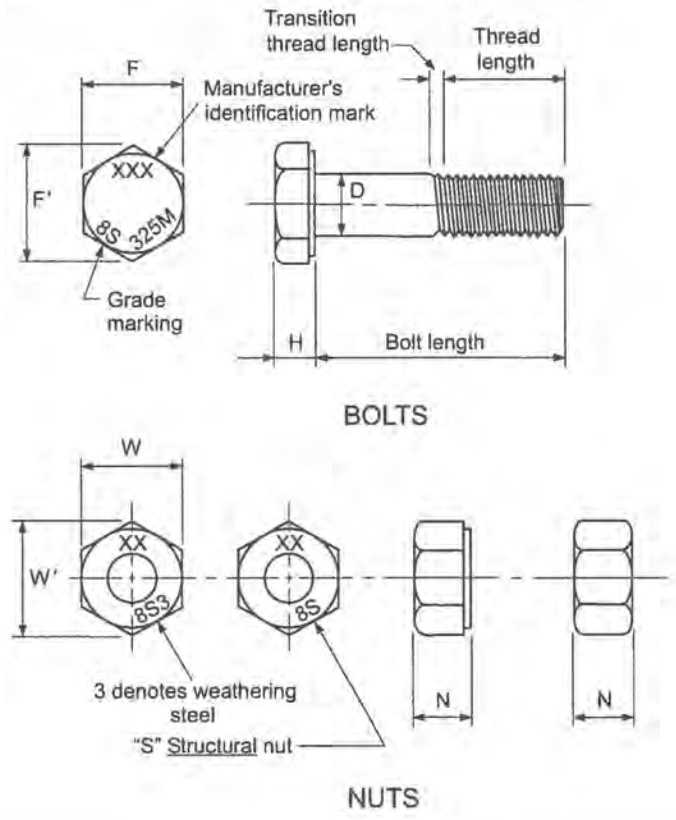
The tables on the following pages contain design data on metric-size high-strength bolts (ASTM A325M and A490M) and accessories (ASTM F436M washers) found in the 10th edition of the Handbook. This material is reprinted herein without revision for historical reference. Metric bolt sizes have not been in common use in Canada, and designers considering their use should first check for their availability.

Tables

The following tables are included in this section:

- ASTM A325M and ASTM A490M – High-Strength Bolts and Nuts
- Minimum and Maximum Grips for Metric Heavy Hex Structural Bolts
- Mass of ASTM A325M Bolts, Nuts and Washers
- ASTM F436M Metric Washer Dimensions
- Fasteners – Miscellaneous Detailing Data:
  - Thread Data, Designations, and Slotted Hole Dimensions
  - Bolt Length Tolerances, Minimum Edge Distance for Bolt Holes, and Usual Gauges
- Erection Clearances – Bolt Impact Wrenches

ASTM A325M AND ASTM A490M**
HIGH-STRENGTH BOLTS AND NUTS



DIMENSIONS

Nominal Bolt Size	Heavy Hex Bolt or Nut Dimension				Heavy Hex Nut Max. Height	Heavy Hex Structural Bolt			
	Across Flats F or W		Across Corners F' or W'			Max Head Height	Thread Length*		Max. Transition Thread Length
	Max.	Min.	Max.	Min.	N	H	Bolt Lengths ≤100	Bolt Lengths >100	
mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
M16 x 2	27.00	26.16	31.18	29.56	17.1	10.75	31	38	6.0
M20 x 2.5	34.00	33.00	39.26	37.29	20.7	13.40	36	43	7.5
M22 x 2.5	36.00	35.00	41.57	39.55	23.6	14.90	38	45	7.5
M24 x 3	41.00	40.00	47.34	45.20	24.2	15.90	41	48	9.0
M27 x 3	46.00	45.00	53.12	50.85	27.6	17.90	44	51	9.0
M30 x 3.5	50.00	49.00	57.74	55.37	30.7	19.75	49	56	10.5
M36 x 4	60.00	58.80	69.28	66.44	36.6	23.55	56	63	12.0

* Does not include transition thread length.
** Strength requirements are based on ASTM Specifications A325M and A490M. See page 3—5.
Bolt dimensions conform to those listed in ANSI B18.2.3.7M-1979 (R2001) "Metric Heavy Hex Structural Bolts", and the nut dimensions conform to those listed in ANSI B18.2.4.6M-1979 (R1998) "Metric Heavy Hex Nuts".

MINIMUM AND MAXIMUM GRIPS FOR METRIC HEAVY HEX. STRUCTURAL BOLTS, IN MILLIMETRES

Nominal Bolt Size	M16		M20		M22		M24		M27		M30		M36	
L Nominal Length (mm)	Min. Grip	Max. Grip	Min. Grip	Max. Grip	Min. Grip	Max. Grip	Min. Grip	Max. Grip	Min. Grip	Max. Grip	Min. Grip	Max. Grip	Min. Grip	Max. Grip
45	14	26		23		20								
50	19	31	14	28		25		24						
55	24	36	19	32	17	29		29		25				
60	29	41	24	37	22	34	19	34		30		27		
65	34	46	29	42	27	39	24	39	21	35		32		
70	39	51	34	47	32	44	29	44	26	40	21	37		31
75	44	56	39	52	37	49	34	49	31	45	26	42		36
80	49	61	44	57	42	54	39	54	36	50	31	47	24	41
85	54	66	49	62	47	59	44	59	41	55	36	52	29	46
90	59	71	54	67	52	64	49	64	46	60	41	57	34	51
95	64	76	59	72	57	69	54	69	51	65	46	62	39	56
100	69	81	64	77	62	74	59	74	56	70	51	67	44	61
110	72	91	67	87	65	84	62	84	59	80	54	77	47	71
120	82	101	77	97	75	94	72	94	69	90	64	87	57	81
130	92	110	87	107	85	104	82	103	79	100	74	97	67	91
140	102	120	97	117	95	114	92	113	89	110	84	107	77	101
150	112	130	107	127	105	124	102	123	99	120	94	117	87	111
160	122	138	117	135	115	132	112	131	109	128	104	125	97	119
170	132	148	127	145	125	142	122	141	119	138	114	135	107	129
180	142	158	137	155	135	152	132	151	129	148	124	145	117	139
190	152	168	147	165	145	162	142	161	139	158	134	155	127	149
200	162	178	157	175	155	172	152	171	149	168	144	165	137	159
210	172	188	167	185	165	182	162	181	159	178	154	175	147	169
220	182	198	177	195	175	192	172	191	169	188	164	185	157	179
230	192	208	187	205	185	202	182	201	179	198	174	195	167	189
240	202	218	197	215	195	212	192	211	189	208	184	205	177	199
250	212	228	207	225	205	222	202	221	199	218	194	215	187	209
260	222	238	217	235	215	232	212	231	209	228	204	225	197	219
270	232	248	227	245	225	242	222	241	219	238	214	235	207	229
280	242	258	237	255	235	252	232	251	229	248	224	245	217	239
290	252	268	247	265	245	262	242	261	239	258	234	255	227	249
300	262	278	257	275	255	272	252	271	249	268	244	265	237	259

1. This table is based on ANSI B18.2.3.7M-1979 (R2006).
2. Bolts with lengths above the heavy solid line are threaded full length.

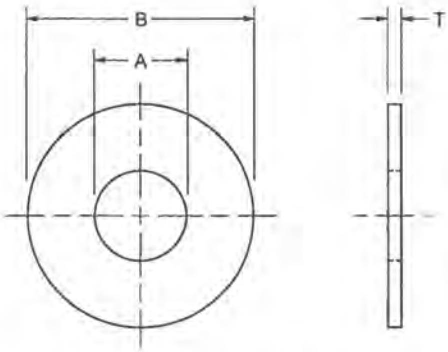
MASS OF ASTM A325M BOLTS, NUTS AND WASHERS

MASS IN KILOGRAMS PER 100 UNITS

HEAVY HEX STRUCTURAL BOLTS WITH HEAVY HEX NUTS (WITHOUT WASHERS)							
Length Under Head, mm	Bolt Diameter, mm						
	M16	M20	M22	M24	M27	M30	M36
45	16.3						
50	17.1	30.4					
55	17.8	31.6	39.2				
60	18.6	32.9	40.7	53.7			
65	19.4	34.1	42.2	55.4	76.8		
70	20.2	35.3	43.7	57.2	79.0	98.0	
75	21.0	36.6	45.2	59.0	81.3	101	
80	21.8	37.8	46.7	60.7	83.5	104	167
85	22.6	39.0	48.1	62.5	85.8	106	171
90	23.4	40.3	49.6	64.3	88.0	109	175
95	24.1	41.5	51.1	66.1	90.2	112	179
100	24.9	42.7	52.6	67.8	92.5	114	183
110	26.3	44.9	55.3	71.0	96.7	120	191
120	27.9	47.4	58.2	74.5	101	125	199
130	29.5	49.8	61.2	78.0	106	131	207
140	31.1	52.3	64.2	81.6	110	136	214
150	32.6	54.7	67.2	85.1	115	142	222
160	34.2	57.2	70.2	88.7	119	147	230
170	35.8	59.7	73.1	92.2	124	153	238
180	37.3	62.1	76.1	95.8	128	158	246
190	38.9	64.6	79.1	99.3	132	164	254
200	40.5	67.0	82.1	103	137	169	262

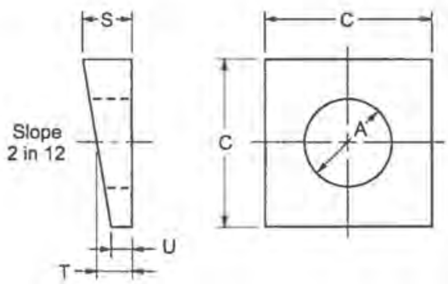
Plain round washers	1.8	2.9	3.2	4.3	5.2	5.9	8.6
Beveled square washers	10.5	9.7	9.3	8.8	15.9	14.9	12.8

ASTM F436M METRIC WASHER DIMENSIONS



PLAIN CIRCULAR WASHERS

Metric Bolt Size	B		A		T	
	Outside Diameter		Hole Diameter		Thickness	
	Max	Min	Max	Min	Max	Min
M16 x 2	34.0	32.4	18.4	18.0	4.6	3.1
M20 x 2.5	42.0	40.4	22.5	22.0	4.6	3.1
M22 x 2.5	44.0	42.4	24.5	24.0	4.6	3.4
M24 x 3	50.0	48.4	26.5	26.0	4.6	3.4
M27 x 3	56.0	54.1	30.5	30.0	4.6	3.4
M30 x 3.5	60.0	58.1	33.6	33.0	4.6	3.4
M36 x 4	72.0	70.1	39.6	39.0	4.6	3.4



BEVELLED SQUARE WASHERS

Metric Bolt Size	C		A		S	T	U
	Width		Hole Diameter		Thickness		
					Thick Side	Mean Nom.	Thin Side
	Max	Min	Max	Min			
M16 x 2	45.0	43.0	18.4	18.0	11.7	8	4.3
M20 x 2.5	45.0	43.0	22.5	22.0	11.7	8	4.3
M22 x 2.5	45.0	43.0	24.5	24.0	11.7	8	4.3
M24 x 3	45.0	43.0	26.5	26.0	11.7	8	4.3
M27 x 3	58.0	56.0	30.5	30.0	12.8	8	3.3
M30 x 3.5	58.0	56.0	33.6	33.0	12.8	8	3.3
M36 x 4	58.0	56.0	39.6	39.0	12.8	8	3.3

FASTENERS – MISCELLANEOUS DETAILING DATA

Metric Fastener Designations

THREAD DATA

Diameter Pitch Combinations

Nominal dia. (mm)	Thread pitch (mm)	Nominal dia. (mm)	Thread pitch (mm)
1.6	0.35	20	2.5
2	0.4	22	2.5
2.5	0.45	24	3
3	0.5	27	3
3.5	0.6	30	3.5
4	0.7	36	4
5	0.8	42	4.5
6.0	1.0	48	5
8	1.25	56	5.5
10	1.5	64	6
12	1.75	72	6
14	2	80	6
16	2	90	6
		100	6

Basic Metric Thread Designation: Metric screw threads are designated by the letter "M" followed by the nominal size (basic major diameter) in millimetres and the pitch in millimetres separated by the symbol "X".

M12	**X**	**1.75**	**– 6g**
Size (mm)		Thread (pitch in mm)	Standard class of fit

Note: In the metric system, the pitch of the thread is given in mm instead of threads per inch – thus a M12 x 1.75 thread has a nominal diameter of 12 mm and the pitch of the thread is 1.75 mm.

PRODUCT DESIGNATION

Metric Bolt Designation: The standard method of designating a metric bolt is by specifying (in sequence) the product name, nominal diameter and thread pitch, nominal length, type, steel property class, and protective coating (if required).

Heavy Hex Structural Bolt, M22x2.5x160,
Type 2, ASTM A325M-09, Zinc Galvanized

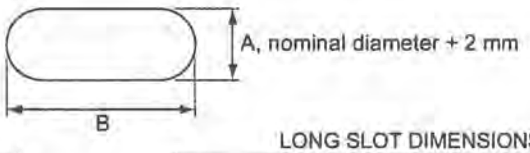
Metric Nut Designation: The standard method of designating a metric nut is by specifying (in sequence) the product name, nominal diameter and pitch, steel property class or material identification, and protective coating (if required).

Heavy Hex Nut, M30x3.5, ASTM A563M
class 105, hot dipped galvanized

Note: It is common practice to omit the thread pitch from the product designation.

Slotted Hole Dimensions

See S16-14 Clause 22.3.5.2 regarding provisions.



SHORT SLOT DIMENSIONS

Nominal Bolt Diameter	Slot Dimensions	
	Width, A	Length, B
mm	mm	mm
16	18	22
20	22	26
22	24	28
24	26	32
27	29	37
30	32	40
36	38	46

LONG SLOT DIMENSIONS

Nominal Bolt Diameter	Slot Dimensions	
	Width, A	Length, B
mm	mm	mm
16	18	40
20	22	50
22	24	55
24	26	60
27	29	67.5
30	32	75
36	38	90

FASTENERS - MISCELLANEOUS DETAILING DATA

Metric-Size Bolt Data

BOLT LENGTH TOLERANCES

Nominal Length	Nominal Bolt Dia. M16 thru 36
to 50 mm	± 1.2
over 50 to 80 mm	± 1.5
over 80 to 120 mm	± 1.8
over 120 to 150 mm	± 2.0
over 150 mm	± 4.0

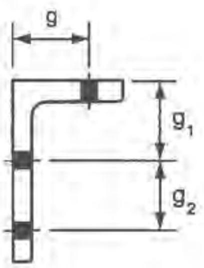
MINIMUM EDGE DISTANCE FOR BOLT HOLES

Bolt Diameter mm	At Sheared Edge mm	At Rolled or Gas Cut Edge† mm
16	28	22
20	34	26
22	38	28
24	42	30
27	48	34
30	52	38
36	64	46
over 36	$1\frac{3}{4}$ x Diameter	$1\frac{1}{4}$ x Diameter

† Gas cut edges shall be smooth and free from notches. Edge distance in this column may be decreased 3 mm when hole is at a point where computed stress under factored loads is not more than 0.3 of the yield stress.

USUAL GAUGES

USUAL GAUGES for Angles, Millimetres



Notes:

Those values shown above the dashed line allow for full socket wrench clearance requirements.

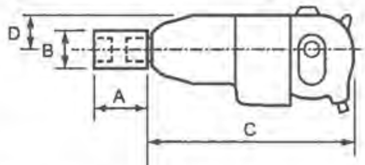
The bolt sizes shown in italics to the left of g and g_1 are the maximum bolt sizes permissible for the dimensions shown.

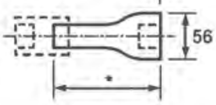
$g_2 \geq 2\frac{2}{3}$ bolt diameters.

Leg \ Gauge	g		g_1		g_2	
200	*M36*	115	*M30*	80		80
150	*M36*	90	*M24*	55		65
125	*M30*	80	*M20*	45		54
100	*M27*	65				
90	*M24*	60				
80	*M24*	50				
75	*M24*	45				
65	*M24*	35				
60	*M24*	30				
55	*M22*	27				
50	*M16*	28				
45	*M16*	23				

ERECTION CLEARANCES
Bolt Impact Wrenches

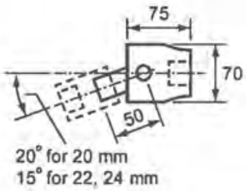
METRIC





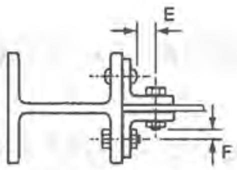
EXTENSION BAR

* Available in lengths 160 to 380 mm



UNIVERSAL JOINT

(for bolts up to 24 mm)



MINIMUM CLEARANCES

	Size	C	D
Light Wrenches	16 to 24	337 to 356	54
Heavy Wrenches	24 to 36	375 to 438	64

Sockets			Min. Clearance	
Bolt size	A	B	E	F
16	80	45	25	28
20	85	54	30	34
22	90	57	32	36
24	95	60	34	38
27	100	70	38	42
30	110	75	41	45
36	130	90	48	52

WELDING

The welding of steel shapes and plates for structural purposes is governed by CSA S16, Design of Steel Structures, and CSA Standard W59, Welded Steel Construction (Metal Arc Welding). In case of conflict between the requirements of CSA W59 and S16, however, S16 shall take precedence (see CSA S16-14 Clause 24.1).

While both standards provide design information on the resistance of welds, CSA Standard W59 extensively covers workmanship, inspection, and acceptance criteria for welded joints in both statically and dynamically loaded structures.

Welding is a process used to join two or more pieces of material together. Arc welding is a process which produces coalescence of metals by heating them with an arc, with or without the application of pressure, and with or without the use of filler metal.

Welding processes used primarily for structural steelwork are:

Shielded Metal Arc Welding	SMAW
Flux Cored Arc Welding	FCAW
Metal Cored Arc Welding	MCAW
Gas Metal Arc Welding	GMAW
Gas Tungsten Arc Welding	GTAW
Submerged Arc Welding	SAW
Electroslag Welding	ESW
Electrogas Welding	EGW
Stud Welding	SW

Welding Definitions

Arc Cutting: a group of cutting processes which melts the metal to be cut with the heat of an arc between an electrode and the base metal.

Arc Spot Weld: a weld made by arc welding between or upon overlapping members in which coalescence may start and occur on the faying surfaces or may proceed from the surface of one member. This is commonly used for thin materials, such as roof and floor deck attachment.

Base Metal: the metal to be welded or cut.

Bevel Angle: the angle formed between the prepared edge of a member and a plane perpendicular to the surface of the member.

Chain Intermittent Welds: intermittent welds on both sides of a joint in which the weld increments on one side are approximately opposite those on the other side.

Coalescence: the growing together or growth into one body of the materials being welded.

Complete Joint Penetration (CJP): a joint welded from both sides or from one side on a backing, having complete penetration and fusion of weld and base metal throughout the thickness of the joint. (Refer to figures in W59)

Edge Joint: a joint between the edges of two or more parallel or nearly parallel members.

Effective Weld Length: the length of weld throughout which the correctly proportioned cross section exists. In a curved weld, it is measured along the axis of the weld.

Effective Throat: the minimum distance from the root of a weld to its face, less any reinforcement.

End Return (Boxing): the continuation of a fillet weld around a corner of a member, as an extension of the principal weld.

Face of Weld: the exposed surface of a weld on the side from which the welding was done.

Fillet Weld: a weld of approximately triangular cross section joining two surfaces approximately at right angles to each other in a lap joint, T-joint, or corner joint.

Groove Angle: the included angle between the weld groove faces.

Groove Weld: a weld made in a groove between two members to be joined.

Intermittent Weld: a weld in which the continuity is broken by recurring unwelded spaces.

Joint Design: the joint geometry together with the required dimensions of the welded joint.

Joint Penetration: the minimum depth a groove weld extends from its face into a joint, exclusive of reinforcement, but including, if present, root penetration.

Leg of a Fillet Weld: the distance from the root of the joint to the toe of the fillet weld.

Partial Joint Penetration (PJP): a groove weld condition in which weld metal extends through a part of joint thickness.

Procedure Qualification: a demonstration that welds made by a specific procedure can meet prescribed standards.

Root of Joint: that portion of a joint to be welded where the members approach closest to each other. In cross section, the root of the joint may be a point, a line or an area.

Root of Weld: the points, as shown in cross section, at which the weld metal intersects the base metal and extends furthest into the weld joint.

Root Penetration: the depth that a weld extends into the root of a joint measured on the centreline of the root cross section.

Size of Weld:

It should be noted that weld symbols and sizes used in North America generally comply with American Welding Society A2.4 "Standard Symbols for Welding, Brazing and Nondestructive Examination". Care should be taken when interpreting other symbol systems.

> **Groove Weld**: See Complete Joint Penetration and Partial Joint Penetration definitions above.
>
> **Fillet Weld**:
>
> For equal-leg fillet welds, the leg lengths of the largest isosceles right triangle which can be inscribed within the fillet weld cross section.
>
> For unequal-leg fillet welds, the leg lengths of the largest right triangle which can be inscribed within the fillet weld cross section.
>
> The preceding definition applies to right-angle connections only. See figure in W59 for the definition of effective size of a fillet weld for connections in which the fusion faces form an angle between 60° and 135°.
>
> Note: When one member makes an angle with the other member greater than 105 degrees, the leg length (size) is of less significance than the effective throat which is the controlling factor for the strength of a weld.

Staggered Intermittent Welds: an intermittent weld on both sides of a joint in which the weld increments on one side are alternated with respect to those on the other side.

Tack Weld: a weld made to hold parts of a weldment in proper alignment until the final welds are made. (Care should be taken to ensure the compatibility of weld metals.)

Throat of a Fillet Weld:

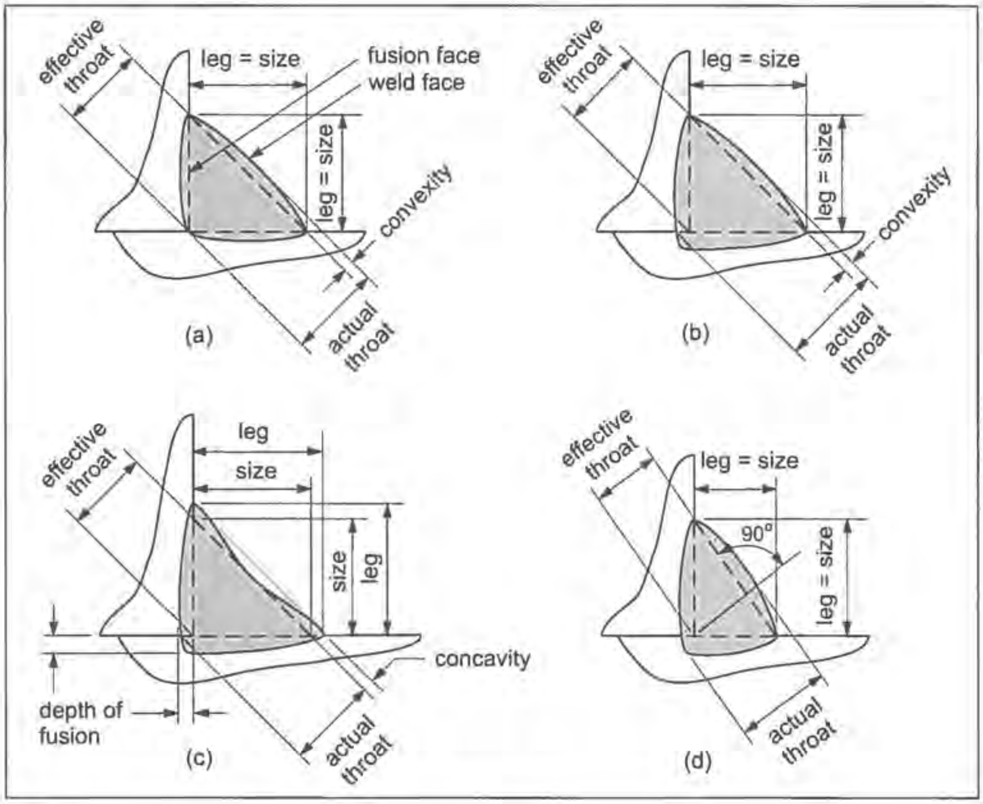
Theoretical Throat: the distance from the beginning of the root of the joint perpendicular to the hypotenuse of the largest right triangle that can be inscribed within the fillet weld cross section. This dimension is based on the assumption that the root opening is equal to zero.

Actual Throat: the shortest distance from the root of weld to its face.

Effective Throat: the minimum distance minus any reinforcement or convexity, from the root of weld to its face.

WELDING PRACTICE

Fillet Welds



Minimum Size

The minimum fillet size as measured should be as shown in the table, unless a larger size is required to meet the calculated resistance. This minimum size requirement need not apply when:

a) welding attachments to members without calculated stress or
b) welding procedures have been established to prevent cracking in accordance with W59-13.

Material thickness t (mm)	Minimum fillet size (mm)
$t \leq 6$	3*
$6 < t \leq 12$	5
$12 < t \leq 20$	6
$20 < t$	8

** For cyclically-loaded structures min. size = 5 mm*

For consumables with a hydrogen content conforming to the H8 requirement or lower, t is the thickness of the thinner part joined. Otherwise, t is the thickness of the thicker part joined; the weld size, however, need not exceed the thickness of the thinner part provided particular care is taken to provide sufficient heat input to ensure weld soundness.

The minimum effective length of a fillet weld should be 38 mm or 4 times the size of the fillet, whichever is larger. Where the geometry of the joint makes it impossible to deposit the minimum effective length, the effective fillet size shall be 0.25 times its effective length.

Maximum Size

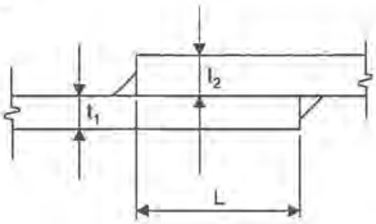
The maximum fillet weld size, D_{max}, recommended by good practice along a sheared edge is:

$D_{max} = t$ when $t < 6$ mm

$D_{max} = t - 2$ when $t \geq 6$ mm

When fillet welds are used in holes or slots, the diameter of the hole or the width of the slot should not be less than the thickness (t) of the member containing it plus 8 mm. The maximum diameter or width shall be t + 12 mm or 2.25 t, whichever is greater.

Lap Joints



$L_{min} = 5\,t_1 \geq 25$ mm when $t_1 \leq t_2$

$L_{min} = 5\,t_2 \geq 25$ mm when $t_2 < t_1$

Partial Penetration Groove Welds

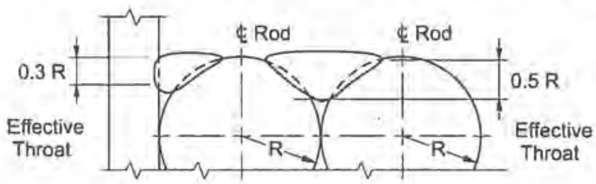
Minimum Groove Depth for Partial Joint Penetration V-, and Bevel Groove Welds †

Thickness, t of Thicker Part Joined (mm)	Minimum Groove Depth, mm	
	Groove Angle, α, at Root $45° \leq \alpha < 60°$	Groove Angle, α, at Root $\alpha \geq 60°$
$t \leq 12$	8	5
$12 < t \leq 20$	10	6
$20 < t \leq 40$	11	8
$40 < t \leq 60$	12	10
$60 < t$	16	12

† Not combined with fillet welds

Effective Throats

Flare Bevel and Flare V-Welds (Flush Welds Only)



Solid or hollow sections with weld filled flush to the curved surface:

Not applicable to flare V-welds using GMAW process except when $R \geq 12$ mm, in which case the effective throat = $0.375R$.

Flare Bevel Groove Weld

When $R > 10$ mm, the effective throat for a joint between a curved and a planar surface shall be 0.3 R. When $R \leq 10$ mm, design as a fillet weld unless an effective throat has been previously qualified as a Flare Bevel (See W59 Clause 4.3.1.6.2.2).

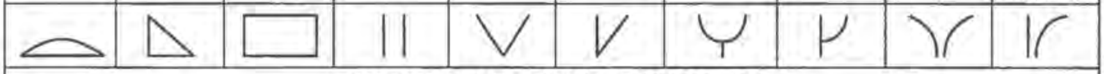
Flare Vee Groove Weld

When $R > 10$ mm, the effective throat for a joint between two curved surfaces shall be 0.5 R.

WELDED JOINTS
Standard Symbols

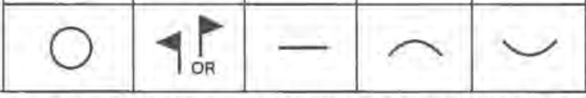
BASIC WELD SYMBOLS

BACK	FILLET	PLUG OR SLOT	GROOVE OR BUTT						
			SQUARE	V	BEVEL	U	J	FLARE V	FLARE BEVEL

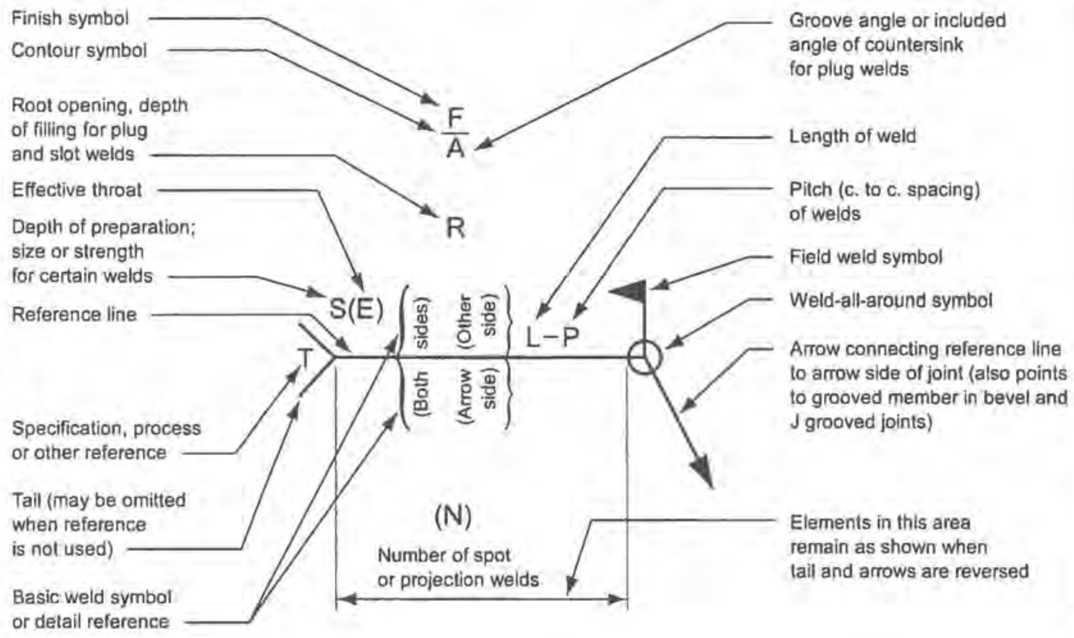


SUPPLEMENTARY WELD SYMBOLS

WELD ALL AROUND	* FIELD WELD	CONTOUR		
		FLUSH	CONVEX	CONCAVE

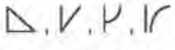


STANDARD LOCATION OF ELEMENTS OF A WELDING SYMBOL



Notes:

Size, weld symbol, length of weld and spacing must read in that order from left to right along the reference line. Neither orientation of reference line nor location of the arrow alter this rule.

The perpendicular leg of  weld symbols must be at left.

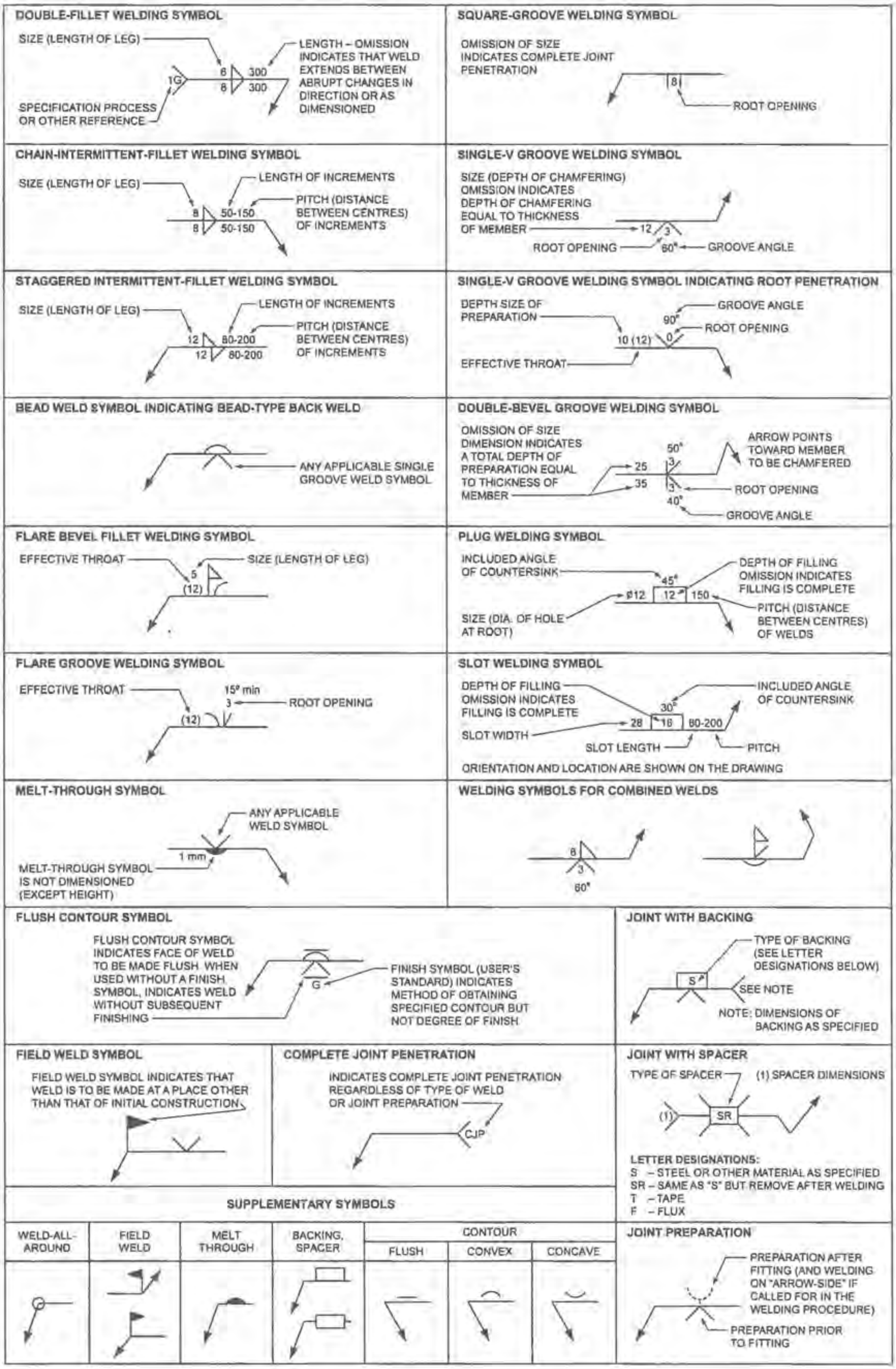
Size and spacing of fillet welds must be shown on both the Arrow Side and the Other Side Symbol.

Symbols apply between abrupt changes in direction of welding unless governed by the "all around" symbol or otherwise dimensioned.

These symbols do not explicitly provide for the case that frequently occurs in structural work, where duplicate material (such as stiffeners) occurs on the far side of a web or gusset plate. The fabricating industry has adopted this convention: when the billing of the detail material discloses the identity of far side with near side, the welding shown for the near side shall also be duplicated on the far side.

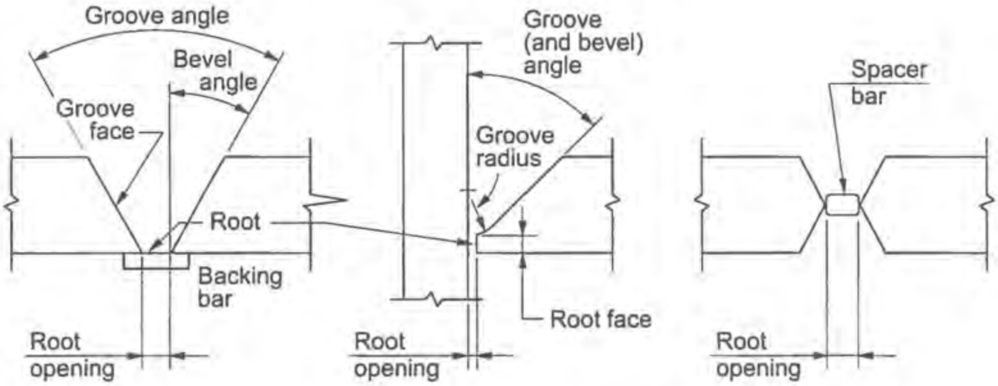
* Pennant points away from arrow.

WELDING SYMBOLS



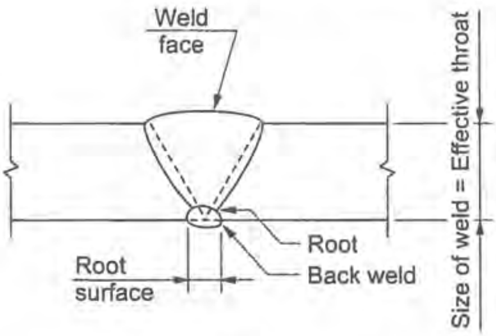
SAMPLE GROOVE WELDS

PREPARATION



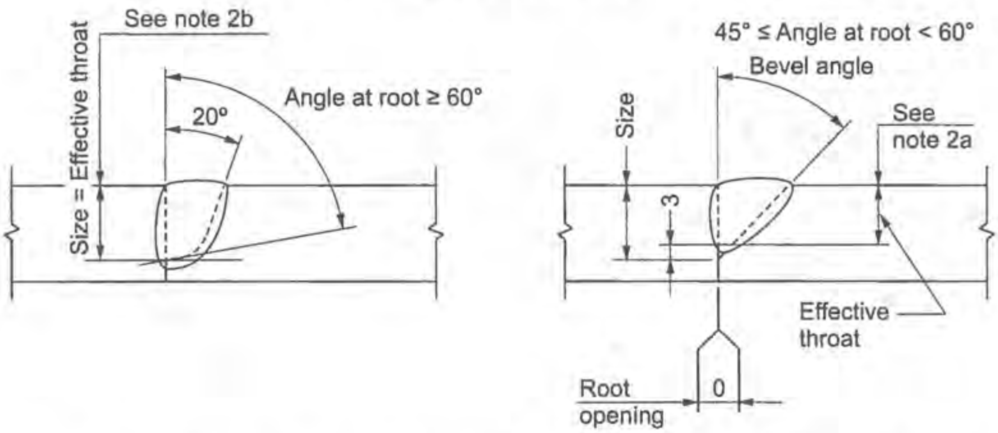
Note 1: For bevel and V-grooves, the groove angle equals the angle at the root. (Does not apply to J and U grooves.)

COMPLETE PENETRATION



See CSA W59 for more details.

PARTIAL PENETRATION



Note 2a: Effective throat = depth of preparation – 3 mm when 45° ≤ Angle at root < 60° *
2b: Effective throat = depth of preparation when angle at root of groove ≥ 60° *
* Applies only to PJPG welds

STEEL PRODUCTS – RECORD OF CHANGES

Following is a chronological record of changes to the list of steel sections included in the CISC Handbook of Steel Construction since the first printing of the Third Edition.

1983 No longer produced by Algoma are:

M100x19
S150x26, 19; S130x22, 15; S100x11; S75x11, 8
All angles except 8" x 8" leg sizes

1985 No longer produced by Algoma are:

WWF550x217; WWF350x385

New shapes and sections produced by Algoma:

WWF1800x632, 548; WWF1600x579, 495
WWF1400x491, 407; WWF550x280

Welded Reduced Flange (WRF) shapes with top flanges narrower than the bottom flanges and intended primarily for composite bridge girders:

WRF1800x543, 480, 416; WRF1600x491, 427, 362
WRF1400x413, 348, 284; WRF1200x373, 309, 244
WRF1000x340, 275, 210

1986 New shapes and sections produced by Algoma:

W610x91, 84; W530x72; W310x31; W250x24; W200x21

1989 Sections produced by Algoma

Sections deleted:

WWF1800x632, 548; WWF1600x579, 495
WWF1400x491, 407; WWF1200x403, 364
WWF1100x335, 291, 255, 220; WWF1000x324, 280, 244
WWF900x293, 249, 213; WWF800x332–154; WWF700x222–141

Sections added:

WWF2000x732–542; WWF1800x700–510; WWF1600x622–431
WWF1400x597–358; WWF1200x418, 380, 333
WWF1100x351, 304, 273, 234; WWF1000x340, 293, 262, 223
WWF900x309, 262, 231; WWF800x339–161; WWF700x245–152
WWF650x864–400; WWF600x793–369

Sections not available from Canadian mills added:

W1000–All sizes
W920x1262–488; W840x922–392; W760x865–350; W760x134
W690x802–289; W610x732–262; W530x599–248; W460x464–193

1991 Sections no longer available from Canadian mills:

W310x283, 253
C380x74–50; C310x45–31

1993 The following shapes are no longer produced:

HP330x149–89
M150x29.8–6.5; M100x19

1995 Sections deleted:

W1000x488–286, 976, 790–483; W920x1072, 876, 722; W840x922–577
W760x865, 783, 644, 531; W690x735, 605, 500, 419
W610x670, 551, 455; W530x599–331
HP330x149–89
M150x29.8, 6.5; M100x19
S180x30, 22.8; S130x22
C130x17
MC250x9.7; MC180x26.2; MC150x22.8
L152x102x4.8; L127x127x4.8; L127x89x11, 4.8; L127x76x16, 4.8
L102x102x4.8; L102x89x16, 11, 4.8; L102x76x4.8; L89x89x16, 4.8
L89x76x16, 11; L89x64x16, 11; L76x76x16; L76x64x16, 11
L76x51x16, 11; L64x64x3.2; L64x51x3.2; L51x38x9.5, 3.2
L32x32x9.5; L25x25x9.5, 7.9
L200–L25 (All metric angles)

Sections added:

W1100x499–342; W1000x749–478, 259, 693–314; W920x381, 345
W840x251; W760x220; W690x192; W610x153; W360x1202
M310x16.1; M250x11.9; M100x8.9
SLB100x5.4, 4.8; SLB75x4.5, 4.3
L203x102x22, 16, 11; L178x102x11; L19x19x3.2

1997 Sections deleted:

W1000x478, 259, 693; W920x1262; W760x710; W690x667
W610x732, 608; W460x464–286; W360x1202
L203x203x14; L203x152x22, 16, 14; L203x102x22, 16, 14, 11
L152x102x6.4; L152x89x6.4; L89x76x4.8; L89x64x4.8; L64x38x7.9–4.8
L51x38x7.9; L44x44x9.5, 7.9; L38x38x9.5, 7.9, 4.0; L32x32x7.9
HSS51x51x2.5; HSS38x38x2.5; HSS32x32x3.8–2.5; HSS25x25x3.2, 2.5
HSS127x64x9.5–4.8; HSS127x51x9.5–4.8; HSS51x25x2.5
HSS48x2.8; HSS42x3.2, 2.5; HSS33x3.2, 2.5; HSS27x3.2, 2.5

Sections added:

W1000x591, 539, 486, 483; W840x576; W760x531; W690x500, 419
W610x551, 455; W150x13
L152x152x6.4

HSS127x127x13; HSS102x102x3.8, 3.2; HSS89x89x3.8, 3.2
HSS76x76x9.5, 3.8, 3.2
HSS152x102x13; HSS152x76x9.5–4.8; HSS127x76x3.8
HSS102x76x3.8, 3.2; HSS76x51x3.2
HSS610x13–6.4; HSS559x13–6.4; HSS508x13–6.4

2000 Sections deleted:

HP310x174, 152, 132

2004 Sections deleted:

W840x576; W760x531
WT230x33.5, 30.5
L203x152x11
HSS305x305x11; HSS254x254x11; HSS203x203x11; HSS178x178x11
HSS152x152x11; HSS127x127x11; HSS102x102x3.8; HSS89x89x3.8
HSS76x76x3.8; HSS64x64x3.8; HSS51x51x3.8; HSS38x38x3.8
HSS305x203x11; HSS254x152x11; HSS203x152x11; HSS203x102x11
HSS178x127x11; HSS152x102x11; HSS127x76x3.8; HSS102x76x3.8
HSS102x51x3.8; HSS89x64x3.8; HSS76x51x3.8
HSS610x13, 11, 9.5, 8.0, 6.4; HSS559x13, 11, 9.5, 8.0, 6.4
HSS508x13, 11, 9.5, 8.0, 6.4; HSS406x11, 8.0; HSS356x11, 8.0
HSS324x11, 8.0; HSS273x11, 9.5, 8.0; HSS219x11, 8.0; HSS141x8.0
HSS114x8.0, 6.4; HSS102x3.8; HSS89x3.8; HSS73x3.8; HSS60x3.8
HSS48x3.8

Sections added:

M310x14.9; M250x11.2; M200x9.2; M150x6.6, 5.5
SLB100x5.1; SLB75x5.6, 3.8; SLB55x6.4
C100x7, C75x5
MC150x22.8
L203x203x14; L203x152x22, 16, 14; L102x89x11; L51x38x3.2
HSS305x305x16; HSS254x254x16; HSS203x203x16; HSS178x178x16
HSS114x114x13, 9.5, 8.0, 6.4, 4.8, 3.2; HSS102x102x13; HSS64x64x8.0
HSS356x254x16, 13, 9.5; HSS305x203x16; HSS254x152x16
HSS152x76x13; HSS102x51x9.5; HSS51x25x4.8
HSS356x16; HSS273x4.8; HSS219x16; HSS178x13, 9.5, 8.0, 6.4, 4.8
HSS168x13, 3.2; HSS152x9.5, 8.0, 6.4, 4.8, 3.2
HSS127x13, 9.5, 8.0, 6.4, 4.8, 3.2; HSS114x9.5, 3.2; HSS102x3.2
HSS89x3.2; HSS76x6.4, 4.8; HSS64x6.4, 4.8, 3.2

2006 Sections deleted:

W920x1188, 967, 784, 653, 585, 534, 488, 446, 417, 387, 365, 342

Sections added:

W1000x438; W920x1191, 970, 787, 725, 656, 588, 537, 491, 449, 420, 390, 368, 344; W840x576; W760x531; W460x464, 421, 384, 349, 315, 286

2010 Sections deleted:

W920x1191, 970, 787, 725; W690x802; W310x31; W250x24; W200x21
WT460x223, 208.5, 193.5, 182.5, 171
M200x9.2, M150x5.5, M130x28.1

Sections added:

WT460x224.5, 210, 195, 184, 172

2016 Sections deleted:

WRF1800x543–416; WRF1600x491–362; WRF1400x413–284
WRF1200x373–244; WRF1000x340-210
WWF2000x732–542; WWF1800x700–510; WWF1600x622–431
WWF1400x597–358; WWF1200x487–263; WWF1100x458–234
WWF1000x447–200; WWF900x417–169; WWF800x339–161
WWF700x245–152; WWF650x864–400; WWF600x793–369
WWF550x721–280; WWF500x651–197; WWF450x503–177
WWF400x444–157; WWF350x315–137
W610x91, 84; W460x67, 61
SLB100x5.4–4.8; SLB75x5.6–3.8; SLB55x6.4
L152x152x6.4, L152x89x16, L127x127x6.4, L102x89x11, L102x76x11
HSS114x114x13, 9.5, 8.0, 6.4, 4.8, 3.2; HSS89x89x3.2; HSS64x64x8.0
HSS356x254x16, 13, 9.5; HSS89x64x8.0, 3.2
HSS356x16; HSS219x16; HSS178x8.0, 6.4, 4.8
HSS168x8.0, 3.2; HSS152x9.5, 8.0, 6.4, 4.8, 3.2; HSS141x4.8
HSS127x13, 8.0, 4.8, 3.2; HSS114x9.5, 4.8, 3.2
HSS102x8.0, 6.4, 4.8, 3.2; HSS89x8.0

Sections added:

W1000x976; W920x1377, 1269, 1194, 1077, 970, 787, 725; W690x802
W530x409, 369, 332; W360x1299, 1202
M318x18.5, 17.3; M200x9.2; M150x5.5; M130x28.1; M100x6.1; M75x4.3
HP460x304, 269, 234, 202; HP410x272, 242, 211, 181, 151, 131
HP310x132
MC310x21.3; MC250x9.7; MC150x10.4, 9.7; MC100x20.5; MC75x10.6
L254x254x32, 29, 25, 22, 19; L203x152x11; L203x102x22, 16, 14, 11
L89x76x11; L76x64x11; L64x38x6.4, 4.8; L38x38x4.0
HSS559x559x19; HSS508x508x22, 19, 16, 13
HSS457x457x22, 19, 16, 13; HSS406x406x22, 19, 16, 13, 9.5
HSS356x356x16, 13, 9.5, 7.9; HSS254x254x4.8; HSS203x203x4.8
HSS127x127x3.2
HSS305x152x16, 13, 9.5, 7.9, 6.4; HSS254x203x16, 13, 9.5, 7.9, 6.4
HSS254x152x4.8; HSS203x152x16; HSS152x102x3.2
HSS152x76x3.2; HSS127x76x13, 3.2; HSS76x38x6.4, 4.8, 3.2
HSS64x38x6.4, 4.8, 3.2
HSS508x13, 9.5, 6.4; HSS457x13, 9.5, 6.4; HSS406x16
HSS273x9.5, 7.9; HSS245x9.5, 6.4; HSS141x13; HSS76x3.2; HSS42x3.2

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CISC

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for Structural Steel

Eighth Edition

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CISC CODE OF STANDARD PRACTICE

for Structural Steel

PREFACE

The CISC Code of Standard Practice for Structural Steel is a compilation of usual industry practices relating to the design, fabrication and erection of structural steel. These practices evolve over a period of time and are subject to change as improved methods replace those of an earlier period. The Code is revised whenever a sufficient number of changes have occurred to warrant a new edition.

The first edition of the Code was adopted and published in November 1958. A second edition incorporating minor revisions was published in October 1962. The third edition, published in September 1967 and revised in May 1970, incorporated minor changes throughout with principal changes in Section 2 - Definition of Structural Steel and Section 3 - Computation of Weights for Unit Price Bids.

The fourth edition adopted in June 1980, revised December 1980, broadened the scope to include bridges and other structures. It also incorporated the CISC "Guide to Tendering Procedures" into Section 3 and Appendices B and C. The Code was converted to SI (metric) units and provided conversion factors and Imperial units in Appendix E.

The fifth edition (1991) reflected the steel standard's recognition of the preparation of five types of fabrication and erection documents which may be produced in fulfilling a steel construction contract. These documents may be in the form of drawings, diagrams, sketches, computer output, hand calculations and other data which can be supplied by the fabricator/erector. This data is generally referred to in contract documents as "shop drawings". The computation of mass has been changed by deleting the mass of welds and the allowances for paint and other coatings. Appendix B, Guideline for Unit Price Application for Changes, and Appendix C, A Suggested Format for Price-Per-Unit of Mass or Price-Per-Item Contracts were substantially revised. To foster uniformity, two new appendices were added: Miscellaneous Steel and A Suggested Format for a Monthly Progress Claim Form.

The sixth edition (1999) clarified the role of the fabricator, the information required, and where that information is expected, as stipulated in the governing technical standards. Added were: definitions of Design Drawings and Quotations, clauses on quotations, discrepancies, shims for bearing surfaces, the allowance for return of documents, the information required when painting is specified, and Appendix H - Suggested Definitions for Progress Invoicing and Substantial Performance. Changes were also made to Appendix C, the terminology for Unit Price contracts, connection types, and anchor rods – the latter two to be consistent with the changes in CSA Standard CAN/CSA-S16-01.

The seventh edition (2008) added two new appendices: I - Architecturally Exposed Structural Steel (AESS) and J - Digital Modelling, in order to give guidance to designers, owners, and contractors on these two important topics. As each of these topics involved issues that vary widely and approaches differ, the Code endeavoured to identify and clarify the main points that should be addressed by the interested parties to avoid conflicts during actual construction. In addition, definitions of AESS, Steel Detailer and Work, and a time frame for accepting erected steelwork were added.

This eighth edition (2015) was updated by a consensus of stakeholders within the Canadian steel construction industry. Committee members included steel fabricators, erectors, detailers, engineers, architects and general contractors. The Code underwent major revisions reflecting this consensus approach with noted changes including BIM (electronic documents), temporary bracing, conditions where lintels would be included in a steel contract, computations of units and mass, and erection stability.

Whenever a gender-specific term is used, it shall be read as gender-neutral.

By documenting standard practices, the CISC Code of Standard Practice aims to provide guidance on current practices in the Canadian structural steel fabrication and erection industry and its clients.

The latest edition of the Code can be found on the CISC website (*www.cisc-icca.ca*).

Canadian Institute of Steel Construction

Adopted September 23, 2015

1. General Provisions

1.1 Scope

This Code covers standard industry practice with respect to the furnishing of structural steel, joist, and platework, in the absence of provisions to the contrary contained in the Contract.

1.2 Definitions

Architect	As defined under the appropriate provincial Architect's Act.
Architecturally Exposed Structural Steel	Structural steel which is specifically designated as architecturally exposed and the appearance of which is governed by Appendix I, Architecturally Exposed Structural Steel.
BIM Administrator	The BIM Administrator is responsible from the pre-design phase onwards to develop and to track the object-oriented BIM against predicted and measured performance objectives, supporting multi-disciplinary building information models that drive analysis, schedules, take-off and logistics.
BIM Execution Plan	The document that defines the expected BIM deliverables and guides the coordination of the project teams. (Includes the BIM Responsibility Matrix).
Building Information Model (BIM)	A digital representation of the physical and functional characteristics of a facility. A BIM is a shared knowledge resource for information about a facility, forming a reliable basis for decisions during its life-cycle – defined as existing from earliest conception to demolition. Note: the term Digital/Electronic Model may be used in lieu of the term BIM in some sectors.
Client	A person, corporation, or authority with whom the Fabricator and/or Erector have contracted.
Change Directive	A written instruction signed by the General Contractor directing the Fabricator and/or Erector to proceed with a change in the Work within the general scope of the Contract Documents, prior to the General Contractor and the Fabricator and/or Erector agreeing upon adjustments in the contracted price and the contracted time.
Change Order	A written amendment to the Subcontract signed by the Contractor and the Subcontractor stating their agreement upon: • A change in the Subcontract Work • The method of adjustment or the amount of the adjustment in the Subcontract price, if any, and • The extent of the adjustment in the Subcontract time, if any.

Connection Design Details	Documents which provide details of standard and non-standard connections and other data necessary for the preparation of shop details.
Construction Documents	The most recent of IFC drawings, specifications, computer output, models and electronic/digital data used to govern the construction of the Works.
Construction Specifications	The IFC Specifications used to govern the construction of the Works.
Contract	The undertaking by the parties to perform their respective duties, responsibilities and obligations as prescribed in the Contract Documents; represents the entire agreement between the parties.
Contract Documents	Include the Construction Documents and all commercial terms and conditions governing the Work (including schedule).
Cost-Plus Contract	An Agreement whereby the Fabricator and/or Erector agrees to fulfill the contract for a consideration which is calculated on the basis of the Fabricator's and/or Erector's costs plus a specified fee as defined in the contract.
Design Documents	Drawings and specifications, including computer models, electronic documents and other data, as prepared by the Engineer of Record, showing member sizes and dimensions and all required forces for connection design, i.e. shears, axial forces, moments and torsions. (Refer to the governing technical standard for the entire list of mandatory requirements.)
Engineer	As defined under the appropriate provincial Professional Engineer's Act.
Engineer of Record	The Professional Engineer that assumes responsibility for the design. (Note: terminology of this individual varies from province to province depending on the local Engineering Act.)
Erection Bracing	Bracing materials or members which are used to plumb, align and stabilize structural members or the structure during construction and are removed when the structural members or the structure is secured by bolting or welding of structural members (not to be confused with Temporary Bracing).
Erection Diagrams	General arrangement drawings and/or models showing all information necessary for the assembly of the steel structure.

Erection Procedures	Outline the construction methods, erection sequence, erection and temporary bracing requirements, and other engineering details necessary for shipping, handling, erecting, and maintaining the stability of the structural steel frame.
Erector	The party responsible for erection of the steelwork.
Fabrication and Erection Documents	A collection of documents (hard copy, electronic and/or models) prepared by the Fabricator and/or Erector related to steel fabrication and erection.
Fabricator	The party responsible for furnishing the Structural Steel.
Field Work Details	Details that provide complete information for modifying fabricated members – both new and existing – in the field.
General Contractor, Constructor or Construction Manager	The person or corporation constructing, coordinating, and supervising the Work.
General Terminology e.g. Beams, Joists, Columns, etc.	These terms have the meanings stated or implied in CSA-S16 (latest edition), CSA-S6 (latest edition) and Appendix A of this Code.
Industry Foundation Class Model	A platform-neutral, open-file format specification that is not controlled by a single vendor or group of vendors. It is an object-based file format with a data model developed by building SMART (formerly the International Alliance for Interoperability, IAI) to facilitate interoperability in the architecture, engineering and construction (AEC) industry, and is a commonly used collaboration format in Building information modelling (BIM) based projects. The Industry Foundation Class model specification is open and available. It is registered by ISO and is an official International Standard ISO 16739:2013.
Issued-for-Construction Documents (IFC)	The initial milestone set of drawings, specifications and other documents (including hard copy, electronic and/or models) produced by the Engineer of Record to be used by the Contractor, Fabricator and/or Erector and other trades for construction. Issued-for-Construction Documents shall conform to the requirements of CSA S16 or CSA S6.

Level of Development (LOD)	A specification that enables practitioners in the AEC Industry to specify and articulate with a high level of clarity the content and reliability of Building Information Models (BIMs) at various stages in the design and construction process.
Lump Sum Price Contract	Also called Stipulated Price Contract; an agreement whereby the Fabricator and/or Erector contracts to fulfill the Contract terms for a lump sum (stipulated price) consideration.
Manufacturing Model	A 3D model created from the LOD that represents the "as fabricated" or "as shop issued" status. The manufacturing model is typically prepared by the detailer and should include all material in the accurate sizes, locations and profiles to represent what is fabricated in the assembled state, including bolts but not necessarily welds.
Miscellaneous Steel	Steel items described and listed in Appendix F of this Code.
Others	A party or parties other than the Fabricator and/or Erector.
Owner	The Owner of a structure, and shall include his authorized agent and any person taking possession of a structure on the Owner's behalf. Depending on the circumstances, an authorized agent may be the architect, engineer, general contractor, construction manager, public authority or other designated representative of the Owner.
Quotations	Proposals by the Fabricator based on Structural Steel as defined in Clause 2.1 and as included in the Tender Documents, and in accordance with the documents outlined in Clause 3.1.1.
Revision	A change in the Contract Documents.
Shop Details	Documents which provide complete information for the fabrication of various members and components of the structure, including the required material and product standards; the location, type, and size of all mechanical fasteners; bolt installation requirements and welds.
Steel Detailer	Those responsible for the preparation of shop details and other data necessary for fabrication and/or erection. May also be the Fabricator.

Steel Erection Execution Plan	Processes and procedures for the safe positioning, aligning and securing of the structural steel components on prepared foundations to form a complete frame.
Stipulated Price Contract	See Lump Sum Price Contract.
Structural Design Documents	May include drawings, specifications, computer output, and electronic and other data. The Structural Design Documents shall show a complete design of the structure with members suitably designated and located, including such dimensions and details as necessary to permit the preparation of Fabrication and Erection Documents. Documents shall be in accordance with CSA S16 and CSA S6.
Structural Steel	Those items listed under Clause 2.1
Structural Steel Frame	An assemblage of Structural Steel components (beams, columns, purlins, girts, etc.) for the purpose of resisting loads and forces. See Clause 2.1.
Structural Steel Specifications	The portion of the Tender Specifications containing the requirements for the fabrication and erection of the Structural Steel.
Temporary Bracing	Members that are designed by the Engineer of Record or a third party, to be removed at a later date at their instruction (not to be confused with Erection Bracing).
Tender Documents	Drawings, BIM files, specifications, general conditions, addenda, etc., used as the basis for preparing a tender.
Tender Drawings	Drawings used as the basis for preparing a tender.
Tender Specifications	Specifications used as the basis for preparing a tender.
Unit-Price Contract	Also called Price-per-Unit Contract. An agreement whereby the Fabricator and/or Erector contracts to fulfill the contract terms for a consideration which is based on the units of steel calculated in accordance with the CISC Code of Standard Practice for Structural Steel.
Work	The product and/or services provided by the Steel Fabricator and/or Erector.

1.3 Governing Technical Standards

The provisions of the latest edition of CSA-S16 "Design of Steel Structures" shall govern the design, fabrication and erection of steel structures except bridges. The provisions of the latest edition of CSA-S6 "Canadian Highway Bridge Design Code", the "Ontario Highway Bridge Design Code" (in Ontario) or the American Railway Engineering Association's "Specifications for Steel Railway Bridges" shall govern the design, fabrication and erection of structural steel for bridges. The provisions of the latest edition of CSA Standard W59 "Welded Steel Construction (Metal-Arc Welding)" shall govern arc welding design and practice. The provisions of other standards shall be applicable if called for in the Tender Drawings and Tender Specifications.

1.4 Responsibility for Design

When the Client provides the structural drawings and specifications, the Fabricator and the Erector shall not be responsible for determining the adequacy of the design nor be liable for the loss or damage resulting from an inadequate design. Should the Client desire the fabricator to assume any responsibility for design beyond that of proposing adequate connections and details, and, when required, components, members, or assemblies standardized by the Fabricator, the Client shall state clearly his requirements in the invitation to tender or in the accompanying Tender Drawings and Tender Specifications. Even though proposed connections and design details may be prepared by the Fabricator's technical staff, the overall behaviour of the structure remains the responsibility of the designer of the structure. (See also Clause 5.6).

1.5 Responsibility for Erection Procedure

When the erection of Structural Steel is part of his Contract, the Fabricator shall be responsible for determining the Erection Procedure, for checking the adequacy of the connections for the uncompleted structure, and for providing Erection Bracing or connection details. When the erection of the Structural Steel is not part of his Contract, the Fabricator shall not be responsible for determining the Erection Procedure, for checking the adequacy of the connections for the uncompleted structure, or for providing Erection Bracing or connection details not included in the Structural Design Documents, nor shall the Fabricator be liable for loss or damage resulting from faulty erection. However, the steel Fabricator shall be informed by the Client of the erection sequence to be used, which may influence the sequence and process of the manufacturing. (See also Clauses 5.1 and 5.4).

1.6 Patented Devices

Except when the Contract Documents call for the design to be furnished by the Fabricator and/or Erector, the Fabricator and/or Erector assume that all necessary patent rights have been obtained by the Client and that the Fabricator and/or Erector will be fully protected by the Client in the use of patented designs, devices or parts required by the Structural Design Documents.

1.7 Scheduling

The Client should provide a construction schedule in the Tender Documents. In the absence of such a schedule, one should be mutually agreed upon between the contracting parties, prior to the Contract award.

2. Classification of Material

2.1 Structural Steel

Unless otherwise specified in the Tender Documents, a Contract to supply, fabricate and deliver Structural Steel shall include only those items from the following list which are clearly indicated as being required by the Structural Design Documents. (See Appendix A)

2.1.1

Anchors for Structural Steel.

Base plates and bearings for Structural Steel members.

Beams, purlins, girts forming part of the Structural Steel frame.

Bearing plates and angles for Structural Steel members and steel deck.

Bins and hoppers of 6 mm plate or heavier, attached to the Structural Steel frame(s).

Bracing for Structural Steel members, steel trusses or steel frames.

Brackets attached to the Structural Steel.

Bridge bearings connected to the Structural Steel members.

Cables for permanent bracing or suspension systems.

Canopy framing if attached to the Structural Steel frame.

Cold-formed channels when used as structural members as listed in the CISC Handbook of Steel Construction.

Columns.

Conveyor galleries and supporting bents (exclusive of conveyor stringers, deck plate and supporting posts which are normally part of the conveyor assembly).

Crane rails and stops, excluding final alignment of the rails, unless otherwise noted on the Drawings.

Curb angles and plates attached to the Structural Steel frame where shown on the Structural Design Documents.

Deck support angles at columns, walls, where shown on the Structural Steel drawings.

Diaphragms for bridges.

Door frame supports attached to the Structural Steel frame.

Expansion joints connected to the Structural Steel frame (excluding expansion joints for bridges).

Field bolts to connect Structural Steel components.

Floor plates, roof plates (raised pattern or plain) and steel grating connected to the Structural Steel frame.

Girders.

Grillage beams of Structural Steel.

Hangers supporting Structural Steel framing.

Jacking girders.

Lintels shown, detailed and dimensioned on the Structural Design Documents.

Mechanical roof support and floor opening framing shown on Structural Design Documents.

Monorail beams of standard Structural Steel shapes.

Open-web steel joists, including anchors, bridging, headers and trimmers; also, when specified to be included in the Structural Steel Design Documents, light-gauge forms and temperature reinforcement.

Sash angles shown, detailed and dimensioned on the Structural Design Documents.

Separators, angles, tees, clips and other detail fittings essential to the Structural Steel frame.

Shear connectors/studs, except when installed through the sheet steel floor or roof deck by the deck installer.

Shelf angles shown, detailed and dimensioned on the Structural Design Documents.

Shop fasteners or welds, and fasteners required to assemble parts for shipment.

Steel connection plates or fixtures for Structural Steel embedded or anchored on site in concrete or masonry.

Steel tubes or cores for composite columns or braces.

Steel window sills attached to the Structural Steel frame.

Struts.

Suspended ceiling supports of Structural Steel shapes where shown on the Structural Design Documents.

Temporary components to facilitate transportation to the site.

Tie, hanger and sag rods forming part of the Structural Steel frame.

Trusses.

2.1.2 Only if shown and designed on the Structural Design Documents and specifically noted by the Tender Documents to be supplied by the Structural Fabricator:

Steel stairs, walkways, ladders and handrails forming part of the structural steelwork. (See Appendix A)

2.2 Field Connection Material

2.2.1 When the erection of the Structural Steel is part of the Fabricator's Contract, he shall supply all material required for temporary and for permanent connection of the component parts of the Structural Steel.

2.2.2 When the erection of the Structural Steel is not part of the Fabricator's Contract, unless otherwise specified in the Tender Documents, the Fabricator shall furnish appropriate bolts and nuts (plus washers, if required) or special fasteners, of suitable size and in sufficient quantity for all field connections of steel to steel which are specified to be thus permanently connected, plus an over-allowance of two per cent of each size to cover waste.

Unless otherwise specified in the Tender Documents, welding electrodes, back-up bars, temporary shims, levelling plates, fitting-up bolts and drift pins required for the Structural Steel shall not be furnished by the Fabricator when the erection of the Structural Steel is not part of the Fabricator's Contract.

2.3 Items Supplied by Others

Unless otherwise specified in the Tender Documents, the following steel or other items shall not be supplied by the Structural Steel Fabricator.

Bins and hoppers not covered in Clause 2.1 of this Code.

Bolts for wood lagging.
Bridge bearings not connected to Structural Steel items.
Canopy framing not attached to Structural Steel.
Catch basin frames.
Concrete for filling HSS or pipe sections. Concrete is to be supplied and poured by others in the shop or field with the cooperation of the Fabricator and/or Erector.
Connection material for other trades.
Conveyor stringers, deck plate and supporting posts.
Door and corner guards.
Door frames not covered in Clause 2.1 of this Code.
Drain pipes.
Drilling of holes into masonry or concrete, including core drilling of anchor rods for bridges and drilling for deck support angles.
Edge forming less than 3.2 mm thick for steel deck and not covered in Clause 2.1 of this code.
Embedded steel parts in precast concrete.
Embedded steel parts not required for Structural Steel or steel deck.
Flagpoles and supports.
Floor plates, roof plates and grating not covered in Clause 2.1 of this Code.
Grout.
Hoppers and chutes.
Hose and tire storage brackets.
Installation of structural steel parts embedded in concrete or masonry.
Lag bolts, machine bolts and shields or inserts for attaching any non-Structural Steel item
Lintels not shown, detailed and dimensioned on the Structural Design Documents.
Lintels which are an integral part of door frames.
Machine bases, rollers and pulleys.
Members made from gauge material except cold-formed channels indicated in Clause 2.1.
Metal-clad doors and frames.
Miscellaneous Steel; see Appendix F.
Shear connectors through sheet steel deck by deck installer.
Sheet steel cladding.
Sheet steel deck.
Sheet steel flashing.
Shelf angles not shown, detailed and dimensioned on the Structural Design Documents.
Shoring under composite floors and stub girders.
Steel doors.
Steel sash angles not shown, detailed and dimensioned on the Structural Design Documents.
Steel stacks.
Steel stairs, landings, walkways, ladders and handrails, not covered in Clause 2.1.2 of this Code.

Steel tanks and pressure vessels.

Steel window sills not covered in Clause 2.1 of this Code.

Support for sheet steel deck at column cut-outs and for openings not requiring framing, connected to Structural Steel.

Temporary bracing for other trades.

Trench covers.

Trim angles, eave angles or fascia plates not directly attached to the structural steel frame.

2.4 Custom Items

The responsibility for the supply and/or installation of items not conforming to the above lists shall be clearly identified by the Client at the time of tender.

3. Quotations and Contracts

3.1 Standard Form of Contract

Unless otherwise agreed upon, a Contract to fabricate, deliver and/or erect Structural Steel shall be the appropriate unaltered Standard Construction Document contract issued and duly sealed by the Canadian Construction Association (CCA) as listed at *www.cca-acc.com*.

3.1.1 Quotations

Unless otherwise stated, Quotations from Fabricators and/or Erectors are based on the following documents:

(1) The appropriate unaltered CCA Contract Document with copyright seal with no additional conditions, as issued by the Canadian Documents Committee.

It is accepted that alterations and/or additions to the standard CCA Contract Document by the General Contractor, Constructor or Construction Manager after Quotation may have implications not originally anticipated by the Fabricator and/or Erector. The use of non-standard Contracts, altered or modified CCA Contract Documents shall allow the Fabricator and/or Erector to incorporate related costs and implications into a new Quotation for consideration.

(2) Canadian Institute of Steel Construction (CISC) Code of Standard Practice for Structural Steel, latest edition.

3.1.2 Progress Payment Claim Form

A suggested format for a progress payment claim form is provided in Appendix G.

3.1.3 Progress Invoicing and Substantial Completion

For suggested recommended progress invoicing terms and definitions, see Appendix H.

3.2 Types of Contracts

3.2.1 For Lump Sum Price Contracts stipulating a "lump sum price", the work required to be performed by the Fabricator and/or Erector must be completely defined by the Tender Documents.

3.2.2 For Unit-Price Contracts stipulating a "price per unit", the scope of the Work, type of materials, character of fabrication, and conditions of erection are based upon the Tender Documents which must be a representative sample of the Work to be performed. Final unit rates may be subject to adjustment, based on the complexity of the Issued-for-Construction (IFC) Documents. For methods of computing mass, area, or quantity, see Clause 3.5. Also see Appendix C of this Code for a suggested unit rate catalogue.

3.2.3 For Cost-Plus Contracts stipulating "cost plus fee", the Work required to be performed by the Fabricator and/or Erector is indefinite in nature at the time the Tender Documents are prepared. Consequently the Contract Documents should define the method of measurement of Work performed, and the fee to be paid in addition to the Fabricator's and/or Erector's costs.

3.3 Revisions to Contract Documents

3.3.1 Revisions to the Contract Documents shall be made by the issue of dated new or revised documents. All Revisions shall be clearly indicated. Such Revisions should be issued by a Change Notice. Revisions to the Work shall not be noted on Shop Details submitted for review but should be issued on revised Construction Documents.

3.3.2 The Fabricator and/or Erector shall advise the Client or Client's representative of any impact that such Revision or change will have on the price and/or schedule of the existing Contract. The response to the Change Notice shall be accompanied by a description of the impact change in sufficient detail to permit evaluation and prompt approval by the Client.

3.3.3 Upon agreement between the Fabricator and/or Erector and the Client or Client's representative as to the Revision's impact, the Client or his representative shall issue a Change Order or Extra Work Order for the Revision to the Contract for the change in the Work.

3.3.4 Unless specifically stated to the contrary, the issue of revised Contract Documents or Revisions indicated on the review documents is not authorization by the Client to release these Revisions for construction. Upon receipt of revised Construction Documents, the Fabricator and/or Erector shall notify the Client that a Revision to the Contract scope has been received, and a time frame shall be agreed for the Fabricator and/or Erector to advise the cost and schedule impact that the Revision will have on the Contract. Upon mutual agreement, and the Client's acceptance of the cost and schedule impact, the Fabricator and/or Erector will proceed with the Revision to the Work.

3.4 Discrepancies

Unless otherwise stated in the Construction Documents, the Structural Design Documents and Construction Specifications for buildings, the Construction Specifications govern. For bridges, the Structural Design Documents govern over Construction Specifications. In case of discrepancies between the Structural Design Documents and Design Documents for other trades or disciplines, the Structural Design Documents shall govern. When it has been agreed to use an electronic Building Information Model (BIM) as part of the Construction Documents, the BIM model shall govern for dimensions and geometry, while drawings shall govern for section sizes.

3.5 Computation of Units and Mass

Unless another method is specified and fully described at the time Tenders are requested, the computed mass of steel required for the structure shall be determined by the method of computation described herein. (Although the method of computation described does not result in the actual mass of fabricated Structural Steel and other items, its relative simplicity results in low computational cost and is based on quantities which can be readily computed and checked by all parties involved to establish the basis of payment). No additional mass for welds or mass allowance for painting, galvanizing, and metallizing is to be included in the computation of mass.

a) *Mass Density.* The mass density of steel is assumed to be 7850 kilograms per cubic metre.

b) *Shapes, Bars and Hollow Structural Sections.* The mass of shapes, bars and hollow structural sections is computed using the finished dimensions shown on shop details. No deductions shall be made for holes created by cutting, punching or drilling, for material removed by coping or clipping, or for material removed by weld joint preparation. No cutting, milling or planning allowance shall be added to the finished dimensions. The mass per metre of length for shapes and hollow structural sections is the nominal published mass. The mass per metre of length for bars is the published mass, or if no mass is published, the mass computed from the specified cross-sectional area.

c) *Plates and Slabs.* The mass/area of plates and slabs is computed using the rectangular dimensions of plates or slabs from which the finished plate or slab pieces shown on the shop details can be cut. No burning, cutting, trimming or planning allowance shall be added.

   Only when it is practical and economical to do so, and the nesting configuration is agreed to between the Fabricator and/or Erector and the Client in advance of fabrication (or defined clearly in the Tender Documents), several irregularly-shaped pieces may be cut from the same plate or slab. In this case, the mass shall be computed using the rectangular dimensions of the plate or slab from which the pieces can be cut. No cutting or trimming allowance shall be added. In all cases, the specified plate or slab thickness is to be used to compute the mass. The mass of raised-pattern rolled plate is that published by the manufacturer.

d) *Bolts.* The mass of shop and field bolts, nuts and washers is computed on the basis of the Shop Details and/or Erection Documents and the nominal published mass of the applicable types and sizes of fastener.

e) *Studs.* Unless included in the contract on a "price-per-unit basis", the mass of studs is computed on the basis of the Shop Details and/or Erection Diagrams and the published mass of the studs.

f) *Grating.* The mass/area of grating is computed on the basis of the Shop Details and/or Erection Documents, and the published mass of the grating. The area to be used is the minimum rectangular area from which the piece of grating can be cut.

g) Where supplied, such items as shims, levelling plates, temporary connection material, back-up bars and certain field "consumables" shall be considered as part of the Structural Steel whether or not indicated specifically in the Contract Documents. Such items then will be added to, and become a part of, computed mass of steel for the structure.

3.6 Contract Price Adjustments by Unit Price

3.6.1 Lump Sum Price Contracts

When the responsibility/scope of the Fabricator and/or Erector is changed from that which was previously established by the Contract Documents, an appropriate modification of the contract price shall be made and specified in a Lump Sum Contract; prices for additions or deletions of materials to the Work may be made on a unit-price basis. In computing the Contract price adjustment, the Fabricator and/or Erector shall consider the quantity of Work added or deleted, modifications in the character of the Work, the timeliness of the change with respect to the status of material ordering, the detailing, fabrication and erection operations, and related impact costs. A suggested format for application of Unit Rates for changes to Work is provided in Appendix B.

3.6.2 Requests for contract price adjustments shall be presented by the Fabricator and/or Erector and shall be accompanied by a description of the change in sufficient detail to permit evaluation and prompt approval by the Client.

3.6.3 Unit-Price Contracts

Generally they provide for minor revisions to the quantity of Work prior to the time Work is approved for construction. Minor revisions to the quantity of Work should be limited to an increase or decrease in the quantity of any category not exceeding ten percent. For Unit-Price Contracts, should the quantity of steel of any category vary by more than twenty percent, then the contract unit price of that category may require adjustment. Changes to the character of the Work or the mix of the Work, at any time, or changes to the quantity of the Work after the Work is approved for construction, may require a contract price adjustment. The unit-price cost of an item subject to changes made after the date of approved Issued–for-Construction Documents shall be evaluated based on the Fabricator's Work in progress at the time of the change, as described in Appendix B.

3.6.4 A suggested format for accommodating contract price adjustments is contained in Appendix B.

3.7 Scheduling

3.7.1 The Contract Documents should specify the schedule for the performance of the Work. This schedule should state when the approved Issued-for-Construction Documents will be issued, and when Shop Details will be submitted and returned from Client review, when the job site, foundations, cores, walls, piers and abutments will be ready, free from obstructions and accessible to the Erector, so that erection can start at the designated time and continue without interference or delay caused by the Client or other trades.

3.7.2 The Fabricator and/or Erector has the responsibility to advise the Client of the effect any revision may have on the Contract schedule.

3.7.3 If the fabrication and erection schedule is significantly delayed due to revisions, or for other reasons which are the Client's responsibility, the Fabricator and/or Erector shall advise the Client in accordance with the requirements of the Contract and the Contract schedule, and the price shall be adjusted as applicable.

4. Contract Documents

4.1 Tender Documents – Tender Drawings and Tender Specifications

4.1.1 At the time tenders are called, the steel Fabricator shall receive a complete set of Tender Documents. In order to ensure adequate and complete tenders for Lump Sum Price Contracts[1], these documents shall include, at minimum, complete Structural Design Documents

[1] *For other types of contracts, it is desirable for the contract documents to be as complete as possible.*

conforming to the requirements for design drawings established in CSA S16, *Design of steel structures* or S6 *Canadian highway bridge design code*, as applicable. Structural Steel Construction Specifications should include any special requirements controlling the fabrication and erection of the Structural Steel, surface preparation and coating, and should indicate the extent of non-destructive examination, if any, to be carried out.

4.1.2 Design drawings shall be drawn to a scale adequate to convey the required information. The drawings shall show a complete design of the structure with members suitably designated and located, including such dimensions and detailed description as necessary to permit the preparation of Fabrication and Erection Documents. Floor levels, column centres, and offsets shall be dimensioned. The term "drawings" may include computer output and other data. Stiffeners and doubler plates required to maintain stability and which are an integral part of the main member shall be shown and dimensioned.

4.1.3 Structural Design Documents shall designate the design standards used, shall show clearly the type or types of construction to be employed, shall show the category of the structural system used for seismic design, and shall designate the material or product standards applicable to the members and details depicted. Drawings shall give the governing combinations of shears, moments, pass-through forces, and axial forces to be resisted by the connections. Refer to CSA S16, *Design of steel structures* or S6 *Canadian highway bridge design code* for mandated requirements.

4.1.4 Where connections are not shown, the connections shall be assumed to be in accordance with the requirements of the governing technical standard/code (see Clause 1.3). The Tender Documents shall clearly define the scope of Work with respect to the responsibility to design Structural Steel connections. If the Work includes design of Structural Steel connections, the Tender Documents must include all connections forces as required by CSA S16, *Design of steel structures* or S6 *Canadian highway bridge design code.* Refer to the applicable standard for mandated requirements.

4.2 Architectural, Electrical and Mechanical Drawings

Architectural, electrical, additional specialty consultant, and mechanical drawings may be used as a supplement to the Structural Design Documents to define detail configurations and construction information, provided all requirements for the Structural Steel are noted on the Structural Documents. Refer to the applicable standard for mandated requirements.

4.3 IFC Construction Documents

4.3.1 At the time specified in the Tender Documents or pre-award negotiations (if different), the Client shall furnish the Fabricator and/or Erector with a plot plan of the construction site, and a set of complete Issued-for-Construction Documents approved for construction consistent with the Tender Documents and any addenda or revisions thereto. These Issued-for-Construction Documents are required by the Fabricator and/or Erector for ordering the material and for the preparation and completion of fabrication and erection documents. The Issued-for-Construction Documents shall conform to the requirements of CSA S16, *Design of steel structures* or S6 *Canadian highway bridge design code* and shall show the following:

a) The complete design of the structure with members suitably designated and located, including such dimensions and detailed description as necessary to permit preparation of the Fabrication and Erection Documents. Floor levels, column centres, and offsets shall be dimensioned;

b) All Revisions from the Tender Documents clearly indicated on the IFC Construction Documents

c) All materials to be furnished by the Fabricator, together with sufficient information to prepare Fabrication and Erection Documents, including the design standards used, the type or types of construction to be employed, the category of the system used for seismic design, the applicable material or product standards, and the governing combinations of shears, moments and axial forces to be resisted by connections. Refer to the applicable standard for mandated requirements.

4.4 Architecturally Exposed Structural Steel

In addition to the preceding requirements, all structural elements, or parts thereof, to be treated as Architecturally Exposed Structural Steel must be in accordance with the requirements of Appendix I and clearly indicated on the Structural Design Documents.

4.5 Building Information Digital Modelling

4.5.1 When a project utilizes BIM as part of the Structural Design Documents, Appendix J shall be used as a guide to define the wording, extents and deliverables of BIM in Contract Documents.

4.5.2 The designated Owner of each digital model shall be responsible for the accuracy and maintenance of the model, unless otherwise stated in the Contract Documents.

4.5.3 The Contract shall clearly stipulate the party designated as the Owner of each Building Information Model to be used as part of the Contract Documents.

5. Fabrication and Erection Documents

Note: The term "shop drawings", frequently used in the construction industry, is replaced in this Code of Standard Practice by the terms "Fabrication and Erection Documents". These terms more correctly describe the following five separate and distinct documents that may be prepared by a Fabricator/Erector. See also Clause 1.2 for definitions. Not all of these documents will be required for every project.

5.1 Erection Diagrams

Unless provided by the Client, the Fabricator will prepare Erection Diagrams from the approved Issued-for-Construction Documents. In this regard, the Fabricator may request reproducible copies of the Structural Design Documents which may be altered for use as Erection Diagrams. When using reproducible copies of the Structural Design Documents, the Engineer of Record's name and seal shall be removed. Erection Diagrams shall be submitted to the Designer for review and approval. Erection Diagrams are general arrangement drawings showing the principal dimensions of the structure, piece marks, sizes of the members, size (diameter) and type of bolts, bolt installation requirements, elevations of column bases, all necessary dimensions and details for setting anchor rods, and all other information necessary for the assembly of the structure. Only one reproducible copy, or electronic file, of each diagram will be submitted for review and approval, unless a BIM or a larger number of copies is required by the Client as specified in the Tender Documents.

5.2 Connection Design Details

5.2.1 When so specified in the Contract Documents, Connection Design Details shall be prepared in advance of Shop Details and submitted to the Engineer of Record for confirmation that the intent of the design is met. Connection Design Details shall provide details of standard and non-standard connections, and other data necessary for the preparation of Shop Details. Connection Design Details shall be referenced to the Design Drawings and/or Erection Diagrams. In the event that the design of connections for Structural Steel is the responsibility of the Fabricator, and the Fabricator's Connection Design Details meet the requirements of the Contract and the governing technical standard, any change to the Fabricator's Connection Design Details required by the Engineer of Record shall be considered as a Revision to the scope of Work.

5.2.2 Clipped Double Connections

Where two beams or girders, framing at right angles from opposite sides of a supporting member, share the same bolts, a clipped double connection shall be used unless a seated connection or other detail is used to facilitate safe erection of the beams or girders. A clipped double connection is not applicable to a two-bolt connection or when the beams are equal to or deeper than half the depth of the girder. For a description of a clipped double connection, see Appendix A.

5.3 Shop Details

Unless provided by the Client, Shop Details shall be prepared in advance of fabrication from the information on the approved Issued-for-Construction Drawings, the Connection Design Details, and the Erection Diagrams. Shop Details shall provide complete information required by the Fabricator to complete the fabrication of various members and components of the structure, including the required material and product standards; the location, type, and size of all attachments, mechanical fasteners, and welds. When Shop Details are required to be submitted for review and approval, only one reproducible copy of each Shop Detail will be submitted, unless a digital file or a larger number of copies is required by the Client as part of the Tender Documents. If mentioned in Contract Documents, shop drawing approval can be done using an appropriate BIM approval tool.

5.3.1 Shop Details Furnished by the Client

When the Shop Details are furnished by the Client, he shall deliver them in time to permit fabrication to proceed in an orderly manner according to the time schedule agreed upon. The Client shall prepare these Shop Details, insofar as practicable, in accordance with the detailing standards of the Fabricator. The Client shall indicate, in the Tender Documents, if the BIM and digital manufacturing data will be made available to the Fabricator, and if so, the digital file format that will be provided. The Client shall be responsible for the completeness and accuracy of Shop Details so prepared, and accuracy of the BIM model and digital manufacturing data.

5.4 Erection Procedures

Erection Procedures shall outline the construction methods, erection sequence, Erection Bracing, Temporary Bracing if required, and other engineering details necessary for shipping, erecting, and maintaining the stability of the steel frame; they shall be prepared in accordance with CSA S16, *Design of steel structures* or S6 *Canadian highway bridge design code*. Erection Procedures shall be supplemented by drawings and sketches to identify the location of stabilizing elements. Erection Procedures shall be submitted for review when so specified.

5.5 Field Work Details

Field Work Details shall be prepared in accordance with CSA S16, *Design of steel structures* or S6 *Canadian highway bridge design code* and submitted to the designer for review and approval. Field Work Details shall provide complete information for modifying fabricated members on the job site. All operations required to modify the member shall be shown on the Field Work Details. If extra materials are necessary to make modifications, Shop Details shall be required.

5.6 Fabrication and Erection Document Review

Erection Diagrams, non-standard Connection Design Details, Shop Details, and Field Work Details are normally submitted for review by the Engineer of Record. The duration required for such review shall be stated in the Tender Documents so that the Fabricator can prepare his schedule accordingly. Review of submitted documents by the Engineer of Record indicates that the Fabricator has interpreted correctly the design and Construction requirements. Connection Design Details and Shop Details are reviewed by the Engineer of Record for structural adequacy and to ensure conformance with the loads, forces and special instructions contained in the Structural Design Documents. Review by the Engineer of Record of Shop Details submitted by the Fabricator does not relieve the Fabricator of the responsibility for accuracy of the detail dimensions on Shop Details, nor of the general fit-up of parts to be assembled.

5.7 Additions, Deletions or Changes

Additions, deletions or changes, when approved, will be considered as Contract revisions and constitute the Client's authorization to release the additions, deletions or revisions for construction. See also Clauses 3.3 and 3.6.

5.8 Fabricator Models

When a Fabricator uses self-prepared three-dimensional software (BIM) specifically for his Work, the Fabricator owns the model and data.

6. Material, Fabrication, Inspection, Painting and Delivery

6.1 Quality Certification

For projects requiring a demonstrated level of quality control, CISC Certification of Steel Structures or CISC Certification of Steel Bridges may be specified.

CISC Certification is a third-party audited quality certification program specific to the fabrication of steel structures or steel bridges.

6.2 Materials

Materials used by the Fabricator for structural use shall conform to those listed in CSA S16, *Design of steel structures* or *S6 Canadian highway bridge design code*, or to other published material specifications, in accordance with the requirements of the Construction Documents.

6.3 Identification

The method of identification stipulated in CSA S16, *Design of steel structures* or S6 *Canadian highway bridge design code* shall form the basis for a Fabricator's identification of material. Control and identification procedures may differ to some extent from Fabricator to Fabricator.

6.4 Preparation of Material

Preparation of Material shall conform to the requirements of CSA S16, *Design of steel structures* or S6 *Canadian highway bridge design code.* Flame or plasma cutting of Structural Steel may be done by hand, by mechanically guided means, or automatically as permitted by the applicable governing Code.

6.5 Fitting and Fastening

6.5.1 Projecting elements of connection attachments need not be straightened in the connecting plane if it can be demonstrated that installation of the connectors or fitting aids will provide adequate contact between faying surfaces.

6.5.2 When runoff tabs are used, the Fabricator and/or Erector need not remove them unless specified in the Structural Design Documents, required by the governing technical Code or the steel is exposed to view. When their removal is required, they may be hand flame-cut close to the edge of the finished member with no more finishing required, unless other finishing is specifically called for in the Structural Design Documents or the governing technical Code.

6.6 Dimensional Tolerances

Tolerances on fabricated members shall be those prescribed in CSA S16, *Design of steel structures* or S6 *Canadian highway bridge design code*, as applicable. Tolerances on steel material supplied by the Fabricator shall meet those prescribed in CSA Standard G40.20 or the applicable ASTM Standard.

6.7 Inspection of Steelwork

Should the Client wish to have an independent inspection and/or non-destructive examination of the steelwork, he shall reserve the right to do so in the Tender Documents. Inspections shall be coordinated between the Fabricator and/or Erector and the Client's inspector. Inspectors are to be appointed prior to the start of fabrication, and the Client is to advise the Fabricator of the arrangement made. The cost of this inspection and testing is the responsibility of the Client. Deficiencies in the Work of the Fabricator and/or Erector requiring re-inspection or re-testing shall have costs borne by the Fabricator and/or Erector. Third-party inspectors shall be duly certified and have sufficient experience for the type of inspection performed.

The Fabricator and/or Erector is responsible for providing a conforming product through internal inspection, quality control, quality assurance and any other means necessary. The Fabricator and/or Erector's personnel used for internal visual inspection, QC or QA shall not be required to hold a visual certification to a National Standard, provided the company has assessed their competency for the Work performed.

The Canadian Welding Bureau Letter of Validation is proof that the Fabricator and/or Erector is certified for welding to CSA Standard W47.1. The applicable welding procedure standards, welding procedure data sheets, and personnel qualifications shall be available for review and verification by the Client or his representative at the place of Work, and are not intended for submission to the Client.

6.8 Surface Preparation

Unless required for a specified coating system, fabricated steelwork will not be cleaned. Surface preparation for a specified coating system shall be described in the Structural Design Documents.

If paint is specified, the Fabricator shall clean all steel surfaces to be painted of loose rust, loose mill scale, prominent spatter, slag or flux deposit, oil, dirt and other foreign matter by wire brushing or other suitable means. Unless specified in the Construction Documents, the Fabricator shall not be obliged to blast-clean, pickle or perform any specific surface preparation operation aimed at total or near-total removal of tight mill scale, rust or non-deleterious matter.

6.9 Paint Coatings

When Structural Steel is specified to receive a shop coating, the coating requirements specified in the Tender Documents shall include the identification of the members to be painted, surface preparation, application specification, the manufacturer's product identification, and the required minimum (and maximum) dry film thickness, if required. The Fabricator shall be responsible only to the extent of performing the surface preparation and painting in the specified manner. To the extent that the Fabricator has met these requirements, the Fabricator is not responsible for the performance of the specified coating system in the service conditions and duration to which the steelwork is exposed.

The expected performance of steel with a shop coat of primer depends on the environment. The primer will provide temporary limited corrosion protection to the steel in an essentially non-corrosive atmosphere for durations not exceeding 6 or 12 months for a CISC/CPMA 1-73a or CISC/CPMA 2-75 primer respectively, or according to the manufacturer's specifications and limitations. These durations apply to installed steel or steel that is not subjected to a corrosive environment in its erected state. Uninstalled steel stored flat with the potential for water accumulation on horizontal surfaces may, in some situations, be considered a corrosive environment. The presence of minor rust bleed-through, especially between unpainted faying surfaces, is not to be considered as a failure of the paint system and is not a cause for rejection or corrective action by the Fabricator.

Unless otherwise specified, coating systems applied by steel Fabricators are for temporary corrosion protection and are not intended for esthetic or final architectural purposes. For complex anti-corrosive multi-coat industrial coating systems or architecturally exposed Structural Steel paint systems, the Fabricator's inspection and test plan for coating applications shall be approved by the Client prior to commencement of the Work. The use of samples may be agreed upon as acceptance criteria. The Client is required to approve the coating application process on an ongoing basis throughout the execution of the project.

6.10 Marking and Shipping

6.10.1 Except for weathering steel surfaces exposed to view and for architecturally exposed Structural Steel (AESS) (see also Appendix I), erection marks shall be painted or otherwise legibly marked on the members. Preferably, members which are heavy enough to require special erection equipment shall be marked to indicate the computed or scale mass, and the centre of gravity for lifting.

6.10.2 Bolts of the same length and diameter, and loose nuts and washers of each size shall be packaged separately. Pins, bolts, nuts, washers, and other small parts shall be shipped in boxes, crates, kegs or barrels, none of which are to exceed 135 kg gross mass. A list and description of material contained therein shall be marked plainly on the outside of each container.

6.10.3 When requested by the Erector, long girders shall be loaded and marked so that they will arrive at the job site in position for handling without turning. Instructions for such delivery shall be given to the carrying agency when required.

6.10.4 For each shipment, the Fabricator shall furnish a shipping bill listing the items in the shipment. Such bill shall show the erection mark, the approximate length, the description (whether beam, column, angle, etc.) of each item. Such bill shall be signed by the receiver and returned to the Fabricator within 48 hours of receipt of the shipment with a note regarding shortages or damages, if any, and the bill shall act as a receipt for the shipment. When the shipments are made by truck transport, the bills should accompany the shipment. When shipments are made by rail or water, the bills shall be sent to the receiver to arrive on or before receipt of the shipment.

6.10.5 Unless otherwise specified at time of tender, steel during shipment will not be covered by tarpaulins or otherwise protected. When such protection is specified, the shipper is to notify the carrier of the protection requirements.

6.11 Delivery of Materials

6.11.1 Fabricated Structural Steel shall be delivered in a sequence which will permit the most efficient and economical performance of shop fabrication and erection. If the Client contracts separately for delivery supply and erection, he must coordinate planning between the Fabricator, Erector and General Contractor as applicable.

6.11.2 Anchor rods, washers and other anchorages, grillages, or materials to be built into masonry or concrete should be shipped so that they will be on hand when needed. The Client must give the Fabricator sufficient notice to permit fabrication and shipping of materials before they are needed.

6.11.3 The size and mass of Structural Steel assemblies may be limited by the shop capabilities, the permissible mass and clearance dimensions of available transportation or government regulations, and the job site conditions. The Fabricator determines the number of field splices consistent with economy. The Engineer of Record shall review and accept splice locations prior to implementation.

6.11.4 On supply-only Contracts, the unloading of steel is the responsibility of Others. Unless stated otherwise, the unloading of steel is part of the steel erection.

7. Erection

7.1 Method of Erection

Unless otherwise specified or agreed upon, erection shall proceed according to the most efficient and economical method available to the Erector on the basis of continuous operation consistent with the Construction Documents.

7.2 Erection Stability

7.2.1 Design

7.2.1.1 The Engineer of Record shall identify the following in the Tender Documents:

a) The lateral-load-resisting system and connecting diaphragm elements that provide for lateral strength and stability in the completed structure.

b) Any special erection conditions or other considerations that are required by the design concept, such as the use of shores, jacks, or loads that must be adjusted as erection progresses to set or maintain camber, position within specified tolerances, or pre-stress.

7.2.1.2 The General Contractor shall indicate to the Fabricator and/or Erector the general construction execution plan, including the installation schedule for non-structural steel elements of the lateral-load-resisting system and connecting diaphragm elements. The General Contractor shall indicate requirements for Temporary Bracing to accommodate this plan.

7.2.1.3 Based upon the information provided in Sections 7.2.1 and 7.2.2, the Fabricator and/or Erector shall determine, furnish, and install all Erection Bracing required for the erection operation. This Temporary Bracing shall be sufficient to secure the skeletal Structural Steel framing or any portion thereof against loads that are likely to be encountered during erection, including those due to wind and those that result from erection operations.

7.2.1.4 The Fabricator and/or Erector need not consider loads during erection that result from the performance of Work by, or the acts of Others, except as specifically identified by the Engineer of Record and/or the General Contractor, nor those that are unpredictable, such as loads due to hurricane, tornado, earthquake, explosion, or collision.

7.2.1.5 Temporary Bracing that is required during or after the erection of the Structural Steel Frame, including steel deck, for the support of loads caused by non-Structural Steel elements, including cladding, interior partitions, and other such elements that will induce or transmit loads to the Structural Steel frame during or after erection, shall be the responsibility of the Engineer of Record or General Contractor, as applicable.

7.2.1.6 The Structural Steel Fabricator and/or Erector shall engage the steel deck contractor to provide a bundle layout (including locations and weights) for the landing of deck bundles based on the Structural Steel erection plan.

7.2.2 Steel Erection Execution

7.2.2.1 The Steel Erection Execution Plan provides for a sequentially erected structure. The full stability of the structure is not achieved until all of the lateral support systems are in place. Proceeding with subsequent non-structural construction prior to completion shall be at the instruction and sole risk of the General Contractor, who shall make the Erector aware of the special provisions in place to accommodate any collateral building loads.

7.2.2.2 The instruction/request to proceed with the Structural Steel erection and steel deck installation will be given by the General Contractor following agreement between all parties that the following events have taken place.

a) The Erection Diagrams and steel deck drawings have been reviewed by the Engineer of Record.

b) The Steel Erection Execution Plan has been reviewed by the General Contractor and approved in principle for compliance with his construction execution plan.

At this time, a formal review shall be completed by all parties, and Work may proceed.

7.2.2.3 During the construction period, any other trade contractor placing a load on a steel framing member shall ensure that the load is distributed so as not to exceed the carrying capacity of the subject steel framing member.

7.2.2.4 Prior to placement of steel deck bundles, communication between the General Contractor, the steel deck installer, and the structural Fabricator and/or Erector has taken place to ensure that all requirements of the Steel Erection Execution Plan have been met and it is agreed that the structure (full or partial) is ready to accept the construction loads of the steel deck.

7.2.2.5 Once the deck installation and required inspections have been completed, and deficiencies addressed, responsibility for structural stability is assumed by the General Contractor.

7.2.2.6 The erection execution plan may be modified, and costs accommodated, to suit specific project requirements, pre- or post-bid, providing the Owner's designated representative for construction has clearly stated these requirements, and they may be accomplished in a safe manner.

Temporary Bracing of the steel frame shall only be removed on instruction from the Engineer of Record.

7.3 Erection Safety

Erection shall be done in a safe manner and in accordance with applicable provincial legislation.

7.4 Site Conditions

The Client shall provide and maintain adequate, all-weather access roads cleared of snow and ice and other material that impedes entry into and through the site for the safe delivery of derricks, cranes, other necessary equipment, and the material to be erected. The Client shall provide for the Erector a firm, properly graded, drained, convenient and adequate space and laydown area for steel of sufficient load-carrying capacity at the site for the operation of erection equipment, and shall remove at the Client's cost all overhead obstructions such as power lines, telephone lines, etc., in order to provide a safe and adequate working area for erection of the steelwork. The Erector shall provide and install the safety protection required for his own operations or for his Work forces to meet the safety requirements of applicable Acts or Codes. The General Contractor shall install protective covers on all protruding rebar, machinery, anchor rods, etc., which are a hazard to workers and shall be installed by other trades prior to commencement of steel erection. Any protection for pedestrians, property, other trades, etc., not essential to the steel erection activity is the responsibility of the Client. When the structure does not occupy the full available site, the Client shall provide adequate storage space to enable the Fabricator and Erector to operate at maximum practicable speed and efficiency. Cleaning of steelwork required because of site conditions, mud, site worker traffic, etc., shall not be to the Fabricator's and/or Erector's account.

7.5 Foundations

Neither the Fabricator nor the Erector shall be responsible for the accurate location, strength and suitability of foundations.

7.6 Bearing Surfaces

Levelling plates shall be set by other trades true, level and to the correct elevation.

7.7 Building Lines and Bench Marks

The Erector shall be provided with a plot plan accurately locating building lines and bench marks at the site of the structure. A survey bench mark establishing elevation and horizontal coordinates shall be provided by the Client at the site.

7.8 Installation of Anchor Rods and Embedded Items

7.8.1 Anchor rods and foundation rods shall be set by others in accordance with the Construction Documents. They must not vary from the dimensions shown on the Construction Documents by more than the following (see also Appendix D):

a) 3 mm centre-to-centre of any two rods within an anchor rod group, where an anchor rod group is defined as the set of anchor rods which receives a single fabricated steel shipping piece; 6 mm centre-to-centre of adjacent anchor rod groups;

b) Maximum accumulation of 6 mm per 30 000 mm along the established column line of multiple anchor rod groups, but not to exceed a total of 25 mm. The established column line is the actual field line most representative of the centres of the as-built anchor rod groups along a line of columns;

c) 6 mm from the centre of any anchor rod group to the established column line through that group. Shims: the finished tops of all footings shall be at the specified level which will not exceed the maximum specified grouting allowance to predetermine the amount of shimming that will be required.

The tolerances of paragraphs (a), (b), and (c) also apply to offset dimensions, shown on the Construction Documents, measured parallel and perpendicular to the nearest established column line for individual columns shown on the drawings to be offset from established column lines.

7.8.2 Unless shown otherwise, anchor rods shall be set perpendicular to the theoretical bearing surface, threads shall be protected, free of concrete, and nuts should run freely on the threads. Shear pockets shall be cleaned of debris, formwork, ice and snow by the Client prior to steel erection.

7.8.3 Other embedded items or connection materials between the Structural Steel and the Work of Others shall be located and set by Others in accordance with approved Construction Documents. Accuracy of these items must satisfy the erection tolerance requirements of Clause 7.12.

7.8.4 All Work performed by Others shall be completed so as not to delay or interfere with the erection of the Structural Steel.

7.9 Bearing Devices

The Client shall set to lines and grades all levelling plates and loose bearing plates. The Fabricator and/or Erector shall provide the wedges, shims or levelling screws that are required, and shall scribe clearly the bearing devices with working lines to facilitate proper alignment. Promptly after the setting of any bearing devices, the Client shall check lines and grades, and grout as required. The final location and proper grouting of bearing devices are the responsibility of the Client.

When steel columns, girders or beams which will be supported on concrete or masonry have base plates or bearing plates fabricated as an integral part of the member, the bearing area of the support shall be suitably prepared by Others so as to be at exact grade and level to receive the steelwork.

7.10 Site Errors or Discrepancies – Examination by Erector

The Erector shall report to the Client any errors or discrepancies in the Work of Others, as discovered, that may affect erection of Structural Steel before or during erection. The accurate placement and integrity of all anchor rods/embedment etc., remain the responsibility of the Client.

7.11 Adjustable Shelf Angles and Sash Angles

The Erector shall position at time of erection all adjustable shelf angles and sash angles attached to the steel frame true and level, within the tolerances permitted by the governing technical standard. Any subsequent adjustment that may be necessary to accommodate the Work of Others shall be performed by other trades.

7.12 Tolerances

Unless otherwise specified, tolerances on erected Structural Steel shall be those prescribed in CSA S16, *Design of steel structures* or S6 *Canadian highway bridge design code* as applicable.

7.13 Checking Erected Steelwork

Prior to the placement or applying of any other material of any other trades, the Client shall:

- Confirm with the Erector that the structure is complete and conforming to the Construction Documents, and
- Confirm that any third-party inspection and testing and necessary corrective action have been completed, and
- Ensure that the Erector is given timely notice of acceptance by the Client or a listing of specific items to be corrected in order to obtain acceptance, and
- Ensure such notice is rendered immediately upon completion of any part of the Work and prior to the start of Work by other trades that may be supported, attached or applied to the structural steelwork.

Should such notice not be received within 14 days, or the Client commences use, occupancy, or improvement to the steelwork, then the Work is taken to have been accepted.

The Erector is not responsible for determining or effecting the stability of the structure due to temporary loads resulting from construction activities of Others.

7.14 Removal of Bracing

7.14.1 Removal of Erection Bracing

Guys, braces and falsework or cribbing supplied by the Erector shall remain the property of the Erector. The Erector shall remove them when the steel structure is otherwise adequately braced, unless other arrangements are made. Guys and braces temporarily left in place under such other arrangements shall be removed by Others, provided prior permission by the Erector for their removal has been given and they are returned to the Erector in good condition. See Clause 7.14.2.

7.14.2 Removal of Temporary Bracing

Temporary Bracing required by the structural designer shall only be removed on instruction from the Engineer of Record.

7.15 Correction of Errors When Material Is Not Erected by the Fabricator

Correction of minor misfits and a moderate amount of cutting, welding, and reaming for the project as a whole shall be considered a part of the erection, in the same manner as if the Fabricator were erecting the Work. Any major rework required due to incorrect shop Work shall be immediately reported to the Fabricator before rework commences. The Fabricator shall then either correct the error, resupply the item within a reasonable time period, or approve the method of correction including applicable costs, whichever is the most economical. The definitions of major and minor rework should be agreed to prior to the commencement of the project.

7.16 Field Assembly

Unless otherwise specified, the Fabricator shall provide for suitable field connections that will, in his opinion, afford the greatest overall economy.

7.17 Accommodation of Other Trades

Neither the Fabricator nor the Erector shall cut, drill or otherwise alter the Work of Others or his own Work to accommodate other trades, unless such Work is clearly defined in the Structural Steel and Tender Documents, and detailed information is provided before the Erection Documents are approved. Any subsequent cutting, drilling or other alteration of the Structural Steel performed by the Fabricator or the Erector for the accommodation of other trades shall be specifically agreed upon and authorized by the Client before such Work is commenced.

7.18 Temporary Floors and Access Stairs

Unless otherwise required by law or in the Tender Documents, all temporary access stairs shall be provided by Others, except for the floor upon which erecting equipment is located. On this floor, the Erector shall provide such temporary flooring as he requires, moving his planking, etc., as the Work progresses.

7.19 Touch-Up of Shop Paint Coatings

Touch-up may also be required for unfinished field bolts or at masked connection areas. It is normal to expect that painted or coated Structural Steel surfaces will be subject to damage due to handling from loading, off- loading and installation, and due to abrasions during shipment. Unless so specified, the Fabricator and/or Erector will not perform any field coating touch-ups, spot-paint field fasteners and field welds, nor touch-up abrasions to the shop paint.

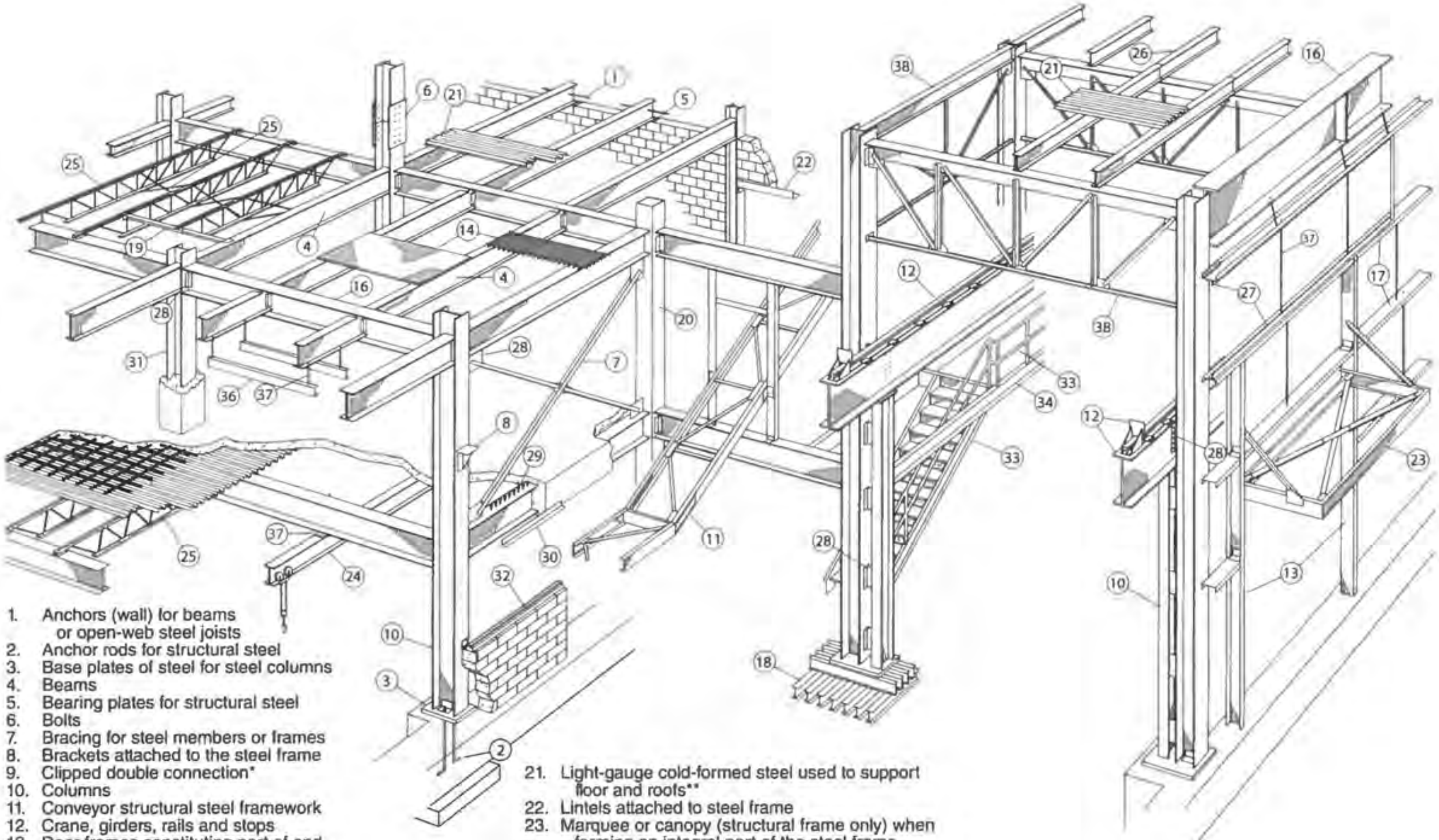
7.20 Final Painting

Unless so specified, the Fabricator and/or Erector will not be responsible for cleaning the steel after erection in preparation for field painting, nor for any general field painting that may be required.

7.21 Final Cleanup

Except as provided in Clause 7.14, upon completion of erection and before final acceptance, the Erector shall remove all falsework, rubbish and temporary building furnished by him.

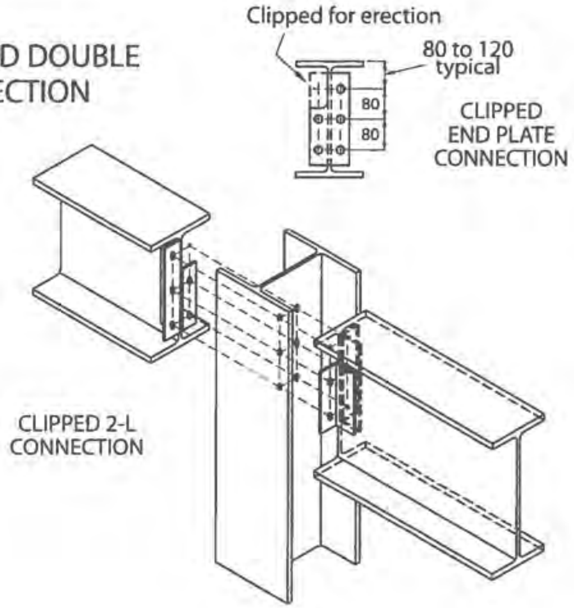
APPENDIX A – Structural Steel in Buildings



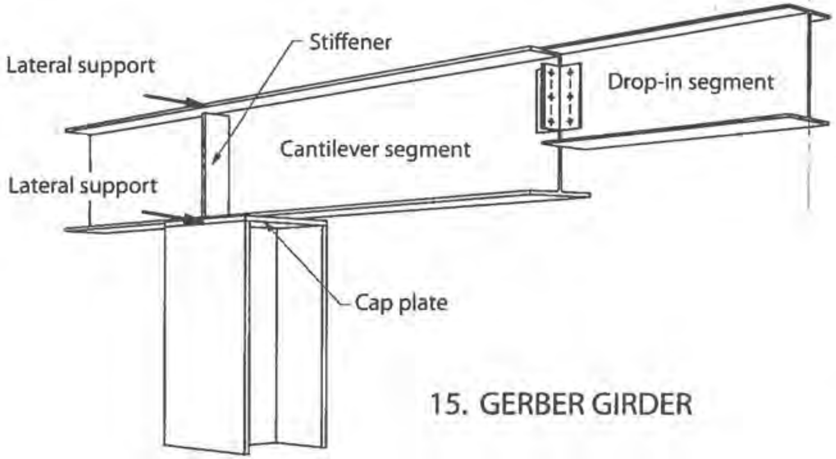
1. Anchors (wall) for beams or open-web steel joists
2. Anchor rods for structural steel
3. Base plates of steel for steel columns
4. Beams
5. Bearing plates for structural steel
6. Bolts
7. Bracing for steel members or frames
8. Brackets attached to the steel frame
9. Clipped double connection*
10. Columns
11. Conveyor structural steel framework
12. Crane, girders, rails and stops
13. Door frames constituting part of and connected to the steel frame
14. Floor and roof plates (raised pattern or plain), grating, connected to steel frame
15. Gerber girder*
16. Girders
17. Girts
18. Grillage beams of steel
19. Headers or trimmers for support of open-web steel joists where such headers or trimmers frame into structural steel members
20. Hollow structural section (HSS) column
21. Light-gauge cold-formed steel used to support floor and roofs**
22. Lintels attached to steel frame
23. Marquee or canopy (structural frame only) when forming an integral part of the steel frame
24. Monorail beams of standard structural shapes, attached to steel frame
25. Open-web steel joists, bridging and accessories when supplied with steel joists
26. Purlins
27. Sash angles connected to the steel frame
28. Separators, angles, tees, clips and other detail fitting essential to the structural steel frame
29. Shear connectors
30. Shelf angles attached to the steel frame
31. Steel cores for composite columns
32. Steel window sills attached to the steel frame
33. Steel stairs and handrails
34. Struts
35. Stub girders*
36. Suspended ceiling supports of structural steel shapes 75 mm or greater in depth
37. Ties, hangers and sag rods forming part of the structural frame
38. Trusses and braced frames

** See separate diagram* *** Supplied by others*

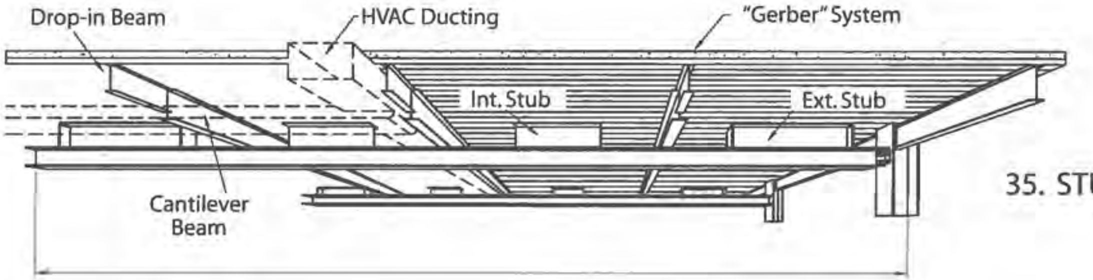
9. CLIPPED DOUBLE CONNECTION



15. GERBER GIRDER



35. STUB-GIRDER



APPENDIX B

Guideline for Unit Price Application for Changes

B1. Unit rates for Changes shall apply on their own, only up until commencement of material order or shop detail drawings, whichever is the earlier.

B2. It is accepted that Unit Rates for additions will be higher than for rates for deletions. Unit Rates for both additions and deletions should be requested in the Tender Documents if Unit Prices are to be used for the project.

B3. The following amounts, additional to the unit rate, shall be charged on additions at the various stages of the contract.

a) If the addition affects drawings (e.g. of support members) already in progress or complete, then the changes to such drawings or re-detailing shall be charged extra at an agreed hourly rate.

b) If the addition requires additional Work to material manufacture or erection (e.g. supporting members) in progress or complete, then such additional Work shall be charged extra at an agreed hourly rate.

c) "Detail" or "Connection" materials added to existing or supporting members, whether due to an additional member or not, shall be charged on a cost-plus basis.

d) If the timing of the addition causes the added material to be shipped as a part load, then transportation shall be charged extra at cost plus an agreed percentage markup.

B4. The following amounts, additional to the unit rate, shall be charged for deletions at the various stages of the contract.

a) If the deleted material has been ordered or delivered and cannot be used elsewhere, then a restocking charge shall be levied.

b) If the deleted member has been detailed or drawings are in progress, then the cost of such drawings shall be charged extra at an agreed hourly rate.

c) If the deletion affects drawings already completed or in progress, then the changes to such drawings or the re-detailing shall be charged extra at an agreed hourly rate.

d) If the deleted member has been manufactured or erected, or manufacture or erection is in progress, then the cost of such manufacture or erection shall be charged extra at an agreed hourly rate or lump sum cost.

e) If the deletion affects members already manufactured (e.g. supporting members), then the changes to such members shall be charged extra at an agreed hourly rate or lump sum cost.

f) If the deleted member has already been shipped, then no credit shall be given.

B5. All unit rates shall be applied in accordance with the CISC Code of Standard Practice, Clause 3.5.

B6. Hourly Rates for additions are as follows:

a) Engineering Design – $ / labour hour

b) Detailing Labour – $ / labour hour

c) Shop Labour – $ / labour hour

d) Field Labour – $ / labour hour

e) Equipment used for revisions will be charged at negotiated rental rates, according to Canadian Construction Association standard practice.

B7. Revisions involving the use of grades of steel, sources of supply, or types of sections other than specified will be subject to price adjustments.

B8. Mass will be computed in accordance with Clause 3.5 of the CISC Code of Standard Practice for Structural Steel.

APPENDIX C

A Suggested Format for Price-per-Unit Contracts

Category List C1

It is common practice in the industry to limit categories for structural steel to light, medium and heavy steel members. These very general categories require the Fabricator to make allowance for the very large degree of complexity that may be encountered in the final project design. This comprehensive category list removes variability of complexity from each category, enabling a more economical price evaluation for each category.

CAT NUM	CLASSIFICATION	PAY UNIT
	Columns and Beams - Rolled Sections	
100	0 to 15 kg/m	tonne
101	16 to 30 kg/m - 0-3 m	tonne
102	16 to 30 kg/m - 3-9 m	tonne
103	16 to 30 kg/m - >9 m	tonne
104	31 to 60 kg/m - 0-3 m	tonne
105	31 to 60 kg/m - 3-9 m	tonne
106	31 to 60 kg/m - >9 m	tonne
107	61 to 90 kg/m - 0-3 m	tonne
108	61 to 90 kg/m - 3-9 m	tonne
109	61 to 90 kg/m - >9 m	tonne
110	91 to 155 kg/m - 0-3 m	tonne
111	91 to 155 kg/m - 3-9 m	tonne
112	91 to 155 kg/m - >9 m	tonne
113	>155 kg/m - 0-3 m	tonne
114	>155 kg/m - 3-9 m	tonne
115	>155 kg/m - >9 m	tonne
	Columns and Beams - HSS/RHS Sections	
116	0 to 30 kg/m - 0-3 m	tonne
117	0 to 30 kg/m - 3-9 m	tonne
118	0 to 30 kg/m - >9 m	tonne
119	31 to 60 kg/m - 0-3 m	tonne
120	31 to 60 kg/m - 3-9 m	tonne
121	31 to 60 kg/m - >9 m	tonne
122	>60 kg/m - 0-3 m	tonne
123	>60 kg/m - 3-9 m	tonne
124	>60 kg/m - >9 m	tonne

APPENDIX C

A Suggested Format for Price-per-Unit Contracts

Category List C1 (Cont'd)

	Monorails and Crane Rails	
150	S Shapes - Straight - 0-30 kg/m	tonne
151	S Shapes - Straight - over 30 kg/m	tonne
152	S Shapes - Curved - 0-30 kg/m	tonne
153	S Shapes - Curved - over 30 kg/m	tonne
154	30 lb Crane Rail c/w Clips	tonne
155	60 lb Crane Rail c/w Clips	tonne
156	85 lb Crane Rail c/w Clips	tonne
	Bracing	
201	Rld Sec - 0 to 30 kg/m - <3 m	tonne
202	Rld Sec - 0 to 30 kg/m - 3-9 m	tonne
203	Rld Sec - 0 to 30 kg/m - >9 m	tonne
204	Rld Sec - >30 kg/m - <3 m	tonne
205	Rld Sec - >30 kg/m - 3-9 m	tonne
206	Rld Sec - >30 kg/m - >9 m	tonne
210	HSS Sec - 0 to 30 kg/m - <3 m	tonne
211	HSS Sec - 0 to 30 kg/m - 3-9 m	tonne
212	HSS Sec - 0 to 30 kg/m - >9 m	tonne
213	HSS Sec - >30 kg/m - <3 m	tonne
214	HSS Sec - >30 kg/m - 3-9 m	tonne
215	HSS Sec - >30kg/m - >9 m	tonne
220	WT Sec - 0 - 30 kg/m - <3 m	tonne
221	WT Sec - 0 - 30 kg/m - 3-9 m	tonne
222	WT Sec - 0 - 30 kg/m - >9 m	tonne
223	WT Sec - >30 kg/m - <3 m	tonne
224	WT Sec - >30 kg/m - 3-9 m	tonne
225	WT Sec - >30 kg/m - >9 m	tonne
	Built-Up Members	
250	3 Plate Girders <90 kg/m	tonne
251	3 Plate Girders 90-155 kg/m	tonne
252	3 Plate Girders >155 kg/m	tonne
260	Fireproofing Corner Angles	tonne
261	Continuous Support Angles for Deck, etc.	tonne
262	Bent Plates	tonne

APPENDIX C

A Suggested Format for Price-per-Unit Contracts

Category List C1 (Cont'd)

	Cold-Formed Channels and Z-Shapes	
301	0 - 5.75 kg/m - 0-3 m	tonne
302	0 - 5.75 kg/m - 3-9 m	tonne
303	0-5.75 kg/m - >9 m	tonne
304	> 5.75 kg/m - 0-3 m	tonne
305	> 5.75 kg/m - 3-9 m	tonne
306	> 5.75 kg/m - >9 m	tonne
320	Sag Rods - specify diameter and finish	tonne
	Connection Materials and Welding	
401	Welded Plates - Gusset Plates, Wrap Plates, Shear Tabs	tonne
402	Welded Plates - Moment Plates	tonne
403	Welded Plates - End Plates, Clip Angles	tonne
404	Welded Plates - Base/Cap Plates	tonne
405	Welded Plates - Stiffeners under W310	tonne
406	Welded Plates - Stiffeners W360 to W460	tonne
407	Welded Plates - Stiffeners W460 to W610	tonne
408	Welded Plates - Web Doubler Plates	tonne
409	Welded Plates - Shop Welded Lifting Lugs	tonne
410	Welded Plates - Bolted Lifting Lugs	tonne
411	Loose Plates - Field-Installed	tonne
412	Prepared Groove Welds	cm^3
413	Seal Welding	cm
414	Welded Shear Studs	ea
	Miscellaneous	
501	Stair Stringers	tonne
502	Shop Assembled Stairs - Stringers and Bolted Treads	tonne
503	Ladders (without safety cage)	tonne
504	Ladder (with safety cage)	tonne
505	Checkerplate: 6 mm thick - specify installation location and method	tonne
506	Checkerplate: 8 mm thick - specify installation location and method	tonne
507	Handrail (straight)	tonne
508	Handrail (sloped)	tonne
509	Handrail (circular)	tonne
510	Safety gates: Premanufactured	Ea
511	Safety gates: Steel Fabricated	Ea

APPENDIX C

A Suggested Format for Price-per-Unit Contracts

Category List C1 (Cont'd)

	Grating and Treads	
601	Stair Treads (specify Bearing Bar Size, tread size surface type, finish)	Ea
603	Grating (specify Bearing Bar Size, tread size surface type, finish)	m^2
605	Cold-Formed Walkway Channels	m
606	Cold-Formed Walkway Channel fasteners	Ea
607	Grating - Straight Banding (shop)	m
608	Grating - Circular Banding (shop)	m
609	Grating - Straight Toe Plate (shop)	m
610	Grating - Circular Toe Plate (shop)	m
611	Grating - Grating Clip (specify type)	Ea
612	Grating - Checkerplate Nose to Grating	m
	Welded Frames (2 or more shop-welded framing members)	
701	Members - 0 - 15 kg/m	tonne
702	Members - 16 - 30 kg/m	tonne
703	Members - 31 - 60 kg/m	tonne
704	Members - 61 - 90 kg/m	tonne
705	Members - 90 - 155 kg/m	tonne
	Bolts	
801	A307 16 mm (⅝) dia. (Black) or 10 mm (⅜) dia. (Plated) x length	Ea / tonne
802	A325 Bolt (Black): 20 mm (¾) dia. x length	Ea / tonne
803	A325 Bolt (Black): 22 mm (⅞) dia. x length	Ea / tonne
804	A325 Bolt (Black): 25 mm (1) dia. x length	Ea / tonne
805	A490 Bolt (Black): 32 mm (1¼) dia. x length	Ea / tonne
806	B307 (Button Head): 16 mm (⅝) dia. x length	Ea / tonne
	Hourly Rates for Extra Work	
	Extra Engineering Design	hour
	Extra Drafting Labour	hour
	Extra Shop Labour	hour
	Extra Field Labour	hour
	Extra Administration Labour	hour

APPENDIX C

A Suggested Format for Price-per-Unit Contracts

Category List C2

In the event that the comprehensive Category List C1 is deemed too onerous to manage, Category List C2 provides an alternate approach. The categories in this list include a greater variation in complexity but may be deemed easier to manage.

CAT NUM	CLASSIFICATION	PAY UNIT	Comments
	Columns and Beams - Rolled Sections		
100	0 to 15 kg/m	tonne	
101	16 to 30 kg/m	tonne	
104	31 to 60 kg/m	tonne	
107	61 to 90 kg/m	tonne	
110	91 to 155 kg/m	tonne	
113	>155 kg/m	tonne	
	Columns and Beams - HSS/RHS Sections		
116	0 to 30 kg/m	tonne	
119	31 to 60 kg/m	tonne	
122	>60 kg/m	tonne	
	Monorails and Crane Rails		
150	S Shapes – Straight - 0-30 kg/m	tonne	
151	S Shapes – Straight - over 30 kg/m	tonne	
152	S Shapes – Curved - 0-30 kg/m	tonne	
153	S Shapes – Curved - over 30 kg/m	tonne	
154	30 lb Crane Rail c/w Clips	tonne	
155	60 lb Crane Rail c/w Clips	tonne	
156	85 lb Crane Rail c/w Clips	tonne	
	Bracing		
201	Rld Sec - 0 to 30 kg/m	tonne	
204	Rld Sec - >30 kg/m	tonne	
210	HSS Sec - 0 to 30 kg/m	tonne	
213	HSS Sec - >30	tonne	
220	WT Sec - 0 - 30 kg/m	tonne	
223	WT Sec - >30 kg/m	tonne	

APPENDIX C

A Suggested Format for Price-per-Unit Contracts

Category List C2 (Cont'd)

	Built-Up Members		
250	3 Plate Girders <90 kg/m	tonne	
251	3 Plate Girders 90-155 kg/m	tonne	
252	3 Plate Girders >155 kg/m	tonne	
260	Fireproofing Corner Angles	tonne	
261	Continuous Support Angles for Deck, etc.	tonne	
262	Bent Plates	tonne	
	Cold-Formed Channels and Z-Shapes		
301	0 - 5.75 kg/m - 0-3m	tonne	
304	> 5.75 kg/m - 0-3m	tonne	
320	Sag Rods - specify diameter and finish	tonne	
	Connection Materials and Welding		
401	Welded Plates - Gusset Plates, Wrap Plates, Shear Tabs	tonne	
402	Welded Plates - Moment Plates	tonne	
403	Welded Plates - End Plates, Clip Angles	tonne	
404	Welded Plates - Base/Cap Plates	tonne	
405	Welded Plates - Stiffeners Under W310	tonne	
406	Welded Plates - Stiffeners W360 to W460	tonne	
407	Welded Plates - Stiffeners W460 to W610	tonne	
408	Welded Plates - Web Doubler Plates	tonne	
409	Welded Plates - Shop-Welded Lifting Lugs	tonne	
410	Welded Plates - Bolted Lifting Lugs	tonne	
411	Loose Plates - Field installed	tonne	
412	Prepared Groove Welds	cm^3	
413	Seal Welding	cm	
414	Welded Shear Studs	ea	

APPENDIX C

A Suggested Format for Price-per-Unit Contracts

Category List C2 (Cont'd)

	Miscellaneous		
501	Stair Stringers	tonne	
502	Shop Assembled Stairs - Stringers and Bolted Treads	tonne	
503	Ladders (without safety cage)	tonne	
504	Ladder (with safety cage)	tonne	
505	Checkerplate: 6 mm thick - specify installation location and method	tonne	Specify thickness, installation location, and method
507	Handrail (straight)	tonne	
508	Handrail (sloped)	tonne	
509	Handrail (circular)	tonne	
510	Safety gates: Premanufactured	Ea	
511	Safety gates: Steel Fabricated	Ea	
	Grating and Treads		
601	Stair treads (specify Bearing Bar Size, tread size surface type, finish)	Ea	
603	Grating (specify Bearing Bar Size, tread size surface type, finish)	m^2	Includes banding, kickplate, and fasteners. Details required.
605	Cold-Formed Walkway Channels	m	
606	Cold-Formed Walkway Channel Fasteners	Ea	
	Welded Frames (2 or more shop-welded framing members)		
701	Members - 0 - 15 kg/m	tonne	
702	Members - 16 - 30 kg/m	tonne	
703	Members - 31 - 60 kg/m	tonne	
704	Members - 61 - 90 kg/m	tonne	
705	Members - 90 - 155 kg/m	tonne	

APPENDIX C

A Suggested Format for Price-per-Unit Contracts

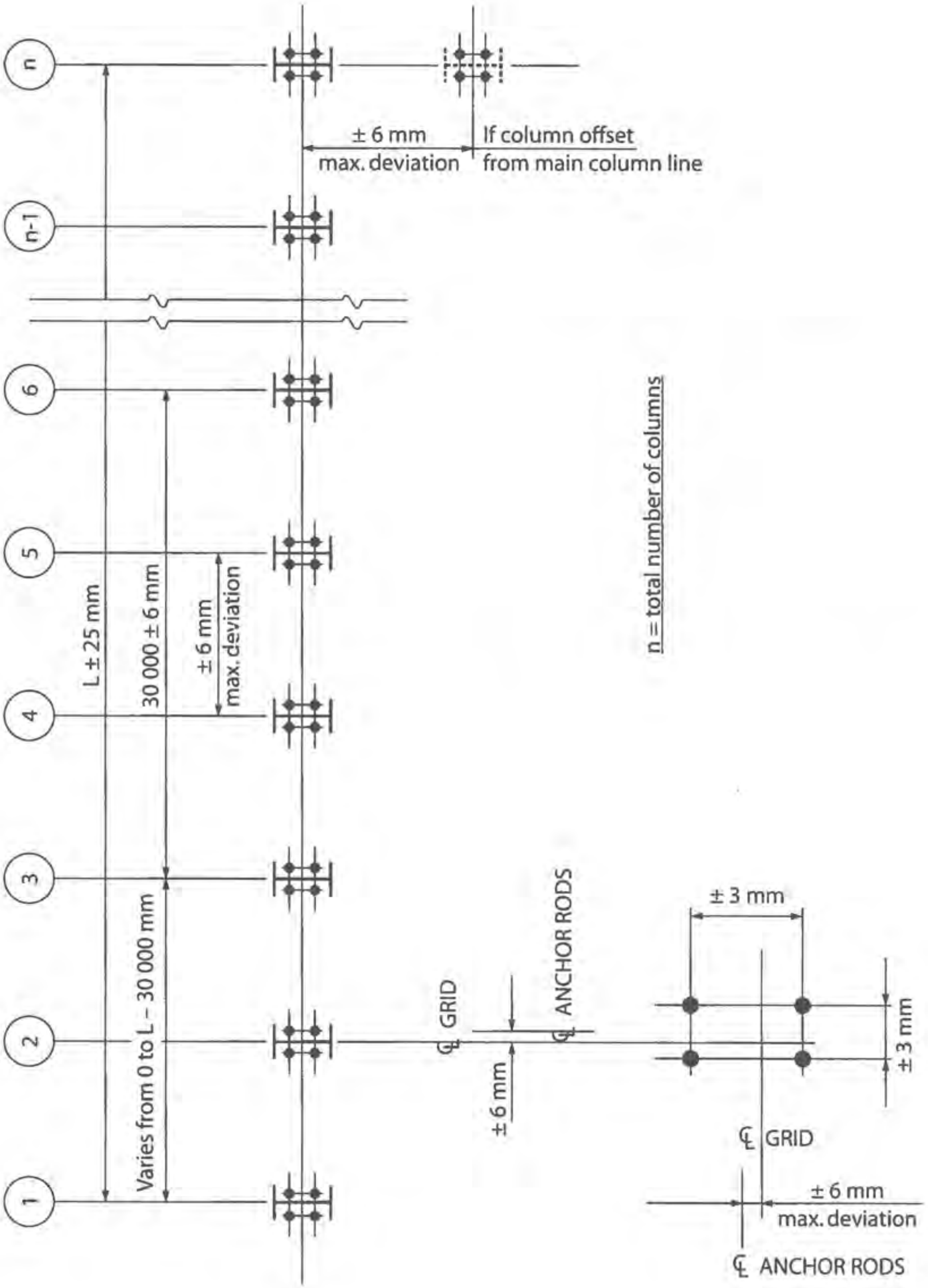
Category List C2 (Cont'd)

	Bolts		
801	A307 16 mm (⅝) dia. (Black) or 10mm (⅜) dia. (Plated) x length	Ea / tonne	
802	A325 Bolt (Black): 20 mm (¾) dia. x length	Ea / tonne	
803	A325 Bolt (Black): 22 mm (⅞) dia. x length	Ea / tonne	
804	A325 Bolt (Black): 25 mm (1) dia. x length	Ea / tonne	
805	A490 Bolt (Black): 32 mm (1¼) dia. x length	Ea / tonne	
806	B307 (Button Head): 16 mm (⅝) dia. x length	Ea / tonne	
	Hourly Rates for Extra Work		
	Extra Engineering Design	hour	
	Extra Drafting Labour	hour	
	Extra Shop Labour	hour	
	Extra Field Labour	hour	
	Extra Administration Labour	hour	

Note:

This Code of Standard Practice for Structural Steel (in PDF format) and the above Category Lists C1 and C2 (in Excel format) may be downloaded from the CISC website at this link: *www.cisc-icca.ca/solutions-centre/publications/publications*

APPENDIX D
Tolerances on Anchor Rod Placement



APPENDIX E

Conversion of SI Units to Imperial Units

When Imperial units are used in contract documents, unless otherwise stipulated, the SI units used in the CISC Code of Standard Practice for Structural Steel shall be replaced by the Imperial units shown, for the clause as noted.

Clause 3.5 (a). Unit Weight. The unit weight of steel is assumed to be 0.2833 pounds per cubic inch.

For other clauses, the standard conversion factors (for length, mass, etc.) stipulated in the CISC Handbook should be used.

Note: Imperial projects should be entirely in the imperial designation including shape sizes. Metric projects should be entirely in the SI designation, including shape sizes. Units should not be intermixed on the same project.

APPENDIX F

Miscellaneous Steel

Unless otherwise specified in the tender documents, the following items are considered miscellaneous steel of ferrous metal only, fabricated from 2.0 mm (14 ga.) and more of metal, including galvanizing, cadmium and chrome plating, but not stainless steel and cast iron items. This list of items is to be read in conjunction with Clause 2.1 Structural Steel and Clause 2.3 Items Supplied by Others, and shall include all steel items not included in Clauses 2.1 and 2.3, unless specified otherwise.

Access doors and frames — except trade-name items and those required for servicing mechanical and electrical equipment.

Angles and channel frames for doors and wall openings — drilling and tapping to be specified as being done by Others.

Benches and brackets.

Bollards, bumper posts and rails

Bolts — only includes those bolts and anchors required for anchoring miscellaneous steel supplied under this list.

Burglar/security bars.

Clothes line poles, custom-fabricated types only.

Coat rods, custom-fabricated types only.

Corner protection angles.

Expansion joint angles, plates custom-fabricated, etc., including types made from steel, or a combination of steel and non-ferrous metal.

Fabricated convector frames and enclosures.

Fabricated items where clearly detailed or specified and made from 2.0 mm (14 ga.) and heavier steel, except where included in another division.

Fabricated steel framing for curtain walls and storefronts where not detailed on structural drawings and not enclosed by architectural metal.

Fabricated wire mesh and expanded metal partitions and screens.

Fire escapes.

Flag poles — steel custom-fabricated. (Excluding hardware)

(Custom-fabricated) Footscrapers, mud and foot grilles, including pans, but less drains.

Frames, grating and plate covers for manholes, catch basins, sumps, trenches, hatches, pits, etc., except cast iron, frames and covers and trade-name floor and roof drains.

Gates, grilles, grillwork and louvres, excluding baked enamel or when forming part of mechanical system.

Grating-type floors and catwalks — excluding those forming part of mechanical system.

Handrails, balusters and any metal brackets attached to steel rail including plastic cover, excluding steel handrails forming part of structural steel framing.

Joist hangers, custom-fabricated types only.

Joist strap anchors.

Lintels, unless shown on structural drawings.

Mat recess frames, custom-fabricated types only.

Mobile chalk and tackboard frames, custom-fabricated types only.

Monorail beams of standard shapes, excluding trade-name items, unless shown on structural drawings.

Shop drawings and/or erection diagrams.

Shop preparation and/or priming.

Sleeves, if specified, except for mechanical and electrical division.

Stair nosings, custom-fabricated types only.

Steel ladders and ladder rungs not forming part of Structural Steel or mechanical work.

Steel stairs and landings not forming part of Structural Steel.

Table and counter legs, frames and brackets, custom-fabricated types only.

Thresholds and sills, custom-fabricated types only.

Vanity and valance brackets, custom-fabricated types only.

Weatherbars — steel.

Miscellaneous Steel Items Excluded

Bases and supports for mechanical and electrical equipment where detailed on mechanical or electrical drawings.

Bolts other than for anchoring items of miscellaneous steel.

Cast iron frames and covers for manhole and catch basins.

Chain link and woven wire mesh.

Glulam connections and anchorages.

Joist hangers, trade-name types.

Metal cladding and covering, less than 2.0 mm (14 ga.).

Precast concrete connections and anchorages in building structure.

Reinforcing steel or mesh.

Roof and floor hatches when trade-name items.

Sheet metal items, steel decking and siding and their attachments, closures, etc., less than 2.0 mm (14 ga.).

Shoring under composite floors and stub-girders.

Steel reinforcement for architectural metal storefronts, curtain walls and windows.

Steel stacks.

Stone anchors.

Stud shear connectors when used with steel deck.

Temporary bracing for other trades.

Thimbles and breeching, also mechanical fire dampers.

Window and area wells.

When the miscellaneous steel fabricator erects miscellaneous steel, all material required for temporary and/or permanent connections of the component parts of the miscellaneous steel shall be supplied.

MONTHLY PROGRESS CLAIM FORM

FIRM NAME: ____________________

PROJECT: ____________________

CONTRACT NO: ____________________

PROGRESS CLAIM NO: ____________________

DATE: ____________________

ITEM	ORIGINAL BASE CONTRACT	APPROVED CHANGES TO DATE	REVISED BASE CONTRACT	PROGRESS TO DATE	PREVIOUS AMOUNT CLAIMED	THIS PROGRESS CLAIM	% COMPLETE
1. ENGINEERING & DETAILING							
2. RAW MATERIALS IN YARD							
3. FABRICATION							
4. FREIGHT TO SITE							
5. ERECTION							
6. PLUMB / BOLT / CLEANUP							
7. TOTAL GROSS AMOUNT							
8. HOLDBACK ________%							
9. NET AMOUNT							
10. APPLICABLE TAX ________% OF LINE 9							
11. TOTAL AMOUNT DUE							
APPROVED CHANGE ORDER(S) TO DATE: ____________________							

APPENDIX H

Suggested Terms for Progress Invoicing and Substantial Performance

H1. Progress Invoicing

Monthly Progress Payments shall be based on the percentage completed of each agreed progress payment criteria during the subject billing period. Suggested progress payment criteria include:

a) Shop Details and/or Erection Diagrams submitted for review.
b) Raw materials received at the fabricators plant.
c) Fabrication of materials.
d) Release for shipment, or shipment to site, as applicable.
e) Erection of materials.
f) Finishing of erected steel Work

H2. Substantial Performance and Statutory Holdback

a) Unless stated otherwise in the Contract, substantial completion criteria and release of statutory holdback shall conform to the requirements of standard construction contracts approved by the Canadian Construction Documents Committee or the Canadian Construction Association, and the governing provincial lien legislation.

b) Contracts for supply only of structural or miscellaneous steel may not be subject to statutory holdback in accordance with the governing provincial lien acts.

c) Substantial completion of Work is be directly related to the Work of the steel Fabricator or Erector, unless stated otherwise in the Contract.

APPENDIX I

Architecturally Exposed Structural Steel (AESS)

I1. Scope and Requirements

I1.1 General Requirements. When members are specifically designated as "Architecturally Exposed Structural Steel" or "AESS" in the Contract Documents, the requirements in Sections 1 through 7 shall apply as modified by this Appendix. AESS members or components shall be fabricated and erected with the care and dimensional tolerances that are stipulated in Sections I2 through I5.

I1.2 Definition of Categories. Categories are listed in the AESS Matrix shown in Table I1, where each Category is represented by a set of Characteristics. The following Categories shall be used when referring to AESS:

AESS 1: Basic Elements

Suitable for "basic" elements which require enhanced workmanship.

AESS 2: Feature Elements Viewed at a Distance > 6 m

Suitable for "feature" elements viewed at a distance greater than six metres. The process involves basically good fabrication practices with enhanced treatment of weld, connection and fabrication detail, tolerances for gaps, and copes.

AESS 3: Feature Elements Viewed at a Distance ≤ 6 m

Suitable for "feature" elements, where the designer is comfortable allowing the viewer to see the art of metalworking. Welds are generally smooth but visible; some grind marks are acceptable. Tolerances are tighter than normal standards. The structure is normally viewed closer than six metres and is frequently subject to touching by the public.

AESS 4: Showcase Elements

Suitable for "showcase or dominant" elements, where the designer intends the form to be the only feature showing in an element. All welds are ground, and filled edges are ground square and true. All surfaces are sanded/filled. Tolerances of fabricated forms are more stringent – generally one-half of the standard tolerance. All surfaces are to be "glove" smooth.

AESS C: Custom Elements

Suitable for elements which require a different set of Characteristics than specified in Categories 1, 2, 3 or 4.

I1.3 Additional Information. The following additional information shall be provided in the Contract Documents when AESS is specified:

a) Specific identification of members or components that are AESS using the AESS Categories listed in I1.2. Refer to Table I1;

b) Fabrication and/or erection tolerances that are to be more restrictive than provided for in this Appendix;

c) For Categories AESS 2, 3 and 4, requirements, if any, of a visual sample or first-off component for inspection and acceptance standards prior to the start of fabrication;

d) For Category AESS C, the AESS Matrix included in Table I1 shall be used to specify the required treatment of the element.

I2. Shop Detail, Arrangement and Erection Drawings

I2.1 Identification. All members designated as AESS members are to be clearly identified with a Category, either AESS 1, 2, 3, 4 or C, on all shop detail, arrangement and erection drawings.

I2.2 Variations. Any variations from the AESS Categories listed must be clearly noted. These variations could include machined surfaces, locally abraded surfaces, and forgings. In addition:

a) If a distinction is to be made between different surfaces or parts of members, the transition line/plane must be clearly identified/defined on the shop detail, arrangement and erection drawings;

b) Tack welds, temporary braces and fixtures used in fabrication are to be indicated on shop drawings;

c) All architecturally sensitive connection details will be submitted for approval to the Architect/Engineer prior to completion of shop detail drawings.

I3. Fabrication

I3.1 General Fabrication. The Fabricator is to take special care in handling the steel to avoid marking or distorting the steel members.

a) All slings will be nylon-type or chains with softeners, or wire rope with softeners.

b) Care shall be taken to minimize damage to any shop paint or coating.

c) If temporary braces or fixtures are required during fabrication or shipment, or to facilitate erection, care must be taken to avoid and/or repair any blemishes or unsightly surfaces resulting from the use or removal of such temporary elements.

d) Tack welds shall be ground smooth.

I3.2 Unfinished, Reused or Weathering Steel. Members fabricated of unfinished, reused or weathering steel that are to be AESS may still have erection marks, painted marks or other marks on surfaces in the completed structure. Special requirements shall be specified as Category AESS C.

I3.3 Tolerances for Rolled Shapes. The permissible tolerances for depth, width, out-of-square, camber and sweep of rolled shapes shall be as specified in CSA G40.20/21 and ASTM A6. The following exceptions apply:

a) For Categories AESS 3 and 4: the matching of abutting cross-sections shall be required;

b) For Categories AESS 2, 3 and 4: the as-fabricated straightness tolerance of a member is one-half of the standard camber and sweep tolerance in CSA G40.20/21.

I3.4 Tolerances for Built-up Members. The tolerance on overall section dimensions of members made up of plates, bars and shapes by welding is limited to the accumulation of permissible tolerances of the component parts as provided by CSA W59 and ASTM A6. For Categories AESS 2, 3 and 4, the as-fabricated straightness tolerance for the built-up member is one-half of the standard camber and sweep tolerances in CSA W59.

I3.5 Joints. For Categories AESS 3 and 4, all copes, miters and butt cuts in surfaces exposed to view are made with uniform gaps, if shown to be open-joint, or in uniform contact if shown without gap.

I3.6 Surface Appearance. For Categories AESS 1, 2 and 3, the quality surface as delivered by the mills will be acceptable. For Category AESS 4, the steel surface imperfections should be filled and sanded.

I3.7 Welds. For corrosive environments, all joints should be seal-welded. In addition:

a) For Categories AESS 1, 2 and 3, a smooth uniform weld will be acceptable. For Category AESS 4, the weld will be contoured and blended.

b) For Categories AESS 1, 2, 3 and 4, all weld spatter is to be avoided/removed where exposed to view.

c) For Categories AESS 1 and 2, weld projection up to 2 mm is acceptable for butt and plug-welded joints. For Categories AESS 3 and 4, welds will be ground smooth/filled.

I3.8 Weld Show-through. It is recognized that the degree of weld show-through, which is any visual indication of the presence of a weld or welds on the opposite surface from the viewer, is a function of weld size and material thickness.

a) For Categories AESS 1, 2 and 3, the members or components will be acceptable as produced.

b) For Category AESS 4, the fabricator shall minimize the weld show-through.

I3.9 Surface Preparation for Painting. Unless otherwise specified in the Contract Documents, the Fabricator will clean AESS members to meet the requirement of SSPC-SP 6 "Commercial Blast Cleaning" (sandblast or shotblast). Prior to blast cleaning:

a) Any deposits of grease or oil are to be removed by solvent cleaning, SSPC-SP 1;

b) Weld spatter, slivers and surface discontinuities are to be removed;

c) Sharp edges resulting from flame cutting, grinding and especially shearing are to be softened.

I3.10 Hollow Structural Sections (HSS) Seams

a) For Categories AESS 1 and 2, seams of hollow structural sections shall be acceptable as produced.

b) For Category AESS 3, seams shall be oriented away from view or as indicated in the Contract Documents.

c) For Category AESS 4, seams shall be treated so that they are not apparent.

I4. Delivery of Materials

I4.1 General Delivery. The Fabricator shall use special care to avoid bending, twisting or otherwise distorting the Structural Steel. All tie-downs on loads will be either nylon strap or chains with softeners to avoid damage to edges and surfaces of members.

I4.2 Standard of Acceptance. The standard for acceptance of delivered and erected members shall be equivalent to the standard employed at fabrication.

I5. Erection

I5.1 General Erection. The Erector shall use special care in unloading, handling and erecting the AESS to avoid marking or distorting the AESS. The Erector must plan and execute all operations in a manner that allows the architectural appearance of the structure to be maintained.

a) All slings will be nylon-strap or chains with softeners.

b) Care shall be taken to minimize damage to any shop paint or coating.

c) If temporary braces or fixtures are required to facilitate erection, care must be taken to avoid and/or repair any blemishes or unsightly surfaces resulting from the use or removal of such temporary elements.

d) Tack welds shall be ground smooth and holes shall be filled with weld metal or body filler and smoothed by grinding or filling to the standards applicable to the shop fabrication of the materials.

e) All backing bars shall be removed and ground smooth.

f) All bolt heads in connections shall be on the same side, as specified, and consistent from one connection to another.

I5.2 Erection Tolerances. Unless otherwise specified in the Contract Documents, members and components are plumbed, levelled and aligned to a tolerance equal to that permitted for structural steel.

I5.3 Adjustable Connections. When more stringent tolerances are specifically required for erecting AESS, the Owner's plans shall specify/allow adjustable connections between AESS and adjoining structural elements, in order to enable the Erector to adjust and/or specify the method for achieving the desired dimensions. Adjustment details proposed by the Erector shall be submitted to the Architect and Engineer for review.

TABLE I1 - AESS Category Matrix

	Category	**AESS C**	**AESS 4**	**AESS 3**	**AESS 2**	**AESS 1**	**SSS**
ID	*Characteristics*	*Custom Elements*	*Showcase Elements*	*Viewed at a distance ≤ 6 m*	*Viewed at a distance > 6 m*	*Basic Elements*	*CSA S16*
1.1	Surface preparation to SSPC-SP 6		√	√	√	√	
1.2	Sharp edges ground smooth		√	√	√	√	
1.3	Continuous weld appearance		√	√	√	√	
1.4	Standard structural bolts		√	√	√	√	
1.5	Weld spatter removed		√	√	√	√	
2.1	Visual samples		optional	optional	optional		
2.2	One-half standard fabrication tolerances		√	√	√		
2.3	Fabrication marks not apparent		√	√	√		
2.4	Welds uniform and smooth		√	√	√		
3.1	Mill marks removed		√	√			
3.2	Butt and plug welds ground smooth and filled		√	√			
3.3	HSS weld seam oriented for reduced visibility		√	√			
3.4	Cross-sectional abutting surfaces aligned		√	√			
3.5	Joint gap tolerances minimized		√	√			
3.6	All welded connections		optional	optional			
4.1	HSS seam not apparent		√				
4.2	Welds contoured and blended		√				
4.3	Surfaces filled and sanded		√				
4.4	Weld show-through minimized		√				
C.1							
C.2							
C.3							
C.4							
C.5							

TABLE I1 - AESS Category Matrix (Cont'd)

	Notes
1.1	Prior to blast cleaning, any deposits of grease or oil are to be removed by solvent cleaning, SSPC-SP 1.
1.2	Rough surfaces are to be deburred and ground smooth. Sharp edges resulting from flame cutting, grinding and especially shearing are to be softened.
1.3	Intermittent welds are made continuous, either with additional welding, caulking or body filler. For corrosive environments, all joints should be seal welded. Seams of hollow structural sections shall be acceptable as produced.
1.4	All bolt heads in connections shall be on the same side, as specified, and consistent from one connection to another.
1.5	Weld spatter, slivers and surface discontinuities are to be removed. Weld projection up to 2 mm is acceptable for butt and plug-welded joints.
2.1	Visual samples are either a 3-D rendering, a physical sample, a first-off inspection, a scaled mock-up or a full-scale mock-up, as specified in Contract Documents.
2.2	These tolerances are required to be one-half of those of standard structural steel as specified in CSA S16.
2.3	Members marked with specific numbers during the fabrication and erection processes are not to be visible.
2.4	—
3.1	All mill marks are not to be visible in the finished product.
3.2	Caulking or body filler is acceptable.
3.3	Seams shall be oriented away from view or as indicated in the Contract Documents.
3.4	The matching of abutting cross-sections shall be required.
3.5	This characteristic is similar to 2.2 above. A clear distance of 3 mm between abutting members is required.
3.6	Hidden bolts may be considered.
4.1	HSS seams shall be treated so that they are not apparent.
4.2	In addition to a contoured and blended appearance, welded transitions between members are also required to be contoured and blended.
4.3	Steel surface imperfections should be filled and sanded.
4.4	The back face of a welded element caused by the welding process can be minimized by hand grinding the back side of the weld. The degree of weld show-through is a function of weld size and material.
C.	Additional characteristics may be added for custom elements.

APPENDIX J

Building Information Modelling

This Appendix is intended to facilitate the understanding and use of digital modelling technology in the design and construction of Steel Structures.

J1. General Provisions

J1.1 Scope

The provisions in this Appendix shall apply when the Contract Documents indicate that a three-dimensional digital Building Information Model (BIM) or Digital/Electronic Model replaces Contract Documents and is to be used as the primary means of designing, representing, and exchanging Structural Steel data for the project. In this case, references to the Design Drawings shall apply to the Design Model, and references to Fabrication and Erection Documents shall apply to the Manufacturing Model.

If the primary means of project communication reverts from a model-based (electronic) system to a paper-based system, the requirements of this Appendix are no longer applicable.

J1.2 Definitions

See Section 1.2 of the CISC Code of Standard Practice for all definitions related to this Appendix.

J2. Supplementary Technical Standards

The following references are provided as a guide to assist in developing a BIM Execution Plan with reference to the Contract Documents. The provisions of other standards shall be applicable if called for in the Project Tender Documents and Construction Specifications.

BIM Execution Plan – *Project Execution Planning Guide V2.0 Released July 2010*

https://bim.psu.edu/

LOD – *2014 LOD Specification*

https://bimforum.org/lod/

LOD Matrix (*also referred to as a model element table*) – AIA Document E203 - 2013

http://www4.fm.virginia.edu/fpc/ContractAdmin/ProfSvcs/BIMAIASample.pdf

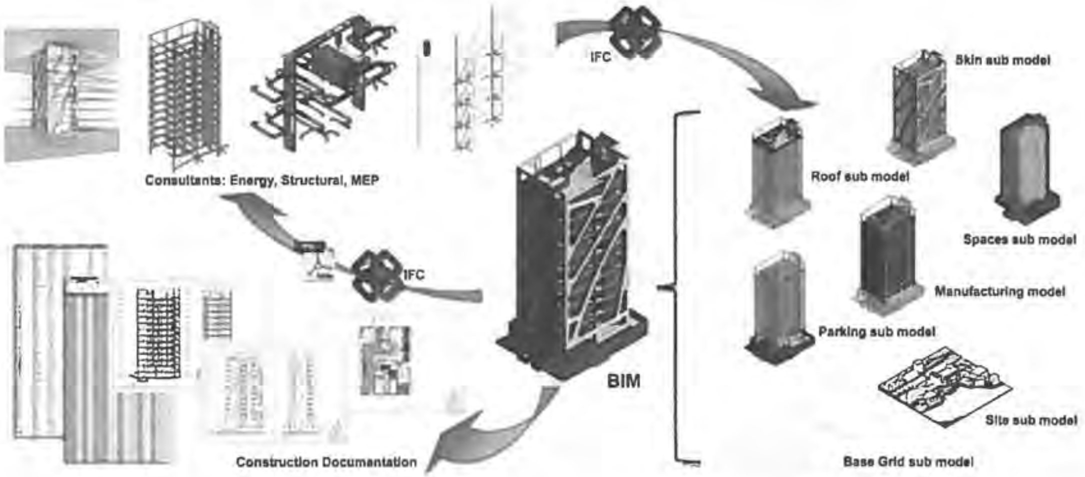
Naming Conventions – *Naming Convention for Structural Steel Products for Use in Electronic Data Interchange (EDI). AISC Document June 25, 2001*

https://aisc.org/WorkArea/showcontent.aspx?id=6444

J3. File Format

The Industry Foundation Class Model should be used, unless otherwise agreed, as the Building Information Model for structural steel. The Industry Foundation Class model for Structural Steel may exist solely as the project's BIM or may be integrated into a multi-disciplinary BIM for projects adopting greater digital model design application.

Refer to the Electronic Data Interchange Project Flowchart (Figure J1) for an example of this interoperability. The figure demonstrates how the BIM file serves several functions. It acts as the repository for project information developed in stand-alone external software platforms such as the manufacturing model. It also acts as a source file from which contract documentation and specific model data can be extracted for further analysis.



Electronic Data Interchange Project Flowchart

Figure J1

Note: Images taken from www.vectorworks.net

J4. Content and Purpose of the BIM Files

In addition to the requirements in Clause 4 related to Contract Documents, the following requirements shall apply to the BIM file.

J4.1 The BIM file is intended to:

a) Govern over all other forms of information, including drawings, sketches, etc., unless specifically noted otherwise in the Construction Documents.

b) Include all steel elements (primary and secondary structural), as well as any other entities required for strength and stability of the completely erected structure.

c) Include entities that fully define each steel element, and the extent of detailing of each element, as would be recorded on an equivalent set of Structural Design Documents (see Clause 4.1.2).

d) Contain Analysis Model data so as to include load calculations as indicated in the Construction Specifications referencing jurisdictional codes.

e) Conform to the required Level of Development (LOD). See Figures J2, J3, and J4.

f) Provide a common reference point and datum (0, 0, 0).

g) Contain all necessary information to comply with downstream user requirements (i.e. design loads, member sizes, dimensions, etc.).

J5. Project Governance

For all BIM projects, a BIM Administrator will be assigned and provided by Others.

J5.1 The BIM Administrator will ensure that the BIM Execution Plan is followed. BIM Administrator responsibilities are intended to include the following:

a) Define control of the BIM by providing appropriate access privileges (read, write, etc.) to all relevant parties.

b) Maintain the security of the BIM.

c) Guard against data loss of the BIM.

d) Be responsible for updates and revisions to the BIM as they occur, and archive all versions with appropriate annotations.

e) Inform all involved parties regarding changes to the BIM.

J6. Usability and Protocol

J6.1 In addition to the requirements in Clause 5 related to Fabrication and Erection Documents, the following requirements shall apply:

a) In the event of a conflict between the BIM and Design Documents, the BIM Execution Plan will determine which document governs. In the absence of this clarification in the BIM Execution Plan, the BIM file shall govern.

b) The responsibility for the development and accuracy of the information added to the BIM file shall be defined in the Contract Documents. In the absence of such terms regarding the information added by the Fabricator (via sharing of the manufacturing model) to the BIM in the Construction Documents, the responsibility will belong to the Fabricator in accordance with the appropriate LOD definition. For clarification related to instructions provided to the Fabricator by other project stakeholders, see LOD Section J8 of this Appendix.

c) During the development of the Manufacturing Model, any relocation of, or adjustments to, members will only be done with approval by the Engineer of Record.

d) The Fabricator and Erector shall accept the use of the Manufacturing Model and the BIM under the same conditions as set forth in Clause 4.3.1, except as modified in J7.

J7. Review

Review of the Manufacturing Model by the Engineer of Record may replace the review of the actual Fabrication and Erection Documents. For this method to be effective, a system must be in place to capture review comments and action items, and to complete the review, correction and final release of the Manufacturing Model for fabrication of structural steel. The versions of the model shall be tracked with review comments permanently attached to the versions of the model to the same extent as such data is maintained with conventional hard copy approvals. The Industry Foundation Class Standard provides this level of tracking.

J7.1 When a review of the detailed material is to be done by using the Manufacturing Model, the version of the submitted model shall be identified. Comments attached to the individual elements as specified in the BIM Execution Plan shall be used to annotate the Manufacturing

Model. The Fabricator will issue the revised Manufacturing Model for review, and the version of the model submitted will be tracked as previously defined.

J8. Level of Development (LOD)

It is important to identify the extent of information that will be provided in the BIM by each stakeholder. The LOD matrix provides a mechanism for defining these responsibilities and commitments. Prior to the development of the LOD matrix specific to any given project, it will be assumed that the detailer will only be responsible for providing information up to the "as-fabricated" state, commonly referred to as LOD 400. Changes beyond the base scope of Work are to be inputted into the BIM by the Owner, unless otherwise agreed to as part of the change management process.

The LOD matrix will determine which project team member is responsible for developing the model to the associated LOD status by assigning a Model Element Author (MEA) for each specific development status number for each line item. An example table taken from AIA document E202 is provided below for general reference.

§ 4.3 Model Element Table *Identify (1) the LOD required for each Model Element at the end of each phase, and (2) the Model Element Author (MEA) responsible for developing the Model Element to the LOD identified.* *Insert abbreviations for each MEA identified in the table below, such as "A – Architect," or "C – Contractor."* *NOTE: LODs must be adapted for the unique characteristics of each Project.*				Conceptualization		Criteria Design		Detailed Design		Implementation Documents		Construction				Note Number (See 4.4)
Model Elements Utilizing CSI UniFormat™				LOD	MEA	LOD	MEA	LOD	MEA	LOD	MEA	LOD	MEA	LOD	MEA	
A SUBSTRUCTURE	A10 Foundations	A1010	Standard Foundations	100		200		300		400		500				
		A1020	Special Foundations	100		100		300		400		500				
		A1030	Slab on Grade *	100		200		300		400		500				
	A20 Basement Construction	A2010	Basement Excavation	100		200		300		300		500				
		A2020	Basement Walls	100		200		300		400		500				
B SHELL	B10 Superstructure	B1010	Floor Construction	100		200		300		300		500				
		B1020	Roof Construction	100		200		300		300		500				

LOD Matrix

Figure J2

LOD definitions are described as follows.

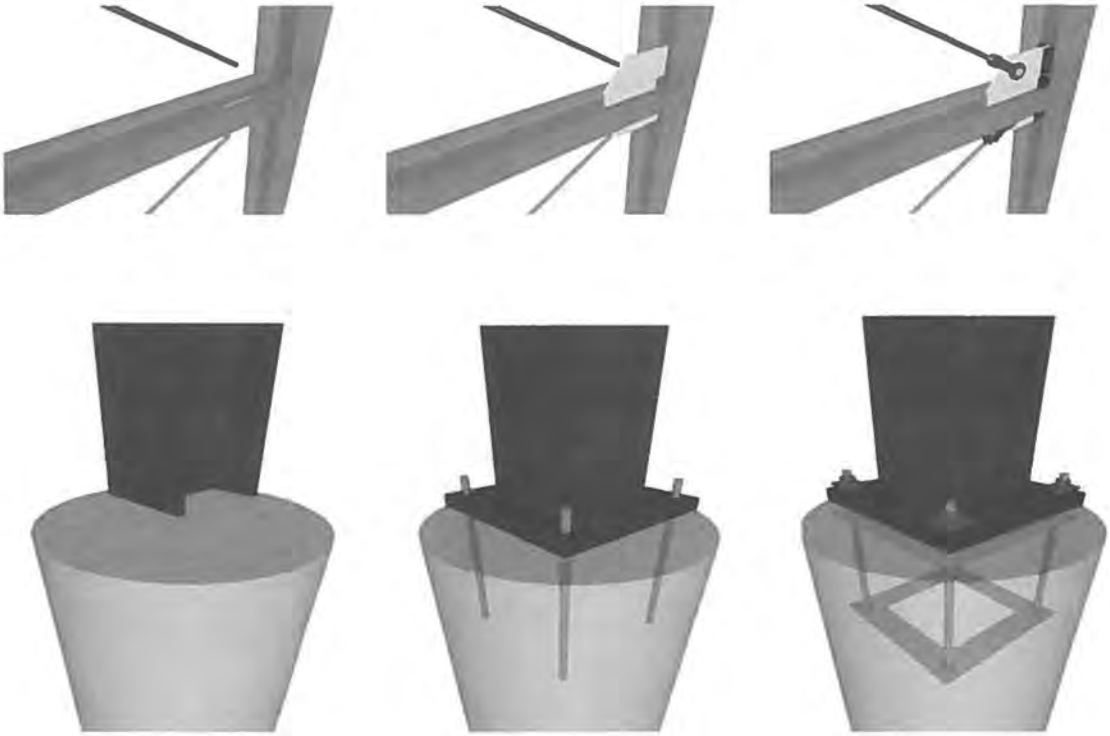
Level of Development (LOD) Descriptions	
LOD 100	The Model Element may be graphically represented in the Model with a symbol or other generic representation but does not satisfy the requirements for LOD 200. Information related to the Model Element (i.e. cost per square foot, tonnage of HVAC, etc.) can be derived from other Model Elements.
LOD 200	The Model Element is graphically represented within the Model as a generic system, object, or assembly with approximate quantities, size, shape, location, and orientation. Non-graphic information may also be attached to the Model Element.
LOD 300	The Model Element is graphically represented within the Model as a specific system, object or assembly in terms of quantity, size, shape, location, and orientation. Non-graphic information may also be attached to the Model Element.
LOD 350	The Model Element is graphically represented within the Model as a specific system, object, or assembly in terms of quantity, size, shape, orientation, and interfaces with other building systems. Non-graphic information may also be attached to the Model Element.
LOD 400	The Model Element is graphically represented within the Model as a specific system, object or assembly in terms of size, shape, location, quantity, and orientation with detailing, fabrication, assembly, and installation information. Non-graphic information may also be attached to the Model Element.
LOD 500	The Model Element is a field-verified representation in terms of size, shape, location, quantity, and orientation. Non-graphic information may also be attached to the Model Elements.

LOD Descriptions

Figure J3

Note: The definitions for LOD 100, 200, 300, 400, and 500 included in this Specification represent the updated language that appears in the AIA's most recent BIM protocol document, G202–2013, Building Information Modelling Protocol Form. *The LOD 100, 200, 300, 400 and 500 definitions are produced by the AIA and have been used by permission. LOD 350 was developed by the BIMForum working group and is copyright to the BIMForum and the AIA.*

Graphical representations of the LOD descriptions are provided for visual reference.



LOD 300 **LOD 350** **LOD 400**

LOD Diagram (Example)

Figure J4

Note: Images taken from BIMFORUM Level of Development Specification 2013.

NOTES

STRUCTURAL SHEET STEEL PRODUCTS

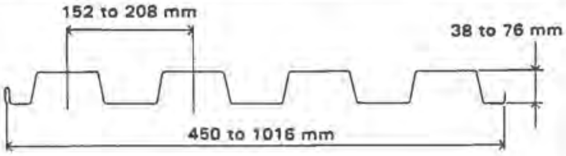
General

Structural sheet steel products such as roof deck, floor deck and cladding complement the structural steel frame of a building. These large-surface elements often perform both structural and non-structural functions, thereby enhancing the overall economy of the design.

TYPICAL STEEL DECK AND CLADDING PROFILES

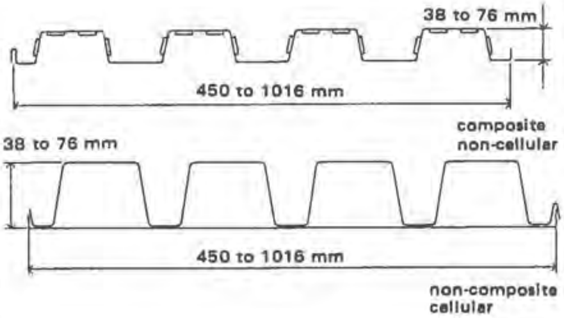
Roof Deck

Can be supplied perforated for acoustical applications.



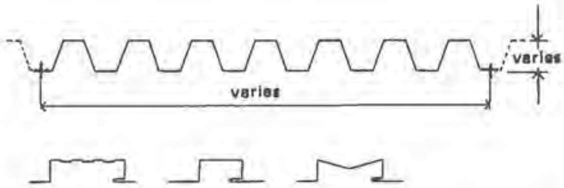
Floor Deck

Available in composite and non-composite profiles and as cellular or non-cellular units.



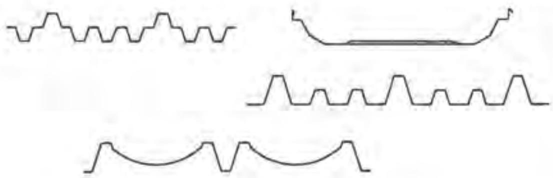
Architectural Cladding

Available in various profiles, widths, coatings and colours.



Steel Building Systems Cladding

Available in various profiles, widths, coatings and colours



Farm Building Cladding

Available in various profiles, widths, coatings and colours

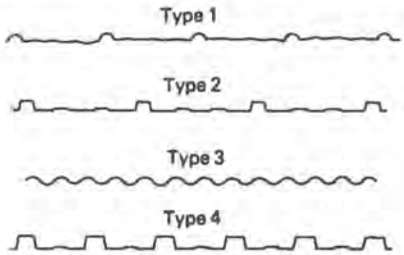


Figure 1: Typical Cladding Profiles

Figure 1 is for general information only, and manufacturers may produce additional profiles which are not represented by any of the types shown.

Many of the sheet steel products used in Canada are supplied by members of the Canadian Sheet Steel Building Institute, a national association of steel producers, zinc producers, coil coaters, fastener manufacturers and fabricators of steel building products, steel building systems and lightweight steel framing components. The Institute promotes the use of sheet steel in building construction by encouraging good design, pleasing form and greater economy.

Sheet steel materials for building construction are metallic coated (zinc or aluminum-zinc alloy) and can be prefinished for extra corrosion protection and aesthetics. Consult fabricators' catalogs for details of available products, profiles, widths, lengths, thicknesses, load capacities and other characteristics.

CSSBI PUBLICATIONS

CSSBI publications include industry product standards, informational bulletins and special publications as well as non-technical promotional material. A selection of current publications is listed below.

CSSBI Standards

Steel Roof Deck – covers design, fabrication and erection of steel roof deck with flutes not more than 200 mm on centre and a nominal 77 mm maximum profile depth, intended for use with built-up roofing or other suitable weather-resistant cover on top of the deck (CSSBI 10M).

Composite Steel Deck – covers design, fabrication and erection of composite steel deck with a nominal 77 mm maximum profile depth, intended for use with a concrete cover slab on top of the deck to create a composite slab (CSSBI 12M).

Sheet Steel Cladding for Architectural, Industrial and Commercial Building Applications – covers design, fabrication and erection of weather-tight wall and roof cladding made from metallic coated, prefinished sheet steel for use on buildings with low internal humidity (CSSBI 20M).

Steel Building Systems – covers the design, fabrication and erection of steel building systems (SBS). Includes definitions, classification of SBS by type, checklist of items normally furnished, criteria for load combinations, design standards, and certification by a registered engineer (CSSBI 30M).

Steel Farm Roofing and Siding – covers the manufacture, load carrying capacity, handling and installation of sheet steel cladding intended for application to walls and/or roofs of farm buildings (CSSBI 21M).

Bulletins and Special Publications

Criteria for the Testing of Composite Slabs – provides the criteria for conducting a series of shear-bond tests necessary to determine the structural capacity of a composite slab (CSSBI S2).

Criteria for the Design of Composite Slabs – contains design criteria, based on limit states design, for composite slabs made of a structural concrete placed permanently over a composite steel deck (CSSBI S3).

Design of Steel Deck Diaphragms – offers a simple and practical approach to the design of steel deck diaphragms supported by horizontal steel framing (CSSBI B13).

Lightweight Steel Framing Design Manual – shows through examples how to design lightweight steel framing structural systems. Detailed calculations are shown for curtain walls, infill walls, and axial load bearing systems as well as all connections (CSSBI 51M).

How-To Series: Insulated Sheet Steel Wall Assemblies – describes the various stages in the selection of sheet steel wall assembly components, architectural and structural design issues, as well as building science topics and material selection (CSSBI S10).

How-To Series: Insulated Sheet Steel Roof Assemblies – describes the various stages in the selection of the sheet steel roof assembly components, architectural and structural design issues, as well as building science topics and material selection (CSSBI S11).

How-To Series: Steel Roof and Floor Deck – describes the various stages in the selection of steel deck products, the different types of deck products, structural design issues and material selection (CSSBI S15).

How-To Series: Lightgauge Steel Roofing and Siding – offers simple and practical recommendations for the selection, application and installation of lightgauge steel cladding (CSSBI S14).

Barrier Series Prefinished Sheet Steel: Product Performance & Applications – presents the features and benefits of the Barrier Series prefinished paint system for sheet steel building products in more aggressive environments (CSSBI B17).

Lightweight Steel Framing Architectural Design Guide – provides information to the architect about the uses and specification of Lightweight Steel Framing (LSF) systems, including details on design, building science, acoustic and fire ratings, as well as extensive references (CSSBI 57).

Contact CSSBI at the address below for a complete listing of publications, copies of publications, or other information concerning sheet steel in construction.

Canadian Sheet Steel Building Institute
652 Bishop St. N., Unit 2A
Cambridge, Ontario N3H 4V6
Tel (519) 650-1285
Fax (519) 650-8081
Website: www.cssbi.ca

MASS AND FORCES FOR MATERIALS

MATERIAL	Mass (kg/m³)	Force (kN/m³)	MATERIAL	Mass (kg/m³)	Force (kN/m³)
METALS, ALLOYS, ORES			TIMBER, AIR-DRY		
Aluminum	2 640	25.9	Birch	689	6.76
Brass	8 550	83.8	Cedar	352	3.45
Bronze, 7.9-14% tin	8 150	79.9	Fir, Douglas, seasoned	545	5.34
Bronze, aluminum	7 700	75.5	Fir, Douglas, unseasoned	641	6.29
Copper	8 910	87.4	Fir, Douglas, wet	801	7.86
Copper ore, pyrites	4 200	41.2	Fir, Douglas, glue laminated	545	5.34
Gold	19 300	189	Hemlock	481	4.72
Iron, cast, pig	7 210	70.7	Larch, tamarack	561	5.50
Iron, wrought	7 770	76.2	Larch, western	609	5.97
Iron, spiegel-eisen	7 500	73.5	Maple	737	7.23
Iron, ferro-silicon	7 000	68.6	Oak, red	689	6.76
Iron ore, hematite	5 210	51.1	Oak, white	753	7.38
Iron ore, hematite in bank	2 560-2 880	25.1-28.2	Pine, jack	481	4.72
Iron ore, hematite, loose	2 080-2 560	20.4-25.1	Pine, ponderosa	513	5.03
Iron ore, limonite	3 800	37.3	Pine, red	449	4.40
Iron ore, magnetite	5 050	49.5	Pine, white	416	4.08
Iron slag	2 760	27.1	Poplar	481	4.72
Lead	11 400	112	Spruce	449	4.40
Lead ore, galena	7 450	73.1	For pressure treated timber add retention to mass of air-dry material.		
Magnesium	1 790	17.6			
Manganese	7 610	74.6			
Manganese ore	4 150	40.7			
Mercury	13 600	133	LIQUIDS		
Monel	8 910	87.4	Alcohol, pure	785	7.70
Nickel	9 050	88.8	Gasoline	673	6.60
Platinum	21 300	209	Oils	929	9.11
Silver	10 500	103	Water, fresh at 4°C (max.		
Steel, rolled	7 850	77.0	density)	1 000	9.81
Tin	7 350	72.1	Water, fresh at 100°C	961	9.42
Tin ore, cassiterite	6 700	65.7	Water, salt	1 030	10.1
Zinc	7 050	69.1			
Zinc ore, blende	4 050	39.7	EARTH, ETC. EXCAVATED		
			Earth, wet	1 600	15.7
MASONRY			Earth, dry	1 200	11.8
Ashlar	2 240-2 560	22.0-25.1	Sand and gravel, wet	1 920	18.8
Brick, soft	1 760	17.3	Sand and gravel, dry	1 680	16.5
Brick, common	2 000	19.6			
Brick, pressed	2 240	22.0	VARIOUS BUILDING		
Clay tile, average	961	9.42	MATERIALS		
Rubble	2 080-2 480	20.4-24.3	Cement, Portland, loose	1 510	14.8
Concrete, cinder, haydite	1 600-1 760	15.7-17.3	Cement, Portland, set	2 930	28.7
Concrete, slag	2 080	20.4	Lime, gypsum, loose	849-1 030	8.33-10.1
Concrete, stone	2 310	22.7	Mortar, cement-lime, set	1 650	16.2
Concrete, stone, reinforced	2 400	23.5	Quarry stone, piled	1 440-1 760	14.1-17.3
SOLID FUELS			MISCELLANEOUS		
Coal, anthracite, piled	753-929	7.38-9.11	Asphaltum	1 300	12.7
Coal, bituminous, piled	641-865	6.29-8.48	Tar, bituminous	1 200	11.8
Coke, piled	368-513	3.61-5.03	Glass, common	2 500	24.5
Charcoal, piled	160-224	1.57-2.20	Glass, plate or crown	2 580	25.3
Peat, piled	320-416	3.14-4.08	Glass, crystal	2 950	28.9
			Paper	929	9.11
ICE AND SNOW *					
Ice	897	8.80			
Snow, dry, fresh fallen	128	1.26	** Consult building code for snow load and density.*		
Snow, dry, packed	192-400	1 .88-3.92			
Snow, wet	432-641	4.24-6.29			

DESIGN DEAD LOADS (kPa) OF MATERIALS

STEEL DECKS	
Steel deck* 38 mm deep	
(up to 0.91 mm thick)	0.10
(1.22 to 1.52 mm thick)	0.15
Steel deck* 76 mm deep (Narrow-Rib)	
(up to 0.91 mm thick)	0.15
(1.22 to 1.91mm thick)	0.30
Steel deck* 76 mm deep (Wide-Rib)	
(up to 0.91 mm thick)	0.10
(1.22 to 1.52 mm thick)	0.15
* for cellular deck, add	0.08
CONCRETE, per 100 mm	
- 2350 kg/m³ (N.D.)	2.31
- 2000 kg/m³ (slag aggregate)	1.96
-1850 kg/m³ (S.L.D.)	1.82
HOLLOW CORE PRECAST (no topping)	
- 200 mm deep (N.D.)	2.60
- 300 mm deep (N.D.)	3.50
WOOD JOISTS (at 400 mm centres)	
- 38 mm x 184 mm joists	0.09
- 38 mm x 235 mm joists	0.12
- 38 mm x 286 mm joists	0.14
PLYWOOD	
- 11 mm thick	0.06
- 14 mm thick	0.08
- 19 mm thick	0.11
CHIPBOARD	
- 12.7 mm thick	0.07
- 15.9 mm thick	0.09
- 19.0 mm thick	0.11
WALLS AND CLADDING	
- Solid brick wall (concrete)	
- 100 mm thick (S.L.D.)	1.40
- 100 mm thick (N.D.)	1.90
- Hollow block (S.L.D.)	
- 100 mm thick	1.10
- 200 mm thick	1.60
- 300 mm thick	2.30
- Hollow block (N.D.)	
- 100 mm thick	1.40
- 200 mm thick	2.10
- 300 mm thick	2.90
- P.C. wall plus glazing	2.40 - 3.80
- Metal curtain wall	0.74 - 1.50
- Insulated sheet steel wall (exclude girts)	0.25 - 0.40
- 38 x 89 wood studs @ 400 mm	0.05
- Gypsum wallboard per 10 mm	0.08
- Stone veneer per 25 mm	0.40

FLOOR FINISHING	
- Vinyl, linoleum or asphalt tile	0.07
- Softwood subfloor per 10 mm	0.06
- Hardwood per 10 mm	0.08
- Carpeting	0.10
- Asphaltic concrete per 10 mm	0.23
- 20 mm Ceramic or quarry tiles on 12 mm mortar bed	0.80
- Terrazzo per 10 mm	0.24
- Mastic floor (20 mm)	0.45
ROOFING	
- 3 ply asphalt, no gravel	0.15
- 4 ply asphalt, no gravel	0.20
- 3 ply asphalt and gravel	0.27
- 4 ply asphalt and gravel	0.32
- Asphalt strip shingles	0.15
- Gypsum wallboard per 10 mm	0.08
INSULATION (per 100 mm thick)	
- Glass fibre, batts	0.05
- Glass fibre, blown	0.04
- Glass fibre, rigid	0.07
- Urethane, rigid foam	0.03
- Insulating concrete	0.06
CEILINGS	
- Gypsum wallboard per 10 mm	0.08
- Tiled ceiling & suspension system, with fixtures, average	0.20
- 20 mm plaster on lath/furring	0.40
- Sprayed fire protection, average	0.07
- Ducts/pipes/wiring allowance (average condition)	0.25
DECK-SLABS (average condition)	
- 38 mm deck with	
- 65 mm N.D. cover*	1.95
- 90 mm N.D. cover*	2.55
- 65 mm S.L.D. cover**	1.55
- 85 mm S.L.D. cover**	1.90
- 75 mm (or 76 mm) "wide-rib" deck with	
- 65 mm N.D. cover*	2.55
- 90 mm N.D. cover*	3.15
- 65 mm S.L.D. cover**	2.15
- 85 mm S.L.D. cover**	2.50
- 76 mm "narrow-rib" deck with	
- 65 mm N.D. cover*	2.20
- 90 mm N.D. cover*	2.80
- 65 mm S.L.D. cover**	1.90
- 85 mm S.L.D. cover**	2.25
* assume 2350 kg/m³ concrete	
** assume 1850 kg/m³ concrete	

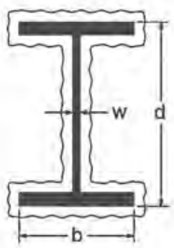
M / D Ratios

How *M/D* Ratios are Calculated

M/D ratios are used to measure the thermal mass resistance of a member under fire. Typically, the higher the *M/D* ratio, the greater the fire resistance. The numbers given in the following table were calculated by dividing the steel member mass per unit length, *M* (kg/m) by the heated perimeter, *D* (m). The resulting units are (kg/m)/m in the Metric system and (lb/ft)/in in the Imperial system.

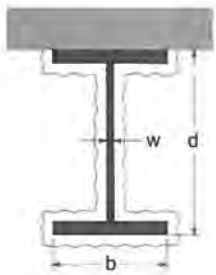
The *D* value is based on the heated perimeter following the contour of the shape, including all flange and web surfaces, and is applicable to fire protection with spray-applied fire-resistive materials. Two separate *M/D* ratios are given for each steel section: (1) one for columns based on the entire perimeter (fire exposure from all sides), and (2) one for beams which typically have the top surface of its top flange shielded from the fire, hence having one less exposed surface.

This heated perimeter (*D*) calculation corresponds to the "contour protection", as described by Gewain *et al* (2006), and is distinct from the "box protection" based on members being boxed up with gypsum board.



$D \approx 4b + 2d - 2w$

(1) Heated perimeter, *D*
for steel columns



$D \approx 3b + 2d - 2w$

(2) Heated perimeter, *D*
for steel beams

Note: The above formulas for the heated perimeter are approximate and do not include the flange-to-web fillets. These have been taken into account when calculating the M/D ratios given in the following pages.

Reference

GEWAIN, R.G., IWANKIW, N.R., ALFAWAKHIRI, F., AND FRATER, G. 2006. Fire Facts for Steel Buildings, Canadian Institute of Steel Construction, American Institute of Steel Construction.

M/D RATIOS FOR CONTOUR PROTECTION
W SHAPES

Designation	SI (kg/m)/m		Imperial (lb./ft.)/in.	
	Beam	Column	Beam	Column
W1100				
x499	149		2.54	
x433	130		2.21	
x390	118		2.01	
x343	104		1.77	
W1000				
x976	291		4.97	
x883	267		4.55	
x748	230		3.92	
x642	200		3.41	
x591	185		3.16	
x554	174		2.98	
x539	170		2.90	
x483	153		2.62	
x443	141		2.41	
x412	132		2.25	
x371	119		2.04	
x321	104		1.77	
x296	96.3		1.64	
W1000				
x584	199		3.40	
x494	171		2.92	
x486	169		2.88	
x438	153		2.60	
x415	146		2.49	
x393	138		2.36	
x350	124		2.11	
x314	112		1.91	
x272	97.4		1.66	
x249	89.6		1.53	
x222	80.5		1.37	
W920				
x1377	404		6.89	
x1269	373		6.37	
x1194	354		6.05	
x1077	324		5.53	
x970	296		5.05	
x787	245		4.18	
x725	228		3.89	
x656	208		3.55	
x588	188		3.21	
x537	172		2.94	
x491	159		2.71	
x449	146		2.49	
x420	137		2.33	
x390	127		2.17	
x368	120		2.05	
x344	113		1.93	
W920				
x381	139		2.37	
x345	127		2.16	
x313	115		1.97	
x289	107		1.82	
x271	101		1.72	
x253	94.5		1.61	
x238	89.1		1.52	
x223	84.2		1.44	
x201	76.0		1.30	
W840				
x576	195		3.33	
x527	180		3.08	
x473	163		2.78	
x433	150		2.56	
x392	137		2.33	
x359	126		2.15	
x329	116		1.98	
x299	106		1.81	
W840				
x251	99.5		1.70	
x226	90.3		1.54	
x210	84.3		1.44	
x193	77.9		1.33	
x176	71.0		1.21	
W760				
x582	211		3.60	
x531	194		3.31	
x484	179		3.05	
x434	161		2.75	
x389	146		2.49	
x350	132		2.26	
x314	119		2.04	
x284	108		1.85	
x257	98.9		1.69	
W760				
x220	96.5		1.65	
x196	86.6		1.48	
x185	81.7		1.39	
x173	77.0		1.31	
x161	71.4		1.22	
x147	65.8		1.12	
x134	59.8		1.02	
W690				
x802	301		5.13	
x548	215		3.68	
x500	198		3.38	
x457	183		3.12	
x419	169		2.88	
x384	156		2.66	
x350	143		2.45	
x323	133		2.27	
x289	120		2.04	
x265	110		1.88	
x240	101		1.72	
x217	91.9		1.57	

M/D RATIOS FOR CONTOUR PROTECTION
W SHAPES

Designation	SI (kg/m)/m		Imperial (lb./ft.)/in.	
	Beam	Column	Beam	Column
W690				
x192	91.3		1.56	
x170	81.4		1.39	
x152	73.4		1.25	
x140	67.6		1.15	
x125	61.1		1.04	
W610				
x551	235		4.00	
x498	215		3.67	
x455	198		3.37	
x415	183		3.12	
x372	165		2.82	
x341	152		2.60	
x307	139		2.37	
x285	130		2.21	
x262	119		2.04	
x241	111		1.89	
x217	100		1.71	
x195	90.7		1.55	
x174	81.3		1.39	
x155	72.6		1.24	
W610				
x153	82.2		1.40	
x140	75.3		1.29	
x125	67.6		1.15	
x113	61.6		1.05	
x101	55.5		0.947	
W610				
x92	54.5		0.931	
x82	48.6		0.830	
W530				
x409	194		3.31	
x369	176		3.01	
x332	160		2.74	
x300	147		2.50	
x272	134		2.29	
x248	123		2.09	
x219	109		1.87	
x196	98.9		1.69	
x182	91.8		1.57	
x165	84.1		1.43	
x150	76.8		1.31	
W530				
x138	82.1		1.40	
x123	73.8		1.26	
x109	65.7		1.12	
x101	61.3		1.05	
x92	56.2		0.959	
x82	50.2		0.856	
x72	44.3		0.757	
W530				
x85	55.7		0.951	
x74	49.5		0.845	
x66	43.8		0.748	

Designation	SI (kg/m)/m		Imperial (lb./ft.)/in.	
	Beam	Column	Beam	Column
W460				
x464	239		4.07	
x421	220		3.76	
x384	203		3.46	
x349	186		3.18	
x315	170		2.90	
x286	156		2.66	
x260	143		2.45	
x235	131		2.23	
x213	119		2.04	
x193	109		1.87	
x177	101		1.72	
x158	90.2		1.54	
x144	83.1		1.42	
x128	74.2		1.27	
x113	65.8		1.12	
W460				
x106	71.8		1.23	
x97	65.9		1.12	
x89	61.2		1.05	
x82	56.5		0.964	
x74	51.4		0.877	
W460				
x68	51.2		0.873	
x60	44.8		0.764	
x52	39.5		0.674	
W410				
x149	93.2		1.59	
x132	83.3		1.42	
x114	72.7		1.24	
x100	63.7		1.09	
W410				
x85	63.9		1.09	
x74	56.7		0.967	
x67	51.3		0.876	
x60	45.5		0.777	
x54	41.2		0.703	
W410				
x46	38.9		0.663	
x39	33.1		0.566	

M/D RATIOS FOR CONTOUR PROTECTION
W SHAPES

Designation	SI (kg/m)/m		Imperial (lb./ft.)/in.	
	Beam	Column	Beam	Column
W360				
x1299		453		7.74
x1202		427		7.28
x1086		394		6.73
x990		366		6.24
x900		339		5.79
x818		313		5.34
x744		289		4.93
x677		267		4.56
x634		253		4.31
x592		238		4.07
x551		224		3.82
x509		209		3.56
x463		192		3.28
x421		177		3.02
x382		162		2.77
x347		148		2.53
x314		135		2.31
x287		125		2.14
x262		115		1.96
x237		104		1.78
x216		96.0		1.64
W360				
x196		90.6		1.55
x179		83.0		1.42
x162	91.2	75.4	1.56	1.29
x147	83.5	69.0	1.42	1.18
x134	76.2	63.0	1.30	1.07
W360				
x122	84.5	71.7	1.44	1.22
x110	76.8	65.1	1.31	1.11
x101	70.9	60.1	1.21	1.03
x91	64.0	54.3	1.09	0.927
W360				
x79	62.2	53.6	1.06	0.914
x72	56.5	48.7	0.965	0.831
x64	50.8	43.7	0.867	0.747
W360				
x57	47.5		0.810	
x51	42.7		0.728	
x45	38.1		0.651	
W360				
x39	37.1		0.633	
x33	31.3		0.535	

Designation	SI (kg/m)/m		Imperial (lb./ft.)/in.	
	Beam	Column	Beam	Column
W310				
x500		239		4.08
x454		220		3.76
x415		204		3.48
x375		187		3.20
x342		173		2.96
x313		160		2.73
x283		146		2.50
x253		132		2.26
x226		120		2.05
x202		108		1.85
x179		96.5		1.65
x158		85.9		1.47
x143		78.5		1.34
x129	86.2	71.5	1.47	1.22
x118	78.6	65.2	1.34	1.11
x107	71.8	59.6	1.23	1.02
x97	65.4	54.2	1.12	0.925
W310				
x86	64.7	54.4	1.10	0.928
x79	59.5	49.9	1.01	0.852
W310				
x74	62.3	53.1	1.06	0.907
x67	56.3	48.0	0.960	0.819
x60	50.5	43.0	0.861	0.734
W310				
x52	47.6		0.812	
x45	40.9		0.699	
x39	35.8		0.611	
W310				
x33	36.3		0.620	
x28	31.7		0.541	
x24	26.9		0.460	
x21	23.9		0.408	
W250				
x167		106		1.82
x149		95.8		1.64
x131		85.4		1.46
x115		75.5		1.29
x101	81.1	67.2	1.38	1.15
x89	72.3	59.9	1.23	1.02
x80	65.1	53.9	1.11	0.920
x73	59.6	49.3	1.02	0.842
W250				
x67	62.1	52.2	1.06	0.892
x58	54.4	45.8	0.929	0.781
x49	46.4	38.9	0.791	0.664
W250				
x45	47.7		0.814	
x39	41.4		0.707	
x33	35.4		0.605	

M/D RATIOS FOR CONTOUR PROTECTION

W SHAPES

Designation	SI (kg/m)/m		Imperial (lb./ft.)/in.	
	Beam	Column	Beam	Column
W250				
x28	35.7		0.610	
x25	32.0		0.547	
x22	28.5		0.486	
x18	22.9		0.391	
W200				
x100	95.9	79.8	1.64	1.36
x86	84.7	70.4	1.45	1.20
x71	70.9	58.9	1.21	1.00
x59	59.6	49.5	1.02	0.844
x52	53.0	43.9	0.904	0.749
x46	46.9	38.9	0.801	0.664
W200				
x42	47.7	40.1	0.814	0.684
x36	41.5	34.8	0.708	0.595
W200				
x31	39.5	33.8	0.675	0.577
x27	33.8	28.9	0.577	0.494
W200				
x22	32.5		0.555	
x19	28.4		0.485	
x15	22.1		0.378	
W150				
x37	49.1	40.8	0.839	0.697
x30	39.9	33.1	0.681	0.565
x22	30.4	25.2	0.519	0.430
W150				
x24	40.0	34.2	0.683	0.584
x18	30.6	26.0	0.522	0.445
x14	23.5	20.0	0.401	0.342
x13	22.0	18.8	0.376	0.320
W130				
x28		37.6		0.642
x24		32.1		0.547
W100				
x19		32.6		0.556

S SHAPES

Designation	SI (kg/m)/m		Imperial (lb./ft.)/in.	
	Beam	Column	Beam	Column
S610				
x180	104		1.78	
x158	91.4		1.56	
S610				
x149	89.9		1.54	
x134	81.4		1.39	
x119	72.3		1.23	
S510				
x143	97.8		1.67	
x128	88.3		1.51	
S510				
x112	79.8		1.36	
x98.2	70.6		1.21	
S460				
x104	81.4		1.39	
x81.4	63.8		1.09	
S380				
x74	67.7		1.16	
x64	58.2		0.993	
S310				
x74	79.9		1.36	
x60.7	65.6		1.12	
S310				
x52	56.8		0.969	
x47	51.7		0.882	
S250				
x52	65.1		1.11	
x38	47.6		0.813	
S200				
x34	52.3		0.892	
x27	41.9		0.715	
S150				
x26	50.0		0.853	
x19	36.5		0.624	
S130				
x15	34.1		0.581	
S100				
x14.1	38.5		0.657	
x11	31.4		0.536	
S75				
x11	37.8		0.645	
x8	28.9		0.494	

M/D RATIOS FOR CONTOUR PROTECTION
M SHAPES

Designation	SI (kg/m)/m		Imperial (lb./ft.)/in.		Designation	SI (kg/m)/m		Imperial (lb./ft.)/in.	
	Beam	Column	Beam	Column		Beam	Column	Beam	Column
M318									
x18.5	20.3		0.347						
x17.3	19.5		0.332						
M310									
x17.6	21.4		0.365						
x16.1	19.7		0.336						
x14.9	17.9		0.306						
M250									
x13.4	19.4		0.330						
x11.9	17.3		0.296						
x11.2	16.2		0.276						
M200									
x9.7	17.2		0.294						
x9.2	16.4		0.280						
M150									
x6.6	15.1		0.259						
x5.5	12.6		0.214						
M130									
x28.1		38.5		0.657					
M100									
x8.9		15.9		0.271					
x6.1	18.0	15.5	0.308	0.265					
M75									
x4.3	15.2	12.8	0.259	0.218					

COEFFICIENTS OF THERMAL EXPANSION

(Linear, per degree × 10^{-6})

METALS	c per °C	c per °F
Aluminum	23	13
Brass	19	10.4
Bronze	18	10.1
Copper	16.7	9.3
Iron, Gray Cast	11	5.9
Iron, Wrought	12	6.7
Lead	28.7	15.9
Magnesium	28.8	16
Nickel	12.6	7
Steel, Cast	11.3	6.3
Steel, Stainless	17.8	9.9
Steel, Structural	11.7	6.5
Zinc, Rolled	31	17.3

NON-METALS	c per °C	c per °F
Cement, Portland	13	7
Concrete, Stone	10	5.7
Glass	7	4
Granite	8.3	4.6
Limestone	7.9	4.4
Marble	9	5
Masonry, Ashlar	6.3	3.5
Masonry, Brick	6.1	3.4
Masonry, Rubble	6.3	3.5
Plaster	16	9
Sandstone	11	6
Slate	10	5.8
Fir (parallel to fibre)	3.8	2.1
Fir (perpendicular to fibre)	58	32

NOTE: Coefficients of thermal expansion indicated are average values from various sources. Minor variations may be expected in metals. Large variations may be expected in concrete and masonry due to the many combinations of constituents possible.

Coefficients apply in general to a temperature range from 0 to 100 degrees Celsius.

The coefficient of linear thermal expansion (c) is the change in length per unit of length for a change of one degree of temperature. The coefficient for surface expansion is approximately two times, and the coefficient of volume expansion is approximately three times, the linear coefficient.

Change in length = $c\,L$ × change in temperature, if member is free to elongate or contract.

Change in unit stress = $c\,E$ × change in temperature, if member is not permitted to elongate or contract (E = modulus of elasticity).

NOTES

CHECKLIST FOR DESIGN DRAWINGS

General

A design does not provide a satisfactory structure unless sufficient information is conveyed to the builder so that the designer's intentions are clearly understood. Furthermore, attempting to prepare an estimate for a structure from plans and specifications which contain insufficient information involves risks which tend to increase the tendered price. Clause 4.2 of CSA S16-14 governs the minimum requirements of design drawings. In addition, the following items are suggested as a checklist of information to be included on design drawings to avoid unnecessary and costly uncertainty at the time of bidding:

1. The type or types of design as defined in CSA S16-14. If plastic analysis is employed, it should be stated. Show the category of the structural system used for seismic design, as well as the seismic design criteria.
2. A list of design and material or product standards used. The grade(s) of structural steel, grade(s) and diameters of bolts.
3. All structural drawings to be adequately dimensioned, preferably in SI metric units. Do not intermix Metric and Imperial systems of units.
4. Centre-to-centre distances for all columns.
5. Outside dimensions of rigid frames and offset dimensions from grid lines to outside of rigid frames.
6. Out-to-out dimension of trusses and offset dimensions from centre line of chords to outside of chords–include any camber requirements.
7. Offset dimensions from centre of column lines to centre of beams for all beams that are not on the grid lines.
8. Relation of outside of exterior walls to centre lines of columns.
9. Relation of the top surfaces of beams to finished floor elevations.
10. Length of bearing for all beams bearing on exterior walls, including the dimension from the outside of the wall to the end of the steel beam and size of bearing plate.
11. Elevations of underside of column base plates.
12. Dimensions of all clear openings for doorways, ducts, stair wells, roof openings, etc., and their relation to adjacent steel members.
13. The specified dead, live, snow, rain, wind, seismic, and special loads, as well as design load criteria and/or parameters. Indicate whether loads and forces shown on drawings are factored or unfactored.
14. Axial loads in beams, columns and bracing members and joint pass-through forces. Forces and member sizes may be identified in beam or column schedules, or bracing elevation drawings.
15. Forces in truss members including moments when members are loaded between panel points.
16. Minimum end reactions required for all connections.
17. Moments for restrained beams and cantilevers. Governing combinations of shears, moments, and axial forces to be resisted by the connections.

18. All information necessary to design and manufacture the open-web steel joists and steel deck diaphragms to suit the loading conditions.
19. When a particular type of connection is required, the location and type of connection. Clear identification of structural connections that are critical for ductile seismic response. Locations and dimensions of protected zones.
20. Type of beam-to-column connection when beams frame over top of columns, including type and location of stiffeners.
21. Any bearing-type connections that are required to be pretensioned. The designation of joints as bearing or slip-critical.
22. For composite beams, the size and location of shear studs and which beams, if any, must be shored.
23. Size of column base plates and size and location of anchor rods or shear lugs. (Column bases require a minimum of four anchor rods unless special precautions are taken.)
24. Size and location of stiffeners, web doubler plates, reinforcement, and bracing required for stability of compression elements.
25. Details and location of built-up lintels.
26. Identify roof cladding systems that do not provide lateral restraint to the roof structure.
27. Reinforcement, where necessary, for openings through beam webs or openings in the steel deck diaphragm for rooftop units.
28. Ledger angles complete with method of attachment.
29. Members requiring prime paint or galvanizing.
30. Identify architecturally exposed structural steel elements requiring special tolerances and finishes. (Also refer to the CISC Code of Standard Practice in Part 7.)
31. Treatment of steel encased in concrete.
32. Fabrication and erection tolerances if other than those specified in CSA S16-14. Special tolerances when interfacing with other materials, i.e., steel attached to concrete.
33. A note that all structural welding is to be performed only by companies certified to Division 1 or 2.1 of CSA W47.1.
34. When weld symbols are shown, refer to "WELDED JOINTS Standard Symbols" in Part 6.

Allow as much time as possible (three weeks for an average job) for preparing bids. During the time allotted for preparing tenders, only those changes necessary to clarify bidding instructions should be issued by addendum. If major changes are included in an addendum, an extension of the tender closing should be considered.

PROPERTIES OF GEOMETRIC SECTIONS
Definitions

Neutral Axis

The line, in any given section of a member subject to bending, on which there is neither tension nor compression.

For pure elastic bending of a straight beam, the neutral axis at any cross-section is coincident with the centroidal axis of the cross-section.

In the case of fully plastic bending, the neutral axis divides the sectional area equally. Therefore, the neutral axis for elastic and plastic bending coincide only in the case of sections symmetrical about the neutral axis.

Moment of Inertia I

The sum of the products obtained by multiplying each of the elementary areas, of which the section is composed, by the square of its perpendicular distance from the axis about which the moment of inertia is being calculated.

Elastic Section Modulus S

The moment of inertia divided by the perpendicular distance from the axis about which the moment of inertia has been calculated to the most remote part of the section.

The elastic section modulus is used to determine the bending stress in the extreme fibre of a section by dividing the bending moment by the section modulus, referred to the neutral axis perpendicular to the plane of bending, both values being expressed in like units of measure.

Radius of Gyration r

The perpendicular distance from a neutral axis to the centre of gyration (i.e., the point where the entire area is considered to be concentrated so as to have the same moment of inertia as the actual area). The square of the radius of gyration of a section is equal to the moment of inertia (referred to the appropriate axis) divided by the area.

The radius of gyration of a section is used to ascertain the load this section will sustain when used in compression as a strut or column. The ratio of the effective unsupported length of the section divided by the least radius of gyration applicable to this length is called the slenderness ratio.

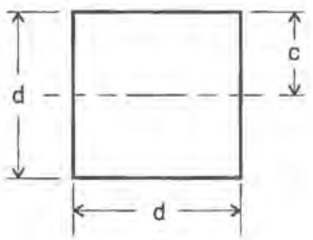
Plastic Modulus Z

The modulus of resistance to bending of a completely yielded cross-section, calculated by taking the combined statical moment, about the neutral axis, of the cross-sectional areas above and below that axis.

In general, the plastic modulus is calculated by simple statics and has been included for only a few of the shapes listed.

SQUARE

Axis of moments through centre



$A = d^2$

$c = \frac{d}{2}$

$I = \frac{d^4}{12}$

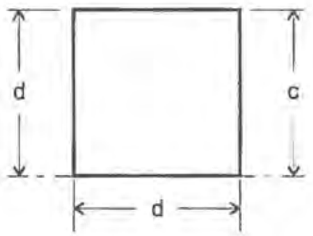
$S = \frac{d^3}{6}$

$r = \frac{d}{\sqrt{12}}$

$Z = \frac{d^3}{4}$

SQUARE

Axis of moments on base



$A = d^2$

$c = d$

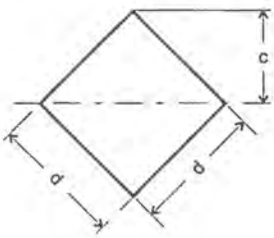
$I = \frac{d^4}{3}$

$S = \frac{d^3}{3}$

$r = \frac{d}{\sqrt{3}}$

SQUARE

Axis of moments on diagonal



$A = d^2$

$c = \frac{d}{\sqrt{2}}$

$I = \frac{d^4}{12}$

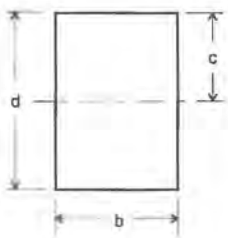
$S = \frac{d^3}{6\sqrt{2}}$

$r = \frac{d}{\sqrt{12}}$

$Z = \frac{2c^3}{3} = \frac{d^3}{3\sqrt{2}}$

RECTANGLE

Axis of moments through centre



$A = bd$

$c = \frac{d}{2}$

$I = \frac{bd^3}{12}$

$S = \frac{bd^2}{6}$

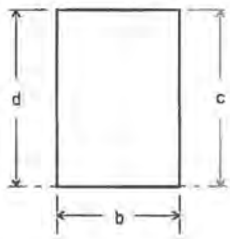
$r = \frac{d}{\sqrt{12}}$

$Z = \frac{bd^2}{4}$

PROPERTIES OF GEOMETRIC SECTIONS

RECTANGLE

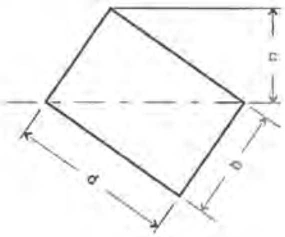
Axis of moments on base



$$A = bd$$
$$c = d$$
$$I = \frac{bd^3}{3}$$
$$S = \frac{bd^2}{3}$$
$$r = \frac{d}{\sqrt{3}}$$

RECTANGLE

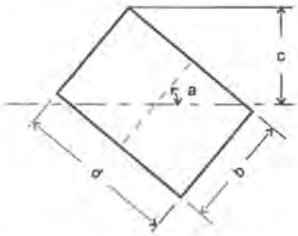
Axis of moments on diagonal



$$A = bd$$
$$c = \frac{bd}{\sqrt{b^2 + d^2}}$$
$$I = \frac{b^3d^3}{6\left(b^2 + d^2\right)}$$
$$S = \frac{b^2d^2}{6\sqrt{b^2 + d^2}}$$
$$r = \frac{bd}{\sqrt{6\left(b^2 + d^2\right)}}$$

RECTANGLE

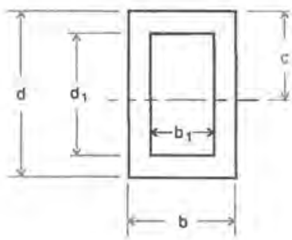
Axis of moments any line through centre of gravity



$$A = bd$$
$$c = \frac{b \sin a + d \cos a}{2}$$
$$I = \frac{bd\left(b^2 \sin^2 a + d^2 \cos^2 a\right)}{12}$$
$$S = \frac{bd\left(b^2 \sin^2 a + d^2 \cos^2 a\right)}{6\left(b \sin a + d \cos a\right)}$$
$$r = \sqrt{\frac{b^2 \sin^2 a + d^2 \cos^2 a}{12}}$$

HOLLOW RECTANGLE

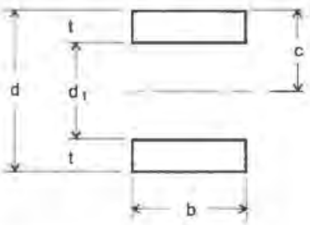
Axis of moments through centre



$$A = bd - b_1d_1$$
$$c = \frac{d}{2}$$
$$I = \frac{bd^3 - b_1d_1^3}{12}$$
$$S = \frac{bd^3 - b_1d_1^3}{6d}$$
$$r = \sqrt{\frac{bd^3 - b_1d_1^3}{12A}}$$
$$Z = \frac{1}{4}\left(bd^2 - b_1d_1^2\right)$$

EQUAL RECTANGLES

Axis of moments through centre of gravity



$$A = b(d - d_1)$$

$$c = \frac{d}{2}$$

$$I = \frac{b(d^3 - d_1^3)}{12}$$

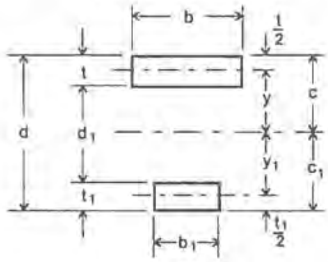
$$S = \frac{b(d^3 - d_1^3)}{6d}$$

$$r = \sqrt{\frac{d^3 - d_1^3}{12(d - d_1)}}$$

$$Z = \frac{b}{4}(d^2 - d_1^2) = bt(d - t)$$

UNEQUAL RECTANGLES

Axis of moments through centre of gravity



$$A = bt + b_1t_1$$

$$c = \frac{\frac{1}{2}bt^2 + b_1t_1(d - \frac{1}{2}t_1)}{A}$$

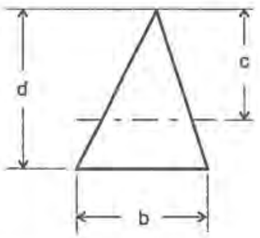
$$I = \frac{bt^3}{12} + bty^2 + \frac{b_1t_1^3}{12} + b_1t_1y_1^2$$

$$S = \frac{I}{c} \qquad S_1 = \frac{I}{c_1}$$

$$r = \sqrt{\frac{I}{A}}$$

TRIANGLE

Axis of moments through centre of gravity



$$A = \frac{bd}{2}$$

$$c = \frac{2d}{3}$$

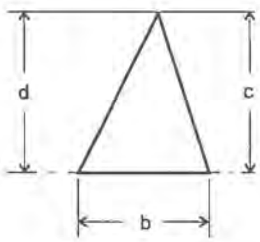
$$I = \frac{bd^3}{36}$$

$$S = \frac{bd^2}{24}$$

$$r = \frac{d}{\sqrt{18}}$$

TRIANGLE

Axis of moments on base



$$A = \frac{bd}{2}$$

$$c = d$$

$$I = \frac{bd^3}{12}$$

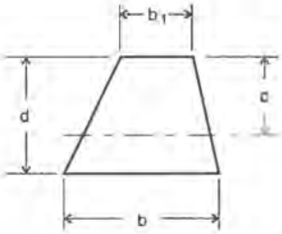
$$S = \frac{bd^2}{12}$$

$$r = \frac{d}{\sqrt{6}}$$

PROPERTIES OF GEOMETRIC SECTIONS

TRAPEZOID

Axis of moments through centre of gravity



$$A = \frac{d(b + b_1)}{2}$$

$$c = \frac{d(2b + b_1)}{3(b + b_1)}$$

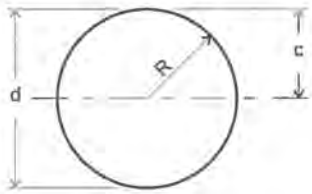
$$I = \frac{d^3(b^2 + 4bb_1 + b_1^2)}{36(b + b_1)}$$

$$S = \frac{d^2(b^2 + 4bb_1 + b_1^2)}{12(2b + b_1)}$$

$$r = \frac{d}{6(b + b_1)}\sqrt{2(b^2 + 4bb_1 + b_1^2)}$$

CIRCLE

Axis of moments through centre



$$A = \frac{\pi d^2}{4} = \pi R^2$$

$$c = \frac{d}{2} = R$$

$$I = \frac{\pi d^4}{64} = \frac{\pi R^4}{4}$$

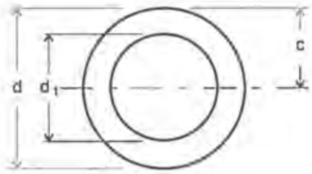
$$S = \frac{\pi d^3}{32} = \frac{\pi R^3}{4}$$

$$r = \frac{d}{4} = \frac{R}{2}$$

$$Z = \frac{d^3}{6}$$

HOLLOW CIRCLE

Axis of moments through centre



$$A = \frac{\pi(d^2 - d_1^2)}{4}$$

$$c = \frac{d}{2}$$

$$I = \frac{\pi(d^4 - d_1^4)}{64}$$

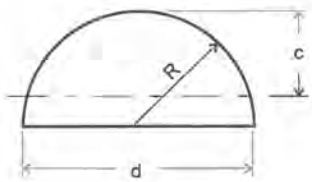
$$S = \frac{\pi(d^4 - d_1^4)}{32d}$$

$$r = \frac{\sqrt{d^2 + d_1^2}}{4}$$

$$Z = \frac{1}{6}(d^3 - d_1^3)$$

HALF CIRCLE

Axis of moments through centre of gravity



$$A = \frac{\pi R^2}{2}$$

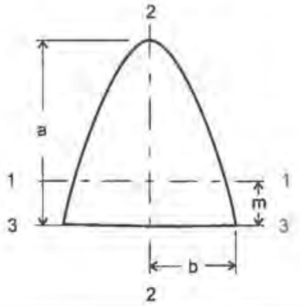
$$c = R\left(1 - \frac{4}{3\pi}\right)$$

$$I = R^4\left(\frac{\pi}{8} - \frac{8}{9\pi}\right)$$

$$S = \frac{R^3}{24}\frac{(9\pi^2 - 64)}{(3\pi - 4)}$$

$$r = R\frac{\sqrt{9\pi^2 - 64}}{6\pi}$$

PARABOLA



$$A = \frac{4}{3}ab$$

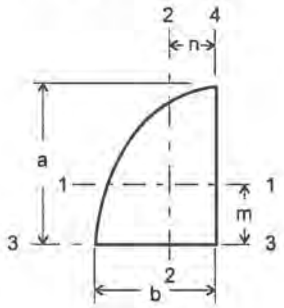
$$m = \frac{2}{5}a$$

$$I_1 = \frac{16}{175}a^3L$$

$$I_2 = \frac{4}{15}ab^3$$

$$I_3 = \frac{32}{105}a^3b$$

HALF PARABOLA



$$A = \frac{2}{3}ab$$

$$m = \frac{2}{5}a$$

$$n = \frac{3}{8}b$$

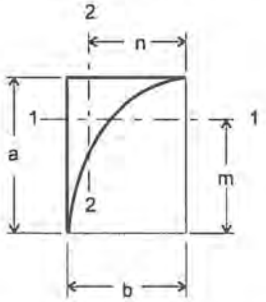
$$I_1 = \frac{8}{175}a^3b$$

$$I_2 = \frac{19}{480}ab^3$$

$$I_3 = \frac{16}{105}a^3b$$

$$I_4 = \frac{2}{15}ab^3$$

COMPLEMENT OF HALF PARABOLA



$$A = \frac{1}{3}ab$$

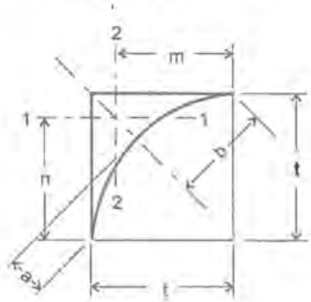
$$m = \frac{7}{10}a$$

$$n = \frac{3}{4}b$$

$$I_1 = \frac{37}{2100}a^3b$$

$$I_2 = \frac{1}{80}ab^3$$

PARABOLIC FILLET IN RIGHT ANGLE



$$a = \frac{t}{2\sqrt{2}}$$

$$b = \frac{t}{\sqrt{2}}$$

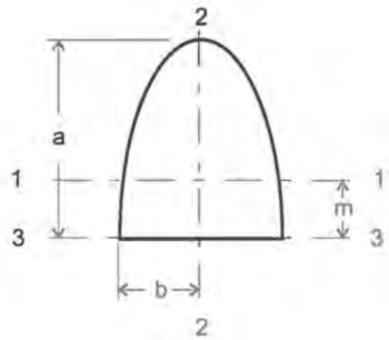
$$A = \frac{1}{6}t^2$$

$$m = n = \frac{4}{5}t$$

$$I_1 = I_2 = \frac{11}{2100}t^4$$

PROPERTIES OF GEOMETRIC SECTIONS

* HALF ELLIPSE



$$A = \frac{1}{2}\pi ab$$

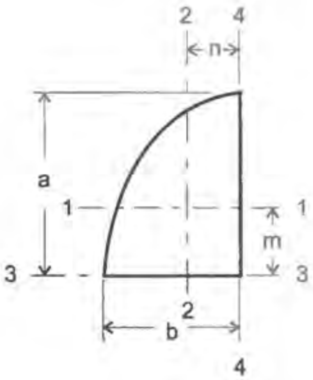
$$m = \frac{4a}{3\pi}$$

$$I_1 = a^3b\left(\frac{\pi}{8} - \frac{8}{9\pi}\right)$$

$$I_2 = \frac{1}{8}\pi ab^3$$

$$I_3 = \frac{1}{8}\pi a^3b$$

* QUARTER ELLIPSE



$$A = \frac{1}{4}\pi ab \qquad I_1 = a^3b\left(\frac{\pi}{16} - \frac{4}{9\pi}\right)$$

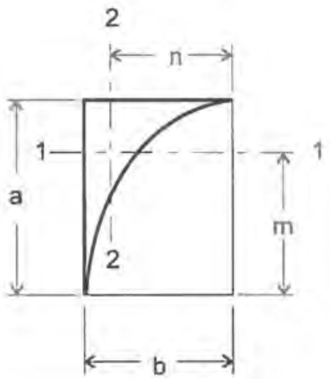
$$m = \frac{4a}{3\pi} \qquad I_2 = ab^3\left(\frac{\pi}{16} - \frac{4}{9\pi}\right)$$

$$n = \frac{4b}{3\pi} \qquad I_3 = \frac{1}{16}\pi a^3b$$

$$I_4 = \frac{1}{16}\pi ab^3$$

* ELLIPTIC COMPLEMENT

$$A = ab\left(1 - \frac{\pi}{4}\right), \quad m = \frac{a}{6\left(1 - \frac{\pi}{4}\right)}, \quad n = \frac{b}{6\left(1 - \frac{\pi}{4}\right)}$$

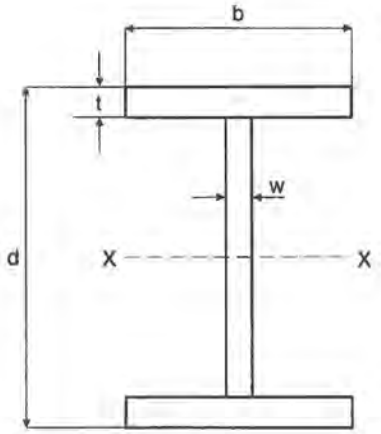


$$I_1 = a^3b\left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{36\left(1 - \frac{\pi}{4}\right)}\right)$$

$$I_2 = ab^3\left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{36\left(1 - \frac{\pi}{4}\right)}\right)$$

* To obtain properties of half circle, quarter circle and circle complement substitute $a = b = R$.

PROPERTIES OF GEOMETRIC SECTIONS AND STRUCTURAL SHAPES



$$A = 2bt + (d - 2t)w$$

$$I = \frac{1}{12}\left[bd^3 - (b - w)(d - 2t)^3\right]$$

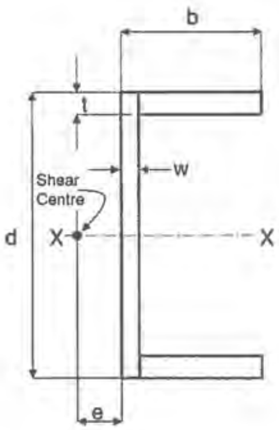
$$S = \frac{1}{6d}\left[bd^3 - (b - w)(d - 2t)^3\right]$$

$$r = \sqrt{\frac{I}{A}}$$

$$Z = \frac{1}{4}\left[bd^2 - (b - w)(d - 2t)^2\right]$$

$$J = \frac{1}{3}\left[2bt^3 + (d - t)w^3\right]$$

$$C_w = \frac{1}{24}(d - t)^2 b^3 t$$



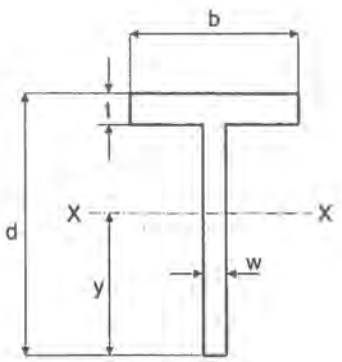
$$A = dw + 2(b - w)t$$

$$I = \frac{1}{12}\left[bd^3 - (b - w)(d - 2t)^3\right]$$

$$S = \frac{1}{6d}\left[bd^3 - (b - w)(d - 2t)^3\right]$$

$$r = \sqrt{\frac{I}{A}}$$

$$e = \frac{3t(b - w/2)^2}{6t(b - w/2) + (d - t)w} - \frac{w}{2}$$



$$A = bt + w(d - t)$$

$$y = \frac{1}{2}\left(\frac{bdt}{A} + d - t\right)$$

$$I = \frac{1}{12}\left[bt^3 + w(d - t)^3 + \frac{3bwtd^2(d - t)}{A}\right]$$

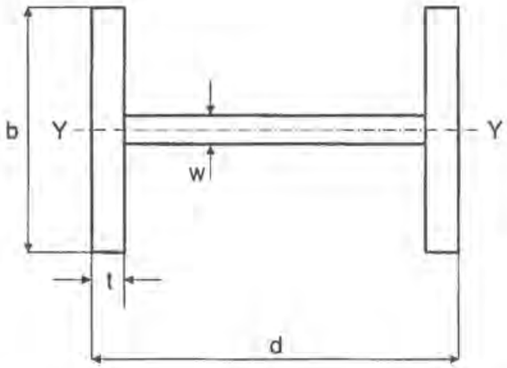
$$S_1 = \frac{I}{y}; \quad S_2 = \frac{I}{d - y}$$

$$r = \sqrt{\frac{I}{A}}$$

$$J = \frac{1}{3}\left[bt^3 + \left(d - \frac{t}{2}\right)w^3\right]$$

$$C_w = \frac{b^3t^3}{144} + \frac{\left(d - \frac{t}{2}\right)^3 w^3}{36}$$

PROPERTIES OF GEOMETRIC SECTIONS AND STRUCTURAL SHAPES



$$A = 2bt + w(d - 2t)$$

$$I = \frac{1}{12}\left[2tb^3 + (d - 2t)w^3\right]$$

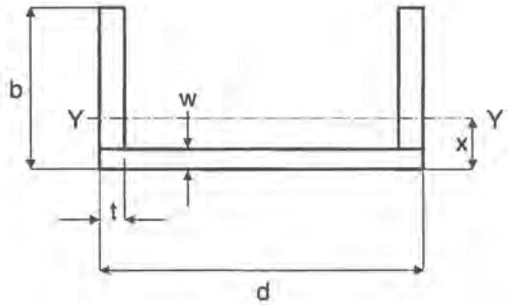
$$S = \frac{1}{6b}\left[2tb^3 + (d - 2t)w^3\right]$$

$$r = \sqrt{\frac{I}{A}}$$

$$Z = \frac{1}{4}\left[2t\left(b^2 - w^2\right) + dw^2\right]$$

$$J = \frac{1}{3}\left[2bt^3 + (d - t)w^3\right]$$

$$C_w = \frac{1}{24}(d - t)^2 b^3 t$$



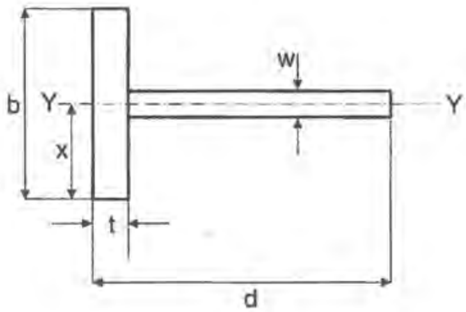
$$A = dw + 2(b - w)t$$

$$x = \frac{1}{2A}\left[(d - 2t)w^2 + 2tb^2\right]$$

$$I = \frac{1}{3}\left[dx^3 + 2t(b - x)^3 - (d - 2t)(x - w)^3\right]$$

$$S_1 = \frac{I}{b - x}; \qquad S_2 = \frac{I}{x}$$

$$r = \sqrt{\frac{I}{A}}$$



$$A = bt + (d - t)w$$

$$x = b/2$$

$$I = \frac{1}{12}\left[tb^3 + (d - t)w^3\right]$$

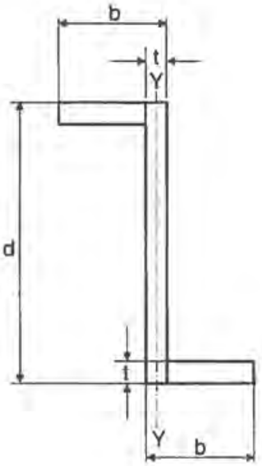
$$S = \frac{2I}{b}$$

$$r = \sqrt{\frac{I}{A}}$$

$$J = \frac{1}{3}\left[bt^3 + \left(d - \frac{t}{2}\right)w^3\right]$$

$$C_w = \frac{b^3t^3}{144} + \frac{\left(d - \frac{t}{2}\right)^3 w^3}{36}$$

PROPERTIES OF GEOMETRIC SECTIONS AND STRUCTURAL SHAPES



$$J = \frac{1}{3}\left[2b + d - 2t\right]t^3$$

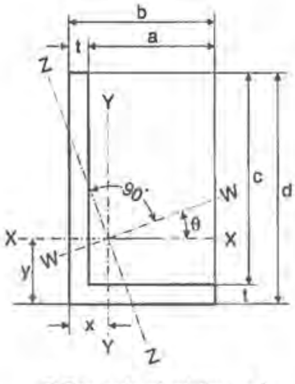
$$C_w = \frac{(d-t)^2(b-t/2)^3 t}{12}\left[\frac{b+2d-5t/2}{d+2b-2t}\right].$$

See next page for other section properties.

ANGLE

Axis of moments through

Centre of gravity



Z-Z is axis of minimum I

$$\tan 2\theta = \frac{2K}{I_y - I_x}$$

$$A = t(b + c) \qquad x = \frac{b^2 + ct}{2(b+c)} \qquad y = \frac{d^2 + at}{2(b+c)}$$

K = Product of Inertia about X-X & Y-Y

$$= \mp \frac{abcdt}{4(b+c)}$$

$$I_x = \frac{1}{3}\left[t(d-y)^3 + by^3 - a(y-t)^3\right]$$

$$I_y = \frac{1}{3}\left[t(b-x)^3 + dx^3 - c(x-t)^3\right]$$

$$I_z = I_x \sin^2\theta + I_y \cos^2\theta + K \sin 2\theta$$

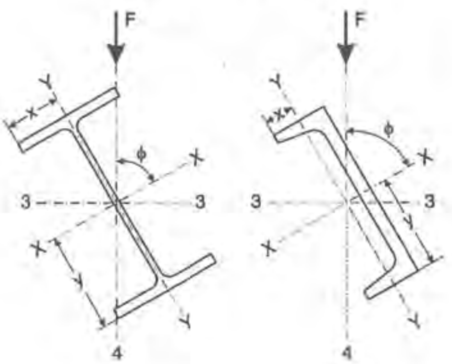
$$I_w = I_x \cos^2\theta + I_y \sin^2\theta \mp K \sin 2\theta$$

K is negative when heel of angle, with respect to c.g., is in 1st or 3rd quadrant, positive when in 2nd or 4th quadrant.

BEAMS AND CHANNELS

Transverse force oblique

through centre of gravity



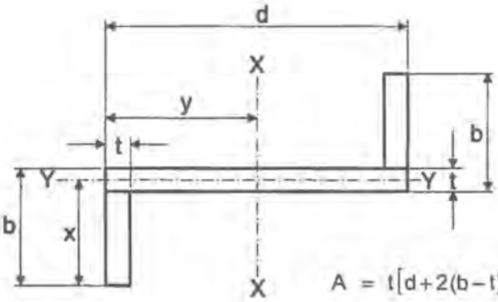
$$I_3 = I_x \sin^2\phi + I_y \cos^2\phi$$

$$I_4 = I_x \cos^2\phi + I_y \sin^2\phi$$

$$f = M\left(\frac{y}{I_x}\sin\phi + \frac{x}{I_y}\cos\phi\right)$$

where M is bending moment due to force F.

PROPERTIES OF GEOMETRIC SECTIONS AND STRUCTURAL SHAPES



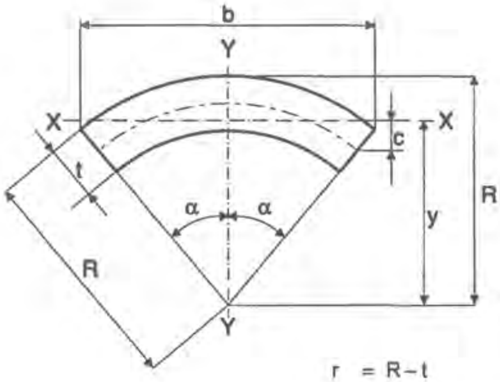
$$A = t[d + 2(b - t)]$$

$$x = b - \frac{t}{2}$$

$$I_x = \frac{1}{12}\left[d^3 b - (d - 2t)^3 (b - t)\right] \qquad I_y = \frac{1}{12}\left[(2b - t)^3 t + (d - t)t^3\right]$$

$$S_x = \frac{I_x}{y} \qquad S_y = \frac{I_y}{x}$$

$$r_x = \sqrt{\frac{I_x}{A}} \qquad r_y = \sqrt{\frac{I_y}{A}}$$



$$r = R - t$$

$$A = \frac{\pi\alpha}{180}(R^2 - r^2)$$

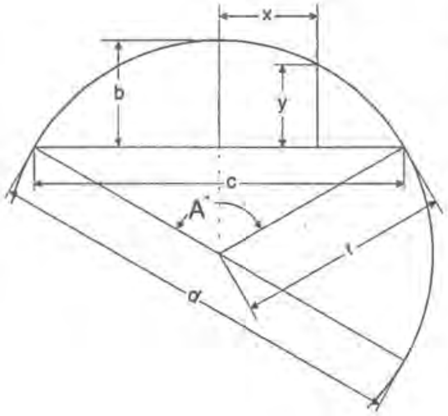
$$b = 2R\sin\alpha$$

$$c = \frac{120\sin\alpha\,(R^3 - r^3)}{\pi\alpha(R^2 - r^2)} - \frac{(R + r)\cos\alpha}{2}$$

$$y = \frac{120\sin\alpha\,(R^3 - r^3)}{\pi\alpha(R^2 - r^2)}$$

$$I_x = \frac{1}{4}(R^4 - r^4)\left(\frac{\pi\alpha}{180} + \sin\alpha\cos\alpha\right) - \frac{80\sin^2\alpha(R^3 - r^3)^2}{\pi\alpha(R^2 - r^2)}$$

$$I_y = \frac{1}{4}(R^4 - r^4)\left(\frac{\pi\alpha}{180} - \sin\alpha\cos\alpha\right)$$



Circumference = 6.28318r = 3.14159d
Diameter = 0.31831 circumference
Area = $3.14159r^2$

Arc a $= \dfrac{\pi r A^\circ}{180^\circ} = 0.017453 r A^\circ$

Angle A° $= \dfrac{180^\circ a}{\pi r} = 57.29578 \dfrac{a}{r}$

Radius r $= \dfrac{4b^2 + c^2}{8b}$

Chord c $= 2\sqrt{2br - b^2} = 2r\sin\dfrac{A}{2}$

Rise b $= r - \dfrac{1}{2}\sqrt{4r^2 - c^2} = \dfrac{c}{2}\tan\dfrac{A}{4}$

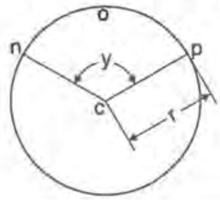
$= 2r\sin^2\dfrac{A}{4} = r + y - \sqrt{r^2 - x^2}$

y $= b - r + \sqrt{r^2 - x^2}$

x $= \sqrt{r^2 - (r + y - b)^2}$

Diameter of circle of equal periphery as square = 1.27324 side of square
Side of square of equal periphery as circle = 0.78540 diameter of circle
Diameter of circle circumscribed about square = 1.41421 side of square
Side of square inscribed in circle = 0.70711 diameter of circle

CIRCULAR SECTOR



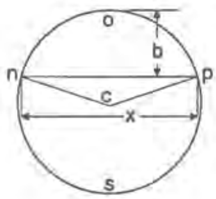
r = radius of circle, y = angle ncp in degrees

Area of Sector ncpo $= \frac{1}{2}(\text{length of arc nop} \times r)$

$= \text{Area of Circle} \times \dfrac{y}{360}$

$= 0.0087266 \times r^2 \times y$

CIRCULAR SEGMENT



r = radius of circle, x = chord, b = rise

Area of Segment nop = Area of Sector ncpo − Area of triangle ncp

$= \dfrac{(\text{Length of arc nop} \times r) - x(r - b)}{2}$

Area of Segment nsp = Area of Circle − Area of Segment nop

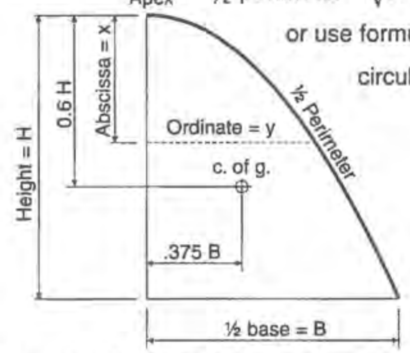
PROPERTIES OF PARABOLA AND ELLIPSE

PARABOLA

When $H \div B = 0.1$ or less, approximate

$\frac{1}{2}$ perimeter $= \sqrt{B^2 + 4/3\,H^2}$

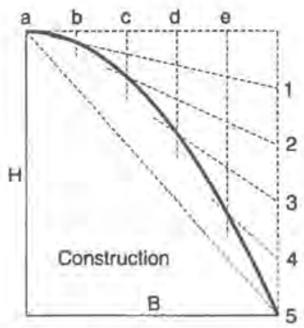
or use formulas for circular arcs



Parameter $P = B^2 \div H$ Area $= \frac{2}{3} HB$

$x = y^2 \div P$

$y = \sqrt{xP}$



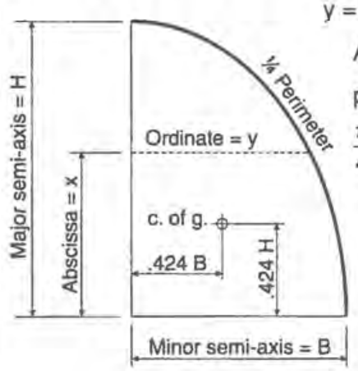
ELLIPSE

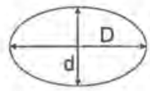
$$\left(x^2 \div H^2\right) + \left(y^2 \div B^2\right) = 1$$

$$x = (H \div B)\sqrt{B^2 - y^2}$$

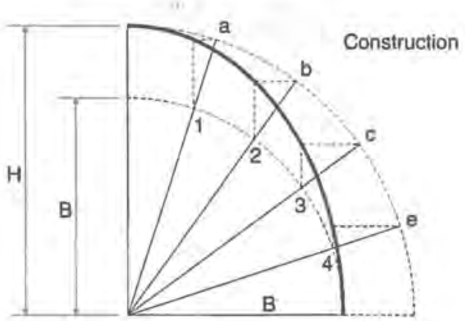
$$y = (B \div H)\sqrt{H^2 - x^2}$$

Approximate ¼ perimeter $= \frac{\pi}{4}\sqrt{2\left(H^2 + B^2\right)}$



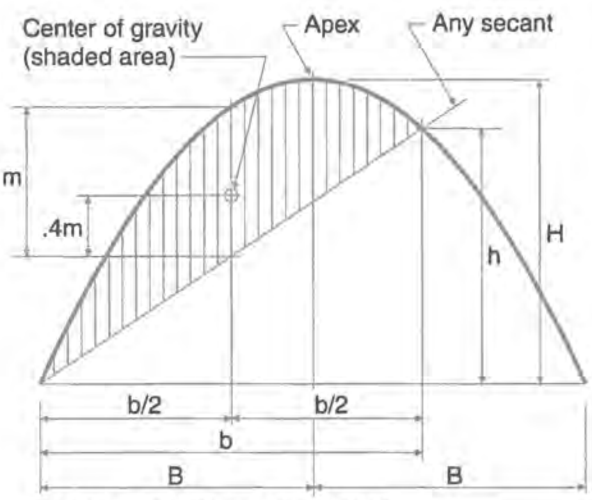


Area = .7854 Dd



Construction

AREA BETWEEN PARABOLIC CURVE AND SECANT



$$h = Hb\left(\frac{2B - b}{B^2}\right)$$

$$m = \frac{Hb^2}{4B^2}$$

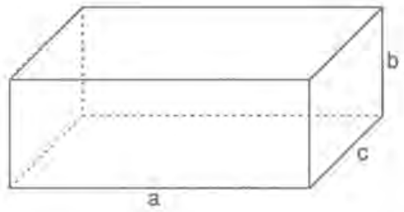
$$\text{Shaded area} = \tfrac{2}{3}bm = \frac{Hb^3}{6B^2}$$

Length b may vary from 0 to 2B

RECTANGULAR PARALLELEPIPED

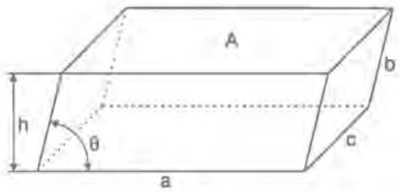
Volume = abc

Surface area = 2(ab + ac + bc)



PARALLELEPIPED

Volume = Ah = abc sinθ

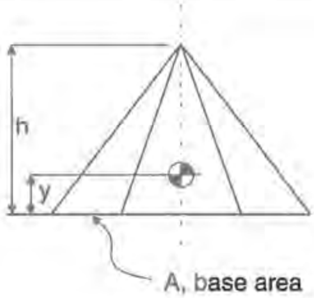


PYRAMID

$$\text{Volume} = \frac{1}{3}Ah$$

The centroid of a pyramid is located y-distance from the base on the line joining the centre of gravity of area A and the apex.

$$y = \frac{h}{4}$$

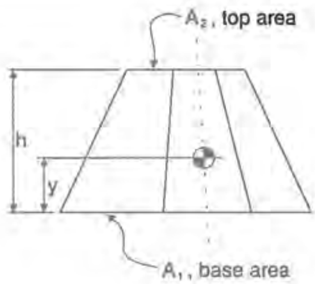


FRUSTUM OF PYRAMID

$$V = \frac{h}{3}\left(A_1 + A_2 + \sqrt{A_1A_2}\right)$$

The centroid is located y-distance up from area A_1 on the line joining the centres of gravity of areas A_1 and A_2.

$$y = \frac{h\left(A_1 + 2\sqrt{A_1A_2} + 3A_2\right)}{4\left(A_1 + \sqrt{A_1A_2} + A_2\right)}$$

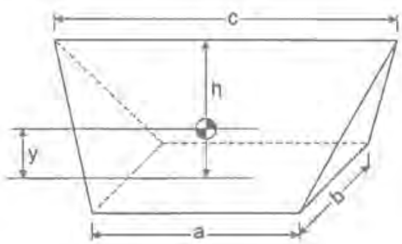


WEDGE

$$V = \frac{(2a + c)\,bh}{6}$$

The centroid is located y-distance from the base on the line joining the centre of gravity of the base area and the mid point of edge, c.

$$y = \frac{h\,(a + c)}{2\,(2a + c)}$$



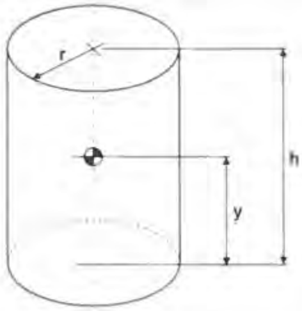
PROPERTIES OF SOLIDS

RIGHT CIRCULAR CYLINDER

Volume $= \pi r^2 h$

Lateral surface area $= 2\pi r h$

$$y = \frac{h}{2}$$

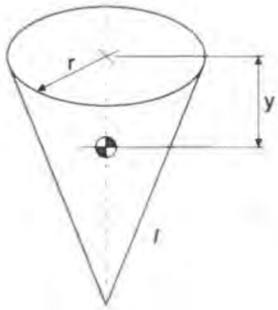


RIGHT CIRCULAR CONE

Volume $= \frac{1}{3}\pi r^2 h$

Lateral surface area $= \pi r\sqrt{r^2 + h^2} = \pi r l$

$$y = \frac{h}{4}$$

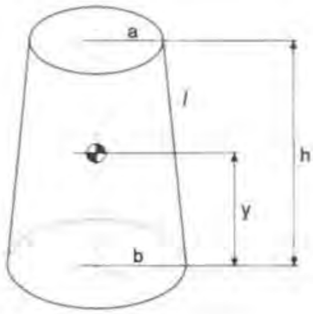


FRUSTUM OF RIGHT CIRCULAR CONE

Volume $= \frac{1}{3}\pi h\left(a^2 + ab + b^2\right)$

Lateral surface area $= \pi(a + b)\sqrt{h^2 + (b - a)^2}$
$= \pi(a + b)l$

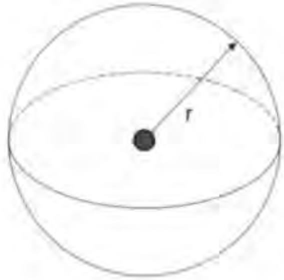
$$y = \frac{h\left(b^2 + 2ab + 3a^2\right)}{4\left(b^2 + ab + a^2\right)}$$



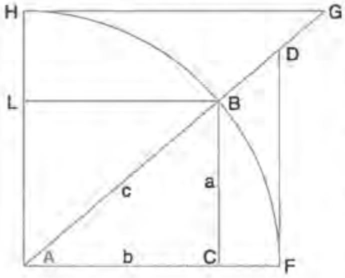
SPHERE

Volume $= \frac{4}{3}\pi r^3$

Surface area $= 4\pi r^2$

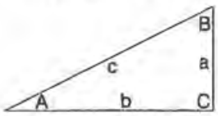


TRIGONOMETRIC FUNCTIONS



Radius AF	$= 1$		
	$= \sin^2 A + \cos^2 A = \sin A \operatorname{cosec} A$		
	$= \cos A \sec A = \tan A \cot A$		
Sine A	$= \dfrac{\cos A}{\cot A} = \dfrac{1}{\operatorname{cosec} A} = \cos A \tan A = \sqrt{1-\cos^2 A}$		$= BC$
Cosine A	$= \dfrac{\sin A}{\tan A} = \dfrac{1}{\sec A} = \sin A \cot A = \sqrt{1-\sin^2 A}$		$= AC$
Tangent A	$= \dfrac{\sin A}{\cos A} = \dfrac{1}{\cot A} = \sin A \sec A$		$= FD$
Cotangent A	$= \dfrac{\cos A}{\sin A} = \dfrac{1}{\tan A} = \cos A \operatorname{cosec} A$		$= HG$
Secant A	$= \dfrac{\tan A}{\sin A} = \dfrac{1}{\cos A}$		$= AD$
Cosecant A	$= \dfrac{\cot A}{\cos A} = \dfrac{1}{\sin A}$		$= AG$

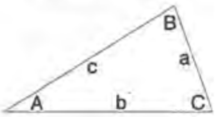
RIGHT ANGLED TRIANGLES



$$a^2 = c^2 - b^2$$
$$b^2 = c^2 - a^2$$
$$c^2 = a^2 + b^2$$

Known	Required A	B	a	b	c	Area
a, b	$\tan A = \dfrac{a}{b}$	$\tan B = \dfrac{b}{a}$			$\sqrt{a^2+b^2}$	$\dfrac{ab}{2}$
a, c	$\sin A = \dfrac{a}{c}$	$\cos B = \dfrac{a}{c}$		$\sqrt{c^2-a^2}$		$\dfrac{a\sqrt{c^2-a^2}}{2}$
A, a		$90° - A$		$a \cot A$	$\dfrac{a}{\sin A}$	$\dfrac{a^2 \cot A}{2}$
A, b		$90° - A$	$b \tan A$		$\dfrac{b}{\cos A}$	$\dfrac{b^2 \tan A}{2}$
A, c		$90° - A$	$c \sin A$	$c \cos A$		$\dfrac{c^2 \sin 2A}{4}$

OBLIQUE ANGLED TRIANGLES

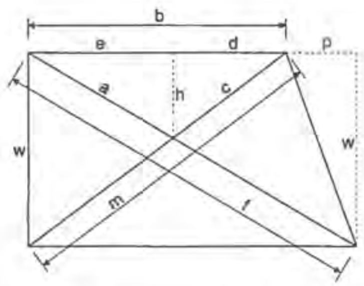


$$s = \frac{a+b+c}{2}$$
$$K = \sqrt{\frac{(s-a)(s-b)(s-c)}{s}}$$

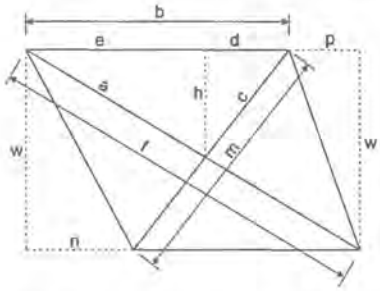
$$a^2 = b^2 + c^2 - 2bc\cos A$$
$$b^2 = a^2 + c^2 - 2ac\cos B$$
$$c^2 = a^2 + b^2 - 2ab\cos C$$

Known	Required A	B	C	b	c	Area
a, b, c	$\tan\frac{1}{2}A = \dfrac{K}{s-a}$	$\tan\frac{1}{2}B = \dfrac{K}{s-b}$	$\tan\frac{1}{2}C = \dfrac{K}{s-c}$			$\sqrt{s(s-a)(s-b)(s-c)}$
a, A, B			$180° - (A+B)$	$\dfrac{a \sin B}{\sin A}$	$\dfrac{a \sin C}{\sin A}$	
a, b, A		$\sin B = \dfrac{b \sin A}{a}$			$\dfrac{b \sin C}{\sin B}$	
a, b, C	$\tan A = \dfrac{a \sin C}{b - a\cos C}$				$\sqrt{a^2+b^2-2ab\cos C}$	$\dfrac{ab \sin C}{2}$

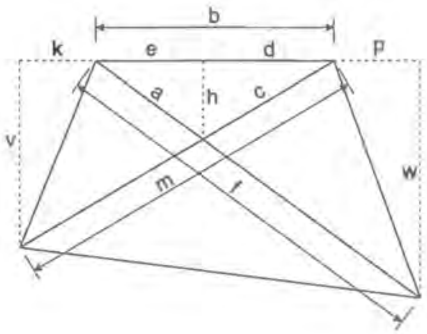
BRACING FORMULAE



Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2 + w^2}$
bw	m	$\sqrt{b^2 + w^2}$
bp	d	$b^2 \div (2b + p)$
bp	e	$b(b+p) \div (2b+p)$
bfp	a	$bf \div (2b+p)$
bmp	c	$bm \div (2b+p)$
bpw	h	$bw \div (2b+p)$
afw	h	$aw \div f$
cmw	h	$cw \div m$

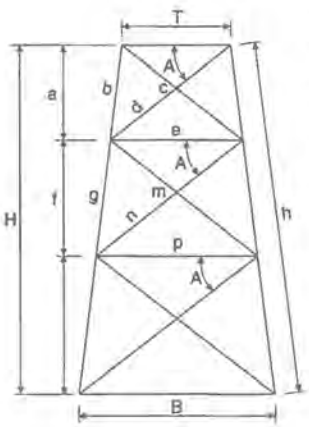


Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2 + w^2}$
bnw	m	$\sqrt{(b-n)^2 + w^2}$
bnp	d	$b(b-n) \div (2b+p-n)$
bnp	e	$b(b+p) \div (2b+p-n)$
bfnp	a	$bf \div (2b+p-n)$
bmnp	c	$bm \div (2b+p-n)$
bnpw	h	$bw \div (2b+p-n)$
afw	h	$aw \div f$
cmw	h	$cw \div m$



Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2 + w^2}$
bkv	m	$\sqrt{(b+k)^2 + v^2}$
bkpvw	d	$bw(b+k) \div [v(b+p) + w(b+k)]$
bkpvw	e	$bv(b+p) \div [v(b+p) + w(b+k)]$
bfkpvw	a	$fbv \div [v(b+p) + w(b+k)]$
bkmpvw	c	$bmw \div [v(b+p) + w(b+k)]$
bkpvw	h	$bvw \div [v(b+p) + w(b+k)]$
afw	h	$aw \div f$
cmv	h	$cv \div m$

PARALLEL BRACING



$k = (\log B - \log T) \div$ no. of panels. Constant k plus the logarithm of any line equals the log of the corresponding line in the next panel below.

$a = TH \div (T + e + p)$

$b = Th \div (T + e + p)$

$c = \sqrt{(\tfrac{1}{2}T + \tfrac{1}{2}e)^2 + a^2}$

$d = ce \div (T + e)$

$\log e = k + \log T$

$\log f = k + \log a$

$\log g = k + \log b$

$\log m = k + \log c$

$\log n = k + \log d$

$\log p = k + \log e$

The above method can be used for any number of panels.
In the formulas for "a" and "b" the sum in parenthesis, which in the case shown is $(T + e + p)$, is always composed of all the horizontal distances except the base.

LENGTH OF CIRCULAR ARCS FOR UNIT RADIUS

By the use of this table, the length of any arc may be found if the length of the radius and the angle of the segment are known.

Example: Required the length of arc of segment 32° 15' 27" with radius of 8 000 mm.

From table: Length of arc (Radius 1) for 32° = .5585054
15' = .0043633
27" = .0001309
.5629996

5629996 X 8 000 (length of radius) = 4504 mm

For the same arc but with the radius expressed as 24 feet 3 inches, the length of arc would be 0.5629996 X 24.25 = 13.65 feet

DEGREES						MINUTES		SECONDS	
1	.017 4533	61	1.064 6508	121	2.111 8484	1	.000 2909	1	.000 0048
2	.034 9066	62	1.082 1041	122	2.129 3017	2	.000 5818	2	.000 0097
3	.052 3599	63	1.099 5574	123	2.146 7550	3	.000 8727	3	.000 0145
4	.069 8132	64	1.117 0107	124	2.164 2083	4	.001 1636	4	.000 0194
5	.087 2665	65	1.134 4640	125	2.181 6616	5	.001 4544	5	.000 0242
6	.104 7198	66	1.151 9173	126	2.199 1149	6	.001 7453	6	.000 0291
7	.122 1730	67	1.169 3706	127	2.216 5682	7	.002 0362	7	.000 0339
8	.139 6263	68	1.186 8239	128	2.234 0214	8	.002 3271	8	.000 0388
9	.157 0796	69	1.204 2772	129	2.251 4747	9	.002 6180	9	.000 0436
10	.174 5329	70	1.221 7305	130	2.268 9280	10	.002 9089	10	.000 0485
11	.191 9862	71	1.239 1838	131	2.286 3813	11	.003 1998	11	.000 0533
12	.209 4395	72	1.256 6371	132	2.303 8346	12	.003 4907	12	.000 0582
13	.226 8928	73	1.274 0904	133	2.321 2879	13	.003 7815	13	.000 0630
14	.244 3461	74	1.291 5436	134	2.338 7412	14	.004 0724	14	.000 0679
15	.261 7994	75	1.308 9969	135	2.356 1945	15	.004 3633	15	.000 0727
16	.279 2527	76	1.326 4502	136	2.373 6478	16	.004 6542	16	.000 0776
17	.296 7060	77	1.343 9035	137	2.391 1011	17	.004 9451	17	.000 0824
18	.314 1593	78	1.361 3568	138	2.408 5544	18	.005 2360	18	.000 0873
19	.331 6126	79	1.378 8101	139	2.426 0077	19	.005 5269	19	.000 0921
20	.349 0659	80	1.396 2634	140	2.443 4610	20	.005 8178	20	.000 0970
21	.366 5191	81	1.413 7167	141	2.460 9142	21	.006 1087	21	.000 1018
22	.383 9724	82	1.431 1700	142	2.478 3675	22	.006 3995	22	.000 1067
23	.401 4257	83	1.448 6233	143	2.495 8208	23	.006 6904	23	.000 1115
24	.418 8790	84	1.466 0766	144	2.513 2741	24	.006 9813	24	.000 1164
25	.436 3323	85	1.483 5299	145	2.530 7274	25	.007 2722	25	.000 1212
26	.453 7856	86	1.500 9832	146	2.548 1807	26	.007 5631	26	.000 1261
27	.471 2389	87	1.518 4364	147	2.565 6340	27	.007 8540	27	.000 1309
28	.488 6922	88	1.535 8897	148	2.583 0873	28	.008 1449	28	.000 1357
29	.506 1455	89	1.553 3430	149	2.600 5406	29	.008 4358	29	.000 1406
30	.523 5988	90	1.570 7963	150	2.617 9939	30	.008 7266	30	.000 1454
31	.541 0521	91	1.588 2496	151	2.635 4472	31	.009 0175	31	.000 1503
32	.558 5054	92	1.605 7029	152	2.652 9005	32	.009 3084	32	.000 1551
33	.575 9587	93	1.623 1562	153	2.670 3538	33	.009 5993	33	.000 1600
34	.593 4119	94	1.640 6095	154	2.687 8070	34	.009 8902	34	.000 1648
35	.610 8652	95	1.658 0628	155	2.705 2603	35	.010 1811	35	.000 1697
36	.628 3185	96	1.675 5161	156	2.722 7136	36	.010 4720	36	.000 1745
37	.645 7718	97	1.692 9694	157	2.740 1669	37	.010 7629	37	.000 1794
38	.663 2251	98	1.710 4227	158	2.757 6202	38	.011 0538	38	.000 1842
39	.680 6784	99	1.727 8760	159	2.775 0735	39	.011 3446	39	.000 1891
40	.698 1317	100	1.745 3293	160	2.792 5268	40	.011 6355	40	.000 1939
41	.715 5850	101	1.762 7825	161	2.809 9801	41	.011 9264	41	.000 1988
42	.733 0383	102	1.780 2358	162	2.827 4334	42	.012 2173	42	.000 2036
43	.750 4916	103	1.797 6891	163	2.844 8867	43	.012 5082	43	.000 2085
44	.767 9449	104	1.815 1424	164	2.862 3400	44	.012 7991	44	.000 2133
45	.785 3982	105	1.832 5957	165	2.879 7933	45	.013 0900	45	.000 2182
46	.802 8515	106	1.850 0490	166	2.897 2466	46	.013 3809	46	.000 2230
47	.820 3047	107	1.867 5023	167	2.914 6999	47	.013 6717	47	.000 2279
48	.837 7580	108	1.884 9556	168	2.932 1531	48	.013 9626	48	.000 2327
49	.855 2113	109	1.902 4089	169	2.949 6064	49	.014 2535	49	.000 2376
50	.872 6646	110	1.919 8622	170	2.967 0597	50	.014 5444	50	.000 2424
51	.890 1179	111	1.937 3155	171	2.984 5130	51	.014 8353	51	.000 2473
52	.907 5712	112	1.954 7688	172	3.001 9663	52	.015 1262	52	.000 2521
53	.925 0245	113	1.972 2221	173	3.019 4196	53	.015 4171	53	.000 2570
54	.942 4778	114	1.989 6753	174	3.036 8729	54	.015 7080	54	.000 2618
55	.959 9311	115	2.007 1286	175	3.054 3262	55	.015 9989	55	.000 2666
56	.977 3844	116	2.024 5819	176	3.071 7795	56	.016 2897	56	.000 2715
57	.994 8377	117	2.042 0352	177	3.089 2328	57	.016 5806	57	.000 2763
58	1.012 2910	118	2.059 4885	178	3.106 6861	58	.016 8715	58	.000 2812
59	1.029 7443	119	2.076 9418	179	3.124 1394	59	.017 1624	59	.000 2860
60	1.047 1976	120	2.094 3951	180	3.141 5927	60	.017 4533	60	.000 2909

SI SUMMARY

General

The following information on SI units is provided to assist those involved in the planning, design, fabrication and erection of steel structures prepared in SI units. Information related to the metric system in general is to be found in CAN3-Z234.1-79, "Canadian Metric Practice Guide" and for terms related to the steel industry in the "Industry Practice Guide for SI Metric Units in the Canadian Iron and Steel Industry". The latter is available from the Task Force for Metric Conversion in the Canadian Iron and Steel Industry, P.O. Box 4248, Station "D", Hamilton, Ontario, L8V 4L6.

The eleventh General Conference of Weights and Measures, in 1960, adopted the name International System of Units for a coherent system which includes the metre as the base unit of length and the kilogram as the base unit of mass. The international abbreviation of the name of this system, in all languages, is SI.

Canada is a signatory to the General Conference on Weights and Measures, and in 1970, the Canadian government stated that the eventual conversion to the metric system is an objective of Canadian policy. Since that time, metric conversion activity in Canada has developed to the point where material and design standards, building codes and technical literature are available in SI units.

The SI system is based on the seven base units listed in Table 7-1. Decimal multiples and sub-multiples of the SI base units are formed by the addition of the prefixes given in Table 7-2.

SI BASE UNITS
Table 7-1

Quantity	Name	Symbol
length	metre	m
mass	kilogram	kg
time	second	s
electric current	ampere	A
thermodynamic temperature	kelvin	K
amount of substance	mole	mol
luminous intensity	candela	cd

SI PREFIXES
Table 7-2

Multiplying Factor	Prefix	Symbol
1 000 000 000 000 = 10^{12}	tera	T
1 000 000 000 = 10^{9}	giga	G
1 000 000 = 10^{6}	mega	M
1 000 = 10^{3}	kilo	k
100 = 10^{2}	hecto	h
10 = 10^{1}	deca	da
0.1 = 10^{-1}	deci	d
0.01 = 10^{-2}	centi	c
0.001 = 10^{-3}	milli	m
0.000 001 = 10^{-6}	micro	µ
0.000 000 001 = 10^{-9}	nano	n
0.000 000 000 001 = 10^{-12}	pico	p
0.000 000 000 000 001 = 10^{-15}	femto	f
0.000 000 000 000 000 001 = 10^{-18}	atto	a

In choosing the appropriate decimal multiple or sub-multiple, the Canadian Metric Practice Guide recommends the use of prefixes representing 10 raised to a power that is a multiple of 3, a ternary power. Thus, common structural steel design units would be:

Force - newton (N), kilonewton (kN)

Stress - pascal (Pa), kilopascal (kPa), megapascal (MPa)

Length - millimetre (mm), metre (m)

Mass - kilogram (kg), megagram (Mg)

The tonne is a special unit, equal to 1 000 kg (or 1 Mg) that will be used in the basic steel industry, but should not be used in structural design calculations.

Designers using SI units must transform loads given in mass (kilograms) to forces, using the relationship force = mass times acceleration. In the design of structures on earth, acceleration is the acceleration due to gravity, designated by "g" and established as 9.806 65 metres per second per second at the third General Conference on Weights and Measures in 1901.

The unit of force to be used in design is the newton (N) (or multiples thereof) where a newton is defined as the force that, when applied to a body having a mass of one kilogram (kg), gives the body an acceleration of one metre (m) per second squared (s^2). The unit of stress is the pascal (Pa), which is one newton per square metre (m^2). Since this is a very small unit, designers of steel structures will generally use megapascals (MPa), where one megapascal is one million pascals and equals one newton per square millimetre (N/mm^2). See also "Structural Loads, Mass and Force".

Properties and dimensions of steel sections are given, in this book, in millimetre units, tabulated to an appropriate ternary power of 10, and millimetres should be used for dimensioning steel structures. Some relationships and values of interest to steel designers are shown below:

SI PREFIXES
Table 7-3

Density of Steel		7 850	kg/m^3
Modulus of Elasticity	E	200 000	MPa
Shear Modulus of Steel	G	77 000	MPa
Coefficient of Thermal Expansion		11.7×10^{-6}	/°C
Acceleration due to Earth's Gravity	g	9.806 65	m/s^2

For a more complete description of SI, the Canadian Metric Practice Guide should be consulted; however, Table 7-4 provides a convenient summary listing selected SI units, the quantity represented, the unit name and typical application.

Structural Loads, Mass and Force

Since most civil engineers have been accustomed to designing structures on earth to withstand loads more variable than the acceleration due to gravity, the pound-force and the kilogram-force have been used as standard units of force. These units were assumed to be numerically equal to their mass counter-parts, the pound-mass and the kilogram-mass respectively.

In SI, the units of mass and force, the kilogram and the newton respectively, are distinctly different both in name and in value. The two are related through the famous Newtonian equation, force = mass times acceleration, or

$$F = ma$$

Thus a newton (N) is defined as the force required to give one kilogram (kg) mass an acceleration of one metre (m) per second (s) squared, or

$$1\ N = 1\ kg{\cdot}m/s^2$$

The standard international value of acceleration due to gravity is $9.806\ 65\ m/s^2$. However, for hand calculations in Canada a value of

$$g = 9.81\ m/s^2$$

may be more acceptable as it retains three significant figures (adequate for most structural design) and produces a numerical value of force distinctly different from the value of mass. Thus, whether or not the mass has been converted to a force will be readily apparent, and errors will tend to be reduced.

SELECTED SI UNITS
Table 7-4

Quantity	Preferred Units	Unit Name	Typical Applications	Remarks
Area	mm²	square millimetre	Area of cross section for structural sections	Avoid cm²
	m²	square metre	Areas in general	
Bending Moment	kN·m	kilonewton metre	Bending moment in structural sections	
Coating mass	g/m²	gram per square metre	Mass of zinc coating on steel deck	
Coefficient of Thermal Expansion	$\frac{1}{°C}$ *	reciprocal (of) degree Celsius	Expansion of materials subject to temperature change (generally expressed as a ratio per degree Celsius)	11.7×10^{-6}/°C for steel
Density, mass	kg/m³	kilogram per cubic metre	Density of materials in general; mass per unit volume	7 850 kg/m³ for steel
Force	N	newton	Unit of force used in structural calculations	1N = 1kg·m/s²
	kN	kilonewton	Force in structural elements such as columns; concentrated forces; axial forces; reactions; shear force; gravitational force	
Force per Unit Length	N/m	newton per metre	Unit for use in calculations	1 kg/m x 9.81 m/s² $= (9.81 \text{ kg·m/s}^2) \times \frac{1}{m}$ = 9.81 N/m
	kN/m	kilonewton per metre	Transverse force per unit length on a beam, column etc.; dead load of a beam for stress calculations	$(1 \text{ kg/m} \times 9.81 \text{ m/s}^2) \times \frac{1\,000}{1\,000}$ $= (9.81 \text{ kg·m/s}^2) \times \frac{1}{m} \times \frac{1\,000}{1\,000}$ $= (9.81 \text{ N/m}) \times \frac{1\,000}{1\,000}$ = 9.81 kN/m x 1/1 000 = 0.009 81 kN/m
Force per Unit Area (See Pressure)				
Frequency	Hz	hertz	Frequency of vibration	1 Hz = 1/s = s^{-1} (replaces cycle per second (cps))
Impact energy	J	joule	Charpy V-notch test	1 N·m = 1 J
Length	mm	millimetre	Dimensions on all drawings; dimensions of sections, spans, deflection, elongations, eccentricity	
	m	metre	Overall dimensions; in calculations; contours; surveys	
	km	kilometre	Distances for transportation purposes	
	μm	micrometre	Thickness of coatings (paint)	
Mass	kg	kilogram	Mass of materials, structural elements and machinery	A metric tonne, t 1t = 10³ kg = 1Mg = 1 000 kg
Mass per Unit Length	kg/m	kilogram per metre	Mass per unit length of section, bar, or similar items of uniform cross section.	Also known as "linear density"
Mass per Unit Area	kg/m²	kilogram per square metre	Mass per unit area of plates, slabs, or similar items of uniform thickness; rating for load-carrying capacities on floors (display on notices only)	DO NOT USE IN STRESS CALCULATION
Mass Density	kg/m³	kilogram per cubic metre	Density of materials in general; mass per unit volume	7 850 kg/m³ for steel
Modulus of Elasticity (Young's)	MPa	megapascal	Modulus of elasticity; Young's modulus	200 000 MPa for carbon, high-strength low alloy and low-alloy wrought steels
Modulus, Shear	MPa	megapascal	Shear Modulus	77 000 MPa assumed for steel
Modulus, Section	mm³	millimetre to third power	First moment of area of cross section of structural section, such as plastic section modulus, elastic section modulus	

* The preferred unit is 1/K, however $\frac{1}{°C}$ is an acceptable unit for the construction industry

SELECTED SI UNITS
Table 7-4

Quantity	Preferred Units	Unit Name	Typical Applications	Remarks
Moment of Inertia	mm^4	millimetre to fourth power	Second moment of area; moment of inertia of a section; torsional constant of cross section	
Moment of Force	kN·m	kilonewton metre	Bending moment (in structural sections); overturning moment	
	N·m	newton metre		
Pressure (see also Stress)	Pa	pascal	Unit used in calculation	1 Pa = 1 N/m^2
	kPa	kilopascal	Uniformly distributed loads on floors; soil pressure, wind loads; snow loads; dead loads; live loads.	1 kPa = 1 kN/m^2
Section Modulus (see Modulus)				
Stress	MPa	megapascal	Stress (yield, ultimate, permitted, calculated) in structural steel	1 MPa = 1 MN/m^2 = 1 N/mm^2
Structural Load (see Force)				
Temperature	°C	degree Celsius	Ambient temperature	0°C = 273.15K However, for temperature intervals 1°C = 1K
Thickness	mm	millimetre	Thickness of web, flange, plate, etc.	
	μm	micrometre	Thickness of paint	
Torque	kN·m	kilonewton metre	Torsional moment on a cross section	
Volume	m^3	cubic metre	Volume; volume of earthworks, excavation, concrete, sand, all bulk materials.	1 m^3 = 1 000 L The cubic metre is the preferred unit of volume for engineering purposes
	L	litre	Volume of fluids and containers for fluids	
Work, Energy	J	joule	Energy absorbed in impact testing of materials; energy in general	1 kWh = 3.6 MJ where kWh is a kilowatt hour.

There are two common areas where the designer of a structure must be alert to the distinction between mass and force:

1. dead loads due to the mass of the structural elements, permanent equipment etc.,
2. superimposed, or live loads due to storage of materials.

In these and other cases where mass is well known since it is the unit of commerce, the designer must convert mass to force by multiplying by g.

COMMON CONVERSION FACTORS
Table 7-5

Item	Imperial – SI	SI – Imperial
Acceleration	1 $ft./s^2$ = 0.304 8 m/s^2	1 m/s^2 = 3.2808 $ft./s^2$
Area	1 acre = 0.404 685 6 ha 1 $ft.^2$ = 0.092 903 04 m^2 1 $in.^2$ = 645.16 mm^2 1 $mi.^2$ = 2.589 988 km^2 1 $yd.^2$ = 0.836 127 4 m^2	1 ha = 2.471 acres 1 m^2 = 10.764 $ft.^2$ 1 mm^2 = 1.55 x 10^{-3} $in.^2$ 1 km^2 = 0.3861 $mi.^2$ 1 m^2 = 1.20 $yd.^2$
Capacity (Canadian Legal Units)	1 oz. = 28.413 062 mL 1 gal. = 4.546 090 L 1 pt. = 0.568 261 L 1 qt. = 1.136 522 L	1 mL = 35.2 x 10^{-3} oz. 1 L = 0.220 gal. 1 L = 1.76 pt. 1 L = 0.880 qt.
Density, Mass	1 lb./ft. = 1.488 16 kg/m 1 lb./yd. = 0.496 055 kg/m 1 $oz./ft.^2$ = 305.152 g/m^2 1 $lb./ft.^2$ = 4.882 43 kg/m^2 1 $lb./in.^2$ = 703.069 6 kg/m^2 1 $lb./ft.^3$ = 16.018 46 kg/m^3 1 $lb./in.^3$ = 27.679 90 Mg/m^3	1 kg/m = 0.672 lb./ft. 1 kg/m = 2.016 lb./yd. 1 g/m^2 = 3.277 x 10^{-3} $oz./ft.^2$ 1 kg/m^2 = 0.205 $lb./ft.^2$ 1 kg/m^2 = 1.42 x 10^{-3} $lb./in.^2$ 1 kg/m^3 = 62.4 x 10^{-3} $lb./ft.^3$ 1 Mg/m^3 = 0.0361 $lb./in.^3$
Force	1 kip = 4.448 222 kN	1 kN = 0.225 kip
Length	1 ft. = 0.304 8 m = 304.8 mm 1 in. = 25.4 mm 1 mile = 1.609 344 km 1 yd. = 0.914 4 m	1 m = 3.28 ft. 1 mm = 0.0394 in. 1 km = 0.622 mi. 1 m = 1.09 yd.
Mass	1 lb. = 0.453 592 37 kg 1 ton (2000 lb.) = 0.907 184 74 Mg	1 kg = 2.205 lb. 1 Mg = 1.10 ton = 2205 lb.
Mass per Unit Area	1 $lb./ft.^2$ = 4.882 43 kg/m^2	1 kg/m^2 = 0.205 $lb./ft.^2$
Mass per Unit Length	1 lb./ft. = 1.488 16 kg/m	1 kg/m = 0.672 lb./ft.
Moment of Inertia a) Second Moment of Area b) Section Modulus	 1 $in.^4$ = 416 231.4 mm^4 1 $in.^3$ = 16 387.064 mm^3	 1 mm^4 = 2.4 x 10^{-6} $in.^4$ 1 mm^3 = 0.061 x 10^{-3} $in.^3$
Pressure or Stress	1 ksi = 6.894 757 MPa 1 psf = 47.880 26 Pa 1 psi = 6.894 757 kPa	1 MPa = 0.145 ksi 1 Pa = 0.0209 psf 1 kPa = 0.145 psi
Torque or Moment of Force	1 ft.·kipf = 1.355 818 kN·m	1 kN·m = 0.738 ft.·kipf
Volume	1 $in.^3$ = 16 387.064 mm^3 1 $ft.^3$ = 28.316 85 dm^3 1 $yd.^3$ = 0.764 555 m^3	1 mm^3 = 0.061 x 10^{-3} $in.^3$ 1 dm^3 = 0.0353 $ft.^3$ 1 m^3 = 1.308 $yd.^3$
Costs	1 \$/ft. = 3.28 \$/m 1 $\$/ft.^2$ = 10.764 $\$/m^2$ 1 $\$/yd.^2$ = 1.20 $\$/m^2$ 1 $\$/ft.^3$ = 35.34 $\$/m^3$ 1 $\$/yd.^3$ = 1.307 $\$/m^3$	1 \$/m = 0.305 \$/ft. 1 $\$/m^2$ = 0.0929 $\$/ft.^2$ 1 $\$/m^2$ = 0.836 $\$/yd.^2$ 1 $\$/m^3$ = 0.0283 $\$/ft.^3$ 1 $\$/m^3$ = 0.765 $\$/yd.^3$

MILLIMETRE EQUIVALENTS
DECIMALS AND EACH 64TH OF AN INCH

FRACTIONS		INCHES	mm
1/64		.015625	.397
	1/32	.03125	.794
		.03937	1
3/64		.046875	1.191
	1/16	.0625	1.588
5/64		.078125	1.984
		.07874	2
	3/32	.09375	2.381
7/64		.109375	2.778
		.11811	3
	1/8	.125	3.175
9/64		.140625	3.572
	5/32	.15625	3.969
		.15748	4
11/64		.171875	4.366
	3/16	.1875	4.763
		.19685	5
13/64		.203125	5.159
	7/32	.21875	5.556
15/64		.234375	5.953
		.23622	**6**
	¼	.25	6.350
17/64		.265625	6.747
		.27559	**7**
	9/32	.28125	7.144
19/64		.296875	7.541
	5/16	.3125	7.938
		.31496	**8**
21/64		.328125	8.334
	11/32	.34375	8.731
		.35433	**9**
23/64		.359375	9.128
	3/8	.375	9.525
25/64		.390625	9.922
		.3937	10
	13/32	.40625	10.319
27/64		.421875	10.716
		.43307	**11**
	7/16	.4375	11.113
29/64		.453125	11.509
	15/32	.46875	11.906
		.47244	**12**
31/64		.484375	12.303
	½	.5	12.700

FRACTIONS		INCHES	mm
		.51181	**13**
33/64		.515625	13.097
	17/32	.53125	13.494
35/64		.546875	13.891
		.55118	**14**
	9/16	.5625	14.288
37/64		.578125	14.684
		.59055	15
	19/32	.59375	15.081
39/64		.609375	15.478
	5/8	.625	15.875
		.62992	**16**
41/64		.640625	16.272
	21/32	.65625	16.669
		.66929	**17**
43/64		.671875	17.066
	11/16	.6875	17.463
45/64		.703125	17.859
		.70866	**18**
	23/32	.71875	18.256
47/64		.734375	18.653
		.74893	**19**
	¾	.75	19.050
49/64		.765625	19.447
	25/32	.781250	19.844
		.7874	20
51/64		.796875	20.241
	13/16	.8125	20.638
		.82677	**21**
53/64		.828125	21.034
	27/32	.84375	21.431
55/64		.859375	21.828
		.86614	**22**
	7/8	.875	22.225
57/64		.890625	22.622
		.90551	**23**
	29/32	.90625	23.019
59/64		.921875	23.416
	15/16	.9375	23.813
		.94488	**24**
61/64		.953125	24.209
	31/32	.96875	24.606
		.98425	25
63/64		.984375	25.003
	1	1.000	25.4

MISCELLANEOUS CONVERSION FACTORS

Area

1 acre	= 0.404 685 6 ha
1 hectare	= **1** hm^2
1 legal subdivision (40 acres)	= 0.161 874 2 km^2
1 section (1 mile square, 640 acres)	= 2.589 988 km^2
1 square foot	= **929.0304** cm^2
1 square inch	= **645.16** mm^2
1 square mile	= 2.589 988 km^2
1 square yard	= 0.836 127 4 m^2
1 township (36 sections)	= 93.239 57 km^2

Linear Density (Mass per Unit Length)

1 pound per inch	= 17.858 kg/m
1 pound per foot	= 1.488 16 kg/m
1 pound per yard	= 0.496 055 kg/m

Area Density (Mass per Unit Area)

1 ounce per square foot	= 305.152 g/m^2
1 pound per square foot	= 4.882 43 kg/m^2
1 pound per square inch	= 703.0696 kg/m^2

Mass Density (Mass per Unit Volume)

1 pound per cubic foot	= 16.018 46 kg/m^3
1 pound per cubic inch	= 27.679 90 Mg/m^3
1 ton (long) per cubic yard	= 1.328 939 Mg/m^3
1 ton (short) per cubic yard	= 1.186 553 Mg/m^3

Energy

1 British thermal unit (Btu) (International Table)	= **1.055 056** kJ
1 foot pound-force	= 1.355 818 J
1 horsepower hour	= 2.684 52 MJ
1 kilowatt hour	= **3.6** MJ

Force

1 kilogram-force	= **9.806 65** N
1 kip (thousand pounds force)	= 4.448 222 kN
1 pound-force	= 4.448 222 N

Heat

1 Btu *foot per (square foot hour °F)	= 1.730 74 W/(m·K)	k-value
1 Btu per (square foot hour °F)	= 5.678 29 W/(m^2·K)	U-value
1 square foot hour °F per Btu	= 0.176 109 m^2·K/W	R-value

* *Based on the Btu IT.*

Length

1 chain (66 feet)	= **20.1168** m
1 foot	= **0.3048** m
1 inch	= **25.4** mm
1 microinch	= **25.4** nm
1 micron	= **1** µm
1 mil (0.001 inch)	= **25.4** µm
1 mile	= **1.609 344** km
1 mile (International nautical)	= **1.852** km
1 mile (UK nautical)	= **1.853 184** km
1 mile (US nautical)	= **1.852** km
1 yard	= **0.9144** m

MISCELLANEOUS CONVERSION FACTORS

Mass

1 hundredweight (100 lb)	= **45.359 237** kg
1 hundredweight (long) (112 lb, UK)	= 50.802 345 kg
1 pennyweight	= 1.555 174 g
1 pound (avoirdupois)	= **0.453 592 37** kg
1 ton (long, 2240 lb, UK)	= **1.016 046 908 8** Mg
1 ton (short, 2000 lb)	= **0.907 184 74** Mg

Mass Concentration

1 pound per cubic foot	= 16.018 46 kg/m^3

Second Moment of Area (Moment of Inertia)

1 $inch^4$	= $0.416\,231\,4 \times 10^6\ mm^4$

Section Modulus

1 $inch^3$	= $\mathbf{16.387\,064 \times 10^3}\ mm^3$

Momentum

1 pound foot per second	= 0.138 255 kg·m/s

Power. See also Energy.

1 Btu (IT)* per hour	= 0.293 072 W
1 foot pound-force per hour	= 0.376 616 1 mW
1 foot pound-force per minute	= 22.596 97 mW
1 foot pound-force per second	= 1.355 818 W
1 horsepower (550 ft·lbf/s)	= 745.6999 W

* *International Tables.*

Pressure or Stress (Force per Area)

1 atmosphere, standard	= **101.325** kPa
1 inch of mercury (conventional, 32°F)	= 3.386 39 kPa
1 inch of water (conventional)	= 249.089 Pa
1 ksi (1000 lbf/in^2)	= 6.894 757 MPa
1 mm mercury (conventional, 0°C)	= 133.322 Pa
1 pound-force per square foot	= 47.880 26 Pa
1 pound-force per square inch (psi)	= 6.894 757 kPa
1 ton-force per square inch	= 13.789 514 MPa
1 ton-force (UK) per square inch	= 15.4443 MPa

Temperature

Scales

Celsius * temperature	= temperature in kelvins – **273.15**
Fahrenheit temperature	= **1.8** (Celsius temperature) + **32**
Fahrenheit temperature	= **1.8** (temperature in kelvins) – **459.67**
Rankine temperature	= **1.8** (temperature in kelvins)

Intervals

1 degree Celsius*	= **1** K
1 degree Fahrenheit	= **5/9** K
1 degree Rankine	= **5/9** K

* "Celsius" replaced "Centigrade" in 1948 to eliminate confusion with the word centigrade, associated with centesimal angular measure.

MISCELLANEOUS CONVERSION FACTORS

Time	
1 day (mean solar)	= **86.4** ks
1 hour (mean solar)	= **3.6** ks
1 minute (mean solar)	= **60** s
1 month (mean calendar, 365/12 days)	= **2.628** Ms
1 year (calendar, 365 days)	= **31.536** Ms
Torque (Moment of Force)	
1 pound-force foot	= 1.355 818 N·m
1 pound-force inch	= 0.112 985 N·m
Volume	
1 acre foot	= 1233.482 m^3
1 barrel (oil, 42 US gallons)	= **0.158 987 3** m^3
1 board foot*	= 2.359 737 dm^3
1 cubic foot	= 28.316 85 dm^3
1 cubic inch	= **16.387 064** cm^3
1 cubic yard	= 0.764 555 m^3
1 gallon	= **4.546 09** dm^3
1 gallon (UK) §	= 4.546 092 dm^3
1 gallon (US)	= 3.785 412 dm^3

* *The board foot is nominally $1 \times 12 \times 12 = 144$ in^3. However, the actual volume of wood is about 2/3 of the nominal quantity.*

§ *Also referred to as the "Imperial gallon."*

Volume Rate of Flow	
1 cubic foot per minute	= 0.471 947 4 dm^3/s
1 cubic foot per second	= 28.316 85 dm^3/s
1 cubic yard per minute	= 12.742 58 dm^3/s
1 gallon per minute	= 75.768 17 cm^3/s
1 gallon (UK) per minute	= 75.7682 cm^3/s
1 gallon (US) per minute	= 63.0902 cm^3/s
1 million gallons per day	= 52.6168 dm^3/s

Notes:

1. The conversion factors give the relationship between SI units and other Canadian legal units as well as commonly encountered units of measure of United Kingdom and USA origin. The yard and the pound are the same throughout the world; by definition they are specified fractions of the metre and the kilogram. The gallons of Canada and Australia, which are identical, differ by a relatively insignificant amount from the gallon of the United Kingdom, whereas that of the USA is a much smaller measure.
2. The conversion factors given in tables apply to Canadian units unless stated otherwise.
3. Conversion factors that are exact are shown in boldface type. Other factors are given to more than sufficient accuracy for most general and scientific work.
4. Conversions are those listed in CAN3-Z234.1-79

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