

B.N. Asthana · Deepak Khare

# Recent Advances in Dam Engineering



Springer

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# Foreword

Water is essential for life. It is required for other uses also such as irrigation, industry, and power development. Its demand is increasing with the increase in population and changes in life style, whereas the availability is practically constant. In India, water is available as precipitation which is concentrated largely in a few months of monsoon season and snowfall in the higher mountains for an equally short time window. Rainfall in monsoon months is also temporally and spatially random. Purely monsoon fed rivers in India have practically no flow in non-monsoon months and even the snow fed rivers of Himalayas have meagre non-monsoon flows of the order of 1 to 2% of the flows during monsoon. In order to maintain water supplies for various purposes in a uniform and assured manner throughout the year, the storages formed by dams are essential for sustainable water management.

Dam design and construction is a time tested solution for storing river flows and is adopted all over the world. This art of damming rivers is as old as civilization. The dam technology development on scientific basis was started in the early years of the 20<sup>th</sup> century. Now it has been possible to design and construct dams upto a height of 300 m of both types i.e. of concrete and earth. Grand Dixence and Nurek dams are examples of large height.

In India, century old dams of small height of earth and stone masonry are existing in the south. Mulla Periyar dam in Periyar basin of about 150 years vintage is still catering to the functions of inter-basin transfer and sustainable water supply for food security. High dams of various types were taken up after the independence of India. Bhakra is one of the first high concrete dam constructed after independence. Recently Tehri rockfill dam in challenging geotectonic environment has been commissioned and is functioning successfully.

Presently, about 5264 large dams (as per ICOLD definition) are functioning in India and some more are under planning and development. The planning and operation of dams require continued surveillance and design inputs for ensuring their safety during construction and operation. Dam design is, therefore, an evolving technology where new solutions and materials are being continually applied for meeting unforeseen challenges. Each of the dams presents a unique problem in

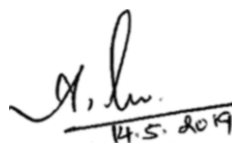
development and construction bound by common principles of engineering and mechanics. The design and construction of Roller Compacted Concrete dams (RCC) and the Concrete Faced Rockfill Dams (CFRD) is the latest in the development of dam engineering and these techniques are being utilized by the engineers world over including India.

Designs and analysis of dams require multiple disciplines of engineering technologies and scientific disciplines. The dam technology including design and construction is developing fast and lot of literature in the form of USBR manuals, IS Codes, books, journals, ICOLD proceedings and trade journal articles etc. is available. With knowledge being distributed amongst a large swath of literature, the collation and assimilation within a short time period of study becomes difficult. The authors of this book, during their long experience in teaching and dealing with field problems, realized the challenges faced by the graduate students and field engineers when exposed to the subject of dam design and construction.

Keeping the experience into consideration the authors have developed this book for the students and new professionals in the field. The book contains the basic design concepts of various types of dams including the latest developments of RCC and CFRD dams. The book covers various other aspects of planning and construction of dams. Specialised topics of reservoir sedimentation, instrumentation, environment, river diversion during construction, foundation treatment, hill slope stabilization, and construction methodology have been covered. The book attempts to provide a comprehensive view of dam engineering. The reader will also get pointers to a large body of references for further specialized knowledge of different issues and disciplines involved.

I congratulate the authors for their efforts in writing such a short and unique book covering theoretical and practical aspects of dam engineering, well illustrated with examples of Indian dams for the benefit of the students and the professionals.

I hope the students and the engineers will find the book useful.



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14.5.2019

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Ashwin B. Pandya

# Preface

The art of constructing dams is as old as the art of harnessing river flows. First, it was done for domestic use and then for irrigating crops. Evidences and records have shown that the earliest dam might be reliably dated to be 5000 years old, which was constructed as rubble walls in Egypt. An earth dam known as Padavil dam was constructed in Sri Lanka in 504 BC. Some earth dams constructed in South India are centuries old. Earlier dams the world over were generally earth embankments of low height. Their design approach was empirical and adhoc. In 20th century, the design and construction technology of dams developed fast and high concrete and embankment dams of different types were constructed in many countries of the world to store monsoon flows for use in non-monsoon period for various purposes. These were developed as multipurpose projects which became growth engines for rapid economic and social development of the area.

In India, before Independence in 1947, some stone masonry or earthfill dams were constructed in Central and South regions of the country. After Independence, large height concrete gravity and embankment type dams were constructed as multipurpose projects. Some of these are Bhakra, Hirakud, Koyna, Rihand, Ramganga, Beas and Idduki. By this time, dam design and construction had become a time tested technology and by the end of 20th century new technologies were developed and widely adopted for roller compacted concrete and concrete faced rockfill dams. These technologies have been adopted recently in India for construction of small and medium height dams.

During 1970s and 1980s, the environmental concerns of storage dams were recognized and taken up by several world organisations as well as the United Nations. This hampered the pace of dam construction activities. In India, considering the need of storing monsoon flows for overall economic and social development, efforts were made by the GOI through the Ministry of Environment and Forest to strike a balance between development and environment impact mitigation. The Ministry issued the directives that, at the project planning stage, a study should be carried out for the environmental impact of project along with the environmental management plan to mitigate adverse impact and it should be part of the DPR.

At present there are about 4900 large dams in India. This activity of storing monsoon flows will have to be accelerated to meet the water demand of the increasing population. As mentioned above, dam design and construction is the most developed and time tested technology of the 20th century and lot of voluminous literature in the form of text books, manuals, codes and journals on different aspects of dam engineering of various kinds of dams is available. These still give only the guidelines to the planners and designers of a dam as each project is site specific and different from others so far as hydrological, geological, foundation type, and availability of construction material aspects are concerned.

The advances and researches in hydrological analysis, geological investigations and analysis procedures in the field of soil mechanics and rock machines, in concrete technology and development of computer-based mathematical models for analysis have made dam engineering highly technical. The advances in the construction technology due to development of high capacity earth moving equipments have mechanised the construction of a dam. These developments have made the planning, design and construction of a dam, a multi-disciplinary subject. A major dam project is capital intensive and so financial management besides technical inputs is also very important. Hence the planning, design and construction of a dam project need a huge team of competent professionals of different disciplines to be headed by a team leader or project manager who may effectively manage and coordinate the activities and the output of his team members as well as the inputs of external consultants, if required.

The authors during their long experience in the field, teaching the subject at the graduate level at the Department of WRD&M, IIT Roorkee (earlier known as WRDTC, University of Roorkee), research and consultancy of various dam projects have felt that the students, who are exposed to the subject during the course of their study are generally not able to comprehend the subject matter from the available literature. Similarly it is seen that the engineers who are for the first time engaged for the dam engineering of a project either in design office or field are not able to do justice with their job by using available voluminous literature due to lack of adequate background of the basics of the subject. It is also experienced that sometimes the project manager is also not fully effective due to lack of exposure to various aspects of dam engineering.

In view of the above experiences, authors have attempted this book to give in a simple format the basics of design and construction of various kinds of concrete and embankment dams which have got wide acceptance in India. The book also covers other associated practical aspects of dam engineering such as foundation treatment, river diversion, hill slope stability, reservoir sedimentation, instrumentation, environmental aspects, and dam safety in separate chapters. The text wherever feasible is supported with practical examples of projects in India.

The book is the outcome of the long field experience of the first author in planning, design and construction of dam projects and the authors' experience in teaching, research and consultancy of the dam projects at the department of WRD&M, IIT Roorkee. It is hoped that this book will be of immense help to the

students and engineers engaged at various levels in different aspects of dam engineering of a project. It will also help them in pursuing advance study of the subject.

The authors gratefully acknowledge the support and cooperation of the department of WRD&M of IIT Roorkee, and the help rendered by Shri Lakhwinder Singh, Research Scholar and Shri Yashpal who undauntedly cooperated in compiling the manuscript of the book. The authors are also grateful to Sri Vivek Tripathi Director (Dams) CWC for his help and suggestions.

All efforts are made to individually acknowledge the source of all data and information used in the book but special mention must be made of organisations like BIS, CWC, CBI&P, ICOLD and USBR whose publications and data are freely made use of in this book.

The authors would also like to gratefully acknowledge collectively their debt to all those consulting firms and corporations with whom they worked on planning and design of several dam projects and whose engineering contributions have been utilized in this text.

Roorkee, India  
15.6.2019

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Deepak Khare

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## Some Useful Conversion Factors

Metre	= 3.28 ft	1 foot	= 0.3048 m
Cubic metre	= 35.32 cubic ft	1 cu. ft.	= 0.0283 m <sup>3</sup>
Hectare	= 2.47 acres	1 acre	= 0.405 ha
Kilogram	= 1.072 seers	1 lb	= 0.4536 kg
	= 2.205 lbs		
Tonne (metric)	= 1000 kg	1 ton	= 1.016 tonne (metric)
	= 0.984 ton		
	= 27.22 maunds		
Million cu. metres	= 810.71	1 acre foot	= 43,560 cu.ft.
	acre ft.		= 1233.38 m <sup>3</sup> .

Hectare metre (ham) = 10,000 m<sup>3</sup> = 8,108 acre feet.

Thousand million cu. ft (1 tmc) = 28.3 million m<sup>3</sup>  
 = 28,300 ha m = 229,456 acre ft.

tmc units are being used in Southern States of India.

The S.I. system of units, now adopted internationally, the unit of Force is a Newton.

1 kilogram (force) = 9.81 Newtons

1 pascal (Pa) = 1 Newton/m<sup>2</sup> (N/m<sup>2</sup>)

1 kg/cm<sup>2</sup> = 98.1 kilo pascals (kPa)

1 t/m<sup>2</sup> = 9.81 kPa

# Abbreviations

BIS	Bureau of Indian Standards
CBI&P	Central Board of Irrigation and Power
CFRD	Concrete Faced Rockfill Dam
COT	Cut-Off Trench
CWC	Central Water Commission
DBE	Design Basic Earthquake
DHI	Danish Hydraulic Institute
DPR	Detailed Project Report
DVC	Damodar Valley Corporation
ECRD	Earth Core Rockfill Dam
FEM	Finite Element Method
FOS	Factor of Safety
FRL	Full Reservoir Level
GOI	Government of India
ICOLD	International Commission on Large Dams
IIT Roorkee	Indian Institute of Technology Roorkee
IMD	Indian Meteorological Department
IS	Indian Standards Code
MCE	Maximum Credible Earthquakes
MDDL	Minimum Drawdown Level
MOEF	Ministry of Environment and Forest
MWL	Maximum Water Level
NEERI	National Environmental Engineering Research Institute
RCC	Reinforced Cement Concrete
RCC Dam	Roller Compacted Concrete Dam
THDC	Tehri Hydro Development Corporation
USBR	United States Bureau of Reclamation

WRD&M	Water Resources Development & Management WRDTC
cm	Centimetre
m	Metre
ha	Hectare
Mham	Million hectare metre
t/m <sup>2</sup>	Tonnes per square metre

# Chapter 1

## Introduction



### 1.1 GENERAL

Life without water is not possible. Hence man started developing techniques to use water for domestic purposes. The technique of harnessing the river flows is as old as the history of mankind. It is linked with the history of development of agricultural practices by the man which required timely watering of crops. Earliest civilizations such as Nile, Indus etc. were largely dependent on use of river flows for irrigation. Besides domestic use and irrigation requirement the uses of water have increased manifold with the development of modern civilization. Now it is required as an input for practically all sectors of economy and for the better living life style of the people. Since the availability of useful water is limited, it is becoming scarce every day because of multiple uses and the increase in population. Thus, water resources development and management is becoming more and more important. The temporal and spatial availability of water is largely varying at most of the places on the earth. Hence, to provide water when and where it is required becomes the objective of water resource planning and development. Therefore, wherever the topography, geology and the hydrology of the river indicate possibility of diverting river flows or creating storage for river flows, it should be planned and developed for various uses. The storage requires construction of a dam across the water courses or rivers. Hence, dams become key component of water resources development plans. There are evidences to show that dams were constructed in earlier civilizations but of small heights. These are being constructed in the present era also. But with time the technology and construction techniques have advanced to the extent that dams as high as 300 m or more have been constructed.

## 1.2 HISTORICAL DEVELOPMENT

One of the earliest dam recorded was in Egypt, reliably about 4800 year old, which was 12 m high and made of rubble masonry. The Padavil dam in Sri Lanka constructed in about 500 BC was 21.4 m high earthen embankment. Pond irrigation in South and Central India is centuries old practice and large number of tanks created through earth dams exist even today which are several hundred years old. Till the end of 19<sup>th</sup> century, rational design and construction practices for dams had not developed to construct high dams, so modest height (less than 30 m) earth dams were generally constructed following empirical methods of design.

With the development of soil mechanics around 1925, the rational design practices for earth dams were developed and high dams upto a height of 160 m could be constructed with confidence the world over. Still higher dams with central clay core and outer shell zones of pervious material known as earth core rockfill dams were constructed after the development of heavy earthmoving equipments in around 1940 A.D. In the second half of 20<sup>th</sup> century, several dams of height between 200 and 300 m were constructed such as Rogun dam (335 m) and Nurek (300 m) in USSR, Tehri Dam (260 m) in India, Mica (262 m) in Canada and Oroville (236 m) in USA.

With the development of rock mechanics, another type of embankment dam known as rockfill dam came in existence. These have the section of rockfill material with impervious membrane on upstream face. By the end of 20<sup>th</sup> century several such rockfill dams of height ranging from 150 to 200 m have been constructed.

Though stone masonry dams of low height were in existence since times immemorial, use of concrete for such dams was started in the beginning of 20<sup>th</sup> century. These were known as gravity dams because these rely on their weight for stability. These low height dams were designed on thumb rule and had very wide base width. After around 1920 AD development of rational design practices for concrete gravity dams started and thereafter large number of high concrete gravity dams have been constructed throughout the world such as Dworshak dam (219 m) in USA, Grand Dixence dam (284 m) in Switzerland and Bhakra dam (226 m) in India.

Another type of concrete dam developed with rational design practices in the first decade of 20<sup>th</sup> century was arch dam in which water load is transferred to the abutments through arch action as the thrust. These are adopted where abutments are strong enough to take this thrust. Such dams have the base width of about 0.15 times of height. Hoover dam in USA which is 221 m high arch gravity type was constructed in 1936 AD. By 1950 AD a large number of arch dams were constructed in several countries and mostly in Europe.

Conventional concrete gravity dams have been replaced by the concept of roller compacted concrete (RCC) after 1970 AD because of economy and expeditious construction. This has gained wide acceptance and large number of RCC dams have been constructed after 1985 AD and several of these are of height above 150 m. These are basically of the same shape and size as of concrete gravity dam but their concrete mix replaces a major portion of cement by fly ash, reducing heat of hydration and efforts of cooling the concrete. In RCC dams concrete can be quickly placed by using dumper, dozers and vibratory rollers.

### ***1.2.1 Development of Dams in India***

In India, to practice pond irrigation, small earth dams to store monsoon flows have been constructed in South and Central region of the country and many of these are several centuries old and are still in use. Many masonry dams of small height have also been constructed in South and Central India. Several of small masonry dams constructed in the first decade of 20<sup>th</sup> century are still in use. A stone masonry dam on river Cauvery called Krishna Raja Sagara dam completed in 1934 is even today a major storage in Karnataka State.

After independence the activity of construction of dams was accelerated and a large number of multipurpose projects were started such as Bhakra, Hirakund, Koyna, Nagarjuna Sagar and Rihand. These storage projects were designed as multipurpose projects for irrigation, hydro power generation, and flood control. These were mostly of concrete gravity type. Some of the projects such as Pong dam on river Beas, Ramganga dam on river Ramganga were constructed as earth core rockfill dam. In 1980's the environmental impact of dam projects was fully realized and the Govt. of India started enforcing the guidelines to include measures to mitigate the adverse impact of dam project on environment in the project report. In the beginning of 1990's the Government policy changed to shift these projects to private sector. This decelerated the pace of the activity of dam construction and only two continuing major multipurpose projects (i) Sardar Sarovar on river Narbada in Gujarat and (ii) Tehri project on Bhagirathi in Uttarakhand have been completed. Some medium height diversion dams for hydropower generation have been completed in this period and some are in various stages of construction and planning in hill states of Himalayas.

## **1.3 FUTURE OF DAM PROJECTS**

Environmentalists and sociologists are against the dam projects because of their impact on environment and rehabilitation. But in India, China and many other developing countries in South East Asia and Africa, where rainfall is seasonal, harnessing river flows through dam projects is considered essential to improve quality of life of the people, food production and overall economy of the country. Hydropower is a relatively low cost, non-polluting and renewable source of green energy, and its generation needs dam projects. Hence dam projects have a future in developing countries including India. However, considering the concerns of environmentalists and sociologists, it has been made mandatory that the dam projects shall take into account measures to mitigate adverse impact on environment. With the advancement in technology, construction technique and equipments the construction of RCC and concrete faced rockfill dams has started in India but till date a few dams have been constructed of height around 50 m or less. It is expected that many more such dams of greater height will be constructed in future. In India,

Himalayas, where a large number of projects are being planned, are geologically young, weak and seismically active, the preference for high dams should be for embankment dams of either earth core rockfill or concrete faced rockfill type.

## 1.4 TYPES OF DAM PROJECTS

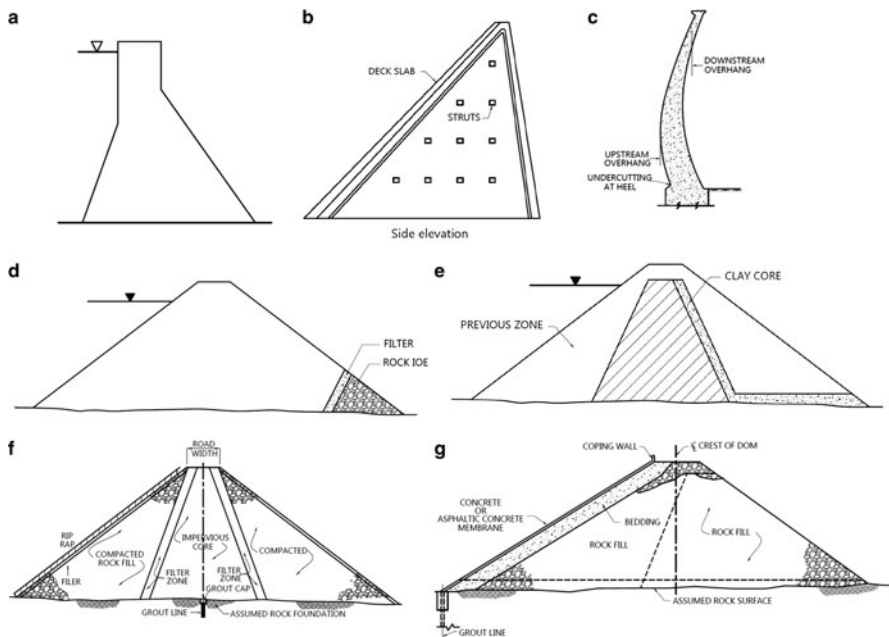
Dam projects are either storage type or run-of-river type. In storage type projects the excess monsoon flows are stored in the reservoir created by the dam and are released when required to meet the demand of water. Such releases from dam generally augment the non-monsoon flows of the river. In a run-of-river type project the dam is constructed to divert the available river flow while maintaining the reservoir at the required elevation.

The storage projects are generally multipurpose projects (a project which serves several purposes). The stored water can be used for consumptive uses such as domestic, industrial, irrigation etc., and for non-consumptive uses such as hydro-power generation, fisheries, tourism etc. and also for flood control. A project can be a single purpose project also i.e. either for irrigation or for power generation or for any other purpose.

## 1.5 CLASSIFICATION OF DAMS

Dams may be divided into two types: rigid and embankment. The rigid type are either of concrete or stone masonry. These are sub-divided into gravity, arch, arch gravity, hollow or buttress type and RCC (roller compacted concrete gravity type). As discussed above the most commonly adopted are gravity dams and arch dams. Due to economy and expeditious construction, RCC dams which are basically concrete gravity type are now being favoured in situations where rigid dams are to be constructed.

Embankment dams are the dams made of naturally occurring material such as earth, sand, gravel, boulders, rock fragments etc. These are generally classified as (i) earth dams of homogenous section of impervious soil i.e. clay or zoned section made of central clay core with outer shell zones of pervious soil such as sand and shingle, and (ii) rockfill dams which are of two types (a) Earth core rockfill dam which is a zoned section with central core of clay and shell of sand, gravel, boulder or crushed rock with suitable filter in between and (b) Concrete faced rockfill dams in which the body of dam is made of graded boulder and sand mix or of crushed rock with impervious membrane (of concrete or asphalt concrete) on the upstream face. Typical sections of various types of dams are shown in Fig. 1.1.

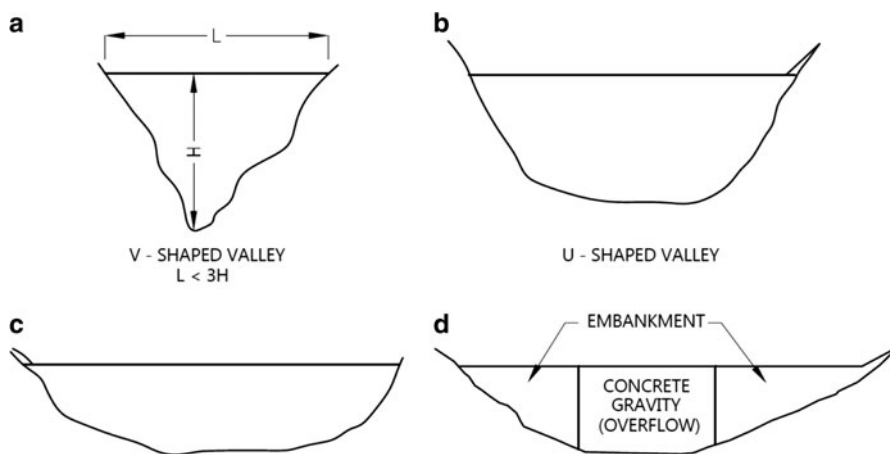


**Fig. 1.1** Typical sections of dams.

## 1.6 CHOICE OF TYPE OF DAM

The choice of type of dam is site specific and depends on a number of factors but the criteria for preliminary selection depends on the valley shape and geology. A narrow V-shaped valley (length at top of dam  $< 3$  times height) with strong abutments is suitable for an arch dam. A moderately wide U-shaped valley with strong rock foundation is suitable for a gravity dam but if foundation is not suitable for gravity dam an embankment dam may be favoured. If the valley is wide (length at top of dam several times the height of dam) like a saucer an embankment dam or a composite dam comprising a central gravity type spillway made of either concrete or stone masonry and embankment on both sides shall be preferred. A large number of such composite dams have been constructed in South and Central India where river valleys are very wide. Advancements in design and construction techniques have made it possible to construct embankment dams in narrow valleys where geologically other types are not feasible. At Tehri, on river Bhagirathi a tributary of river Ganga in Uttarakhand, an embankment dam (earth core rockfill type) has been constructed whose height is 260 m and the length at dam top is about 540 m. Valley shapes and the suitable type of dam for a valley shape, are shown in Fig. 1.2.

Besides above factors, the choice of type of dam is also influenced by the factors like availability of construction material, availability of the kind of equipment



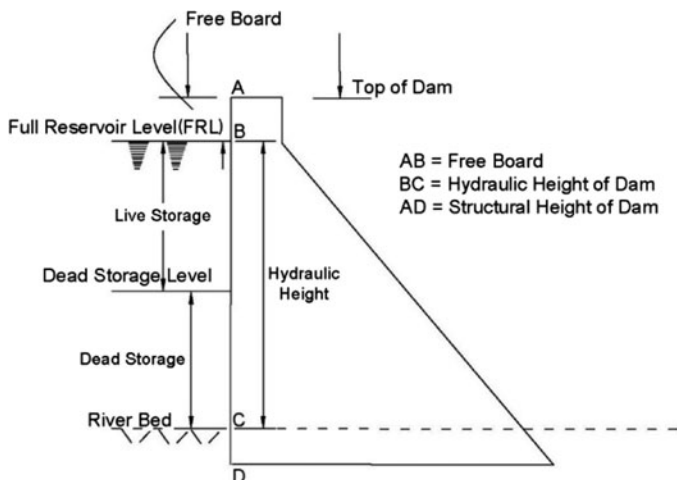
**Fig. 1.2** Valley shapes for different types of dams.

required for the construction of a specific type of dam, suitable location of the spillway and finally the cost which besides the cost of dam shall also include the cost of spillway, river diversion and the intake structure for the outlets. The intake structure and spillway are separate structures in case of an embankment dam whereas these are generally made as part of a concrete dam. River diversion for an embankment dam is for a much larger discharge (floods of return period of 100 years or more) and forms a substantial part of total cost as compared to a concrete dam in which the diversion is provided generally for maximum non-monsoon or in 25 years non-monsoon flows and the monsoon flood is allowed to pass over the incomplete portion of dam.

## 1.7 DAM AND ITS APPURTENANT WORKS

The dam is located at the narrowest gorge section of the river subject to the suitability of the foundation conditions. The axis is generally aligned straight and normal to the direction of flow. Sometimes topographical and geological constraints require curving of the axis. In that case slight upstream curvature may be provided in the axis.

Spillways and intake structures for outlets are the components of every type of dam. Spillway is required to pass excess flood discharge and intake structure is required to pass required discharge in the outlet tunnel or penstock or channel for power generation or irrigation or both. There may be a power house at or in the vicinity of the dam which may be a surface or underground type depending on topography and geology. The spillway and intake structures are located separate



**Fig. 1.3** Definition sketch for dam height and storage.

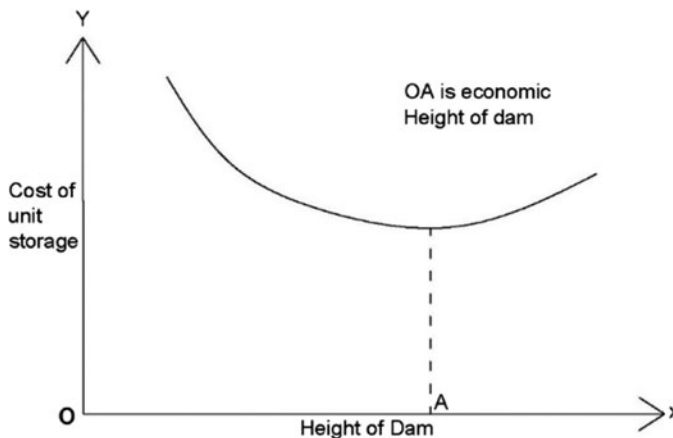
from the dam in case of an embankment dam. The layout of dam and the appurtenant structures shall be planned as a composite project complex satisfying operational, topographical and geological requirements.

## 1.8 HEIGHT OF DAM

The top level of dam is the full reservoir level (FRL) plus the free board which is function of wave height and also includes settlement allowance in case of embankment dams. The FRL above the river bed is decided on the basis of the summation of dead storage required for sediment deposition and live storage required to meet the water use requirement. The difference between the river bed level and the FRL is called hydraulic height of dam. The difference between the deepest foundation level and top level of dam is known as structural height of dam. These terms are illustrated in Fig. 1.3. The height generally associated with the dam is the structural height.

The height which gives minimum cost per unit of storage is called economical height of dam. It can be calculated out by working out the cost of dam and the storage for various heights of dam. It is graphically shown in Fig. 1.4. If other conditions such as storage requirement, foundation characteristic, environmental constraints, water availability etc. permit, the economical height or a height close to it shall be adopted.

Generally dams are classified on the basis of height. A dam less than 30 m is a small dam, between 30 to 90 m is medium height dam and a dam above 90 m is a high dam.



**Fig. 1.4** Economic height of dam and storage.

## 1.9 CHALLENGES IN DAM PROJECTS IN INDIA

As discussed above, India and other developing countries need dam projects in future to meet requirement of food and water for the increasing population and to improve the overall economy of the region. In India, this activity is presently in both public and private sectors and is moving at a slow pace because of several risks and challenges involved in such projects. These are (i) environmental and other clearances, (ii) rehabilitation, (iii) acquisition of land, (iv) location of site being in remote and difficult areas, (v) risk involved in hydrological and geological uncertainties and (vi) difficulties in managing resources, both human and financial. In order to accelerate the construction of dam projects the Government will have to take suitable measures to remove the road blocks as being faced by this sector.

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# Chapter 2

## Concrete Dams



### 2.1 INTRODUCTION

Concrete dams are made of concrete which is a mixture of cement, sand, coarse aggregate and water. Such dams came into existence from the last decade of 19<sup>th</sup> century and in 20<sup>th</sup> century; these were constructed in many countries of the World. Development of rational design practices was started around 1920 and a large number of high concrete dams were constructed in USA and Europe in first half of 20<sup>th</sup> century. The construction of concrete dams has started in India after independence.

### 2.2 TYPES OF CONCRETE DAMS

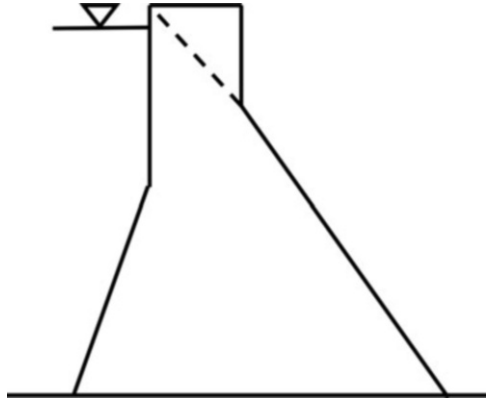
Concrete dams are of three principal types according to shape and design features. These are as below:

#### (i) Gravity Dam

A concrete dam is called gravity dam when it resists the applied forces by its weight. It is a solid concrete section practically triangular in shape (Fig. 2.1). Its axis is generally a straight line. But to take advantage of topographic conditions the axis may be slightly curved upstream or a combination of curves and straight line may be adopted. The section is so designed that stresses at all locations are compressive as the concrete is strong in compression but can take no tension.

All the stresses due to loads and forces are transferred to the foundation, so the gravity dams require reasonably strong rock foundation of sedimentary, volcanic or metamorphic type rocks. But these have been successfully constructed on fractured and stratified geologically weak rocks also. These are generally economical in narrow and U shaped valleys but these have been adopted in wide valleys with

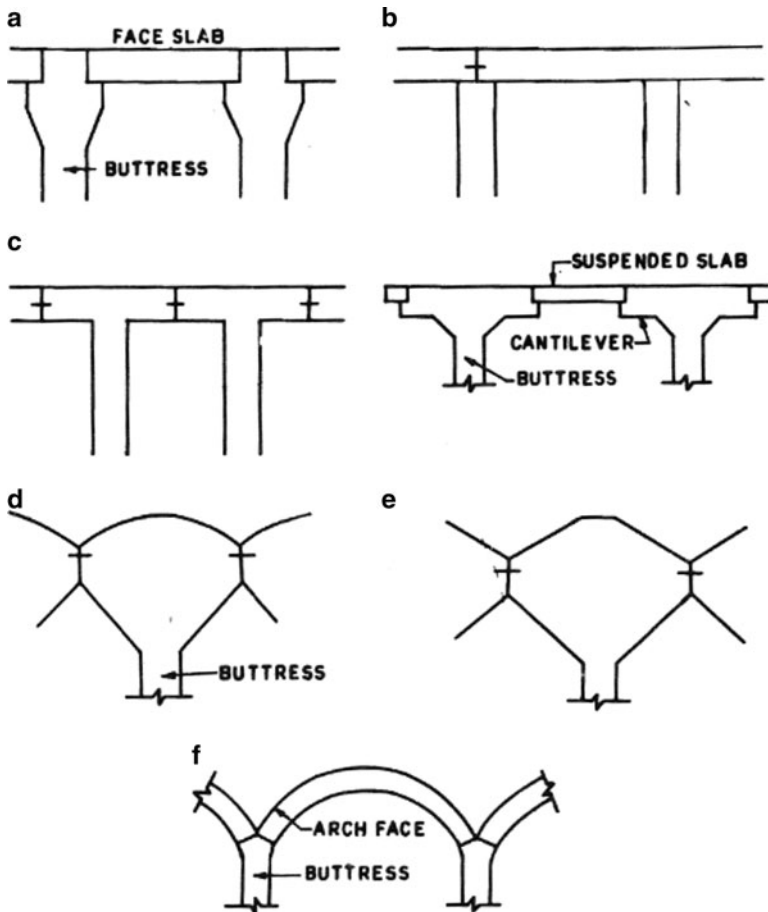
**Fig. 2.1** Section of concrete gravity dam



relatively flat canyon floor. In wide valleys (ratio of length at dam top and height i.e.  $\frac{B}{H} > 3$ , the gravity dam is built in blocks separated by ungrouted transverse joints. The blocks behave as free cantilevers. But in narrow valleys the dam is built as a monolithic structure to transfer part of load to the abutments. In gravity dams generally grout curtain and drainage holes are provided in the foundation to reduce seepage and uplift pressure on dam to economise on concrete volume. Gravity dams, when constructed with lean concrete (no slump concrete) having more of flyash and less cement and placement of concrete is done by using earth moving equipments and compaction is done by vibratory rollers, are called roller compacted concrete (RCC) dams. However in conventional concrete dam revolving cranes or cable ways are used for placement of concrete. The RCC dam is generally known for low cost and rapid construction.

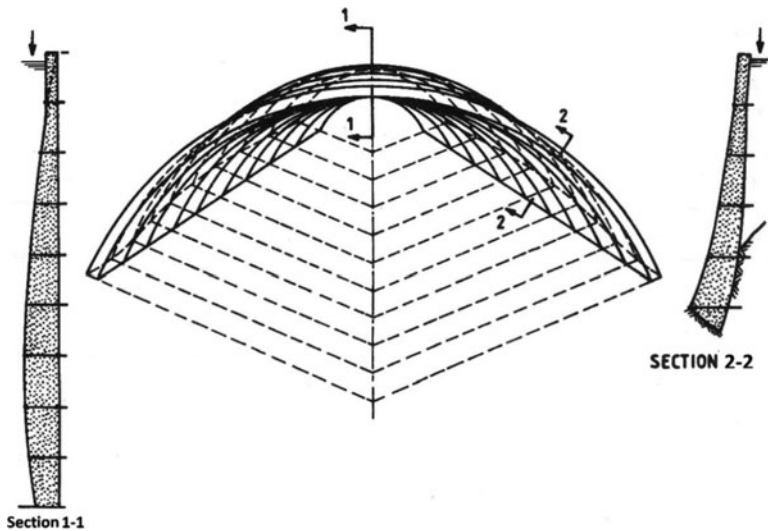
## (ii) Hollow Gravity and Buttress Dams

The concrete gravity dam is acted upon by the uplift force acting vertically at the base and concrete weight has to be provided to counteract it. It is estimated that 1/3 to 1/4 of total concrete volume is required for counteracting the uplift. If the seepage is allowed through the base between the two blocks the uplift will reduce resulting in the decrease in concrete volume. This concept resulted in the development of hollow gravity dam. In addition to it the idea of increasing stability of gravity dam through weight of water on inclined upstream face helped in developing various kinds of buttress dams comprising sloping water supporting deck and the supporting buttress. The sloping upstream deck is of reinforced concrete. The buttresses are vertical walls with their axis normal to the plane of the deck slab. The buttresses may be single wall, hollow or double wall. Various types of buttress dams are shown in Fig. 2.2. A buttress dam with deck slab spanning between two adjacent buttress is shown in Fig. 2.2(a, b) and cantilever deck slab is shown in Fig. 2.2(c). The massive head type deck is shown in Fig. 2.2(d, e). Multiple arch type deck supported on buttress is shown in Fig. 2.2(f).



**Fig. 2.2** Type of buttress dams.

These dams are economical because of comparatively less volume of concrete required and limited foundation excavation. The space between buttresses can be used for economically locating outlet works and even the power plant. The buttress dams usually used good and strong rock foundation better than a gravity dam because the space between buttresses is not loaded. A large number of such dams were constructed in 20<sup>th</sup> century. It was later experienced that saving in concrete volume did not necessarily reduce the cost because of high cost involved in the form work and reinforcement in the structural members of hollow gravity or buttress dams. It was also experienced that buttress dams are more prone to damage during earthquake than other types. Hence these type of dams are not in vogue. No such dam has been constructed or planned in India.



**Fig. 2.3** Constant-angle type arch dam.

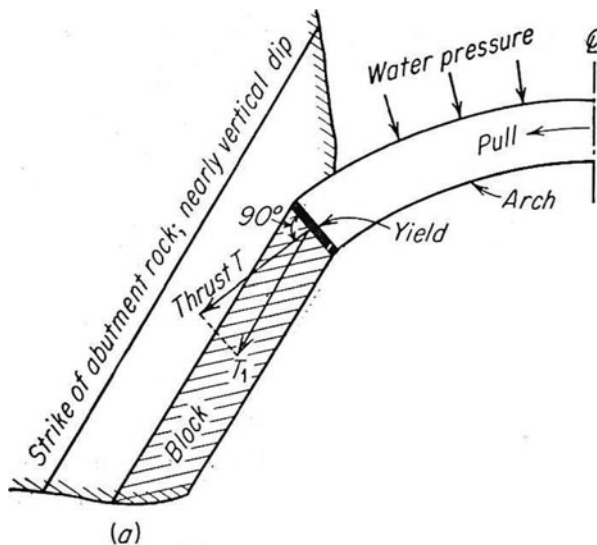
### (iii) Arch Dam

Arch dam is a concrete dam whose axis is curved upstream and most of the water pressure is transmitted to the abutment as horizontal thrust. Therefore, these are preferred in narrow valleys (length-height ratio 3 or less) where abutment rocks are strong enough to take the thrust. These are usually of two types single curvature arch dam and double curvature arch dam. In single curvature, the dam is curved in plan and in double curvature the dam is curved both in plan and elevation. An arch dam in plan and section is shown in Fig. 2.3.

The arch should be well keyed to the abutment which should be as immovable as possible. If the rock modulus of abutment rock is less, yield of the abutment will be more and it will cause a pull in concrete from the centre of the arch. This pull will cause tensile stresses and cracks in the arch. It is shown in Fig. 2.4. Hence the value of rock modulus in the design should be adopted judiciously based on tests. An  $E_c/E_r$  ratio of 2 to 3 is considered desirable.

A large number of high arch dams have been constructed in USA and Europe in 20<sup>th</sup> century. The advantage is reduction in quantity of concrete as compared to other types due to reduced base width (0.1 to 0.15 times of height) but this advantage is to some extent balanced by the varying shape of form work required in construction of such type of dam. Arch dams in wider valleys (length-height ratio upto 5) are also considered. Arches in wider valley will require that a greater percentage of load is transmitted to the foundation by cantilever action. If the distribution of load between arch and cantilever action is approximately equal, then the dam is considered as an arch-gravity or gravity-arch dam. Arch-gravity dams are considered in situations where foundations are not strong enough to support a gravity dam. Sometimes a

**Fig. 2.4** Arch dam abutment condition.



combination of arch dams with gravity tangents (concrete gravity dam sections extending tangentially from the arch dam) or thrust blocks may be provided on one or both ends to ensure better stress distribution in abutments where abutment rock conditions are not satisfactory for an arch dam.

Since an arch type of dam requires a narrow valley with strong abutments, this has not been adopted in India because such sites are generally not available. Only one arch dam has been constructed known as Idukki dam in South India which is 172 m high and is a double curvature type arch dam with base width of 20 m. It is shown in Fig. 2.5.

## 2.3 DAM TYPES IN INDIA

Buttress dams not being necessarily economical and being more prone to seismic damages are not in vogue. Arch dams are not adopted in India as the sites are not generally topographically and geologically suitable. On the other hand gravity dams are suitable for all valley shapes. If the foundation is not free from faults and shears, these can be satisfactorily treated to make foundation suitable for a gravity dam. Therefore, a large number of major concrete dams in India are concrete gravity dams. These may be constructed as RCC dams because of economy and rapid construction. Hence these two types i.e. concrete gravity dam and RCC dams have been dealt in chapters 3 and 4. For arch dams and buttress dams the readers can refer USBR publications 'Design of Arch Dams' and 'Treatise on Dams'.

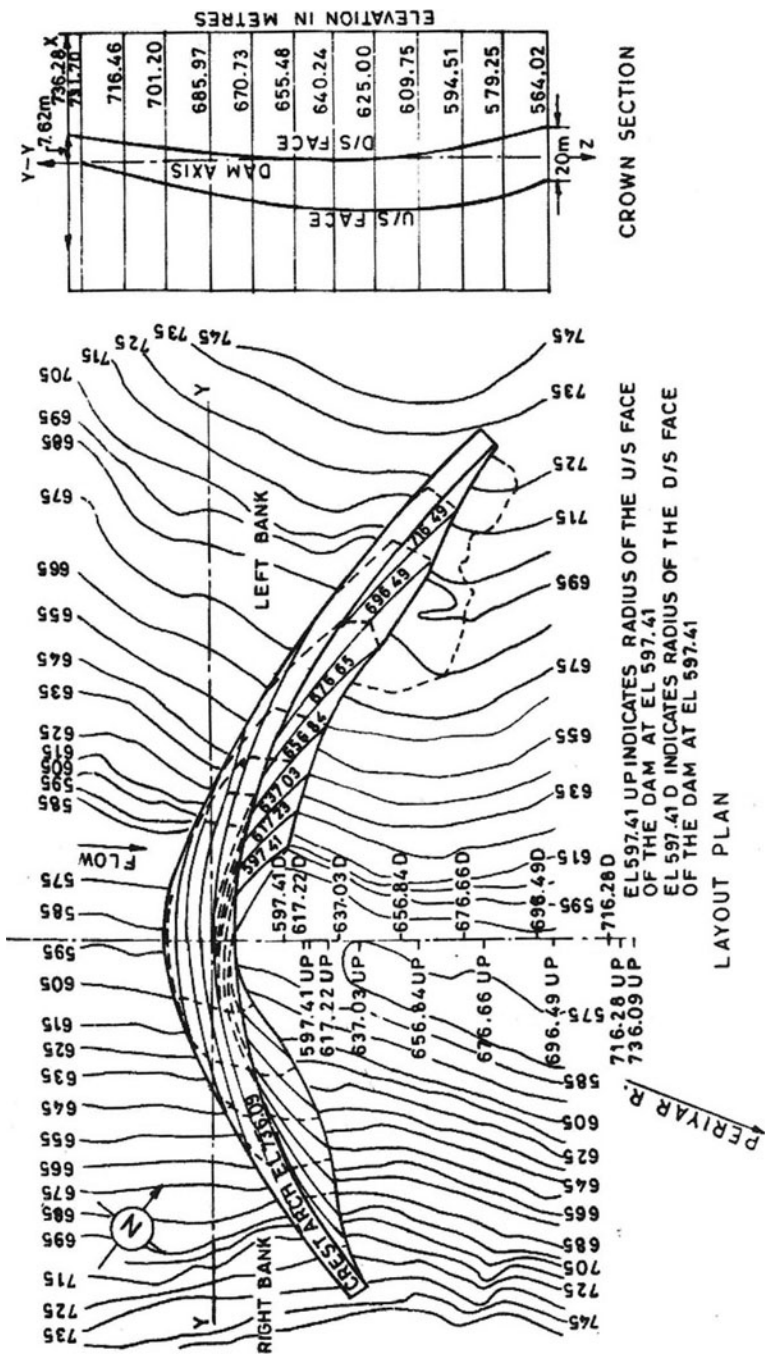


Fig. 2.5 Idukki Dam: Layout plan and crown cantilever section. (Ref. H.D. Sharma)

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## Chapter 3

# Concrete Gravity Dams



### 3.1 GENERAL

A dam which relies on its weight for stability against overturning and sliding due to externally applied forces is a gravity dam. It can be of stone masonry as well as of concrete. In India large number of stone masonry gravity dams were constructed in the end of 19<sup>th</sup> century and in first half of 20<sup>th</sup> century. After independence large multipurpose storage dams were planned and all these were made of concrete except Nagarjuna Sagar dam which is a stone masonry dam of height 114 m and Koyna dam (103 m height) in Maharashtra which is made of rubble concrete. A 20-25 cm of concrete layer when vibrated together with 10 to 30 cm size stones formed rubble concrete. The demerits of stone masonry dams are as follows:

- (i) These are labour intensive
- (ii) Period of construction is long
- (iii) These are less impermeable than concrete
- (iv) The permissible compressive strength limits the dam height to around 100 m.

The advantages of masonry dam are the following:

- (i) No elaborate arrangement and equipment for manufacturing and placing is required as for a concrete dam.
- (ii) No arrangement is required for cooling as required in concrete dam.
- (iii) No requirement of shuttering as is required in a concrete dam.
- (iv) Joints are less as compared to concrete dam.

The advantages in concrete dam are its impermeability, smooth finish for the surfaces of high velocity flow and quick construction. These can be constructed for any height, the highest gravity dam constructed is Grand Dixence dam, 284 m high in Switzerland and the highest in India is 226 m high Bhakra dam. The design criteria for the two types i.e. stone masonry and concrete gravity dams is the same. The dam section of both types should be safe against overturning on the toe and sliding along

the base or any joint plane. The vertical stresses transferred to the base should be compressive at both ends and within permissible bearing capacity of rock. The sections of dam are practically triangular with downstream face sloping at 0.7 to 0.8 H to 1.0 vertical.

### 3.2 SELECTION OF DAM SITE

Following are the considerations for a good site:

- (i) The river valley shall be narrow so that the length of dam (at the top) shall be minimum, but the valley in the upstream should be wide. This will enable maximum storage for minimum height of the dam.
- (ii) Foundation rock should be strong enough to bear the pressure exerted by the external loads at the base of the dam. It should be free from faults, shears and weak zones to reduce the efforts of strengthening the foundation.
- (iii) Abutments and valley slopes shall be stable, otherwise lot of efforts would be required to stabilize the slopes to prevent them from sliding.
- (iv) The permeability of the foundation and abutment rocks shall be low in order to minimize grouting and draining measures.
- (v) The dam and appurtenant works should preferably be founded on one type of rock in order to reduce differential settlement and its adverse impact on the behaviour of structures.
- (vi) The site shall be topographically and geologically such that the spillway and other works namely intake, power house complex etc. could be located economically. The power house may be a river side surface power house at the toe of non-overflow part of dam or an underground one in the abutment but spillway and intake shall be necessarily the part of the main dam. Deviations from the conventional layout are sometimes made due to space limitations.
- (vii) The construction material such as sand and coarse aggregate of desired quality and in required quantity should be available within economical distance.
- (viii) Availability of construction facilities such as communication, connectivity, power supply, nearness to township add to the suitability of the site. It reduces cost and pre-construction time.
- (ix) The reservoir rim shall be above FRL/MWL, geologically stable and of low permeability.
- (x) The submergence shall not affect the important historical structures and structures of high economic value such as factories, mines or townships etc.
- (xi) The site with low sediment yield from catchment should be preferred. Sediment management is a major problem in all the dam projects in Himalayas and it adds substantially to the cost of the project.

The following few examples will illustrate some of the above points in selecting a dam site:

- (i) Rihand dam, 91 m high concrete gravity dam on a tributary of river Sone in southeast Uttar Pradesh when initially planned submerged rare coriundum mines. To save these mines the height was reduced.
- (ii) Rihand dam which is 91 m high has a wide valley in upstream and has a storage of about 10,000 Mcum, practically equal to Bhakra dam on river Sutlej which is 226 m high. Ramganga dam on river Ramganga, 125.5 m high earth and rockfill dam, has a storage of 2200 Mcum and Tehri dam 260.5 m high has a storage of 3500 Mcum. This reveals that river valleys are generally narrow and long in Himalayan rivers.
- (iii) Bhakra Dam foundation rocks were found to contain deep pockets of weak clay stone. These had to be excavated out and filled with concrete through shafts which took 2 to 3 years to strengthen the foundation.
- (iv) Tehri dam which has been recently completed is 260.5 m high, one of the highest earth and rockfill dam on river Bhagirath, a tributary of river Ganga in Uttarakhand, submerged old Tehri town and a new township with all modern amenities for rehabilitation was constructed at the project cost.

During construction of Tehri dam it was found that clay for the central core was not sufficient to meet the requirement in the borrow areas near the dam site. It had to be brought from long distances. Similarly rock for rip-rap for upstream slope protection had to be brought from long distances (more than 35 km) as the rocks in adjoining areas were not found suitable for rip-rap.

- (v) The valley slopes of Ichari dam, a diversion dam on river Tones, a tributary of river Yamuna, and of the dam of Nathpa Jhakri hydro project on river Sutlej, adjacent to the dam sites were found unstable during construction and had to be extensively protected at great cost.
- (vi) Lakhwar dam, a 200 m high proposed concrete gravity dam on river Yamuna in Uttarakhand, was started in last decade of 20<sup>th</sup> century but work was stopped when during construction it was found that the proposed dam would be founded on two types of rocks with a shear zone in between. Studies were carried out to change the type of dam but still design is not finalized and construction has not been started.

### 3.3 SURVEY AND INVESTIGATIONS

In order to select a suitable site for a dam, the site shall be carefully inspected and investigated in respect of topographical, hydrological and geological features. If these are carried out in detail the designer will get required parameters precise enough to evolve a sound and economical design which in turn will ensure a cost effective and timely execution of the project.

The involvement of the specialists is required for topographical, hydrological and geological study of the site. Some aspects regarding these studies for a dam site are

given below. The reader for detailed information regarding these aspects may refer standard texts or concrete dam as well as I.S. Code which are available covering all the relevant topics.

### ***3.3.1 Topographical Surveys***

Survey of India maps shall be used for preliminary selection of a dam site. The chosen site shall be surveyed for following details. IS 5497 for topographical surveys of river valley projects may be referred for more details.

- (i) A contoured plan of the reservoir and the dam site shall be prepared covering the area about 10 m above the expected maximum water level in the reservoir. The survey shall identify the land use in the submergence as well as extent of rehabilitation required and the submergence of important historical structures, industry, mines etc. in the reservoir area.
- (ii) L-Section of the river from the upstream of reservoir to about 5 kms downstream of dam site shall be prepared.
- (iii) The X-sections of the river at 500 m spacing in the length of L-section shall be prepared. The cross sections shall extend about 10 m above MWL. This will help in examining the potentially unstable locations along the rim of the reservoir. The X-sections shall be used to develop area-capacity curve.
- (iv) A bench mark for future reference and use during construction shall be established at dam site.

### ***3.3.2 Hydrological Data and Analysis***

The hydrological information required for the dam project is water availability, design flood and river diversion discharge. If the long-term discharge observations of the river at the proposed dam site or its vicinity are available, these can be analyzed using well established hydrological procedures to arrive at the required information. But generally long-term observed stream flow data at the proposed site are not available. In that case it is developed by using indirect methods for which following data shall be collected:

#### **1. Catchment characteristics**

- (a) Area of catchment
- (b) Altitudes from the highest point of the catchment to the proposed site
- (c) River system
- (d) Shape
- (e) Land use pattern
- (f) Snow cover area, if any
- (g) Soil type

(h) Details of existing projects.

2. Rainfall data of the catchment – location of rain gauge stations and their long-term rainfall data.
3. Rainfall data and observed stream flow data of adjoining hydrologically homogeneous catchment.

The catchment characteristics shall be obtained from Survey of India maps, and records of Forest and Agriculture Departments. Rain gauge and rainfall data and other climatology information can be obtained from Indian Meteorological Department (IMD). River flow data and sediment load can be obtained for major rivers from Central Water Commission.

The gaps in rainfall and discharge data shall be filled using standard methods and the quality checks shall also be carried out on this data. Based on the type and extent of data, a suitable indirect method to develop discharge series at the proposed site shall be selected. It is advisable to start discharge observations at the proposed site at the first opportunity so that indirectly developed discharges could be verified.

The indirectly developed or observed long-term (about 30 years) discharge series at proposed site shall be analyzed for water availability, design flood and river diversion design discharge. The usual practice is to use 10 daily mean discharge for water availability. Generally 75% and 90% dependable year flows are worked out for water availability calculations for irrigation and power projects. The dependable years are worked out by arranging the annual runoff in descending order and using Weibulls formula.

$$P = \frac{m}{N+1} \times 100$$

where  $P$  is dependability percentage,  $m$  is rank of runoff of desired dependability and  $N$  is number of data.  $m$  can be worked out for the desired dependability from the above expression.

Design flood is required for design of spillway which is the structure to pass it safely in the downstream. A cautious and judicious approach is required in selecting the design flood. An over-estimation will result in costly structure and under-estimation may endanger the safety of structure. Any damage to spillway structure may cause loss to life and property in the downstream. The guidelines for the selection of design flood as per 11223 are as below. These are based on the size of dam which is defined in terms of hydraulic head (from normal or annual average flood level in the downstream to the maximum water level) and the gross storage. The overall size of dam would be greater of that indicated by either of the two parameters.

Type of dam		Gross storage (Mcm)	Hydraulic head (m)	Design flood
(i)	Small	0.5 to 10	7.5 to 12	in 100 year flood
(ii)	Intermediate	10 to 60	12 to 30	SPF
(iii)	Large	Greater than 60	Greater than 30	PMF

To work out SPF or PMF, unit hydrograph method is used and generally IMD provides the required storm values. Unit hydrograph which is the characteristic of the catchment is either derived from the observed flood hydrographs and effective rainfall in the catchment or a synthetic hydrograph is developed using catchment characteristics.

Flood frequency analysis of annual flood peaks is carried out to work out the floods of various return periods. Generally for concrete dams a flood of 25 years return period of maximum non-monsoon flows is adopted for design of river diversion works. The diversion discharge for small dam projects is generally adopted as the maximum of non-monsoon flows to be derived from the long-term discharge.

Sediment data observed with discharge observations shall be collected for the catchment or the adjoining catchments which will be useful to carry out reservoir sedimentation studies which are very important for the projects in Himalayas.

### ***3.3.3 Geological Investigations***

The geological investigations are required to have the information about the nature of the foundation, abutments and the rim of reservoir. Foundation investigations are more important for a concrete gravity dam because all the loads are transferred to the foundation. The following information regarding foundation is necessary for design and construction of dam:

- (i) The location of bedrock and the depth of overburden over it.
- (ii) The type of rock and its extent of weathering.
- (iii) The existence and extent of jointing pattern, shears, shear zones, faults and other geological weaknesses in the bed rock such as soluble rocks like limestone, clay stone etc.
- (iv) Permeability of foundation rock
- (v) Rock properties such as compressive strength, shear strength and rock modulus. These shall be estimated through in-situ and laboratory tests.

The geological investigations are very expensive and time consuming; so these shall be carried out in stages. During pre-feasibility and feasibility stage geological mapping and geophysical tests shall be carried out. A few boreholes shall be drilled to verify the findings of the geological mapping and geophysical tests. If these investigations establish the feasibility of the site, detailed investigations shall be carried out through additional boreholes and adits. Visual inspection and in-situ tests in adits/galleries shall be carried out to determine the nature and geological features of rock and the rock properties required for design.

The adits at suitable elevations in the abutments are required for geological investigations of the abutments. The logging of adits and the in-situ tests give the idea about the soundness and impermeability of abutments.

The shear strength parameters of the rock at the slopes of the locations vulnerable to land-slide shall be determined by tests to carry out slope stability analysis.

The geological investigations are generally complemented with geophysical investigation in important dam projects where the geological formations are stratified and different strata have different physical properties. If contrast in properties do not exist geophysical method will not work. Geophysical surveys are very helpful in determining depth of overburden above the bed rock when it is not possible by other methods under perennial river channels beds. These methods are also useful in detecting faults in foundation which play an important role in carrying out safety analysis of dam and in selecting tunnel alignment. The geophysical techniques are of two types: (i) seismic refraction method and (ii) electrical resistivity method. Both have their limitations. For more details refer relevant IS Codes. The geophysical methods shall be performed by skilled personnel and observations shall be interpreted by the experts.

To carry out the investigations and tests, the 'Guidelines for preparation of irrigation and multipurpose projects' issued by MOWR, GOI, 2010, the requirements of Geological Survey of India and relevant IS Codes shall be followed. Some of these IS Codes are:

- IS 15662 for investigation for gravity dam
- IS 13216 for investigations for reservoirs
- IS 15686 for preparation of geological maps
- IS 4453 for exploration of pits, trenches, drifts and shafts
- IS 6926 for diamond core drilling for investigations for river valley projects
- IS 4464 & 5313 for observations and presentation of core drilling data
- IS 5529 (Part 1 & 2) for in-situ permeability tests in overburden and bed rock.
- IS 15681 for geophysical method (seismic vibration)
- IS 15736 for geophysical method (electrical resistivity).

### **3.3.4 Construction Material Survey**

In a concrete gravity dam the volume of concrete required is large. Hence the requirement of coarse and fine aggregate is also large. Either natural river bed material, if available in required quantity and is of desired quality, can be used; otherwise the quarried rock of desired quality is used after proper crushing and screening.

The coarse aggregate shall be tested for specific gravity, absorption, abrasion and soundness. It shall also be tested for alkali-aggregate reactivity (AAR) with the cement to be used. Fine aggregate (sand) shall be free from organic material and fines and shall be of required fineness modulus preferably between 2 and 3. Standard procedures for these tests on aggregates as laid down in relevant IS Codes shall be followed.

The quarry sites satisfying the requirements of quality and quantity shall be identified and these shall be as close to the dam site as possible because transportation of large quantities of quarried material will add to the cost of project.

### 3.4 LAYOUT OF DAM

The axis of a concrete gravity dam is generally the line of upstream face of dam if it is vertical, or the centre line of the road bridge at the top of dam. The dam axis is normally made straight. It may be made slightly curved towards upstream to take advantage of topography to shorten the length of dam or to build the dam with geologically sound abutments. The axis shall be selected with following considerations:

- (i) Entire dam section should fall on one type of rock.
- (ii) Foundation excavation is minimum.
- (iii) Foundation treatment is minimum.
- (iv) Foundation bedrock does not slope in downstream direction.

The concrete gravity dam section, which is a section of dam cut by a plane perpendicular to dam axis, is a solid triangular section. Theoretically the thickness at top is zero but it is widened to accommodate the road bridge or other construction or operational requirements. The base width of dam section depends on the height and stability considerations based on foundation characteristics. Thus, the base width must be large enough to keep the stresses in foundation within permissible value of bearing pressure and to satisfy the requirements of sliding and overturning. The downstream slope is generally uniform. It commonly varies within a range of 0.70 to 0.85: 1 (H:V). Bhakra dam has a downstream slope of 0.8:1 and Hirakud dam spillway has a downstream slope of 0.7:1. Upstream slope is usually vertical but sometimes it is provided with slope for stability of dam due to weight of water over it. The upstream slope provided in dams has varied from 0.1 to 0.4 H to IV. Sardar Sarovar dam on river Narbada has an upstream slope of 0.4:1 and Srisailem dam has a slope of 0.1:1.

The base of dam section is placed on bed rock after removing the overburden. The base is kept generally horizontal or slightly sloping upstream.

The top of dam is fixed after adding free board to the maximum water level. The free board depends on wave height which is the function of reservoir fetch and the wind velocity. The minimum free board is 1.0 m. Sometimes the upstream parapet of concrete dam, if it is solid, is considered part of freeboard.

The dam is divided into a number of blocks separated by contraction joints. These are known as transverse joints and are perpendicular to dam axis. These are provided with waterstops (refer IS 12200) on the upstream to prevent seepage. These joints are normally spaced at a distance of 15 to 20 m in order to prevent cracking due to thermal stresses. These blocks structurally behave as cantilever fixed at base under water load and other horizontal forces. When the height of dam exceeds 100-120 m,

usually the deflection of the cantilever blocks becomes large and adversely affects the operation of dam. In order to reduce the deflection, the transverse joints are grouted. In that case the loads of a block are shared by the adjoining blocks and, in this manner, the dam becomes monolithic in longitudinal direction and some load is transferred to the abutments.

When the dam height is large the base width required becomes large and it becomes difficult to place concrete of a lift in one continuous operation. In such cases a joint is formed in concrete and the possibility of vertical cracking in dam is increased. In that case a longitudinal joint parallel to dam axis is provided in the downstream portion of the block. These are staggered and are made normal to the downstream face near the face. Shear keys are provided at the joint and the joint is grouted to make the dam section monolithic. Experience with several dams with longitudinal joints have shown that the monolithic section is not achieved and so these joints shall be avoided as far as possible.

The concrete gravity dam usually comprises overflow and non-overflow portions. The over flow portion is provided to pass excess river flow and is called spillway and is generally located in the river channel and the non-overflow part is located on both sides of spillway upto the junction with the abutments.

A typical plan of concrete gravity dam showing joints and the overflow and non-overflow sections are shown in Fig. 3.1.

### 3.4.1 Overflow Section

The overflow section is generally an ogee shaped crest conforming the lower nappe of a free falling jet so that negative pressures on concrete surface are not developed. The ogee crest is tangentially joined with the downstream edge of required base width.

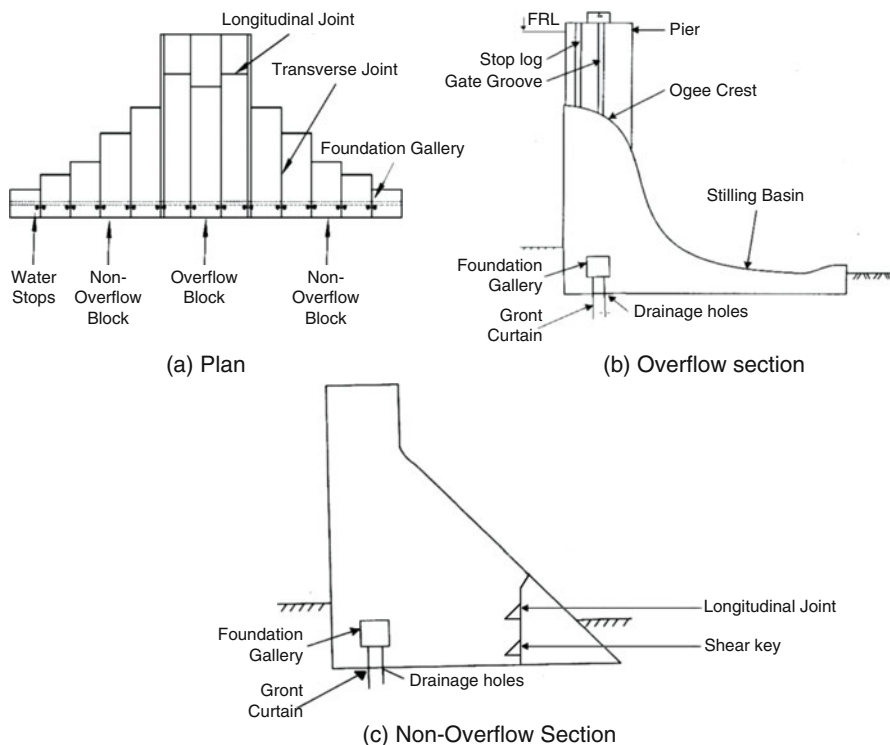
The length of overflow section and the crest level which is kept below FRL from hydraulic and economic considerations as well as the limitation of gate height because the difference between crest level and FRL is the gate height. Gate upto a maximum height of 19-20 m have been installed so far on spillways. Length and head over the crest are related as below:

$$Q = C_d L H^{3/2}$$

where

$Q$  = Design discharge of spillway ( $m^3$ ),  $C_d$  = Coefficient of discharge,  $L$  = Effective length of spillway (m) and  $H$  = Head over crest (m).

Thus, if  $H$  is large the length of spillway will be small and vice-versa.  $C_d$ , the coefficient of discharge, has a maximum value of 2.21 for a ogee crest when crest is not submerged by tail water level and upstream face is vertical. For submergence of crest and slope of upstream face  $C_d$  value is reduced. If  $H$  is large the discharge



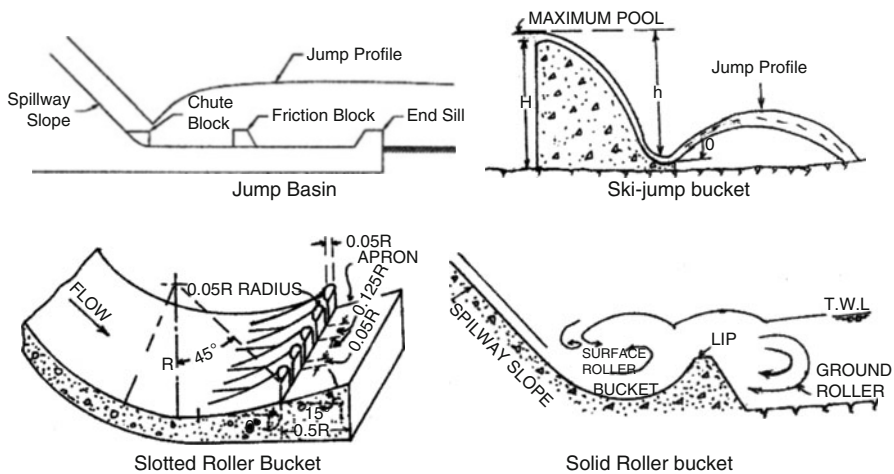
**Fig. 3.1** (a) Pan, (b) Overflow section and (c) Non-overflow section.

intensity (discharge per metre length) will be more which may require deeper stilling basin at the toe of spillway. Hence, combination of the length and head over crest should be judiciously decided considering hydraulic efficiency and economy.

The upstream portion of the ogee is circular curve and the portion downstream of crest is a parabolic curve. For the details of hydraulic design of spillways readers can refer IS Code No. 6934 or any standard text book on hydraulic design of spillway.

In some situations, uncontrolled (ungated) spillways are provided. In that case the crest of spillways is at FRL; so the height of dam becomes more as compared to gated spillway. Hence, if other factors permit, generally a gated spillway is adopted for regulated operation.

The length of the spillway is divided into bays separated by RCC piers. Each bay is provided with regulation gate which is either vertical fixed wheel type generally upto a height of 8 to 10 m or a radial gate for greater heights. The span of gates used so far has been a maximum of 18 m. In upstream of regulation gate generally an emergency gate or stop log is provided to facilitate the repair of regulation gate. Considering the number of bays one or two sets of stop logs are provided which may be dropped in front of the gate which requires repair and maintenance. For design of RCC piers, IS 13551 may be referred.



**Fig. 3.2** Different types of energy dissipation arrangements.

At the top of the piers the roadway and the hoisting arrangement for gates are provided. The width of the roadway is generally kept as 6 to 8 m sufficient for two lane bridge. Hoisting arrangement of gates is separate. The length of pier accommodates both the bridge and gate hoists.

The flow at the toe of the overflow section is supercritical (Froude number  $> 1$ ) and the velocities are high which may scour the river bed in the downstream. This scouring may damage the structure. Hence energy dissipation arrangements are made at the toe of the section. These are of different types as below and are shown in Fig. 3.2.

- (i) **Hydraulic jump basin:** A hydraulic jump is formed in the basin, which is provided at the toe of spillway when supercritical flow at the toe of spillway meets the subcritical flow in the basin where water depths will be more. For a supercritical flow depth ' $d_1$ ' at the toe a conjugate depth ' $d_2$ ' is required for the formation of jump and the energy is lost in the jump. So this type of energy dissipation structure is provided where the downstream depth of flow (tail water level) corresponding to spillway discharge is close to the conjugate depth. The length of the basin and the requirement of other appurtenances such as chute blocks, baffle blocks and end sill and their dimensions depends on the Froude number of the incoming flow depth. A good jump forms when Froude number is 4 or more. If it is less than 4, then a weak jump will form. If the tail water depth is more than conjugate depth then a submerged jump will form with pulsating flow which may travel long distance in the downstream causing scouring and retrogression in long reaches. For detail design of hydraulic jump basin IS Code No. 4997 may be referred.
- (ii) **Slotted roller bucket type:** This type of energy dissipator is provided when tail water level exceeds the conjugate depth. It consists of a circular bucket of

considerable radius with a slotted lip at the end. The high velocity flow is deflected away from the river bed by the lip of the bucket in the form of two rollers which are diffused in the mass of water resulting in dissipation of energy. In cases where tail water depth is substantially more than conjugate design, the solid roller bucket is preferred. For design parameters of the bucket type energy dissipaters readers may refer IS Code No. 7365.

- (iii) Ski jump bucket: This type is suitable when the river bed is of strong rock and the tail water depth is less than the conjugate depth required for jump formation. In this case a bucket of considerable radius and included angle is provided at the toe of spillway with a solid lip at the end of bucket. The high velocity jet is thrown away from the structure at a considerable distance. The jet strikes either the rocky bed or impinges into the pool of water. The energy is dissipated due to impact. For the details of radius, invert level, lip, throw distance and pool size etc. IS Code 7365 may be referred.

River bed protection in the downstream of hydraulic jump basin in the form of CC blocks and boulders etc. is provided because all the energy is not dissipated in the jump. The length of such protection depends on the expected scour. Similarly a concrete apron is provided in the downstream of the buckets to protect river bed from scour in the vicinity of buckets.

### **3.4.2 Non-Overflow Section**

The non-overflow section on either side of the overflow section is basically a triangular section with base width required from stability considerations. The top is widened to accommodate the road bridge. The downstream face is made straight from top of dam upto a depth generally equal to the road width. After that it slopes to meet the downstream edge of required base width as shown in Fig. 3.1.

A divide wall or training wall separates the overflow section from non-overflow section. The height of training wall depends on depth of flow along overflow section and in the basin or bucket corresponding to the design discharge of spillway. To fix the top level of the training wall a freeboard of 1.5 m shall be provided above the water level.

One or two non-overflow blocks on either one side or both sides of the overflow section are widened to accommodate control room for gates, site office, elevator shaft and stair case to reach the galleries, stair case to approach the gate hoist bridge and also sometimes to store the stop logs of the emergency gate and to park the stop log crane for maintenance.

The intake structures for the outlets from the dam for the release of regulated flows for irrigation or power generation are generally located in one or more non-overflow blocks. These intake structures are placed at the upstream face and the conduit or penstock which carries water to the downstream passes through the

body of the dam. The intake structures are generally of two types: (i) straight type and (ii) semicircular type. These are shown in Fig. 3.3. The straight type intakes are provided in small height dams and semicircular type in high dams. In straight type, the trash rack is on the upstream face of dam and in semicircular type the trash rack is in the semi-circular cage built in front of the dam face. The cage is attached with upstream face of dam. The crest level of the invert of the conduit/penstock should be sufficiently below minimum draw down level (MDDL) so that air is not drawn into the conduit but it shall be substantially above the dead storage level so that sediment entry into intake is minimized. The regulation gate is located inside the body of dam near the upstream face and is generally operated from the top of dam through a gate shaft or well. In high dams the gate is operated through a gate gallery located above the gate.

### 3.4.3 Drainage Arrangement

There is always a possibility of seepage of reservoir water through the foundation and the body of the dam despite all measures and precautions taken during construction. This shall be collected as near the upstream face as possible and shall be discharged in the river in the downstream of dam.

The seepage through foundation is collected through the drainage holes provided immediately downstream of the grout curtain which is provided as a measure to check seepage through foundation. The drainage holes are spaced generally about 3 to 4 m and their depth is about  $3/4^{\text{th}}$  the depth of grout curtain and they discharge the seepage flow into a drain which is provided in the foundation gallery through which the grout curtain is constructed. This gallery is provided throughout the length of dam in both overflow and non-overflow portions and is constructed near the upstream face of dam as shown in Fig. 3.1.

The seepage through transverse joints where these joints are not grouted may take place despite the provision of waterstops due to excessive hydraulic pressure. This is collected through a form drain which is provided just downstream of waterstops. This is 20 cm diameter hole with porous or unvibrated concrete around it. The form drains discharge seepage water into the drain provided into foundation gallery. The form drain thus runs from top to bottom in the entire height of the dam.

Seepage may also take place through upstream face of dam due to cracks in concrete which is subjected to large temperature variation. To arrest this kind of seepage, form drains are provided at about 3 m from the upstream face at a spacing of 3 to 4 m. The construction is similar to the form drain of transverse joint and these also drain the seepage water into the drain provided in the foundation gallery. The form drain location and its details are shown in Fig. 3.4. The porous concrete blocks shown in Fig. 3.4 are generally used in measuring dams.

The drain in the foundation gallery, which collects the seepage from the foundation as well as the dam body through form drains, discharges all the water into a

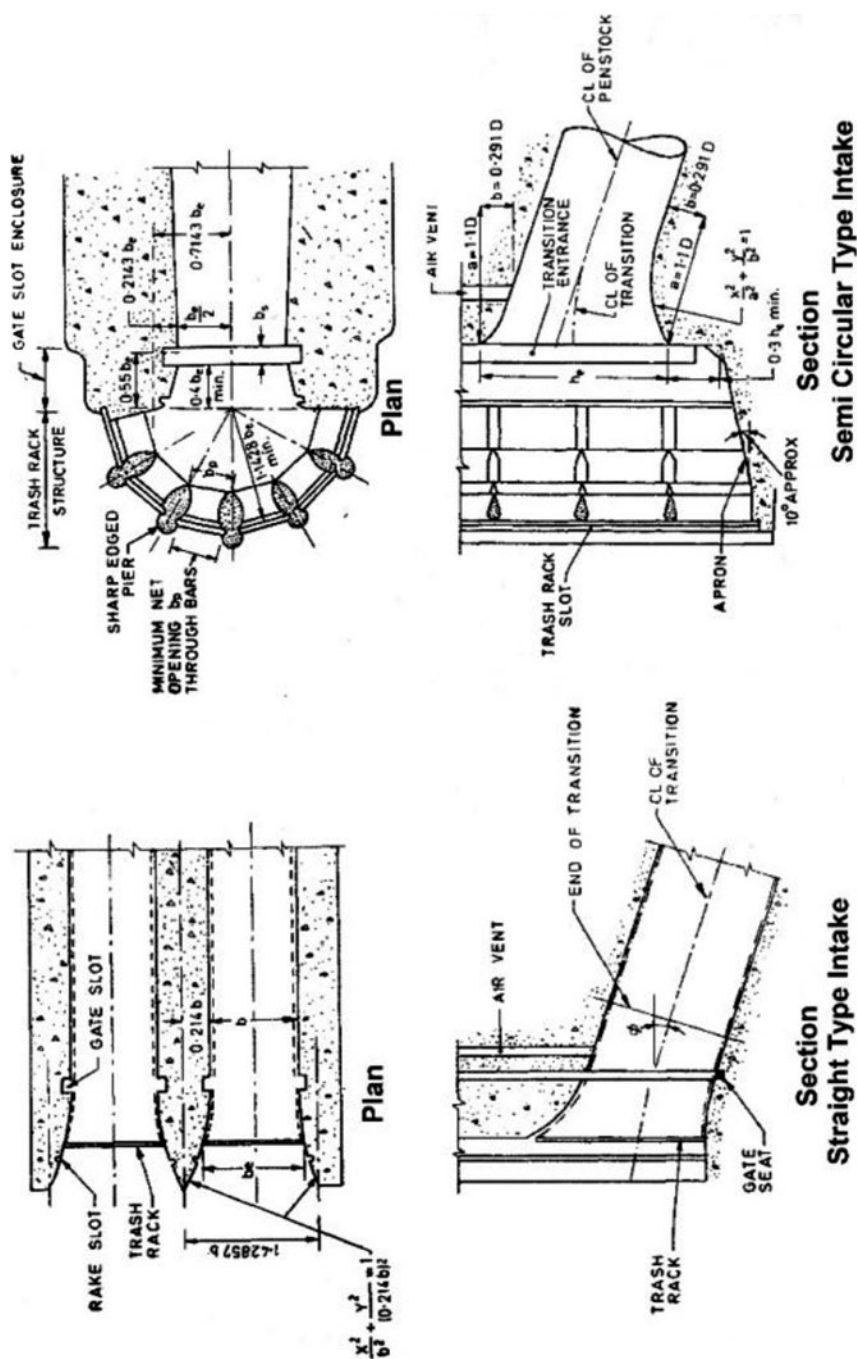
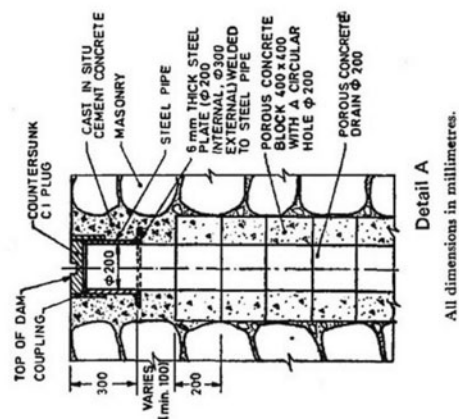
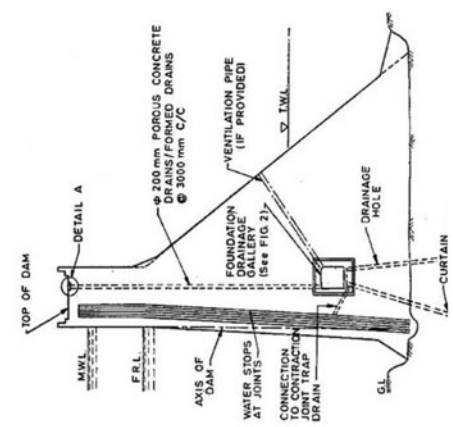


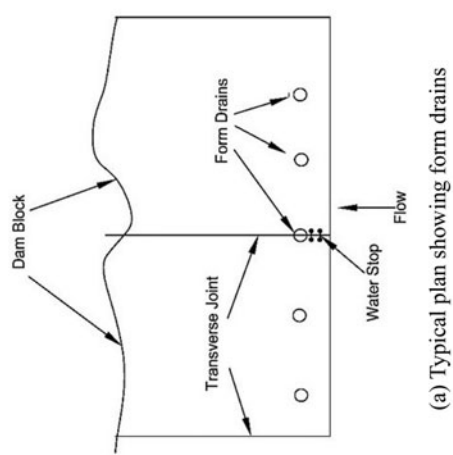
Fig. 3.3 Type of intake structures.



(c) Porous Concrete Drain/Formed Drain



(b) Dam section showing porous concrete drain formed drain



(a) Typical plan showing form drains

Fig. 3.4 Details of Form Drains.

sump located at the lowest point of the foundation gallery. For layout and design of galleries and other openings IS 12966 may be referred. The bottom of sump is kept below the level of foundation gallery.

The sump is fitted with submersible pumps which empty the sump by discharging in the stilling basin preferably above the high flood level. The pump chamber is constructed either at the top of sump or at the top of dam. If it is at dam top it is connected with the sump through a shaft. In some cases sump and pump chamber are made outside the dam body. The size and capacity of sump may be estimated on the basis of a seepage rate of 1 litre/minute/6 sq.m of concrete surface area which is under water. The seepage from foundation may be taken as 1.5 times of the seepage from dam body. However, the capacity of sump should be conservatively fixed. The pump must run atleast for 3 minutes for one time emptying the sump.

#### **3.4.4 Galleries and Openings**

A large number of galleries and openings are provided in a dam for various purposes such as grouting, drainage, operation of gates, valves and pumps, inspection, instrumentation etc. In addition, other type of openings for sluices, conduits, approaches etc. are also provided. According to their alignment these are grouped as below:

(a) *Parallel to dam axis:*

- (i) The foundation gallery runs along the length of dam with its bottom atleast 1.5 to 2.0 m in concrete above the foundation rock. It is parallel to dam axis and at a minimum distance of 1.5 m or 5% of water head from the upstream face of dam conforming to transverse profile of the river valley. It is used for making grout curtain by drilling holes for grouting near the upstream face of gallery and drilling drainage holes in foundation near the downstream face. It is also used for inspection. The minimum size of foundation gallery is 2.25 m in height and 1.5 m in width. The drain to carry seepage from drainage holes shall be made 0.3 m wide and atleast 0.5 m deep. It shall have sufficient slope.
- (ii) Drainage gallery: A supplementary drainage gallery is sometimes provided in the downstream portion of the dam section in some of the deepest blocks of the dam. Arrangement shall be made to drain this gallery also in the drainage sump. At times more than one such drainage gallery is provided.
- (iii) Gate gallery: The gate gallery is provided to house the gate or valve operating arrangement for sluices or penstocks etc. The location and size depends on the requirement of the equipments to be placed and the number of gates.
- (iv) Inspection gallery: Sometimes in high dams a few more galleries above foundation gallery at higher elevations are provided for inspection and to

take observations of embedded instruments which are provided in a few selected blocks and at different elevations to monitor the behaviour of the dam.

(b) *Normal to dam axis:*

These are generally adits to approach the galleries which are parallel to dam axis.

(c) *Vertical shafts:*

The vertical shafts are provided for housing elevators, stairs, intake gates etc. which are operated from the top of dam.

Normally galleries and adits are of minimum size of 1.5 m wide and 2.25 m high. The dimensions of vertical openings depend on the operating requirement. All these galleries, adits and shafts shall be provided with adequate lighting and ventilation. IS. 12966 (Part-1) may be referred for general requirement and layout of galleries and openings.

For design, these openings are considered as openings in infinite mass. Design procedure has been developed on the basis of stress analysis of openings in thin semi-infinite plates. It is dealt separately in para 3.5.8. The design method is also described in IS Code 12966 (Part 2).

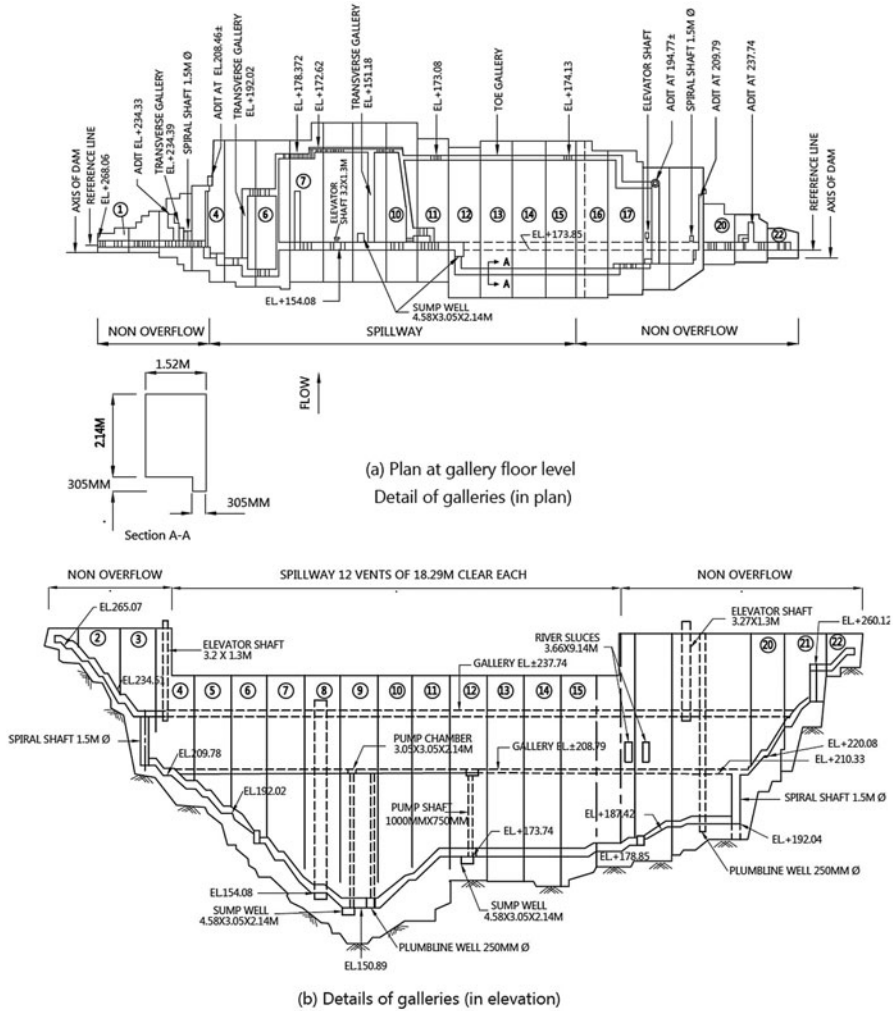
In order to illustrate the layout of a concrete dam the plan, elevation and section of Sirisialam dam are enclosed as Fig. 3.5.

## 3.5 DESIGN OF DAM

### 3.5.1 *Concept and Criteria*

The gravity dam section for height greater than 5-6 m shall be checked for stability. The stability requirements are:

- (i) The stresses in all load combinations (defined in para 3.5.3) in concrete at all elevations and the base shall be within permissible values of compression and tension in concrete. Normally the entire concrete section shall be under compression for which it shall be ensured that the resultant of all forces lies within the middle third of the base.
- (ii) The compressive stress transferred to the foundation rock shall be within the permissible bearing capacity of the rock. The permissible bearing capacity should be worked out by testing the rock. For preliminary studies the values in Table 3.1 may be adopted.
- (iii) The dam section shall be safe against overturning. It means that ratio of stabilizing moment to overturning moment shall be more than one. Usually a factor of safety of 2 shall be ensured.
- (iv) The dam section shall be safe against sliding at the base or at any plane within the dam and the foundation.



**Fig. 3.5** (a), (b) and (c) Details of Sirisalam Dam.

The criteria to check sliding is as below:

- (a) *Sliding on dam base*: It is evaluated as sliding resistance of dam foundation along the base considering either frictional resistance only or considering both frictional and cohesive strength of rock base. The coefficient of static friction  $\mu$  ( $\tan \phi$ ) for normally encountered rock foundations of a concrete dam is 0.65 to 0.75 for normal loading and 0.85 for extreme loading conditions:

Factor of safety =  $\frac{\sum V}{\sum H} > 1.5$  for normal loading conditions and not less than 1.0 for extreme loading conditions where  $\sum V$  is total vertical load and  $\sum H$  is total horizontal

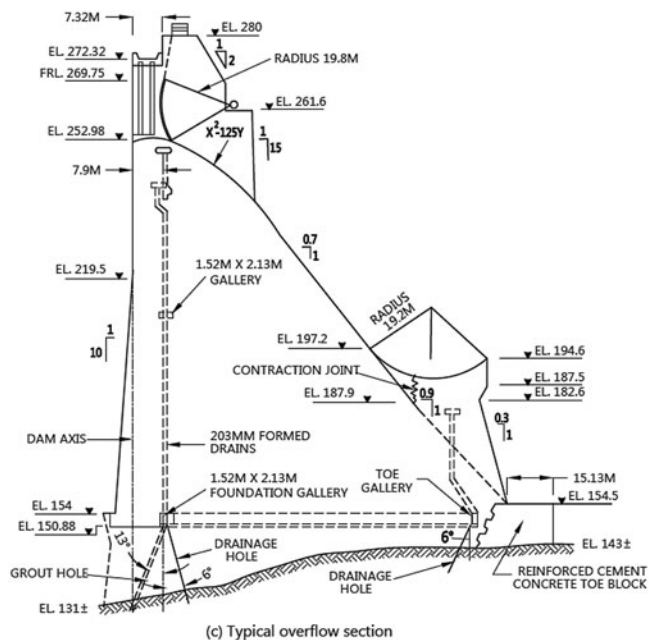


Fig. 3.5 (continued)

Table 3.1 Allowable bearing pressure

Sl. No.	Type of foundation	Allowable pressure tonnes/sq.m
1.	Sand, coarse, loose; sand, fine, compact	30
2.	Gravel, sand-gravel mixture, loose; sand coarse, compact	40
3.	Gravel, sand-gravel mixture, compact	50
4.	Hardpan; residual deposits of shattered or broken bed rock of any kind except shale; and shale in sound condition (some cracks allowed)	100
5.	Laminated rocks in sound condition (some cracks allowed) like slate and schist	350
6.	Massive bed rock without lamination (granite, gneiss, trap, felsite and thoroughly cemented conglomerates, all in sound condition with some cracks allowed), ledge rock	1000

force. For large dams the sliding safety is evaluated considering both frictional and cohesive strength as below:

Shear Friction Factor (SFF) =  $\frac{\mu \sum V + CA}{\sum H} > 1.5$  for normal loading conditions where  $C$  is cohesive resistance of foundation material at the base of dam and  $A$  is the area of dam base. The value of  $C$  and  $\mu$  ( $\tan \phi$ ) shall be determined by actual tests of foundation material. However, for preliminary studies the values in Table 3.2 may be adopted.

**Table 3.2** Values of cohesion and friction

Material	$\tan \phi$	Minimum cohesion $C$ , kg/cm <sup>2</sup>
Rock, sound massive	0.80	30
Rock, fractured, jointed	0.60	7
Gravel	0.50	0

As per IS Code-6512, the safety against sliding in terms of shear friction factor shall be computed as below and it shall not be less than 1.0.

$$\text{Shear Friction Factor} = \frac{\mu \sum V}{\sum H} + \frac{CA}{F_c}$$

where  $F_\phi$  and  $F_c$  are partial factor of safety in respect of friction and cohesion respectively. Their values for different loading conditions (defined in para 3.5.3) are as below:

Loading conditions	$F_\phi$	$F_c$ for dam base with foundation	$F_c$ (for foundation)	
			Fully investigated	Others
A, B, C	1.5	3.6	4.0	4.5
D,E	1.2	2.4	2.7	3.0
F, G	1.0	1.2	1.35	1.5

(b) *Sliding resistance in dam section:*

The expression for shear friction factor (SFF) given above can be used to evaluate the sliding resistance at any horizontal construction joint in the dam section. It is the shear resistance between two concrete surfaces and much depends on the development of bond between two lifts. Considering the uncertainties in construction the value of  $\phi$  is taken as  $45^\circ$  and the value of  $C$  is taken as  $0.05 f_c$  where  $f_c$  is compressive strength of concrete.

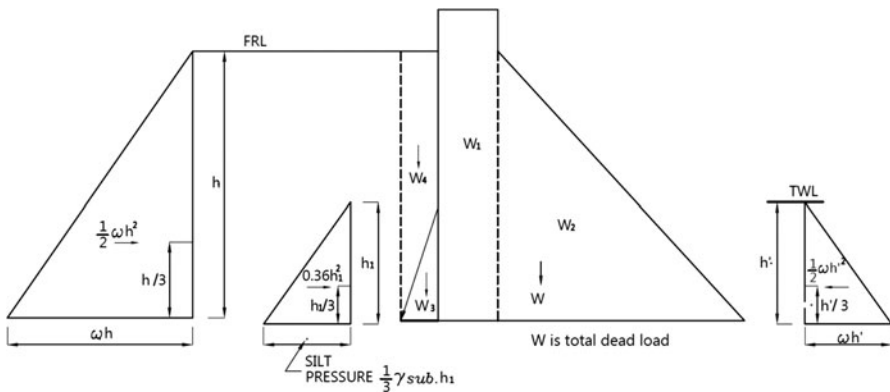
### 3.5.2 Forces Acting on the Dam

The forces acting on a gravity dam are:

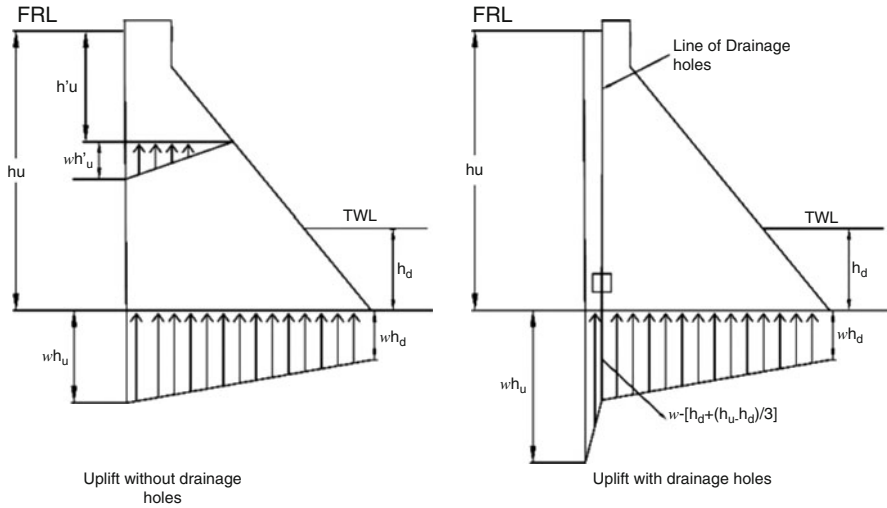
1. Dead load
2. Water pressure
3. Silt pressure
4. Uplift pressure
5. Wave pressure
6. Wind pressure
7. Earthquake forces
8. Ice load.

These forces are estimated considering unit length of dam as described below:

1. **Dead Load:** It includes the weight of concrete plus other loads supported on dam such as gates, bridge, hoist etc. To estimate the weight of concrete section the assumed sectional area is multiplied by unit weight of concrete which is taken as  $2.4 \text{ T/m}^3$ . This load is assumed acting as a point load at the centre of gravity of the section. For ease of computation the dam section is divided into parts as shown in Fig. 3.6.
2. **Water Pressure:** The water pressure at any point of dam section is equal to the hydraulic head at that point. At the base of section the water pressure is equal to the reservoir depth at that section. Hence the total water pressure is equal to the area of the triangular pressure diagram. The total water force is thus, equal to  $\frac{1}{2} wh^2$  where ' $w$ ' is unit weight of water which is taken as  $1 \text{ T/m}^3$ . The total water pressure acts on dam section at  $h/3$  above the base. If the water exerts pressure on dam section from downstream side with depth  $h'$  above base it can be estimated as  $\frac{1}{2} wh'^2$  acting at a height of  $h'/3$  above base. It is illustrated in Fig. 3.6. If the u/s face is inclined then the weight of water acting on dam shall be equal to the area of water above the section and shall be considered as vertical load along with the dead load. It is shown as  $W_4$  in Fig. 3.6.
3. **Silt Pressure:** Silt pressure is assumed to be acting on upstream face corresponding to the level upto which silt is expected to get deposited. It may be considered to be deposited upto dead storage level. If the height of deposited silt above base is  $h_1$  as shown in Fig. 3.6 then silt pressure at base is  $\frac{1}{3} \gamma_{\text{sub}} h_1$  and total silt pressure acting at  $h_1/3$  is  $\frac{1}{6} \gamma_{\text{sub}} h_1^2$ ,  $\gamma_{\text{sub}}$  is taken as  $2.1 \text{ T/m}^3$ . Thus total silt pressure is taken as  $0.36 h_1^2$ . If the face is inclined then weight of silt shall be taken with a density of  $1.9 \text{ T/m}^3$ .
4. **Uplift Pressure:** Percolation invariably takes place through the dam along lift joints and cracks in concrete and through the joints and seams in the foundation. This causes hydrostatic force within the dam body and foundation corresponding to water level in reservoir which acts upwards reducing the weight of dam and



**Fig. 3.6** Loads acting on dam.



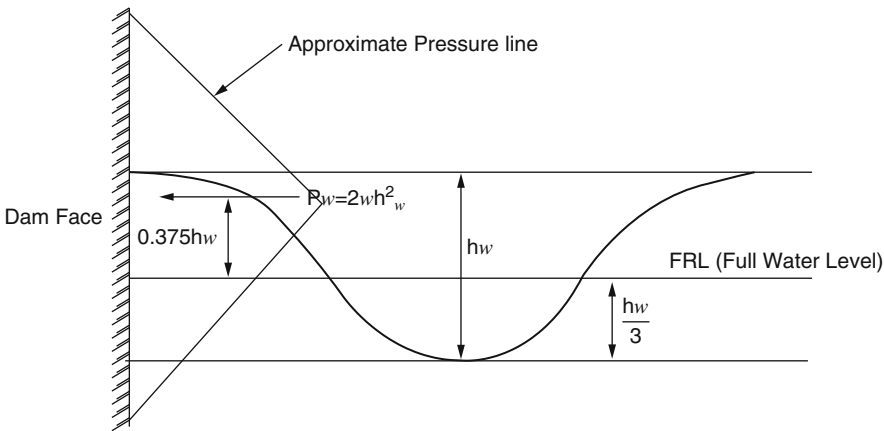
**Fig. 3.7** Uplift diagram.

affecting the stability of dam adversely. This hydrostatic force is termed as uplift pressure and is assumed to be acting on full area of concrete in dam body as well as on the foundation base and its distribution from upstream to downstream face is assumed a straight line. Since it affects the stability adversely, provisions are made in the dam to reduce it. Drainage holes are made near the upstream face. The reduction in uplift pressure depends on the size, depth, spacing of holes as well as the foundation rock characteristics such as joints, permeability etc. and the efficacy of the grout curtain provided in the upstream of the drainage holes. The experience of USBR and other dams of TVA (Tennessy Valley Authority of USA) etc. has shown that the intensity of uplift pressure at the line of drainage holes is around 30% of the net head at line of holes. Therefore, for design purpose the uplift pressures with and without drainage holes (assuming holes choked) are taken as shown in Fig. 3.7.

5. **Wave Pressure:** The waves are produced by the wind blowing over the surface of water in reservoir. When these waves strike against the upstream face of dam they exert pressure called wave pressure. It is an overturning horizontal pressure acting above the FRL in the height provided as free board. Therefore, the wave pressure depends on wave height which depends on the length of reservoir at FRL and the wind velocity. The wave height ' $h_w$ ' can be determined by the following relation.

$$h_w = 0.032 FV + 0.763 - 0.271 F^{1/4} \quad (\text{For } F < 32 \text{ km})$$

$$h_w = 0.032 FV \quad (\text{For } F > 32 \text{ km})$$



**Fig. 3.8** Wave pressure diagram.

where  $h_w$  is wave height (m),  $F$  is reservoir fetch (km) and  $V$  is wind velocity (km/hr). The wave pressure is taken as triangular and the total wave force  $P_w = 2 wh_w^2$  ( $w$  is unit weight of water) and is assumed to be acting at  $0.375 h_w$  above still water level (FRL). It is shown in Fig. 3.8.

6. *Wind Pressure:* Wind will exert pressure depending on velocity when it will strike the exposed surface of the upstream face. So this will act when reservoir level is low. Hence it is not a significant force for the design of dam and is ignored.
7. *Earthquake Force:* If a dam is located in a region which is earthquake prone, it shall be analyzed for the seismic forces. Usually a pseudostatic approach of analysis is followed. The seismic forces are considered as static inertial forces for gravity load combined with water. An earthquake causes acceleration which may be in any direction; so for design purposes it is considered as vertical and horizontal acceleration separately. The acceleration is expressed as a percentage of acceleration due to gravity ( $g$ ) i.e.  $\alpha g$  where ' $\alpha$ ' is called seismic coefficient. The value of  $\alpha$  at a particular site depends on the region in which the dam site is located and the geology of the site. India is divided into five Zones I to V. Zone IV and V in Himalayan belt are more susceptible to earthquake. The basic horizontal seismic coefficient  $\alpha_{ho}$  of each zone is specified (refer IS 1893) and is given below:

Zone	I	II	III	IV	V
$\alpha_{ho}$	0.01	0.02	0.04	0.05	0.08

Horizontal seismic coefficient  $\alpha_h = \beta I \alpha_{ho}$ , where  $\beta$  depends on foundation type and is 1.0 for rock and hard strata and  $I$  is importance factor which is taken as 3.0 for dams.

$$\alpha_v(\text{vertical seismic coefficient}) = 0.75 \alpha_h$$

In pseudo static analysis  $\alpha_h$  and  $\alpha_v$  are considered constant though these vary with the height of dam. Therefore, for medium height dams (say for a height of around 30 m) the pseudo static approach for analysis may be used but for high dams dynamic analysis using response spectrum method shall be used. The seismic forces for pseudo static approach are as given below.

Horizontal acceleration exerts two types of lateral forces on the dam: (i) inertial force and (ii) hydrodynamic force of water.

- (i) Inertial force: The horizontal acceleration causes movement which is resisted by the dam due to inertia and so a horizontal inertial force  $W\alpha_h$  acts at the centroid of the dam section. This force is in opposite direction of acceleration. For stability computation this force is considered acting in the direction of water force from reservoir.

The vertical acceleration  $\alpha_v$  if acting upwards will cause inertial force  $W\alpha_v$  in downward direction. This force, in stability consideration, is considered acting upwards, thereby reducing the vertical weight of dam. These inertial forces are momentary and keep on changing the direction.

- (ii) Hydrodynamic pressure: The hydrodynamic pressure is generated due to lateral movement of upstream face of dam against reservoir water during earthquake. This force is in addition to hydrostatic pressure. The hydrodynamic pressure is computed by Zangers method. This is based on the concept of added mass of water with the dam body. Therefore, when acceleration is towards upstream direction the hydrodynamic force will be acting in downstream direction and so it will be additive with hydrostatic pressure and this condition is considered for stability computation.

The hydrodynamic pressure diagram is parabolic and the pressure at any depth is given by

$$p_e = C \alpha_h w y h$$

$$C = \frac{C_m}{2} \left[ \frac{y}{h} \left( 2 - \frac{y}{h} \right) + \sqrt{\frac{y}{h} \left( 2 - \frac{y}{h} \right)} \right]$$

where

$p_e$  = hydrodynamic pressure at any depth normal to dam face.

$C$  = coefficient

$\alpha_h$  = horizontal acceleration due to earthquake

$w$  = unit weight of water

$y$  = depth of reservoir at dam section for the elevation in question.

$h$  = total depth of reservoir surface at the dam section under consideration.

$C_m$  = maximum value of  $C$ .

The total hydrodynamic force at any elevation  $y$  below reservoir surface is given by

$$P_E = 0.726 p_e y$$

and the moment due to horizontal force  $P_E$

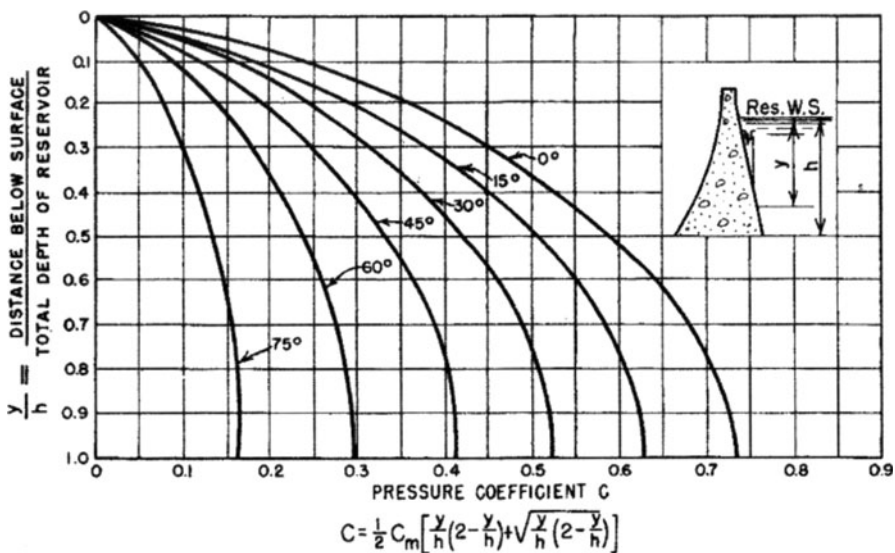
$$M_E = 0.299 p_e y^2$$

$$\text{Point of application of } P_E = \frac{4y}{3\pi}$$

At the dam base where  $y = h$ ,  $P_E = 0.726 C_m \alpha_h w h^2$  and  $M_E = 0.299 C_m \alpha_h w h^3$ . The value of  $C$  for various inclinations of upstream face are given in Fig. 3.9.

For dams with partially inclined upstream face the value of  $C$  is worked out as below:

- If the height of vertical portion of the upstream face of the dam is equal to or greater than one-half of the total height of dam, analyse as if the face is vertical throughout.
- If the height of vertical portion of the upstream face of dam is less than one-half of the total height of dam, use the pressure on sloping line connecting the point of



**Fig. 3.9** Coefficients for pressure distribution for constant sloping faces. (Source: Design of Small Dams, USBR)

inter-section of the upstream face of dam and the reservoir surface with the point of intersection of upstream face of the dam with the foundation.

8. *Ice Load:* Ice load is required to be considered in cold climate where the freezing of the reservoir surface is expected. The pressure is exerted due to thermal expansion of ice due to temperature rise after the cold wave. The magnitude of the ice pressure has been estimated varying from 0 to 80 t / linear m of the contact with vertical face. A value of 25 t/m<sup>2</sup> is widely accepted for analysis.

### 3.5.3 Load Combinations

The following load combinations are recommended for ensuring the safety and economy of the dam section.

- (i) A (construction stage) – dam completed but no water in reservoir.
- (ii) B (normal operating condition) – reservoir full upto FRL, normal tail water level, and normal uplift (drains operative)
- (iii) C (flood condition) – reservoir at MWL, maximum tail water level (all spillway gates open) and normal uplift.
- (iv) D – condition A with earthquake
- (v) E – condition B with earthquake
- (vi) F – condition C with full uplift (drains inoperative)
- (vii) G – condition E with full uplift (drains inoperative)

The above load combinations are grouped as below:

Condition A & B are usual load combinations

Condition C is unusual load combination

Condition D to G are extreme load combinations.

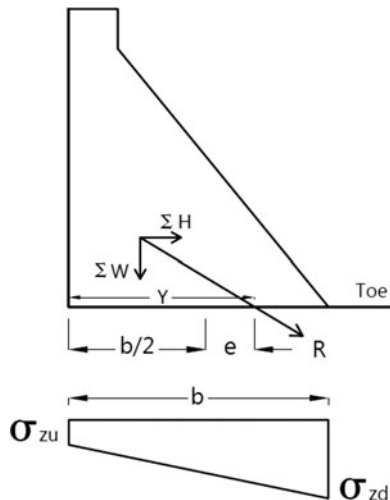
### 3.5.4 Stability Analysis

The stability analysis has following assumptions:

- (i) Dam is divided into blocks and each block behaves as an independent cantilever fixed at base. Loads and forces are worked out per linear metre length of the block.
- (ii) Vertical stress varies linearly at the base or any other horizontal plane from upstream to downstream end.

A dam section is assumed for stability analysis. The downstream slope is assumed on the basis of existing similar height dams. An economical section satisfying stability criterion with adequate FOS is evolved after trial. For stability analysis the loads vertical and horizontal acting on a dam are worked out which are required

**Fig. 3.10** Vertical normal stress at base.



for load combinations A to G. The moment of all the loads about the toe are also worked out. Then for all load combinations the stresses on heel and toe are worked out by

$$\sigma_{zu}, \sigma_{zd} = \frac{\sum W}{b} \left( 1 + \frac{6e}{b} \right)$$

where  $\sigma_{zu}$  and  $\sigma_{zd}$  are normal vertical stress on upstream and downstream ends,  $b$  is base width of section and  $e$  is eccentricity, the distance of the point at which the resultant force cuts the base from the mid-point as shown in Fig. 3.10.

$$e = \bar{y} - \frac{b}{2}$$

$$\bar{y} = \frac{\sum M_{toe}}{\sum W}$$

where  $\sum M_{toe}$  is total of moments of all forces about the toe and  $\sum W$  is total of all vertical loads.

If  $e$  is of negative value, resultant is acting on upstream of the middle point of base and if of a positive value the resultant is passing through a point on the downstream of middle point of base.

These vertical normal stresses should be within permissible compressive value of concrete. The maximum permissible compressive stress limit should be one year strength of concrete  $f_c$  divided by 3.0, 2.0 and 1.0 for normal, unusual and extreme load combinations respectively. In no circumstances the compressive stress in any part of section shall exceed  $70 \text{ kg/cm}^2$  in normal load combinations and  $100 \text{ kg/cm}^2$  in other load combinations. If  $e > b/6$ , tension will develop. No tension shall be

permitted for load combinations A and B. For other load combinations permissible tensile stress shall be as below:

Load combination	Permissible tensile stress
C	$0.01 f_c$
E	$0.02 f_c$
F	$0.02 f_c$
G	$0.04 f_c$

In dams  $f_c$  of concrete normally ranges between 150 to 300 kg/cm<sup>2</sup> and generally a tension more than 5 kg/cm<sup>2</sup> shall not be allowed.

These normal stresses are transferred to the foundation and shall be within permissible compressive stress of the foundation rock. The maximum permissible compressive stress in foundation shall be less than the compressive strength of rock divided by 4.0, 2.7 and 1.3 for normal, unusual and extreme load combinations respectively.

In addition to stress computation, the factor of safety against overturning and sliding shall be worked out for all load combinations by the method described above and the minimum required factor of safety shall be ensured. However, the section designed on no tension basis will be safe in overturning.

### 3.5.5 Principal, Normal Horizontal and Shear Stresses

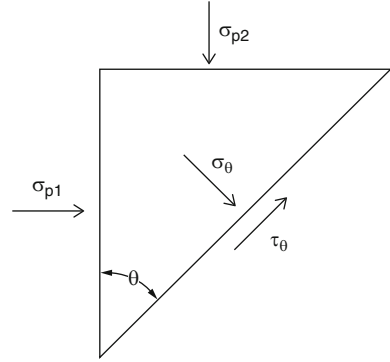
The vertical normal stress at upstream and downstream end of the section at any horizontal plane or the base and its distribution along the plane or base, which is assumed linear, are determined as discussed above. In terms of these normal vertical stresses the principal, normal horizontal and shear stresses at upstream and downstream face can also be worked out by using expressions derived below.

Plane on which stress acts normally and there is no shear stress is called principal plane and the stress is called principal stress. At any point there will be two principal planes perpendicular to each other and the stresses on these two planes are called major and minor principal stresses.

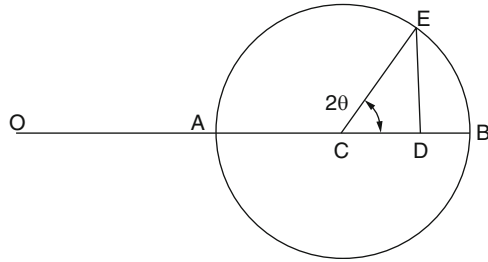
- (i) Determination of normal and shear stress on any oblique plane in an element which is acted upon by the principal stresses:

Consider the element and oblique plane as shown in Fig. 3.11. Here  $\sigma_{p1}$  = Maximum principal stress,  $\sigma_{p2}$  = Minimum principal stress,  $\sigma_\theta$  = Normal stress on an oblique plane inclined at an angle  $\theta$  and  $\tau_\theta$  = Shear stress along the oblique plane.

**Fig. 3.11** Stresses on oblique plane.



**Fig. 3.12** Representation on Mohr's Circle.



By equating the forces we get

$$\sigma_{\theta} = \sigma_{p1} \cos^2 \theta + \sigma_{p2} \sin^2 \theta \quad (1)$$

or

$$\sigma_{\theta} = \frac{1}{2} (\sigma_{p1} + \sigma_{p2}) + \frac{1}{2} (\sigma_{p1} - \sigma_{p2}) \cos 2\theta$$

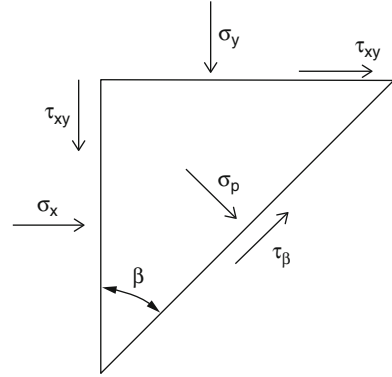
and

$$\tau_{\theta} = (\sigma_{p1} - \sigma_{p2}) \frac{1}{2} \sin 2\theta \quad (2)$$

It can be represented on Mohr's circle as shown in Fig. 3.12, where  $OB = \sigma_{p1}$ ,  $OA = \sigma_{p2}$ ,  $OD = \sigma_{\theta}$  and  $DE = \tau_{\theta}$ .

- (ii) If normal vertical stress  $\sigma_x$ ,  $\sigma_y$  and shear stress  $\tau_{xy}$  are acting on an element as shown in Fig. 3.13, stresses on any inclined plane can be derived by using above Eqs (1) and (2).

**Fig. 3.13** Principal stresses on inclined plane.



$$\sigma_p = \sigma_x \cos^2 \beta + \sigma_y \sin^2 \beta + \tau_{xy} \sin 2\beta$$

$$\tau_\beta = \left( \frac{\sigma_x - \sigma_y}{2} \right) \sin 2\beta - \tau_{xy} \cos 2\beta$$

The plane at angle  $\beta$  can be a principal plane if  $\tau_\beta = 0$ .

$$\therefore \tan 2\beta = \frac{2\tau_{xy}}{\sigma_x - \sigma_y}$$

Then principal stresses (substituting value of  $\beta$  in above equation).

$$\sigma_{p1,2} = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left( \frac{\sigma_x - \sigma_y}{2} \right)^2 + \tau_{xy}^2} \quad (3)$$

$$\therefore \sigma_{p1} + \sigma_{p2} = \sigma_x + \sigma_y$$

Thus, if normal vertical stress  $\sigma_y$ , normal horizontal stress  $\sigma_x$  and the shear stress  $\tau_{xy}$  acting on an element are known, with the above relations principal stresses and principal planes can be determined.

### 3.5.6 Principal Stresses on Dam Faces

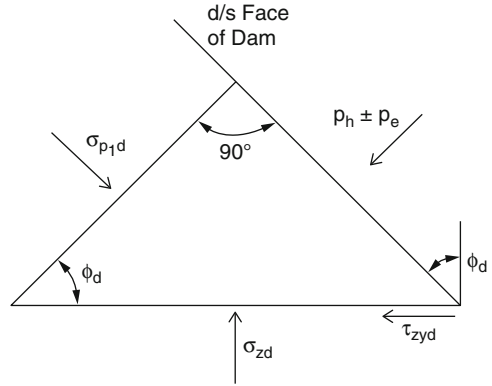
#### (i) On Downstream Face

As shown in Fig. 3.14, force  $(p_h \pm p_e)$  is acting on D/S face.

where  $p_h$  = Tail water pressure and  $p_e$  = Hydrodynamic force due to earthquake.

Since no shear force acts on downstream face, it becomes a principal plane, the other principal plane will be perpendicular to it so the stress  $\sigma_{p1d}$ , acting on this plane is the major principal stress. D/S face is minor principal plane with minimum principal stress equal to  $(p_h \pm p_e)$

**Fig. 3.14** Stresses on an element on D/S face of dam.



$$\sigma_{zd} =$$

$$\tau_{zyd} =$$

Using equation (1)

$$\sigma_{zd} = \sigma_{p1d} \cos^2 \phi_D + (p_h \pm p_e) \sin^2 \phi_D$$

or

$$\sigma_{p1d} = \sigma_{zd} \sec^2 \phi_D - (p_h \pm p_e) \tan^2 \phi_D$$

If there is no tail water level in the downstream, the maximum principal stress on D/S face is

$$\sigma_{p1d} = \sigma_{zd} \sec^2 \phi_D \quad (4)$$

From Eq. (2)

$$\begin{aligned} \tau_{zyd} &= \frac{1}{2} [\sigma_{zd} \sec^2 \phi_D - (p_h \pm p_e) \tan^2 \phi_D - (p_h \pm p_e)] \sin^2 \phi_D \\ &= [\sigma_{zd} - (p_h \pm p_e)] \tan \phi_D \end{aligned}$$

If there is no tail water the shear stress at the base of D/S end is

$$\tau_{zyd} = \sigma_{zd} \tan \phi_D \quad (5)$$

From Eq. (3)

$$\begin{aligned} \sigma_{yD} + \sigma_{zD} &= \sigma_{p1d} + (p_h \pm p_e) \sigma_{yD} \text{ is horizontal normal stress on vertical plane} \\ \text{or } \sigma_{yD} &= \sigma_{p1d} - \sigma_{zD} + (p_h \pm p_e) \end{aligned}$$

$\sigma_{yD} = \sigma_{zD} \sec^2 \phi_D - (p_h \pm p_e) \tan^2 \phi_D + (p_h \pm p_e) - \sigma_{zD}$  (on substituting the value of  $\sigma_{p1D}$ )

$$\sigma_{yD} = \sigma_{zD} \tan^2 \phi_D + (p_h \pm p_e)(1 - \tan^2 \phi_D)$$

If there is no tail water, horizontal normal stress on downstream face is

$$\sigma_{yD} = \sigma_{zD} \tan^2 \phi_D \quad (6)$$

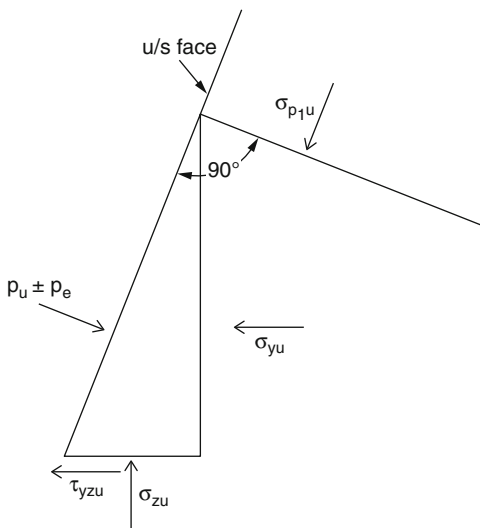
### (ii) On Upstream Force

As in case of D/S face, the U/S face which is acted upon by water pressure from reservoir, say  $p_u$ , with no shear stress is a principal plane and the other plane perpendicular to it is minor principal plane as shown in Fig. 3.15. In this case u/s face is a major principal plane and the other is the minor plane.

The expressions for principal stress, shear stress and horizontal normal stress on U/S base can be worked out in similar manner as derived above for D/S face. These will be as below when hydrodynamics force is not considered.

$$\begin{aligned} \sigma_{p1u} &= \sigma_{zu} \sec^2 \phi_u - p_u \tan^2 \phi_u \\ \sigma_{yu} &= \sigma_{zu} \tan^2 \phi_u - p_u (1 - \tan^2 \phi_u) \\ \tau_{yzu} &= -[\sigma_{zu} - p_u] \tan \phi_u \end{aligned}$$

**Fig. 3.15** Stresses on an element on U/s face.



**Example**

The enclosed appendix at the end of the chapter illustrates the computation procedure for stability analysis and determination of principal stresses at u/s and d/s and of a dam section at foundation level.

**3.5.7 Internal Stresses**

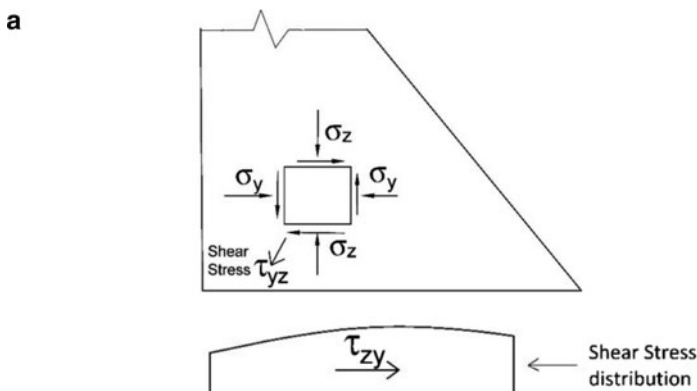
Maximum compressive or tensile stresses occur in a dam section at the faces. But knowledge of stresses at internal locations within the dam section is essential:

- to provide reinforcement around the openings in dam section.
- for aligning the faces of the shear keys at longitudinal contraction joint in line of major principal stress for load transfer at full water load.

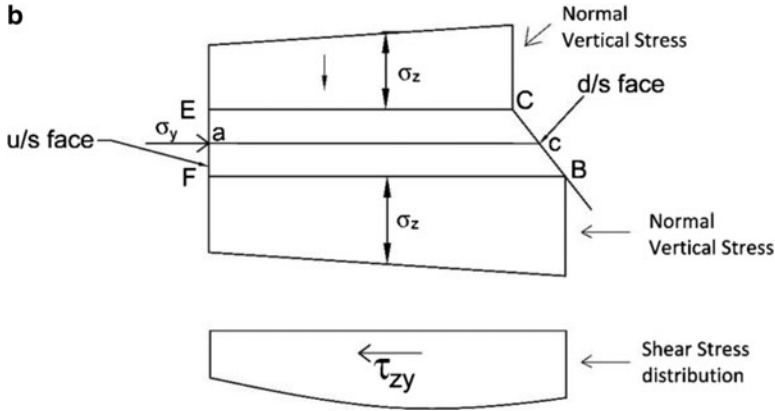
**Computation of Internal Stresses**

At every element in a concrete dam section normal and horizontal vertical stresses along with shear stress will be acting (shown in Fig. 3.16a) and if these can be evaluated, as explained earlier, the principal major and minor stresses and the major and minor principal planes can be worked out.

Suppose at any elevation in the dam the internal stresses at a specific distance from D/S face is required to be worked out, then consider two closely spaced horizontal planes at that elevation as shown in Fig. 3.16b. These are say EC and FB. The figure shows the normal vertical stress diagram at the two planes. The variation of this stress from U/S to D/S is a straight line. The figure also shows the shear stress diagram at the two planes and this is parabolic in variation from U/S to D/S face. The normal horizontal stress  $\sigma_y$  will be difference of shear stress at two planes.



**Fig. 3.16 (a)** Showing stresses on an element in a concrete dam.



**Fig. 3.16 (b)** Computing shear and normal horizontal stresses inside a dam section.

The normal vertical stress diagram can be represented by the equations:

$$\sigma_z = a + by$$

and the area of normal vertical stress diagram at any plane is given by

$$\int \sigma_z dy = \int (a + by) dy$$

The values of  $a$  and  $b$  can be determined by substituting the values of  $\sigma_{zu}$  at  $y = T$  and  $\sigma_{zd}$  at  $y = 0$  (base width of section is taken as  $T$  and  $Y$  is measured from D/S face moving towards upstream).

The shear stress diagram being a parabolic curve can be represented by a second degree equation in  $y$

$$\tau_{yz} = \tau_{zy} = a_1 + b_1y + c_1y^2$$

The values of three constants can be determined by substituting the known values of  $\tau_{zyd}$  and  $\tau_{zyu}$  at  $y = 0$  and  $y = T$  respectively and equating the area of shear stress diagram equal to total horizontal force at the section i.e.  $\int_0^T \tau_{zy} dy = \sum H$ . With these three conditions the constants are calculated as

$$\begin{aligned}
a_1 &= \tau_{yzD} = [\sigma_{zD} - (p_h \pm p_e)] \tan \phi_D \\
b_1 &= -\frac{1}{T} \left[ \frac{6 \sum H}{T} + 2\tau_{zyu} + 4\tau_{zyD} \right] \\
c_1 &= \frac{1}{T^2} \left[ \frac{6 \sum H}{T} + 3\tau_{zyu} + 3\tau_{zyD} \right]
\end{aligned}$$

Normal horizontal stress is the algebraic difference of the parabolic diagrams of horizontal shear stress acting on two faces and the horizontal inertial force due to earthquake. Area of shear stress diagram

$$\int_0^T \tau_{zy} dy = \int_0^T (a_1 + b_1 y + c_1 y^2) dy$$

Therefore, normal horizontal stress at any point can be expressed by a cubic equation.

$$\sigma_y = a_2 + b_2 y + c_2 y^2 + c_3 y^3$$

To evaluate these four constants four equations are required. Two equations can be obtained by putting the known values of  $\sigma_y$  at upstream end ( $T = y$ ) and at D/S end ( $y = 0$ ). The other two equations can be obtained by working out the value of  $\frac{\partial \sigma_y}{\partial y}$  from the following general stress equations

$$\frac{\partial \sigma_z}{\partial z} + \frac{\partial \tau_{yz}}{\partial y} + K_1 = 0 \quad (i)$$

$$\frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{zy}}{\partial z} + K_2 = 0 \quad (ii)$$

$K_1 = w_c$  (unit weight of concrete) and  $K_2 = \alpha w_c$  (the horizontal inertial force). These differential equations of two dimensional structural problems indicate equilibrium in horizontal and vertical directions. Equation (ii) can be written as

$$\frac{\partial \sigma_y}{\partial y} = - \left[ \frac{\partial \tau_{zy}}{\partial z} + K_2 \right] \quad (A)$$

$\tau_{zy}$  is a second degree equation given above and  $\frac{\partial \tau_{zy}}{\partial z}$  can be obtained by differentiating this equation and  $\frac{\partial \sigma_y}{\partial y}$  can be obtained by substituting  $\frac{\partial \sigma_{zy}}{\partial z}$  with known value of  $K_2$  in equation (A).

$$\text{and } \frac{\partial \sigma_y}{\partial y} = b_2 + 2c_2 y + 3d_2 y^2$$

(by differentiating the equation of  $\sigma_y$  given above). (B)

The above two values of  $\frac{\partial \sigma_y}{\partial y}$  at both U/S and D/S faces from (A) when substituted in (B) will give the conditions for evaluation of the other two constants. Thus the four constants are worked out as below:

$$\begin{aligned} a_2 &= \sigma_{yD} = a_1 \tan \phi_D + p' - p'_E \\ b_2 &= b_1 \tan \phi_D + \frac{\partial a'_1}{\partial z} - \alpha w_c \\ \frac{\partial a'_1}{\partial z} &= \tan \phi_D \left[ \frac{\partial \sigma_{zD}}{\partial z} z - w^* + \frac{\partial p'_E}{\partial z} \right] + \frac{\partial \tan \phi_D}{\partial z} [\sigma_{zD} - p' + p'_E] \end{aligned}$$

(\*to be omitted if tail water is absent)

$$\begin{aligned} \frac{\partial \sigma_{zD}}{\partial z} &= w_c + \tan \phi_u \left[ \frac{12 \sum M}{T^3} + \frac{2 \sum M}{T^2} - \frac{2p}{T} - \frac{2p_E}{T} \right] \\ &\quad + \tan \phi_D \left[ \frac{12 \sum M}{T^3} - \frac{1 \sum M}{T^2} + \frac{4p'}{T} - \frac{4p'_E}{T} \right] - \frac{6 \sum M}{T^2} \\ \frac{\partial p'_E}{\partial z} &= \frac{(p'_E - p'^*)}{\Delta z}, \quad \frac{\partial \tan \phi_D}{\partial z} = \frac{(\tan \phi_D - \tan \phi_D^*)}{\Delta z} \end{aligned}$$

(\*The value of the quantity is to be determined at a horizontal plane  $\Delta z$  distance above the horizontal section under consideration.)

$$\begin{aligned} c_2 &= c_1 \tan \phi_D + \frac{1}{2} \frac{\partial b_1}{\partial z} \\ \frac{\partial b_1}{\partial z} &= -\frac{1}{T^2} \left[ 6 \left( \frac{\partial \sum H}{\partial z} \right) - \frac{\partial T}{\partial z} \left( \frac{12 \sum H}{T} + 2\tau_{zyU} + 4\tau_{zyD} \right) \right] \\ &\quad - \frac{1}{T} \left[ 2 \left( \frac{\partial \tau_{zyU}}{\partial z} \right) + 4 \left( \frac{\partial \tau_{zyD}}{\partial z} \right) \right] \\ \frac{\partial \sum H}{\partial z} &= -(p - p' + \alpha w_c T + p_E + p'_E) \\ \frac{\partial T}{\partial z} &= \tan \phi_u + \tan \phi_D \\ \frac{\partial \tau_{zyU}}{\partial z} &= \tan \phi_U \left[ w^* - \frac{\partial \sigma_{zU}}{\partial z} + \frac{\partial p_E}{\partial z} \right] + \frac{\partial \tan \phi_U}{\partial z} [p + p_E - \sigma_{zU}] \end{aligned}$$

(\*w to be omitted if reservoir water is absent.)

$$\begin{aligned}\frac{\partial \sigma_{zU}}{\partial z} &= w_c + \tan \phi_U \left[ \frac{4p}{T^2} + \frac{4p_E}{T} - \frac{4 \sum W}{T^2} - \frac{12 \sum M}{T^3} \right] \\ &\quad + \tan \phi_D \left[ \frac{2 \sum W}{T^2} + \frac{2p_E}{T} - \frac{2p'}{T} - \frac{12 \sum M}{T^2} \right] + \frac{6 \sum H}{T^2} \\ \frac{\partial \tau_{zyD}}{\partial z} &= \frac{\partial a_1}{\partial z} \text{ and } \frac{\partial p_e}{\partial z} = \frac{(p_E - p^*_E)}{\Delta z} \\ \frac{\partial \tan \phi_U}{\partial z} &= \frac{\tan \phi_U - \tan \phi_U^*}{\Delta z}\end{aligned}$$

(\*The value of the quantity is to be determined at a distance  $\Delta z$  above the horizontal section under consideration.)

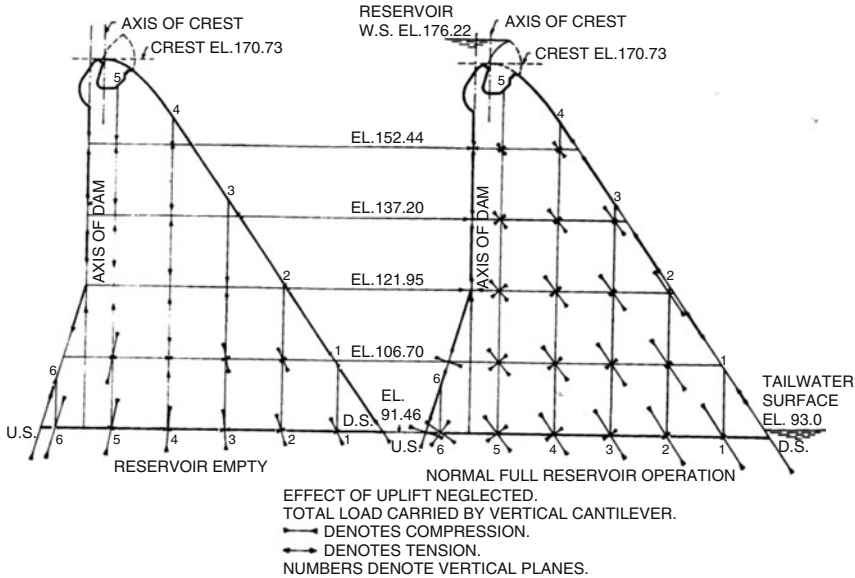
$$\begin{aligned}d_2 &= \frac{1}{3} \frac{\partial c_1}{\partial z} \\ \frac{\partial c_1}{\partial z} &= \frac{1}{T^3} \left[ 6 \left( \frac{\partial \sum H}{\partial z} \right) - \frac{\partial T}{\partial z} \left( \frac{18 \sum H}{T} + 6 \tau_{zyU} + 6 \tau_{zyD} \right) \right] \\ &\quad + \frac{1}{T^2} \left[ 3 \left( \frac{\partial \tau_{zyU}}{\partial z} \right) + 3 \left( \frac{\partial \tau_{zyD}}{\partial z} \right) \right]\end{aligned}$$

The values of  $\alpha$ ,  $p_E$  and  $p'_E$  in above equations correspond to upstream direction of earthquake ground motion and shall change sign if the direction of ground motion is reversed. It is seen that above procedure for computation of internal stresses at any point is very tedious and time consuming, if it is to be carried out manually (refer Sharma H.D).

The analysis of internal stresses can be easily and expeditiously carried out by using FEM for which computer programs are now available. Both 2-D and 3-D static and dynamic analysis can be carried out. Dynamic analysis using response spectrum approach for seismic accelerations can also easily be carried out for dams by using general purpose FEM packages such as ANSYS, ABAQUS etc., considering the foundation with the dam because foundation is not completely rigid as is assumed in the stability analysis given above. As per US Army Corps of Engineers (EM 1110-2-2200) dynamic analysis is necessary for the following conditions.

- Height of dam  $\geq 30$  m and PGA  $> 0.2$  g for maximum credible earthquake (MCE).
- Height of dam  $\leq 30$  m and PGA  $> 0.4$  g for MCE or any other condition considered desirable by experts.

Besides the above, requirements of IS 1893 and IS 6312 shall be followed and site specific seismic studies for dynamic analysis should be got approved by National Committee on Seismic Design Parameters as per guidelines of CWC. For detailed guidelines for seismic analysis of dams ICOLD Bulletin No. 46 and 72 may also be referred.



**Fig. 3.17** Principal stresses on the spillway section of Friant dam for normal condition. (Source: Concrete Dams by Sharma)

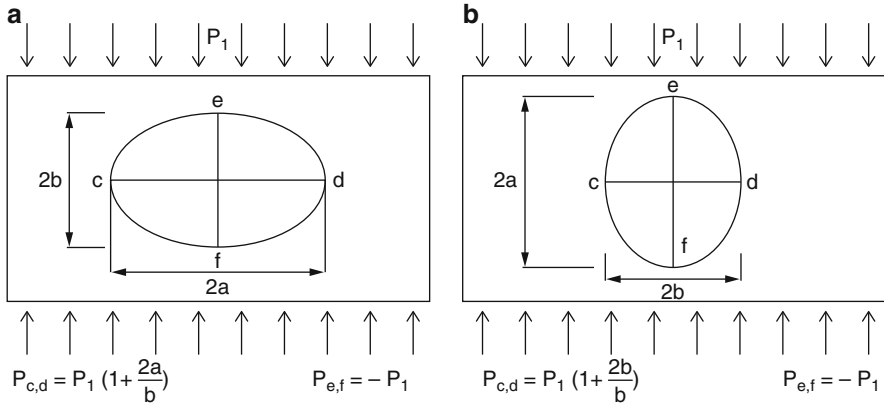
The principal stress diagram of a dam (Friant Dam in USA) in empty and full reservoir conditions are shown in Fig. 3.17.

The figure shows that

- (i) Stresses are maximum at U/S end (heel) when reservoir is empty.
- (ii) Stresses are maximum at D/S end (toe) when reservoir is full.
- (iii) The principal stresses are less inside the body of dam in both cases.
- (iv) The major principal stresses are vertical or slightly inclined to vertical and minor principal stresses perpendicular to major stress are quite small in magnitude.

In high dams the knowledge of internal stresses can help in deciding about zoning of concrete in dam. It means that concrete of different strength can be used in different zones as per stress requirements from economic consideration in high dams.

The magnitude of tensile stress developing on the faces will also give the idea whether cracks will develop. Necessary precautions can be taken and dam section may be suitably modified.



**Fig. 3.18** Stresses around openings.

### 3.5.8 Design of Opening and Galleries

After working out principal stresses at the internal points in the dam, the design of opening and galleries can be approximately worked out using the analysis of stresses around an opening in a thin semi-infinite plate subjected to uniaxial loading. This is applied in dams where it is observed that generally major principal stress is nearly vertical and minor principal stress is of small magnitude and so its impact on stress concentration around opening may be ignored.

Stresses around elliptical openings acted upon by uniaxial vertical stress are given in Fig. 3.18.

If the hole is circular i.e.  $a = b$  and is acted upon by an uniaxial vertical stress  $P_1$ , the stresses at the face of the opening and its distribution along horizontal and vertical axes will be as shown in Fig. 3.19.

It can be seen that stress concentration at face is local and reduces rapidly with distance moving away from the face.

The integrated area of  $p'_x$  at 'a' is zero but tension exists at the face and the total tensile force at 'a' is approximately given as

$$T = \frac{1}{2} \times 0.73 r \times P_1 = 0.36 r P_1$$

At point  $b$  the compressive stress is  $3P_1$  which sharply reduces to  $P_1$ . The area of additional stress can be worked out by integrating the area under the curve as below:

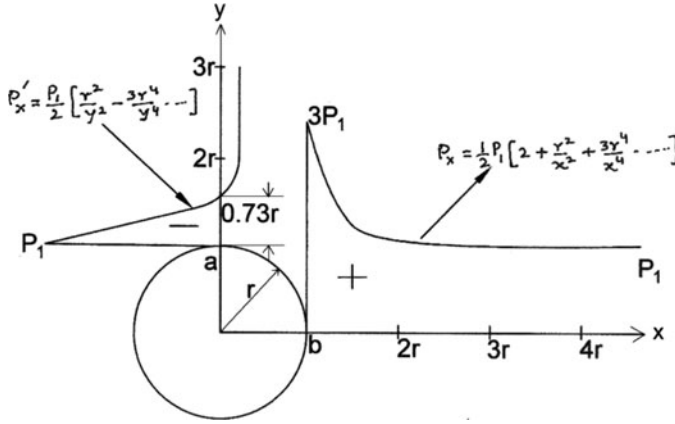


Fig. 3.19 Stress distribution around circular opening.

$$\begin{aligned}
 \int_r^{\infty} (P_x - P_1) dx &= \int_r^{\infty} \left[ \frac{1}{2} P_1 \left( 2 + \frac{r^2}{x^2} + \frac{3r^4}{x^4} \right) - P_1 \right] dx \text{ neglecting higher terms} \\
 &= \frac{1}{2} P_1 \int_r^{\infty} \left( \frac{r^2}{x^2} + \frac{3r^4}{x^4} \right) dx \\
 &= \frac{1}{2} P_1 \left( -\frac{r^2}{x} - \frac{r^4}{x^3} \right)_r^{\infty} = P_1 r
 \end{aligned}$$

Therefore, if  $P_1$  the major principal stress acting on the circular opening the tensile force at top and compressive force on the side can be worked out and the reinforcement for that force may be provided. Generally concrete can take the compressive stresses but the reinforcement required for tensile force is also provided on compression side.

In case both the principal stresses are of significant magnitude then stresses at the face of the opening can be worked out as shown in Fig. 3.20.

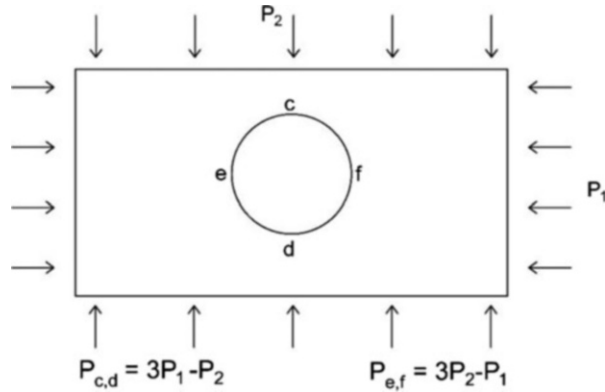
$$P_{c,d} = 3P_1 = P_2$$

$$P_{e,f} = 3P_2 = P_1$$

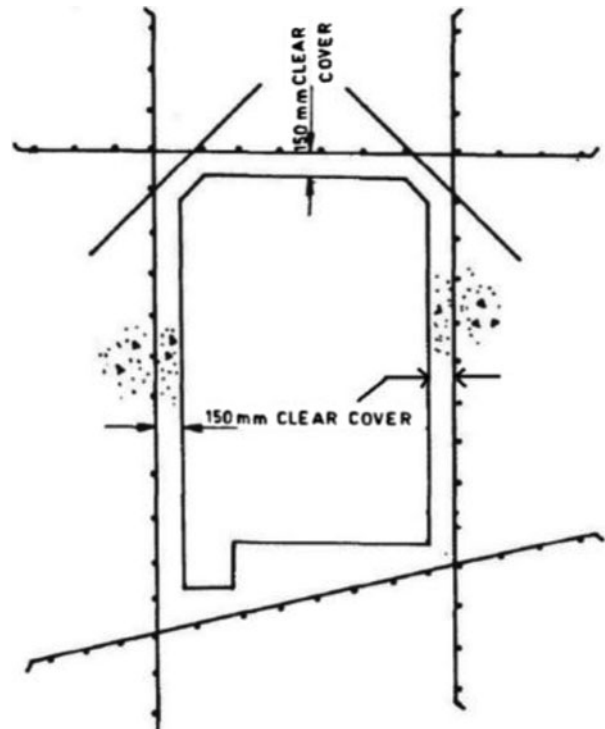
If  $P_1 > 3P_2$ ,  $-P_{e,f}$  will be  $-\vee$ , then stress at e, f will be tensile.

Stress distribution along the axes can be worked out as above. For detailed guidelines for the design of openings IS Code No. 12966 may be referred. Typical reinforcement pattern around an opening is shown in Fig. 3.21.

**Fig. 3.20** Stresses around circular opening subjected to stress from both directions.



**Fig. 3.21** Typical reinforcement around gallery.



**3.6 THERMAL STRESSES AND CONTROL**

Thermal stresses in dams are induced due to restraint to volume change caused by temperature change in concrete mass. When these stresses exceed the tensile stress of concrete, cracks develop. The restraint to volume change can be both external and internal. The external restraint is by the bond of concrete with foundation rock and

the internal restraint is due to change in volume in the concrete mass where volume change is not at the same rate. The range of temperature change is between peak internal temperature which is reached soon after placement and the annual ambient temperature. The placement temperature is largely dependent on the heat of hydration generated during mixing of concrete aggregates.

The cracks when develop due to thermal stresses affect the dam adversely in respect of water-tightness, durability, structural stability, appearance, and stress pattern in the dam body and so these are undesirable and measures should be taken to avoid them.

Thermal stresses in the dam due to restraint in volume change is given by

$$f = RCE_{ff}\Delta T$$

where  $f$  = tensile stress in concrete due to temperature drop  $\Delta T$  and  $R$  = restraint factor. It is the ratio of thermal stress due to actual restraint to that for full restraint.  $C$  = coefficient of thermal expansion of concrete. Usually the value of  $C$  is  $9 \times 10^{-6}$  per degree centigrade.

$$E_{ff} = \frac{E_c}{1 + 0.4\left(\frac{E_c}{E_r}\right)}$$

$E_c$  is sustained modulus of elasticity of concrete and  $E_r$  is modulus of elasticity of foundation rock.  $E_c$  value varies from 1.4 to  $2.1 \times 10^5$  kg/cm<sup>2</sup>.

$$\Delta T = t_p + t_r - t_f$$

where  $t_p$  = placement temperature of concrete mix,  $t_r$  = temperature rise of concrete due to heat of hydration of cement and  $t_f$  = final stable temperature of concrete mass.

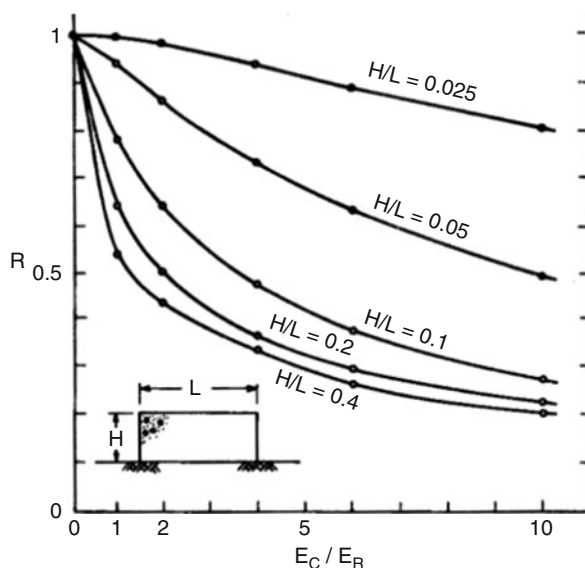
If there is no restraint  $R = 0, f = 0$  and if there is full restraint then  $R = 1$ , and thus  $f = C.E_{ff}\Delta T$ .

It can be seen that,  $C$  and  $E_c$  are the properties of concrete and  $E_r$  is the property of rock foundation and these being constant, the tensile stress and cracking will be the combined effect of restraint  $R$  and temperature drop  $\Delta T$ .

In  $\Delta T$ ,  $t_r$  the temperature rise due to heat of hydration of cement and is usually of the order of 20°C for concrete placed in 1.5 m lift at an interval of 72 hours. Measures can be taken to reduce it.

The restraint  $R$  is unity (i.e. 100%) for the concrete placed at the foundation rock and so tensile stress is maximum near foundation as the concrete is fully resisted to movement. The effect of this external restraint due to rock foundation reduces as the construction of block moves up. The studies carried out using FEM and shown in Fig. 3.22 give the effect of height-length ratio of a concrete block and  $E_c/E_r$  on the restraint factor. It can be seen from the figure that value of  $R$  reduces significantly after H/L value of 0.2 to 0.4 and the effect between  $H/L = 0.2$  to 0.4 is very small. If

**Fig. 3.22** Magnitude of degree of external restraint at bottom of concrete.  
(Source: Concrete Dam by Sharma)



the length of block is 20 m the effect of external restraint on tensile stress is small after a height of 8 m.

Internal restraint is caused because of different temperature change in different areas of concrete. It is induced when the surface and inside of freshly placed concrete has different temperature rise due to heat of hydration. It is also quite significant when new concrete is placed on the old concrete which has cooled to ambient temperature i.e. when the next lift is placed after a long time. This restraint on upper surface of lift can be avoided by quickly placing the next lift but this cannot be avoided for upstream and downstream faces because these faces are exposed to ambient temperature. Hence this type of restraint causes vertical cracks on upstream and downstream faces.

High thermal tensile stresses and resulting cracking can be controlled by taking measures to reducing the restraint and temperature drop  $\Delta T$ . Thus the crack control measures are (i) temperature control and (ii) reduction in restraint.

### 3.6.1 Temperature Control

Temperature drop  $\Delta T$  is equal to  $t_p + t_r - t_f$ . To reduce  $\Delta T$ , measures are taken to reduce  $t_p$  (placement temperature) and  $t_r$  (temperature rise due to heat of hydration of cement),  $t_f$  the ambient temperature is site specific and cannot be changed.  $\Delta T$  usually for gravity dam is limited to 22 °C. If it is exceeded, cracking may take place.

Placement temperature can be reduced by pre-cooling of aggregates. Different methods for cooling of concrete aggregates, coarse and fine such as spraying cold

water, air cooling, vacuum cooling etc., are often used. Ice flakes are added in mixing water to reduce placement temperature. These measures are necessary during summer times when atmospheric temperatures are high. The placement temperatures are usually kept around 10 to 12°C. The placement temperature can be worked out using the weight of various components of the concrete mix and their temperature by the formulae given in IS 14591. It is given below:

$$t_p = \frac{0.22(T_{ca} \cdot W_{ca} + T_{fa} \cdot W_{fa} + T_c \cdot W_c) + T_f \cdot W_f + T_w \cdot W_w - 79.6 W_i}{0.22(W_{ca} + W_{fa} + W_c) + W_f + W_w + W_i}$$

where  $T_{ca}$  = temperature of coarse aggregate at mixing time in °C,  $T_{fa}$  = temperature of fine aggregate °C,  $T_c$  = temperature of cement in °C,  $T_f$  = temperature in °C of force and absorbed moisture in aggregate (it is assumed same as of aggregate unless specified),  $T_w$  = temperature of mixing water in °C,  $W_{ca}$  = drymass of coarse aggregate in kg,  $W_{fa}$  = drymass of fine aggregate in kg,  $W_c$  = mass of cement in kg,  $W_f$  = mass of free and absorbed moisture in aggregate in kg,  $W_i$  = mass of ice in kg and  $W_w$  = mass of batch mixing water in kg.

With the use of this method of working out  $t_p$ , the pre-cooling requirements for achieving desired placement temperature can be worked out.

Post cooling is an effective method of crack control and is adopted in addition to pre-cooling in areas of high restraint near the foundation. It is done by circulating cold water through embedded pipes placed on top of each lift. It is costly and time consuming and so it is adopted when the construction joints are proposed to be grouted. Generally transverse joints are grouted when the dam height is more than 120 m or so.

Temperature rise due to heat of hydration of cement is about 20°C for concrete placed in 1.5 m lifts placed after 72 hours. This can be controlled by reducing the lift height as well as by reducing the quantity of cement. To achieve this the cement required is replaced by about 15% by pozzolana (generally fly ash). The exposure time of three days may be extended to five days or so for cooling of lift surface.

### 3.6.2 Restraint Control

The external restraint is maximum near the foundation. It is beneficial to keep the lift height less near foundation. It may be reduced to 0.75m to minimize rise in temperature. The internal restraint causes vertical cracks on upstream and downstream faces. So to avoid such cracks it is advisable to provide cracks during construction. Hence transverse construction joints at a spacing ranging from 12 to 18 m are generally provided. Proper water-stops to check seepage are provided in these joints.

Foundation restraint is of significance upto a height of  $0.4 L$  where  $L$  is length of block. In high dam the length of block is large approximately equal to the height of dam. In that case it becomes necessary to maintain a rigid temperature and volume change control upto a large height of dam. Hence it is considered desirable to provide a longitudinal joint in dams of large height say 150 m or more. The USBR practice is to provide a longitudinal joint when length of block is more than 60 m. It is provided in staggering way in adjacent blocks. These joints, in order to maintain structural intensity, are provided with shear keys and are grouted before filling the reservoir. In India, longitudinal joints are provided in Bhakra dam which is 226 m high. The observations on existing dams has created doubts about the efficiency of grouted longitudinal joints in respect of integral action of the block. The current practice is thus to omit the longitudinal joint and to adopt more intensive temperature and crack control measures. In Chamera dam (145 m high) no longitudinal joints are provided.

### 3.6.3 *Curing*

Curing is a measure of controlling cracking of exposed surfaces after removal of the form work due to drying shrinkage. The hairline shrinkage cracks if formed may later develop into more extensive cracking causing harm to the structure. Thus proper curing arrangement after removal of forms is necessary for crack prevention.

### 3.6.4 *Temperature Distribution inside Gravity Dam*

In order to plan and develop more precise temperature control and crack prevention measures in an important high concrete dam, temperature distribution studies in the mass concrete with different range of parameters should be carried out by solving basic heat conduction differential equation which is given below for one dimensional heat flow. For details the readers may refer 'Design of Gravity Dam', USBR.

$$\frac{\partial \theta}{\partial t} = h^2 \frac{\partial^2 \theta}{\partial x^2}$$

where  $\theta$  is temperature at any distance  $x$  from the surface at time  $t$  and  $h$  is the diffusion coefficient of concrete. A simple method for frequent use for determining the temperature distribution in mass concrete structures is given by Schmidt (refer USBR Engineering Monograph No. 34, Water Resources Publication, USBR).

### 3.7 DAMS ON PERVIOUS FOUNDATION

Generally concrete gravity dams are made on sound rock foundation. When sound rock is found to be buried under thick layer of alluvium, it is excavated out and filled with concrete. Several dams have been constructed by excavating 30 – 40 m deep alluvial strata above rock foundation. But sometimes due to site constraints and other requirements a concrete gravity storage or diversion dam of small and medium height may have to be constructed on pervious foundation. The complexity of foundation treatment and dam design depends on the properties of foundation material such as size, type, bearing capacity, sliding coefficient, permeability etc. It requires extensive laboratory and field investigations. The problems with such foundations are of erosion of foundation material, seepage under the structure and settlement. For the control of erosion, seepage, uplift etc. the following measures are suggested:

- Upstream concrete floor with cutoff at upstream end
- Downstream concrete floor with cutoff for scour.
- Drains with filter under the downstream floor.
- Cutoff at both upstream and downstream ends of overflow section with filter drains under the section.

The base width of the dam should be made wide enough with mild slopes to reduce the pressures on foundation to the permissible bearing capacity and to make the section stable against sliding. The long downstream floor increases the path of seepage and provides safe exit gradient at the end. The floor length should also be enough to satisfy energy dissipation requirements. A longitudinal joint between dam section and the downstream floor should be provided with waterstops. The dam section stability should be checked without downstream floor. The feasibility of providing a seepage gallery below the crest of overflow section should be examined as this will reduce the uplift. Adequate river bed protection should also be provided in the downstream of the concrete floor.

The cut off walls at the upstream and downstream end of concrete floor are provided to reduce seepage. These can be in the form of concrete walls, steel sheet pile, cement bound curtains etc. The depth of downstream cut off wall should be safe against scour.

A typical section incorporating the above measures for a dam on alluvial foundation is shown in Fig. 3.23. It is under consideration for a run-of-river power project with 35 m high (above river bed) diversion dam. The entire length of dam is over flow section of a length of about 90 m. It is resting on thick alluvial foundation over highly sheared rock mass. The permissible bearing capacity is taken as 50 t/m<sup>2</sup>.

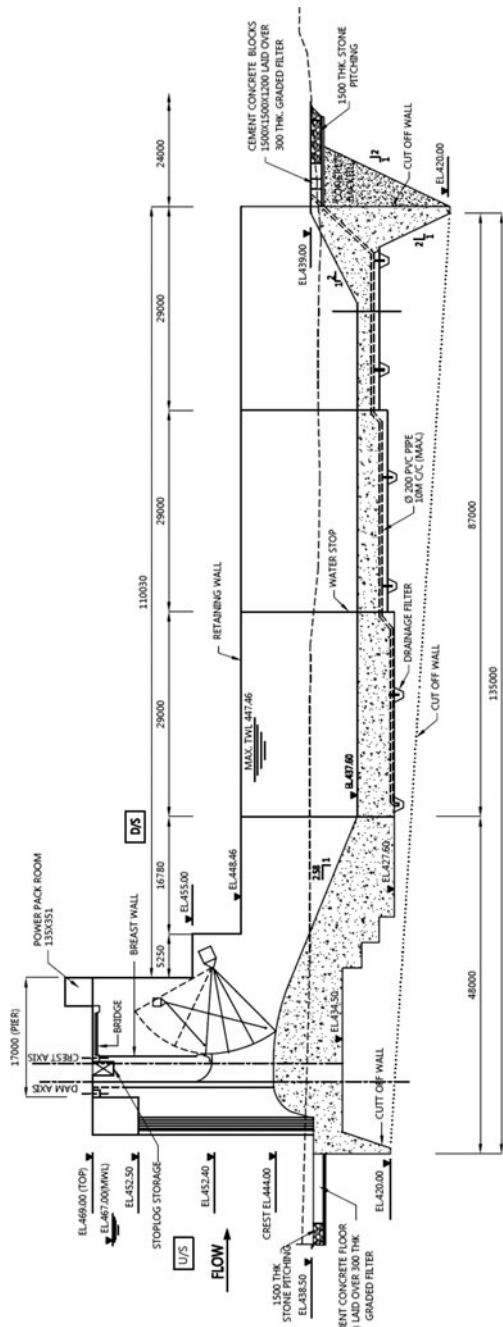


Fig. 3.23 Section of dam on alluvial foundation.

## APPENDIX

### *Computing Stability Analysis and Principal Stresses in a Dam Section*

The dam section is shown in Fig. 3.24. The forces and moments are computed and given in Table 3.3.

#### 1. Normal stresses in usual loading condition

##### (i) Stresses due to loads and forces

$\Sigma V$  = Resultant of horizontal forces

Items  $[2(A) + 4(i)]$

$$= -2149.875 - 109.375 = -2259.250 \text{ t}$$

$\Sigma W$  = Resultant of vertical forces

= items  $[1 + 2(B) + 4]$

$$= 5118 + 290 + 56.25 = (+) 5464.25 \text{ t}$$

$\Sigma M_{\text{toe}}$  = item  $[1 - 2(A) + 2(B) + 4]$

$$= 198772.5 - 47845.687 + 15961.25 + 2482.1$$

$$= 169810.163 \text{ t}$$

Distance of resultant from the toe

$$\bar{Y} = \frac{\Sigma M_{\text{toe}}}{\Sigma W} = \frac{169370.163}{5464.25} = 30.996 \text{ m}$$

$$\text{Eccentricity } (e) \bar{Y} - \frac{B}{2} = 30.996 - \frac{62}{2} = -0.004 \text{ m}$$

Resultant stresses at D/S and U/S base of dam

$$b_{zu} = \frac{\Sigma W}{B} \left[ 1 + \frac{6e}{B} \right] = \frac{5464.25}{6.2} \left[ 1 + \frac{6(-0.004)}{62} \right] = 881.167 \text{ t/m}^2$$

$$b_{zd} = \frac{\Sigma W}{B} \left[ 1 + \frac{6e}{B} \right] = \frac{5464.25}{6.2} \left[ 1 + \frac{6(-0.004)}{62} \right] = 881.167 \text{ t/m}^2$$

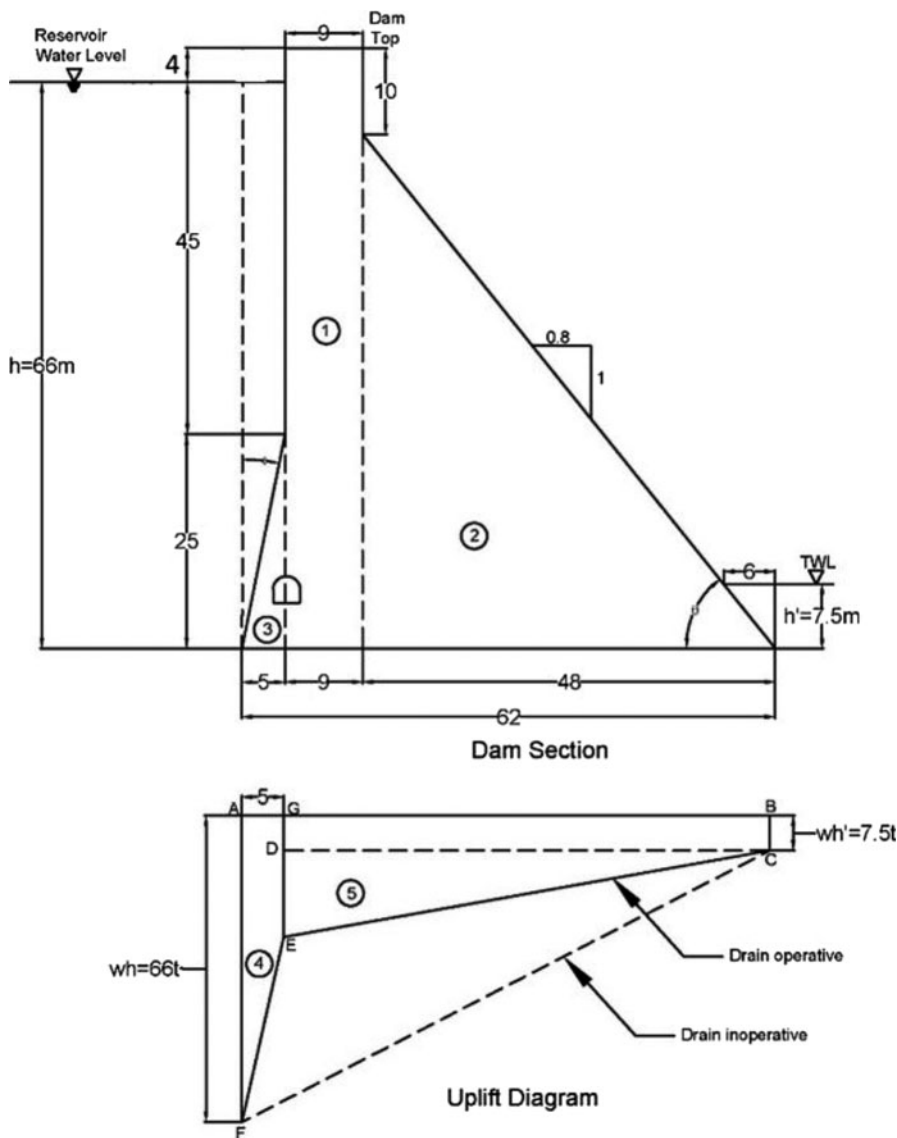


Fig. 3.24 Dam section.

**Table 3.3** Computation of loads, forces and moments

Sl. No.	Description of loads	Force computation (in tonnes)	Magnitude of force		Moment ARM (m)	Moment about toe (in Tonnes-m)
			Vertical downwards (+) upward (-)	Horizontal D/S (-ve) U/S (+ve)		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
	For unit length of dam section					
1.	Dead load ( $W_c$ ) Portion – 1 Portion – 2 Portion – 3	$9 \times 70 \times 2.4$ $0.5 \times 60 \times 48 \times 2.4$ $0.5 \times 5 \times 25 \times 2.4$	1512 3456 150 (+) 5118		$[48+9/2] = 52.5$ $2/3 \times 48 = 32$ $[49+9+5/3] = 58.7$	(+) 79380 (+) 110592 (+) 8800.5 (+) 198772.5
2.	Water loading					
A	Horizontal component hydrostatic pressure (i) Headwater U/S (V) (ii) Tail water D/S (V')	$0.5 \times 66^2 \times 1$ $0.5 \times 7.5^2 \times 1$		(-)2178.00 (+) 28.125 (-) 2149.875	$66/3 = 22$ $7.5/3 = 2.5$	(-) 47916 (+) 70.313 (-) 47845.68
B	Vertical component (i) Head water ( $W_w$ ) (ii) Tail water ( $W'_w$ )	$0.5 \times (66+41) \times 5 \times 1$ $0.5 \times 7.5 \times 6 \times 1$	267.5 22.5 (+)290.00		59.5 2.0	(+) 15916.25 (+) 45.00 (+) 15961.25
3.	Uplift force A. Drains operative: (i) Portion – 4 (ii) Portion – 5	$0.5 \times (66+27) \times 5$ $0.5 \times (27+7.5) \times 57$	(-) 232.5 (-)983.25 (-) 1215.75		59.849 33.87	(-) 13914.893 (-) 33302.68 (-) 47217.573
	B. Drains inoperative full trapezium	$0.5 \times (66+7.5) \times 62$	(-)2278.5		39.22	(-) 89362.77
4.	Silt load Excess horizontal pressure Excess vertical load	$0.35 \times 25 \times 25 \times 0.5$ $25 \times 5 \times 0.5 \times 0.9$	(+) 56.25 (+) 56.25	(-) 109.375 (-) 109.375	$25/3 = 8.33$ $57+10/3 = 60.33$	(-) 911.458 (+) 3393.562 (+) 2482.1
5.	Earthquake forces A. Hydrodynamic	$P_E = c\alpha w z = 0.73 \times$		(-) 346.289		(-)9412.71

(continued)

Table 3.3 (continued)

Sl. No.	Description of loads	Force computation (in tonnes)	Magnitude of force		Moment ARM (m)	Moment about toe (in Tonnes-m)
	forces U/S headwater [ $a_h = 0.15\text{ g}$ ] $F_E = 0.726 \times P_e \times h$ $M_E = 0.299 \times P_E \times h^2$	$0.15 \times 1 \times 66 = 7.227$ $F_E = 0.726 \times 7.223 \times 66 = 346.26$ $M_E = 0.299 \times 7.227 \times (66)^2$				
	D/S Tail water $f_D = 38.66^\circ$ $C = 0.45\text{ t/m}^2$	$P_E = 0.45 \times 0.15 \times 1 \times 7.5 = 0.5063$ $F_E = 0.726 \times 0.5063 \times 7.5 = 2.757$ $M_E = 0.239 \times 0.563 \times (7.5)^2$		(-) 2.757		(-) 8.515 (-) 9421.275
	B. Inertial horizontal force due to earthquake Portion – 1 Portion – 2 Portion-3	$0.15 \times 1512$ $0.15 \times 3456$ $0.15 \times 150$		(-) 226.8 (-) 518.4 (-) 22.5 (-) 767.7	$70/2 = 35$ $60/3 = 20$ $25/3 = 8.33$	(-) 7938 (-) 10368 (-) 187.425 (-) 18493.423

(ii) Stresses due to uplift pressure diagram

(a) Drains operative:

$\Sigma W = \text{Total uplift forces} = [\text{item } 3A] = -1215.75\text{ t}$   
 $\Sigma M_{\text{toe}} = \text{item } B(A) = -47217.573\text{ t}$   
 $\bar{Y} = \frac{-47217.573}{-1215.75} = 38.84\text{ m}$   
 $e = \bar{Y} - \frac{B}{2} = \left[ -38.84 - \frac{62}{2} \right] = 7.84\text{ m}$   
 $p_{zu} = \frac{-1215.75}{62} \left[ 1 + \frac{6 \times 7.84}{62} \right] = -34.486\text{ t/m}^2$   
 $p_{zD} = \frac{-1215.75}{62} \left[ 1 + \frac{6 \times 7.84}{62} \right] = -4.731\text{ t/m}^2$

(b) Drains inoperative

$$\Sigma W = \text{item 3}[B] = -2278.5 \text{ t}$$

$$\Sigma M_{\text{toe}} = \text{item 3}[B] = -89362.77 \text{ t.m}$$

$$\bar{Y} = \frac{(-) 89362.77}{(-) 2278.5} = 39.22 \text{ m}$$

$$e - \bar{Y} - \frac{B}{2} = [39.22 - 31] = 8.22 \text{ m}$$

$$p_{zU} = \frac{-2278.5}{62} \left[ 1 + \frac{6 \times 8.22}{62} \right] = -65.88 \text{ t/m}^2$$

$$p_{zD} = \frac{-2278.5}{62} \left[ 1 - \frac{6 \times 8.22}{62} \right] = -7.516 \text{ t/m}^2$$

(iii) Resultant normal vertical stresses including uplift

(a) Drains operative

$$p_{zU} = 88.1 - 34.486 = 53.614 \text{ t/m}^2$$

$$p_{zD} = 88.167 - 7.516 = 80.651 \text{ t/m}^2$$

(b) Drains inoperative

$$p_{zU} = 88.1 - 65.98 = 22.12 \text{ t/m}^2$$

$$p_{zD} = 88.167 - 7.516 = 80.651 \text{ t/m}^2$$

2. Principal stress at faces (Maximum values)

(i) U/S Force: Calculations indicate a normal stress of 22.12 t/m<sup>2</sup> at heel of the dam (under drain inoperative condition). The inclined (or minor) principal stress parallel to upstream face will be higher and needs to be checked.

$$\begin{aligned} \sigma_{pzU} &= p_{zU} \sec^2 \phi_U - (p + p_E) \tan^2 \phi_U \\ &= 22.12 \times \sec^2(11.31^\circ) - 65.98 \tan^2(11.31^\circ) \\ &= 22.12 \times 1.04 - 65.98 \times 0.04 = 20.37 \text{ t/m}^2 \end{aligned}$$

- (ii) D/S force: Maximum value of inclined (or major principal stress) will occur on the downstream face and needs to be checked.

$$\begin{aligned}
 \sigma_{pzU} &= p_{zD} \sec^2 \phi_D - (p' + p'_E) \tan^2 \phi_D \\
 &= 88.167 \times \sec^2(38.66^\circ) - 7.516 \tan^2(38.66^\circ) \\
 &= 144.59 \times 4.81 = 139.78 \text{ t/m}^2
 \end{aligned}$$

- (iii) Maximum shear stress

Maximum shear stress occur at D/S end of base and works out as

$$\begin{aligned}
 T_{yzD} &= T_{xyD} = (p_{zD} - p' + p'_E) \tan \phi_D \\
 &= (88.167 - 7.516) \tan(38.66^\circ) = 64.52 \text{ t/m}^2
 \end{aligned}$$

The difference between usual and unusual combinations is that in the latter case the stability of the dam is tested for maximum design reservoir elevation instead of normal design reservoir elevation in the usual case. The stability calculations are otherwise identical. In this case both conditions are same.

- (iv) Stresses without uplift

$$\Sigma W = 5118 + 290 + 56.25 = 5464.25 \text{ t}$$

$$\Sigma V = \text{items } [2(A) + 4(i) + 5A + 5B]$$

$$= - (2149.875 + 109.375 + 346.289 + 2.757) = 2608.296 \text{ t}$$

$$\Sigma M_{\text{toe}} =$$

$$= 198772.5 - 47845.687 + 15961.25 + 2482.1 - 9421.275 - 18493.415$$

$$= 141455.463 \text{ t/m}$$

$$\bar{Y} = \frac{\Sigma M_{\text{toe}}}{\Sigma W} = \frac{141455.463}{5464.25} = 25.887 \text{ m}$$

$$e = \bar{Y} - \frac{B}{2} = 25.887 - \frac{62}{2} = -5.113 \text{ m}$$

$$p_{zU} = \frac{\Sigma W}{B} \left[ 1 + \frac{6e}{B} \right] = \frac{5464.25}{62} \left[ 1 + \frac{6 \times (-5.114)}{62} \right] = 44.52 \text{ t/m}^2$$

$$p_{zD} = \frac{\Sigma W}{B} \left[ 1 - \frac{6e}{B} \right] = \frac{5464.25}{62} \left[ 1 - \frac{6 \times (-5.113)}{62} \right] = 131.74 \text{ t/m}^2$$

## (v) Resultant stresses including uplift

## (a) Drains operative:

$$p_{zU} = 44.52 - 34.486 = 10.034 \text{ t/m}^2$$

$$p_{zD} = 131.74 - 4.731 = 127.009 \text{ t/m}^2$$

## (b) Drains inoperative:

$$p_{eU} = 44.52 - 65.98 = -21.46 \text{ t/m}^2$$

$$p_{zD} = 131.74 - 7.516 = (+) 124.224 \text{ t/m}^2$$

It may be noted that no increase in uplift pressure is allowed for due to hydrodynamic pressure because of transient and oscillatory nature of earthquake forces. The same values of uplift pressure as obtained for hydrostatic pressure have, therefore, been used for working out the stresses.

## (vi) Principal stress at dam faces

$$\begin{aligned}\sigma_{pzU} &= p_{zU} \sec^2 \phi_U - (p + p_E) \tan^2 \phi_U \\ &= (-) 21.460 \sec^2 (11.31^\circ) - (65.98 + 7.227) \tan^2 (11.31^\circ) \\ &= (-) 21.460 \times 1.04 - 73.207 \times 0.04 = (-) 25.25 \text{ t/m}^2 \\ \sigma_{pzD} &= p_{zU} \sec^2 \phi_D - (p' - p'_E) \tan^2 \phi \\ &= 131.74 \sec^2 (38.66^\circ) - (7.516 - 0.5063) \tan^2 (38.66^\circ) \\ &= 131.74 \times 1.64 - 7.01 \times 0.64 = 211.57 \text{ t/m}^2\end{aligned}$$

## 3. Shear stress at toe

$$\begin{aligned}T_{yzD} &= (p_{zD} - p' + p'_E) \tan \phi_D \\ &= (131.74 - 7.01) \tan (38.66^\circ) \\ &= 124.73 \times 0.8 = 99.78 \text{ t/m}^2\end{aligned}$$

## 4. Stability against sliding

Stability against sliding is evaluated by computing the value of shear friction factor given by

$$Q = \left[ \frac{C.A + (\sum W - U) \tan \phi}{\sum V} \right]$$

The following values are assumed:

- (a) Average value of cohesion along concrete-rock contact = 100 t/m<sup>2</sup>
- (b) Coefficient of static friction of the foundation (or  $\tan \phi$ ) = 0.7
- (i) Usual loading combination
  - (a) Drains operative:

$$Q = \left[ \frac{100 \times 62 \times 1 + (5464.25 - 1215.75) \times 0.7}{2259.25} \right]$$

$$= 4.06$$

- (b) Drains inoperative

$$Q = \left[ \frac{100 \times 62 \times 1 + (5464.25 - 2278.5) \times 0.7}{2259.25} \right]$$

$$= 3.73$$

- (ii) Extreme loading combination

- (a) Drains operative:

$$Q = \left[ \frac{100 \times 62 \times 1 + (5464.25 - 1215.75) \times 0.7}{2608.296} \right]$$

$$= 3.517$$

- (b) Drains inoperative

$$Q = \left[ \frac{100 \times 62 \times 1 + (5464.25 - 2278.75) \times 0.7}{2608.296} \right]$$

$$= 3.232$$

It can be inferred that the computed values are within allowable limits as specified for different loading combinations. The dam section is therefore safe against permissible stresses and sliding.

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## Chapter 4

# Roller Compacted Concrete Dams



### 4.1 GENERAL

The design concept and construction technique of concrete gravity dam, evolved after 1930, comprised of making the dam in a series of blocks of width generally varying from 15 to 18 m separated from each other by contraction joints provided with waterstops, and placing concrete in each block in lifts of 1 to 1.5 m in thickness and putting the next lift after 72 hrs in order to reduce cracking because of temperature rise due to heat of hydration. The requirement of cement depending on strength requirement of concrete varies from 250 to 300 kg per cum of concrete. To manage heat of hydration further arrangements for precooling of aggregates are also required. These measures made the dam construction costly and time consuming. The concept of roller compacted cement (RCC) which is a lean concrete (less cementitious material) and can be placed and compacted by using earth moving equipments which are generally used in embankment dam construction, was used for construction of concrete gravity dam for the first time on Shimajigawa Dam in Japan in 1978. Because of the advantages of use of RCC in concrete dam in reducing construction time and cost, this gained acceptance the world over and since 1980s a large number of RCC dams of height ranging from 100 to about 200 m have been constructed in many countries such as Japan, USA, England, China, South Africa etc. In India, in the end of 20<sup>th</sup> century RCC dams were investigated, planned and designed for Jamrani dam (150 m high) on river Gola and Koteswar Dam (103.5 m) on river Bhagirathi downstream of Tehri Dam both in Uttarakhand but could not be constructed. But in the beginning of 21<sup>st</sup> century India slowly started adopting this technology of dam construction and a small height (12 m high) RCC dam was first completed in 2003 as a part of Ghatghar pumped storage scheme in Maharashtra. Since then a couple of such dams have been constructed. This technology has a better future in India.

## 4.2 RCC DAMS

Roller compacted concrete is a low cement content and low workability concrete. It is able to support heavy equipment during transport, placement and compaction. It is transported in large trucks, spreaded by dozers and compacted by vibratory rollers. Hence it has been used in structures of mass concrete and so widely used in construction of concrete gravity dams. The gravity dams made of RCC are called RCC dams.

The main advantages of RCC dams over conventional concrete gravity dams are as below:

1. Low cement content reduces heat of hydration, thereby resulting in minimizing the measures required to check thermal cracking in concrete dams.
2. Speedy construction due to easy transportation and placement.
3. Reduction in transverse joints, efforts in cleaning and preparation of lift joints and requirement of form work on dam faces.
4. Due to smaller construction period the cost on river diversion during construction is reduced.

The above advantages result in low cost and speedy construction of an RCC dam.

## 4.3 DESIGN APPROACHES IN RCC DAMS

RCC dams were developed simultaneously in different countries like Japan, USA, South Africa, Australia and China. Each country has slightly different approach to design and construction and hence the advantages of each approach in respect of cost and time are also varying. The concrete mix design to suit the construction technique is also different. The different concepts of design and construction of RCC dams based on type of mix are as below:

- (i) Lean RCC Dams: The lean RCC dams are those which are made of low cementitious material content say upto  $100 \text{ kg/m}^3$ . The pozzolana content may vary between 0 to 40 percent. The concrete is placed generally in layers of thickness of approximately 300 mm. Because of low cementitious content measures are required to check seepage through horizontal construction joints. To check seepage bedding mixes are used between lifts and also near upstream face.
- (ii) Medium paste RCC Dams: In these dams the cementitious content is between 100 and  $150 \text{ kg/m}^3$  with about 40 to 60 percent of pozzolana. The concrete is placed in thin layers of thickness of about 300 mm.
- (iii) High Paste RCC Dams: In these dams the cementitious content is usually above  $150 \text{ kg/m}^3$ . The pozzolana content is generally 70 to 80%. The high paste concrete has a high density, the pozzolana content is high and cement content is low and so the heat of hydration is low. In high paste concrete RCC dams, it has

been possible to get improved joint properties and good bonding between the two lifts placed after a gap of three days. This avoided the need of surface treatment and bedding mix. This concrete is placed in layers of thickness of 300 mm.

The following table gives the idea of the properties of different concrete mixes which are achieved in in-situ testing.

Sl. No.	Properties (at 91 days)	Unit	Type of concrete		
			Lean RCC	Medium paste RCC	High paste RCC
1.	Density	%	95–98	96–98.5	98.5–99.5
2.	Permeability	m/sec	$10^{-4}$ – $10^{-8}$	$10^{-6}$ – $10^{-9}$	$10^{-9}$
3.	Shear strength at joints	MPa	0.5–1.0	1.0–2.0	2.2
4.	Compressive strength	MPa	8–10	12–20	20–40

Thus on the basis of mix type, RCC dams described above are of three types.

Sl. No.	RCC dam type	Cementitious content (cement + pozzolana)
1.	Lean RCC	Less than 100 kg/m <sup>3</sup> (less than 40% pozzolana)
2.	Medium paste RCC	100–150 kg/m <sup>3</sup> (40–60% Pozzolana)
3.	High paste RCC	150 or more kg/m <sup>3</sup> (70–80% Pozzolana)

The choice between the above three types depends on the requirement. Because of less cement content high density and better bonding properties the high paste and medium paste RCC dams are preferred for medium and high storage dams. The lean RCC may be used for low height dams and the dams which may store water for short period such as diversion dams or dams for flood control. The present trend is to construct high paste content RCC dams.

The construction approach rationalized by Japan to achieve speed and economy was different from the approach adopted by USA, UK and other countries. In Japan RCC is generally used in the inner part of dam body and conventional mass concrete and structural concrete is used in other parts of dam. The upstream and downstream faces are made of conventional concrete. Transverse joints are provided in the same manner as in conventional concrete dam. The bedding mix is also placed between two lifts. Thus, the RCC dams are similar in design, structural integrity and appearance to the conventional concrete gravity dams. In such RCC dams the speed of construction is slow and economy in cost is also limited.

In USA, UK and other countries the approaches for the construction of RCC dams are different than Japan. In these approaches the transverse joints are either eliminated or provided at larger distances say 40–60 m and dam faces form work is reduced to minimal requirement. The layout of works and design is adjusted in each specific case such that there is maximum saving in time and cost and the structural integrity and performance is same as that of a conventional concrete gravity dam.

## 4.4 CONSIDERATIONS IN LAYOUT

The RCC dams are basically concrete gravity dams and the only difference is in type of concrete and method of transport, placement and compaction. Hence the requirements of site, layout and design are basically same as in case of a conventional concrete gravity dam. The site selection requirements regarding topography and geology are also the same. The type and properties of rock foundation requirements are also same in both these types of concrete dams.

The RCC dams like a conventional concrete gravity dam also comprises over flow (spillway) and non-overflow sections. The RCC dam also has outlets and intake structure, galleries and other openings, sluices, gate shafts etc. which are generally constructed in conventional concrete with reinforcement. This prevents speedy construction of RCC. Therefore, RCC dam construction discourages provision of many outlets, galleries, vertical shafts etc. So in finalizing the layout of RCC dam such structures should be minimum and located in a group at one side. If conditions allow, the outlets, intake structure, diversion structure should be installed along the abutments of the dam. If feasible, attempt shall be made to avoid horizontal alignment of outlet works in the dam body so that equipment for RCC may move freely. Foundation gallery necessarily has to be provided for curtain grouting and drainage to reduce uplift. It shall be provided sufficiently away from upstream face to allow free movement of equipment between upstream face and gallery.

## 4.5 DESIGN CONSIDERATIONS

The conventional concrete gravity dams are usually constructed and designed as independent blocks and the stability analysis is carried out as two-dimensional structure. The RCC dam in narrow valley is generally a three-dimensional structure and is designed accordingly. But in wide valley the RCC dam with transverse joints also behaves as two-dimensional structure and is analyzed in the same way as the conventional concrete gravity dam.

The two-dimensional RCC dam shall thus satisfy the criterion of safety of conventional concrete gravity dam for permissible stresses in concrete (both compressive and tensile) and in rock foundation, overturning and sliding along foundation contact with dam base, along weak joint matrix in foundation and along any plane or horizontal joint between two layers. The stability analysis procedure and computation for safety factors are the same as discussed in the chapter of gravity dams. The loads which are considered for the stability analysis and their estimation and the various loading combinations to be analyzed are also the same as for a gravity dam. In RCC dams a lower value of concrete density is generally used say  $2.2 \text{ T/m}^3$ . The shear strength due to cohesion and friction between two layers of RCC or between the dam base and foundation needs to be evaluated judiciously using engineering judgement by the designer because these depend on the mix to be used

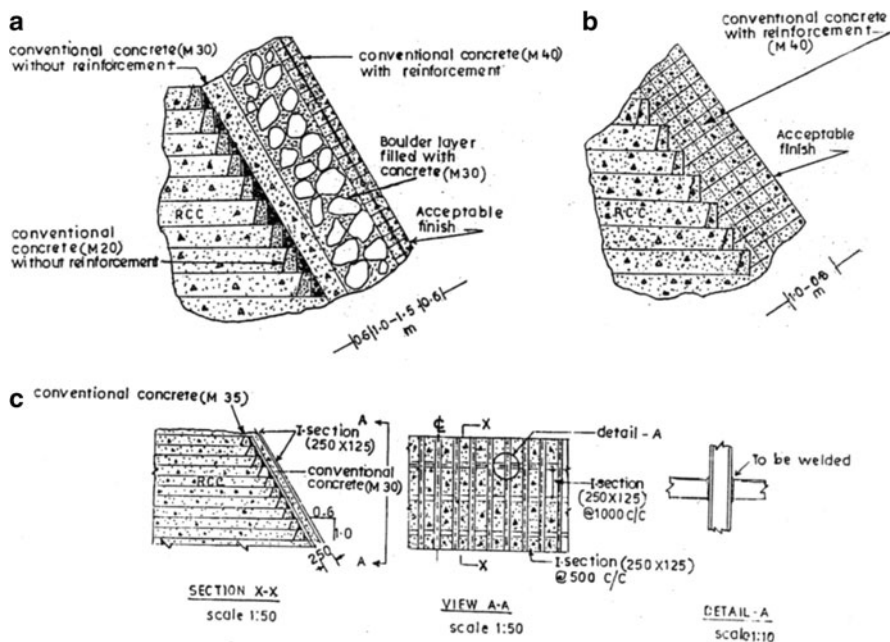
and the construction technique and control to be exercised in construction. These factors vary largely between laboratory tests and in-situ test. The in-situ tests results of the test section constructed at site may be considered more realistic. If the test results give a value of 1.4 MPa, the effective value for design for cohesion between two layers shall be taken 75 percent of the test value to account for the bonding efficiency and in case of dam base and rock it may be reduced by 50%. The friction value ( $\tan \phi$ ) is generally taken as unity or less corresponding to  $\phi = 40^\circ$  for a plane between two layers and 0.7 to 0.8 for contact plane between dam base and rock foundation. In case of mixes with low paste content, effective bonding may not be achieved between the two lifts, the sliding friction factor for safety against sliding shall be worked out ignoring the cohesion factor. As a result of stability analysis a downstream slope of 1:07 to 1:1 (V : H) are found adequate. In low and medium height dams which are constructed with any type of mix a flatter slope shall be adopted.

The spillway in RCC dams to pass excess discharge is the same as it is the part of a conventional gravity dam. Hence the hydraulic design of a spillway and the energy dissipation in the downstream in RCC dam is similar to the conventional concrete gravity dam. It has an ogee crest and the stilling basin or bucket at the downstream end. The surface of the crest and the top of the stilling basin floor or bucket needs to be smooth to avoid cavitation and strong enough to resist abrasion due to high velocity flow which generally carries large quantity of sediment during floods. The designer for an RCC dam spillway has to make provision for smoothness of the surface and the resistance to abrasion. It is generally achieved by providing conventional rich concrete with reinforcement at the crest, sloping surface of spillway and the floor of stilling basin or bucket. The thickness of such rich concrete may vary from 0.6 to 1.0 m. The upstream face of RCC dam should also be provided with say 1.0 m thick conventional concrete to ensure water tightness of dam. A typical section of spillway profile is shown in Fig. 4.1.

In case of medium or low height RCC dams ogee crest and the slope are shaped after RCC placement with conventional concrete. Loose RCC is removed to provide sound surface for placing conventional concrete in required shape. The conventional concrete is generally reinforced. A typical section is shown in Fig. 4.2.

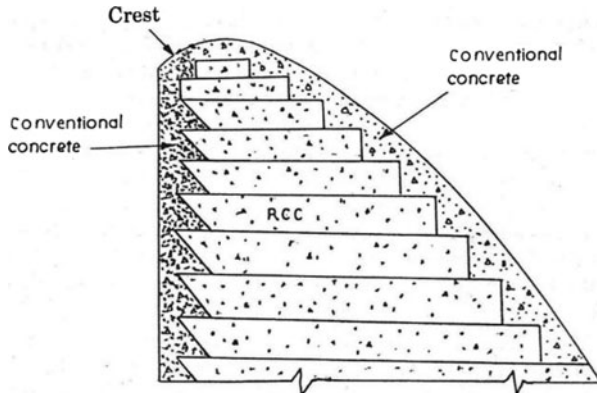
In case of ungated spillway where overflow depth is small the energy dissipation may be achieved by stepping the spillway profile. The slopes are formed during the placement of RCC and these are faced with conventional concrete to provide resistance to erosion. The steps may be 0.5 to 1.0 m high.

Thermal stresses and cracking of concrete in RCC dams is expected less than in case of conventional concrete because heat of hydration is less due to use of less quantity of cement and more of pozzolana. In view of this fact the transverse joints in RCC dams are either eliminated or placed at large distances. Thermal studies to work out the thermal stresses in the dam are quite complicated and are carried out in high dams with the help of FEM packages. Thermal stresses largely depend on placement temperature of concrete which can be controlled by cooling the aggregates. It is achieved by spraying cool water on aggregate stockpiles during the day and using chilled water for concrete mix. Since the water requirement in RCC mix is quite less,



**Fig. 4.1** Details of typical spillway profiles for a high RCC dam.

**Fig. 4.2** Typical design of spillway for small and medium RCC dam.



the effect of using chilled water for mix is quite minimal. Placing concrete in the night is quite effective in reducing thermal stresses. Sometimes cooling of exposed surface with water after placement is quite effective. Post-cooling of concrete as is done in conventional concrete dams has not been used in any of the RCC dams constructed so far.

In transverse joints provided in RCC dam, waterstops shall be placed to check seepage. Generally PVC waterstops are used. These are placed as close to the upstream face as possible to minimize interference with RCC placement. The

waterstops shall be installed in conventional concrete. The PVC waterstops used in RCC dams is of same specification as used in conventional concrete dams and relevant IS Code may be referred.

## 4.6 RCC MATERIALS

The materials required for RCC are very much the same as those required for conventional mass concrete. These are aggregates coarse and fine, cementitious material (cement plus pozzolana), admixtures and water. These shall meet the quality specifications as per relevant IS codes.

- (a) **Aggregates:** The grading and quality of aggregates affect the properties of the mix and the hardened RCC. Wide range of grading and quality of aggregates have been used in RCC varying from fully processed aggregates to unwashed pit-run aggregates.

Coarse aggregates are generally natural gravel or crushed rock or a combination of both properly blended. Fine aggregate is natural sand or natural sand mixed with crushed sand to produce well graded sand. The sand shall be free from elongated particles, clay and organic material.

The quality of aggregates shall be tested as per standards laid down in the IS Codes for strength, durability, soundness, mineral compaction etc. Mineral composition test will give the idea regarding the potential of alkali silica reaction. This activity in RCC is greatly reduced due to use of large proportion of pozzolana. The aggregate should be chemically inert. It shall be strong enough to withstand the impact of vibratory roller. The gradation of both coarse and fine aggregates should be such that these together form a compact mass with minimum voids. The two comprise about 85–90 percent of total volume of mix. Since the layer thickness in RCC dams is generally limited to 300 mm, the maximum size of coarse aggregate is also limited to 75 mm. Thus, the coarse aggregate grading should vary from 4.75 mm to 75 mm. The fines primarily fill the voids in coarse aggregate. A well graded mass of coarse and fine aggregates will require minimum cementitious paste for the required strength and density of concrete.

- (b) **Cementitious material:** It is a mix of cement and pozzolana.

- (i) *Cement:* Any type of cement of normal strength, low heat, moderately sulphate resistant and suitable for exposure conditions can be used in RCC. Slag cement or cement blended with pozzolana of specified quantity and quality of flyash is recommended for RCC dams. The low alkali percentage in cement is an optional requirement because of use of large amount of pozzolana. However it should be ensured that total quantity of alkali is cement, pozzolana and the aggregate does not exceed the specified limit.

(ii) *Pozzolana*: Pozzolana is a siliceous inert material. But when used in concrete it slowly reacts with  $\text{Ca(OH)}_2$  produced during hydration of cement and forms calcium silicate hydrate. This reaction takes place at normal temperature and takes years to complete. The types of pozzolana which can be used in RCC are:

1. Fly ash produced from burning coal (Bye product of thermal power stations)
2. Silica fume
3. Raw or calcined natural pozzolana such as volcanic ashes

**Fly ash:** As it is available in large quantities as a waste product from thermal power stations it can economically be used in RCC dams. It is an inert material and does not have the cementing property of its own but after reacting with free lime produced in the hydration of cement it contributes to long term strength of concrete and its water tightness. The spherical shape of its particles contributes to the workability of concrete. The physical and chemical properties of flyash required for use in concrete are specified in IS 3812 (Part-1).

Depending on the type of coal burnt in the power plant the quality of flyash collected at the plants is varying. Hence it is necessary to test the quality as specified in IS 3812. It is also essential to ensure that total quantity of flyash required for a RCC dam is available of uniform quality. Special attention should also be given to the percentage of lime in the flyash because it will decide on the quality of cement for the RCC mix.

**Silica Fume:** It is a bye-product of silicon and ferro-silicon industry. These are extremely fine (about 100 times finer than cement) particles and are highly reactive with  $\text{Ca(OH)}_2$  produced during hydration of cement. It improves the strength of concrete as well as permeability and resistance to corrosion. It is an expensive material and so not used in RCC dams where requirement of pozzolana is large. It is used in small quantities with cement to produce corrosive resistant concrete which is placed on the surfaces of outlets and spillways which are exposed to high velocity sediment laden flows.

**Raw or Calcinad Natural Pozzolana:** These are volcanic ashes, shales, tuffs and burnt clays. These, when finely crushed, chemically react to  $\text{Ca(OH)}_2$  in presence of moisture to form compounds which have cementitious properties. Each of these materials has its own physical and chemical properties, so before using any of such materials as pozzolana in RCC, it shall be tested to meet the requirements as per ASTM 618. The following tests must be carried out on the samples (atleast one sample for a quantity of 400 T).

- Fineness
- Moisture content
- Specific gravity
- Soundness and
- Loss on ignition.

The choice of material for pozzolana should depend on its meeting the minimum requirements of physical and chemical properties as well as sufficiency of the source regarding continuous satisfactory supply needed for the structure. Generally, the world over for RCC dams fine flyash from coal burning with low alkali and calcium content has been widely used.

- (c) **Water:** The quality requirement of water to be used in RCC is the same as for conventional concrete. However, the water to be used in RCC mix should not have high turbidity, objectionable organic matter, salts and other impurities. The ice to be used for chilling the mixing water if required shall also be made of the water of good quality meeting the above specification.
- (d) **Admixtures:** Admixtures are chemical compounds which are added in RCC mix for improving or imparting certain property. These are classified as
  - Water reducers
  - Set retarders and
  - Air entraining agents

The use of water reducing agents in RCC mix is essentially made to increase the workability at a given water cement ratio. It reduces compaction efforts. It results in economy because less cement will be used for same workability, required for particular strength. The choice of the water reducing agent and its dose shall be determined from the laboratory tests as well as the trials on test section.

Some chemicals are used as retarders of setting time of RCC. It is used when there are difficult conditions of transportation and delay in placement of concrete. The use of set-retarders permits more time to employ full compaction effort. Its use also contributes in better binding between the two layers as the setting of first layer is delayed.

The use of an air-entraining agent increases workability but reduces density and strength. So the choice of chemical and its dose shall be decided after proper tests in adequate numbers. Its use in a RCC has been limited. It has been found useful in increasing freezing and thawing resistance of RCC.

## 4.7 RCC MIX DESIGN

The mix design is primarily the proportioning of the ingredients of the RCC to meet the workability and strength requirements. RCC is known as no slump concrete which is to be compacted by vibratory rollers. The proportioning of ingredient should be such that in a RCC dam no elaborate precooling and post-cool arrangements are required. Hence quantity of pozzolana should be large and cement should be less in cementitious content of the mix. Before starting the exercise of working out the proportioning of the ingredients in RCC mix the parameters such as strength, workability and durability should be predetermined.

**Strength:** The strength required to be achieved at specified age has to be decided by the designer. The water cementitious ratio has to be worked out for this predetermined strength.

**Workability:** The RCC is stiff concrete for the use of earth moving equipment during construction. The workability of the mix can be determined only by a consistency meter (like Vebe Apparatus) and not by usual method of measuring slump. The vibratory effort for compaction required for achieving maximum specified density is also a measure of consistency.

**Durability:** The durability of RCC is related to the permeability. If the voids are more, then the permeability is more. A durable concrete should have less voids.

The proportioning of ingredients of RCC mix for specific strength, workability and durability is decided by testing the sample mixes in the laboratory by varying water content, sand-aggregate ratio, cement-pozzolana ratio, and air entraining agent if to be used to achieve desired workability for the placing conditions. After optimizing the proportion for workability, the  $W/(C+P)$  (water and cementitious content ratio) ratio is varied for required design strength and durability.

The cement and pozzolana ratio shall also be varied in the cementitious paste because high percentage of pozzolana will reduce the cost and thermal heat rise but beyond a ratio of unity the pozzolana will not contribute to strength and the strength development rate is slowed down though long time age strength is not affected. Hence balancing the strength versus heat rise is the part of cementitious material proportioning process. Hence for laboratory trials of mix proportioning the following shall be initially decided:

- (i) The desired strength and workability (Vebe consistency time of 15 to 20 seconds).
- (ii) The type of paste (low, medium or high) to be designed depending on the type selected; an initial cement plus pozzolana (C+P) content shall be decided.
- (iii) Maximum volume of coarse aggregate and the maximum size of that aggregate. Generally the maximum size used is 50 mm to 75 mm.
- (iv) Select the cement to pozzolana ratio. Initially it may be taken as unity.
- (v) Select initial content of water:  $W / (C+P)$  ratio may be initially taken as 0.7.
- (vi) Select the sand content: It usually ranges from about 30 to 50 percent of the combined gradation. It is generally more than typically used in conventional concrete. It is to avoid segregation and prevent aggregate breakage during compaction.

Then during trials water content shall be varied to decide water content for required workability. Sand content shall also be varied to see the effect on consistency and segregation. Then pozzolana ratio in cementitious content is varied. Then total paste content (C+P) is varied to see effect of  $W/(C+P)$  on compressive strength. From the results of all these parametric trials an economical and workable mix can be selected.

The designed RCC mix should be tested on a test section to be constructed at site using the ingredients available at site and any adjustment required in the mix as a result of test carried out at test section should be made. In actual construction the

water content decided on the basis of mix design should be slightly increased to account for the evaporation during transportation and placement.

## 4.8 CONSTRUCTION ASPECTS

There are basically two approaches developed for the construction of RCC dams. In Japanese approach, the RCC is used in the interior of the dam and it is protected by 2 to 3 m thick conventional concrete on both U/S and D/S faces. Transverse joints are provided with waterstops. Drainage systems are also provided similar to conventional concrete gravity dam. The lean paste cementitious content ( $C + P = 100\text{--}120 \text{ kg/m}^3$ ) with 20 to 30 percent of pozzolana is used and the RCC is placed in dam in 60–90 cm thick layers.

The prevailing trend in American approach to RCC dams has been to modify the dam design in such a way that structural safety of a gravity dam is ensured with maximum economical benefits. In this approach maximum use of RCC is ensured which may be laid uninterrupted in long reaches which requires either elimination of transverse joints or the joints are at large spacing. Attempt is also made to eliminate formwork at dam faces. This approach generally uses high and medium paste cementitious material with large proportion of pozzolana. The American approach is simple, practical and safe and has been adopted in many other countries with slight changes suitable to their conditions.

The design and construction of an RCC dam is site specific, but the stiff character of RCC and the mode of transport and compaction are the same in all cases. Hence some general construction aspects are described below.

### (i) General

The construction of RCC dam is done in thin layers placed over a large area unlike a conventional concrete gravity dam where construction is done vertically in independent blocks. Therefore, any problem in the process from mixing to compaction may stop the whole work of concrete placement till the problem is resolved. It is therefore necessary that each activity involved in construction from aggregate production to procurement of essentials such as cement, pozzolana and other items such as embedded parts and transportation system shall be carefully planned in advance. It is considered advisable that RCC mix ingredients are stockpiled in sufficient quantity (say 50% of the total requirement of the season) near the mixing site before starting the work. Similarly sufficient standby equipment for transportation, placement and compaction should be available at site for uninterrupted construction.

### (ii) Aggregate Production

The quarry sites for both coarse and fine aggregates which may economically yield material of specified quality and quantity should be identified. The processing of aggregate to specified grade, if required, and stockpiling near the mixing plant

should also be organized before the work is started. If concreting is to be done in hot weather, precooling of aggregates may be required. It can be done by sprinkling cold water on coarse aggregate stockpiles during the day.

### (iii) **Batching and Mixing**

Batching and mixing are both important features in producing a RCC mix of specified strength and consistency which should have good homogeneity of ingredients. Both conventional plants and continuous feed plants are used to produce RCC. Conventional batch plants provide accurate control on weights of aggregates but are slower than continuous feed plants. Conventional batch plants have the advantage of producing other type of concrete required on the job. Continuous feed plants may be belt-scale feed plants or volumetric plants. Plants equipped with weight scales on the materials feed belts provide a check on mix proportion. The limits of accuracy of weighing the material may vary within  $\pm 2$  percent for aggregate and  $\pm 1$  percent for other ingredients.

The mixer should be such that it produces homogeneous mix of specified workability and consistency. The mixers should be regularly examined for accumulation of hardened concrete and wear and tear of the blades and corrective measures shall be taken. Transit mix trucks should not be normally used for mixing and transportation.

The RCC batching and mixing plant should be sized for the job requirement. The plant capacity should normally be adequate to provide for the requirement of RCC for two lifts in a working shift. A capacity of about 150 to 200 m<sup>3</sup>/hr is generally found adequate.

### (iv) **Transportation**

The transportation system should be carefully planned to meet the rate of production and placing of RCC. Normally two systems of transportation are used for transporting RCC from mixing plant to placement area, one is conveyor belt system and the other is hauling vehicle which may be end or bottom dump trucks, trailer or rail car carrying number of buckets. Sometimes cable ways may also be used. The selection of transporting system depends on topography, shape of valley etc. which may put constraints on layout of haul roads with adequate turning space and the overall economy. The time required in transportation is also an important consideration in selecting transportation equipment. It is considered desirable to complete dumping of RCC on placement area within 15 minutes after mixing. The general rule is that entire process of transportation, placing, spreading and compaction should be completed within 40 minutes of mixing of unretarded RCC mixes.

The transportation by continuous high speed belt conveyors from mixing plant to the placement area on the dam is ideal because of fast delivery, no requirement of constructing and maintaining haul roads, less maintenance and less labour requirement. Covered belts are preferred if the distance is large to avoid drying of mix. When hauling trucks are used, the problems of drying of mix during transportation and segregation of coarse aggregate are severe and needs to be mitigated. Segregation is less in bottom dump trucks than that in end dump trucks. Movement of

hauling trucks in placement area generally causes damage to lift surface. So efforts shall be made to avoid such damage.

Finally the aim of selecting and planning the transportation system is to maintain a continuous supply of RCC mix to the placement area within specified time limit.

#### **(v) Placing and Spreading**

The concrete is placed from abutment to abutment in lifts generally 30 cm (after compaction) thick. Thickness before compaction may be 35 cm. The hauling equipment deposits material (RCC mix) in heaps are spread on the already placed lift. Measures shall be taken to ensure proper blending between the two lifts. Spreading can be done either by bulldozers or motor graders or front end loaders. Spreading by bulldozers is generally advisable. The spreading shall be accomplished quickly after placement preferably within 10 to 15 minutes. A flat spread surface should be left as a result of spreading for compaction.

If conventional concrete is to be placed on the faces and RCC in the interior, it shall be properly demarcated. The conventional concrete shall be placed after placing RCC but within 20–25 minutes. If required a richer mix of high paste content may be placed between conventional concrete and RCC.

#### **(vi) Joint Cutting**

In case transverse joints are to be provided at specified distances, these shall be made in each lift after spreading and proper finishing but before compaction. The vibratory cutter can cut the joint immediately after completion of compaction. A joint filler board is inserted in the groove. A galvanized steel sheet 3 mm thick can also be used in the groove at the joint. Plastic sheeting can also be used as bond breaking material.

#### **(vii) Compaction**

Vibratory rollers ranging from 5 to 15 ton capacity are usually used. The selection of capacity for specific use depends on compaction effort required, thickness of lift, mix design, the location or area to be compacted etc. A vibratory roller will need a clearance of about 30–50 cm from the vertical shuttering or any other obstacle. So near the form work or in tight areas or near rock out-crop etc. small hand operated compactors shall be used.

The minimum number of passes of the vibratory roller required to achieve desired compaction depends on type of mix and the lift thickness. The number of passes required should be determined from the tests conducted on test section outside the dam before starting the construction of dam.

Normally 3 to 6 passes are required for about 30 cm thick lift to achieve desired density. Over compaction shall be avoided as this will lead to reduction in the density.

#### **(viii) Lift Joint Treatment**

Bond between the lifts is an important design requirement for providing resistance to sliding and minimizing water seepage through lift joint. The time between

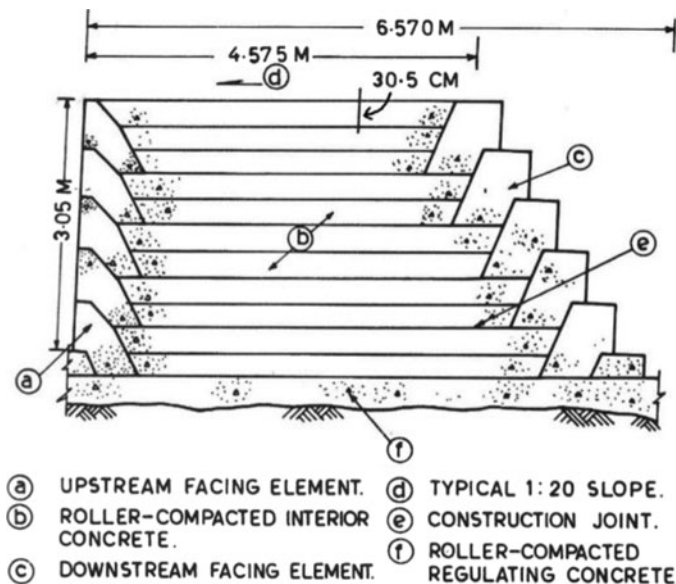
the placement of lifts is the important factor which affects the bond between the lifts. RCC does not bleed and bring up lattice to the surface. So no clean up of surface is required and no joint treatment for bonding is required if the next lift is placed within six hours. A cold joint is said to be developed if the time elapsed in placing the next lift is more than 24 hours (if maturity index is measured in degree-hours it is 2000 degree (F) hours). In case the cold joint has been formed it shall be treated in the manner as in conventional concrete structure i.e. cleaning by using high pressure water jet or sand blasting followed by washing and drying. The next lift of RCC shall be placed on roughened surface which should be in a saturated dry condition. Before placing the next lift, generally a bedding layer of rich concrete is placed for better bonding. This may be used for better bonding in case, if considered necessary, the cold joint is not formed. The bedding layer thickness may range between 30 to 50 mm or biggest size aggregate in the mixture. The bedding mix should be high sand conventional rich concrete with measurable slump of 4 to 6 cm using maximum 20 mm size aggregate. The W/C ratio shall be 0.45. Bedding layer must be covered by RCC before it is allowed to set. Bonding mortar layer 12 to 20 mm thick can also be used. The mortar usually consists of 1 part cement and 2½ parts sand with maximum W/C ratio of 0.45 by weight.

#### (ix) Constructing Galleries and Openings

As stated earlier the galleries and openings in a RCC should be minimized as they create lot of interference in construction. These are to be constructed separately in conventional reinforced concrete. Foundation grouting and drainage gallery is essential. Several methods have been developed to construct gallery and openings to minimize the interference with construction. These methods use sand fill in entire section of gallery/opening or timber blocking in lifts which are removed when RCC has gained strength. Later, formed conventional reinforced concrete or concrete precast panels are used for constructing gallery wall and roof. In small height dams such as Clear Lake Dam (USA) (USBR, 2005), a collector pipe is used instead of gallery through which drainage holes have been drilled from the dam top.

#### (x) Forms and Facings

The slopes in U/S or D/S of a RCC dam can be made to desired slope by the use of conventional formwork. The experience has shown that RCC can be placed at a slope flatter than of 0.75 : 1 (H : V) without formwork but about last 0.3 m wide concrete will not be fully compacted. Hence in case of placing RCC without formwork a slope sufficiently flatter than required should be adopted. For vertical or steep slopes concreting with formwork may be used but the resulting surface will be rough and of poor quality. If better surface with good water tightness is desired then anchored precast concrete, interlocking stays or in-place form erected by the help of steel stays or facing units using curbs can be used. Concrete curbs or facing units of conventional concrete 60 to 75 cm high can be made in entire length against which RCC can be placed after the curbs are hard enough (8 hours time is adequate for hardening of curb concrete). Laying RCC lifts and simultaneously raising of the conventional concrete curbs is shown in Fig. 4.3.



**Fig. 4.3** Cross-section of the Upper Stillwater dam trial sections.

The precast concrete panels anchored and supported internally or externally are used for both upstream and downstream slopes. In upstream, for water tightness the flexible impervious membrane can be attached to the rear of the panel.

In some projects upstream face is made of thick conventional concrete for which formwork is erected. Facing conventional concrete is placed against the formwork to be compacted by needle vibrators. The placement of conventional concrete should immediately follow the placement of RCC lift so that both can be compacted together to be monolithic. The thick-conventional concrete becomes helpful in placing waterstops at the transverse joints. In this system, problems arise in effectively treating the joint between two lifts of conventional concrete. A poor interface often results between two layers of conventional concrete.

## 4.9 TEST SECTION

The construction of test section and subsequent tests generally carried out on it have been very beneficial and this has been done on all RCC dam projects before taking up actual construction.

The test section should simulate actual operations of mixing, transporting, placing and compaction of designed RCC mix in lifts of specified thickness. Test sections are about 25 to 30 m long and 6 to 8 m wide. Lift placement should also simulate the expected time interval between two lifts. The core cutting for tests shall be done after 28 days of construction. The usual inspection of core and its laboratory tests will

verify the design of the mix as well as give the idea about segregation expected in construction and bonding between lifts. The requirement of number of passes of the vibratory roller to give desired compaction and density can also be determined. These results of tests on test section give an opportunity to modify both the design and construction procedure.

## 4.10 RCC DAMS IN INDIA

RCC dam was first constructed in 1978 in Japan. After that over 500 RCC dams have been constructed world over. India slowly adopted this technology and the first 12 m high RCC dam (a component of Ghatghar project) was constructed in 2003. It was the saddle dam of upper reservoir of Ghatghar project. Later the upper dam 15 m high was completed in 2004 and the lower dam 84 high was completed as RCC dam in 2006. The second RCC dam project was middle Vaitatna. It is 102 m high and 15 lac m<sup>3</sup> RCC was placed in two years and work was completed in 2012. The non-overflow portion has stepped slope and the spillways face was made of conventional concrete. The third RCC dam is Teesta IV Low Dam Project. The dam is 30 m above river bed level and 45 m above deepest foundation. In this dam 1.68 lac m<sup>3</sup> RCC was placed in 196 days using conveyor belt system for transportation of RCC to dam site. The project was completed in 2015. It is hoped that many more high RCC dam projects will be taken up in future.

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# Chapter 5

## Embankment Dams



### 5.1 GENERAL

Embankment dams are water impounding structures which are constructed of naturally available material (soil or rock) without addition of any binding material other than those inherent in the natural material. These materials are usually obtained at or near the dam site. These materials form a relatively flexible structure which can slightly deform to conform to the foundation deflection without causing deflection failure. This makes the construction of embankment dams possible on any type of foundation.

### 5.2 ORIGIN AND DEVELOPMENT

The history and the surviving remnants of ancient structures and other evidences have shown that embankment dams have been used for storing water since the early days of civilization. Some of these structures were of considerable size. Some embankment dams are found in southern part of India and Sri Lanka which are centuries old. Rao has described ancient earth dams of India constructed from 800 to 1600 AD. One such earth dam about 17.5 km long, 21.4 m high and containing about 13.0 Mcm of earth with a facing of stones was constructed in Sri Lanka in the year 504 BC. About 2800 BC, a 12 m high dam of rubble walls was constructed in Egypt. Even, today, embankment dams continue to be the most common type of dams, because its construction utilizes locally available materials in natural form with minimum processing.

Until modern times all earth dams were designed by empirical methods. Many such dams failed. The lessons from these failures helped in developing rational design procedures and Bassell in 1907 (Ref. USBR) suggested the rationale of selecting the slopes for an earth dam but until 1930's little progress was made and

**Table 5.1** High embankment dams

Sl. No.	Name of dams	Height of dam (m)
1.	Pong Dam on Beas (India)	116
2.	Ramganga Dam (India)	125
3.	Oroville Dam (USA)	235
4.	Mica Dam (Canada)	244
5.	Tehri (India)	260
6.	Chicoasen Dam (Mexico)	262
7.	Nurek Dam (USSR)	300
8.	Rogun (USSR)	335 (under construction)

dams of height less than 30 m were constructed. The rapid advances in science of soil mechanics since that time has resulted in development of greatly improved design procedures. This helped in developing zoned embankment and rock fill dams. Alongwith the advances in science of soil mechanics and rock mechanics rapid developments have also taken place in the technology of moving the earth i.e. in earth moving machinery during first half of 20<sup>th</sup> century. And both these advances have made it possible to construct high (100 m and more in height) embankment dams with confidence. Table 5.1 shows a few examples out of hundreds of high embankment dams constructed in the second half of 20<sup>th</sup> century. Failures have been rare and the recorded failures in India (Chapter 8) are of small height dams of improper design and due to lack of care in construction and maintenance.

The design of an embankment dam, today, is based on rational design methods and construction is properly planned and controlled; hence these dams can favourably compete with other possible types on a particular site. The following are the important stages in constructing an embankment dam:

- (i) Thorough pre-construction investigations of foundation conditions and construction materials.
- (ii) Application of engineering skill and design techniques.
- (iii) Employing carefully planned and controlled method of construction.

### 5.3 SELECTION OF SITE

The basic requirements of a suitable dam site are as below:

- (i) *Bottleneck configuration of contours*: This means a narrow gorge at dam site and a wide reservoir in its upstream. It is a natural gift and if available a good storage can be developed much more economically than otherwise. The following examples (Table 5.2) indicate wide disparity in height and storage capacity of some embankment dams. It can be seen from the table that the storage capacity of Ramganga dam and Tehri dam is practically same but the height of Tehri dam is around 2.75 times of that of Ramganga dam. Hence a site giving maximum storage for minimum height should be selected for a project.

**Table 5.2** Storage capacity of some dams

Sl. No.	Name of dams	Height (m)	Storage capacity ( $10^9$ cum)
1.	Ramganga Dam (India)	125	2.2
2.	High Aswan Dam (Egypt)	97	156.2
3.	Oroville (USA)	235	4.3
4.	Nurek (USSR)	300	10.5
5.	Akosombo (Ghana)	112.5	148.0
6.	Tehri Dam (India)	260	2.1

- (ii) *Type of foundation:* A site with sound rock foundation is desirable for a high concrete dam. But an embankment dam can be constructed on any type of foundation. Poor rock foundations may need elaborate treatment. An embankment dam can be constructed on alluvial foundation but with adequate measures to check seepage and prevent piping failure. Adequate investigations of the foundation are required to develop a plan of foundation treatment.
- (iii) *Availability of material:* For an embankment dam it is necessary to utilize material available within reasonable distance. The material characteristics such as grain size, moisture content, shear parameters ( $c$  &  $\phi$ ) influence the design and economy of dam. Hence, adequate investigations and field and laboratory tests should be conducted to determine these soil parameters.
- (iv) *Suitable spillway arrangement:* Embankment dam needs a separate spillway. Most common is chute spillway. These are placed on abutments requiring lot of excavation which can possibly be economically used in dam construction. If a saddle is available chute spillway can be economically placed in it with minimum excavation. Other type of spillways provided are shaft, tunnel, side channel etc. The selection of type of spillway is site specific and depends on a number of factors such as flow discharge, topography, geology and economy.  
 Another arrangement in wide valleys is to have an overflow spillway in the river channel and embankment section in rest of the length of dam. These are known as composite dams. There are a large number of such dams in India on Southern rivers e.g. Ukai, Obra, Hirakud, Matalila and Rajghat.
- (v) *Reservoir rim above maximum water level:* If there are gaps and saddles below MWL, these are to be closed or plugged. This increases project cost. Pervious seams and solution channels through abutment may leak stored water and need investigations and treatment. Stability of reservoir slopes along the reservoir rim shall be investigated in drawdown condition and necessary stabilizing measures may be taken if required. The selected site for a project should therefore have minimum of such issues.
- (vi) *Submergence:* Ecological and environmental problems in reservoir submergence area shall be investigated. It includes loss of flora, fauna, submergence of forest land, rehabilitation etc. These may need measures to mitigate the adverse impact and the cost of such measures shall be properly accounted for in

the cost of project. Hence the selected site for a project should have minimum environment problems.

- (vii) *Infrastructural facilities:* A site is good and economical if it is well connected with rail/road and has power facilities. It is better if the site is near to an established township having amenities like school, colleges, hospital, etc.

## 5.4 TYPES OF EMBANKMENT DAMS

There are basically two types of embankments dam:

- (i) Earth-fill dam
- (ii) Rock-fill dam

Earth-fill dam is made of soil whereas rock-fill dam consists of well graded rock fragments or a mix of boulder and shingle. The design principles of the two types are basically the same.

### 5.4.1 *Earth-Fill Dam*

Earth-fill dams are made of soil and are commonly constructed on alluvial foundation. Earth-fill dams are further divided into following two types:

- (a) Homogeneous earth-fill dams
- (b) Zoned earth-fill dams
  - (a) Homogeneous earth-fill dams are made of one type of soil. The soil used is impervious (clayey) type. The height of such dams is not very large. It is generally limited to 6 to 8 m. A homogeneous dam for storage of height exceeding 8 m to 10 m should always have some type of drainage arrangement. The drainage should be made of material more pervious than the embankment soil. Such drainage reduces pore pressures in the dam section and increases stability of downstream slope. The drainage also collects seepage and checks development of piping action. Different types of drainage arrangements usually used are shown in Fig. 5.1. Dam sections with drainage arrangement are also called ‘modified homogeneous’ type embankment dam.
  - (b) Zoned earth-fill dams have central or inclined impervious core which is flanked from either side by zones of pervious material. These are commonly used as being economical and more stable. High dams are constructed of such a zoned section. These have been constructed on all type of foundations. Figure 5.2 shows zoned dam section with different positions of impervious core.

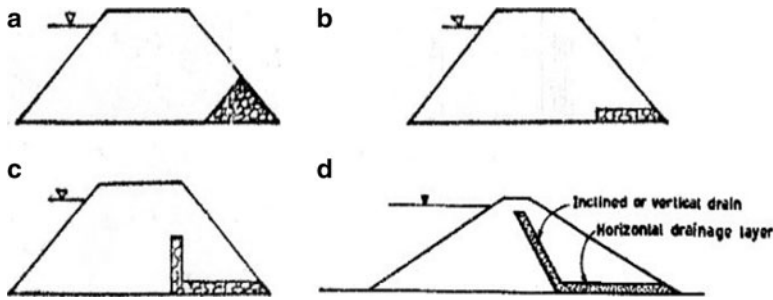


Fig. 5.1 Homogeneous earth dam – typical section.

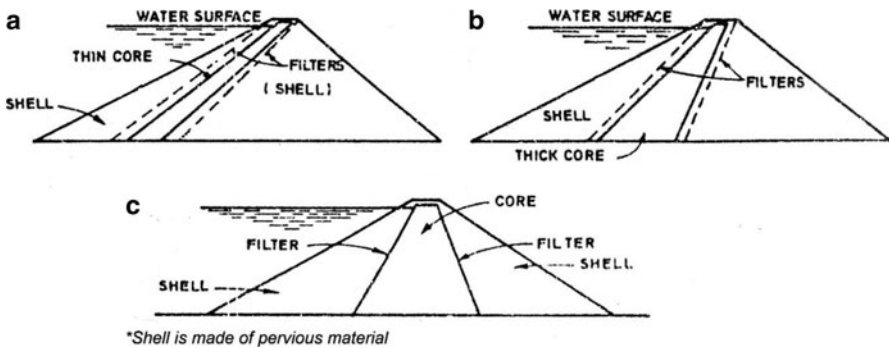


Fig. 5.2 Zoned earth dam – typical sections.

The impervious (clay) core provides imperviousness to the embankment and reduces seepage. The pervious zones are also known as shell zones. These enclose, support and protect the impervious core. Upstream shell provides stability against sudden or rapid draw-down of reservoir level whereas downstream shell acts as a drainage for seepage through dam body and provides stability in steady state condition. The shell zones are made of pervious material which has better shear strength than impervious material. Hence the slopes of zoned section are steeper than those of the homogeneous section. The impervious core thickness depends on the type and quantity of clayey material available for construction in the vicinity of dam site. The thickness cannot be theoretically worked out. A few meter thick core of highly impervious material is enough to reduce seepage. The shear strength of core material is less than shell material. The core also develops appreciable pore pressures. Hence a thin core is provided with a filter zone on both sides. Thick cores are more resistant to piping and seismic forces. The dimensions of top width and base width of core generally provided are given below:

Top width	6 to 12 m (minimum 3 m)
Base width	(a) 30 to 50% of water head (for any type of soil) (b) 15 to 20% of water head (with proper filter) (c) should not be less than 10% of water head.

The core as shown in Fig. 5.2 can be vertical or sloping upstream. The vertical core has higher contact pressure at foundation level and so there is less possibility of seepage. For a given quantity of material available it has more base width than the inclined core. The inclined core has constructional advantage. The D/S shell can be constructed independently. Grouting below core can also be done while D/S shell is being constructed.

### 5.4.2 Rockfill Dams

Rockfill dams are of two types: (i) concrete faced rockfill dam and (ii) earth core rockfill dam. The shell zones are made of well graded rock fragments or boulder, gravel and sand mix. These are commonly constructed on rock foundation, but these have also been constructed on weak foundations made of RBM etc.

#### (a) Concrete Faced Rockfill Dam (CFRD)

CFRD section is made of well graded rock fragments or natural boulder, gravel, sand mix for stability of dam under loads. On its upstream face an impervious membrane is placed for providing water tightness as shown in Fig. 5.3. Membrane can be made of concrete, steel, asphalt, wood etc. Now commonly reinforced concrete or asphaltic concrete membranes are used. The membrane is usually placed on the upstream face though it can be placed inside the dam section replacing the core. The upstream membrane has the following advantages:

- It prevents seepage inside the rock mass.
- It serves as protection against wave action.
- It can be constructed after completing the dam section.
- It can be easily inspected and repaired.

The slopes provided are steeper than the earth core rockfill dams and can be provided nearly equal to the natural slope of dumped rockfill. It is generally 1.4 (H) to 1 (V). The upstream slope may be a little flatter to facilitate construction of impervious facing slab.

#### (b) Earth Core Rockfill Dam (ECRD)

These are like zoned section with core of clay to act as barrier to seepage and the shell zones on both sides of core. The shell zones are made of rock fragments or a mix of boulder and gravel. A well designed filter is provided between core and shell. The upstream slope is protected from wave section by rip-rap. The section is practically the same as shown in Fig. 5.2.

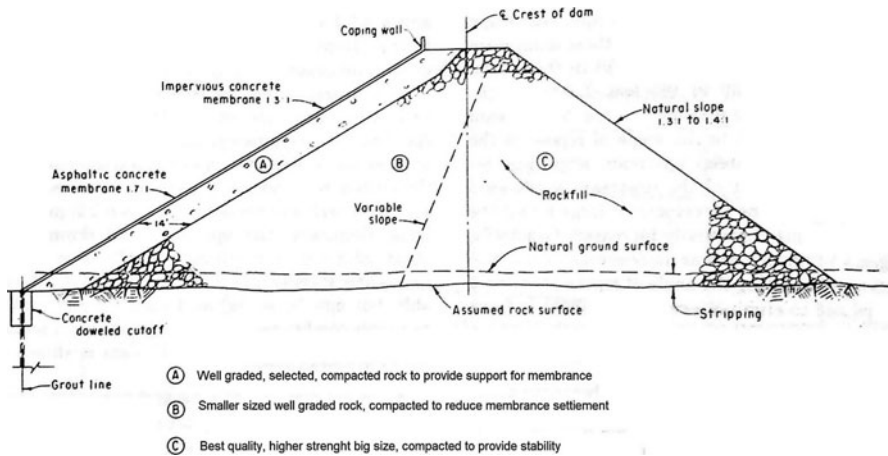


Fig. 5.3 Typical section of a rockfill dam.

## 5.5 METHOD OF CONSTRUCTION OF EMBANKMENT DAMS

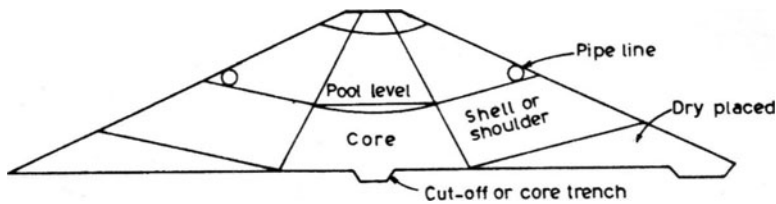
Embankment dams may be classified according to type of construction as below:

- (i) Hydraulic fill dams
- (ii) Rolled fill dams
- (iii) Dumped fills dams.

### 5.5.1 Hydraulic Fill Dams

The hydraulic fill dam is made of material which is excavated, transported and placed by hydraulic methods. The material is mixed with water and pumped from borrow pits into flumes or pipes and carried to the embankment site. Flumes, sluices or pipes extending along outer edges of embankment are provided with outlets at intervals along their lengths and the discharge through these outlets flows inwards to a central pool. Coarse material is automatically deposited on outer edges of the embankment, the finer moves to the centre and finest and most impervious is deposited in the pool to form the central impervious core. It is illustrated in Fig. 5.4.

An alternative is semi-hydraulic fill type which has been adopted on sites where adequate water is not available to transport material over long distances. In this method material is hauled from borrow area by other means, and then it is moved to embankment by water. The outer zones are generally made by car dumped fills and fine material is sluiced from inner slopes of these fills by water jets.



**Fig. 5.4** Hydraulic fill construction.

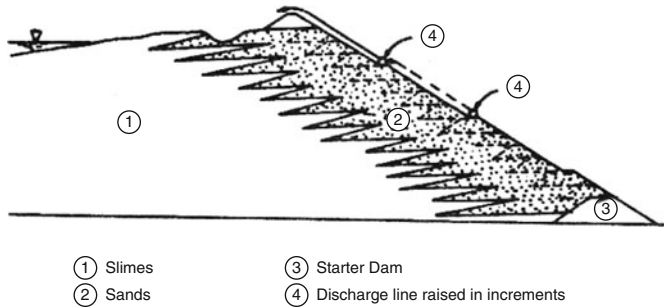
These are practically zoned embankments. This type of construction started in 1850 in USA from gold rush discovery in California and has been practically abandoned after the failure of Fort Peck dam during construction in 1938 due to liquefaction of foundation sand and saturated upstream slope. This has been replaced by mechanized construction technique. High Aswan dam in Egypt (111 m high) was constructed by hydraulic fill method. The suspended mixture of sand was carried to the dam body by pumping into pipes. Compaction of submerged sand was done by vibrators placed on floating installations.

In China this method of hydraulic fill has been used where borrow area is above crest of dam. Water soil slurry (7 soil and 1 water) was pumped with ditches, cut in the ground. The slurry was contained in dam by constructing small bunds on U/S and D/S slopes. Dianshi dam (36 m high) was constructed and was made in five months ( $4.9 \text{ lac m}^3$ ). The soil suitable for this type shall have less than 20% clay and 50 to 60% silt. Such dams with fine sand with uniformity coefficient less than 5 have high risk of liquefaction during earthquake.

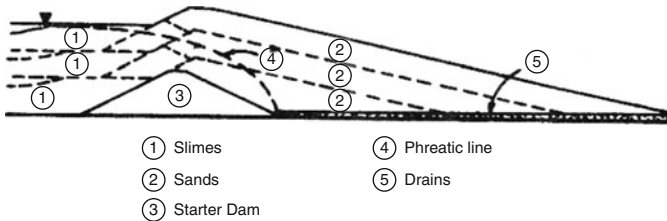
Another type of hydraulic fill dams are Tailing dams. Tailings in slurry form are transported in pipes or flumes to suitable storage areas and stored by tailing dams. These are generally made from sandy fractions of the wastes and finer fractions known as slimes. These fractions along with transported water are pooled behind the dam. The functions of tailings dams are:

- (i) To store waste slurry U/S of dam.
- (ii) To provide temporary storage for certain minimum volume of water so that effluent is clarified prior to discharge into adjacent stream.

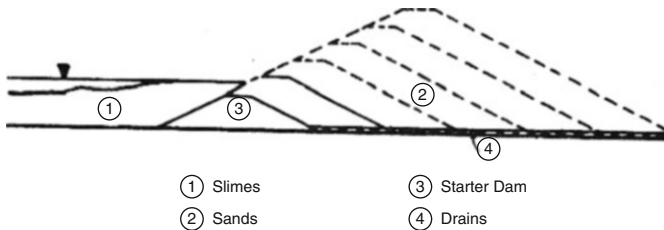
One of the methods of constructing tailing dam depending on quantity of sand in the effluent is shown in Fig. 5.5. In all cases a starter dam from borrowed earth is made to provide initial sufficient pool. When surface of tailings reaches upto the crest of starter dam, the dam is built from tailings by raising in the U/S, at crest or in the D/S. Overflow facilities are important point of these dams. The risk of failure specially due to liquefaction during earthquake is high.



Upstream method of tailings dam construction.



Centre-line method of tailings dam construction using cycloned sand.

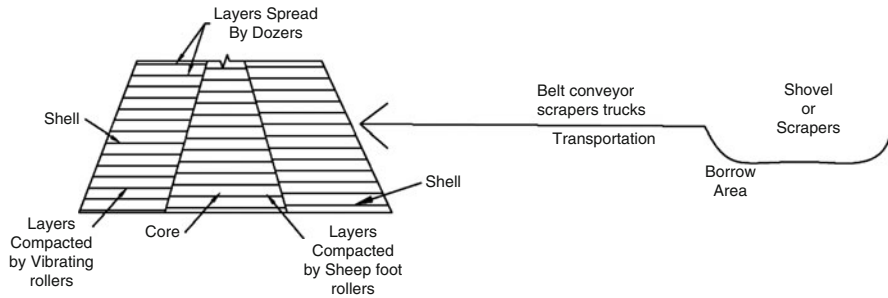


Downstream method of tailings dam construction.

**Fig. 5.5** Construction method of tailing dam. (Source: H.D. Sharma)

### 5.5.2 Rolled Earthfill Dam

It is the embankment which is constructed in mechanically compacted layers. Material is excavated from borrow pits by excavators and is brought to placement site by earth moving machinery. It is spread by dozers in thin layers after moisture adjustment, if necessary. Each layer is compacted with power operated rollers of proper design. Practically, all dams after 1940 have been constructed by this method. A line sketch of the process is shown in Fig. 5.6.



**Fig. 5.6** Process of construction of a rolled embankment dam.

### 5.5.3 Dumped Rock-Fill Dams

The dumped rock fill dam is constructed by dumping rock from cableways or end tipping in high lifts. Large size rock pieces could be used. Sluicing of rock during and after dumping is common practice. It softens the rock, induces early settlement, lubricates stones and helps them to rest in stable position and washes small size pieces into large voids. Sluicing requires water about twice the volume of rock.

Now this practice is rarely adopted and rolled rockfill method is in vogue since 1940.

## 5.6 FACTORS INFLUENCING LOCATION, LAYOUT, SHAPE, TYPE AND DESIGN OF AN EMBANKMENT DAM

### 5.6.1 Topography

Embankment dam is suitable for any topography. There may be some problems in case of a narrow valley with steep abutments such as (a) availability of suitable construction material, (b) location of spillway, (c) haulage of material from borrow areas to the fill, (d) transverse cracking due to cross valley movement and (e) providing sufficient abutment contact and safeguarding against piping. These problems have been dealt with satisfactorily in many cases and high dams have been constructed successfully in narrow valleys. Some examples are given below. This has resulted in increasing trend to construct embankment dams in narrow valleys:

- (a) 237 m high Esmeralda (Chivor) dam in Columbia ( $L/H = 1.3$ )
- (b) 240 m high Guavio dam in Columbia ( $L/H = 1.6$ )
- (c) 264 m high Chicoasen dam in Mexico ( $L/H = 1.1$ )
- (d) 260 m high Tehri dam in India ( $L/H = 2.2$ )

The axis of dam is generally straight but topography and foundation geology may influence the axis of dam and slightly curved axis may be provided. If foundation along axis is not geologically suitable, a sloping core instead of a central core dam may be preferred.

### **5.6.2 *Foundation Conditions***

Generally embankment dams can be built on any type of foundation. Most rock types are acceptable foundation for high rockfill dams except very low strength shales. On good rock type foundations, because of high bearing capacity and resistance to erosion and seepage, there is no restriction to type of dam and selection depends on overall economy. The locations of shear zones and faults in foundation may need relocation of core resulting in change of type of dam. Slope stability and seepage are of primary concerns for the designer in case of high embankment dams on weak bed rock.

An alluvial foundation requires provision of seepage cut off and other seepage control measures. Upstream cutoff may indicate sloping core. Loose overburden is prone to liquefaction or excessive settlement. It may have to be either removed if economically possible or improved by artificial means such as vibro compaction or tamping techniques. Clayey foundations are generally suitable for an earth-fill dam but due to low shear strength, these require special foundation treatment such as horizontal and vertical drainage (sand drains) for quick consolidation of foundation. Very flat slopes are also required for stability of dam section. Generally small height dams are constructed on such foundations.

### **5.6.3 *Material Availability***

It is an important consideration and influences the selection of type of dam the most. If impervious material is available in abundance, homogeneous type is the obvious choice if height is not more. If sufficient quantity of both impervious and pervious soil is available, the choice should be for a zoned embankment and if impervious soil is not available and suitable rock is available, the choice should be for a concrete faced rock fill dam. If the soil available has variable or inconsistent properties it may be used in random zones in D/S shell. Excavations from tunnels, spillway and foundation may be used in random zones for reasons of economy.

### 5.6.4 Climate

It is generally difficult to handle fine-grained soils during the rainy season, and the control over moisture content of fine grained soil in arid regions. If construction of embankment is to be continued during rainy season, provided haul roads should be well maintained, the dam section should have minimum requirement of fine grained soil and a maximum of pervious material. It is also advisable to have sloping core so that work on downstream shell may continue during rainy season. In arid region extra provision for sufficient quantity of water should be made. One extra year may have to be provided for creating a small reservoir for storing flood run-off to meet the construction requirement of water in arid region.

### 5.6.5 Spillway Location

In a narrow valley, either a chute spillway or tunnel type spillway is provided. A chute spillway is widely used and the use of its excavation influences the type and design of dam. If suitable site for locating chute spillway on abutments is not available a tunnel spillway is provided. Usually diversion tunnels are used as spillway by providing either an inclined or a vertical shaft. Provision of an inclined shaft in a dam is shown in Fig. 5.7. When design flood is too large (13,500 cumec in case of Tehri) a combination of chute spillway (for 550 cumec) and tunnel spillways (for 8000 cumec) using all the four diversion tunnels has been adopted for economy. In Ramganga and Pong (Beas) dams only chute spillway is provided.

If valley is wide, a composite type dam with ogee shape overflow spillway of concrete in the river section and embankment dam in rest of the valley is generally adopted as shown in Fig. 5.8.

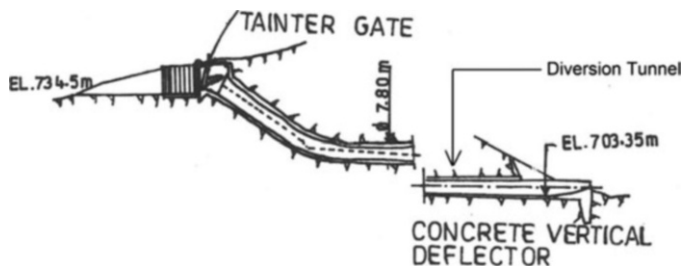
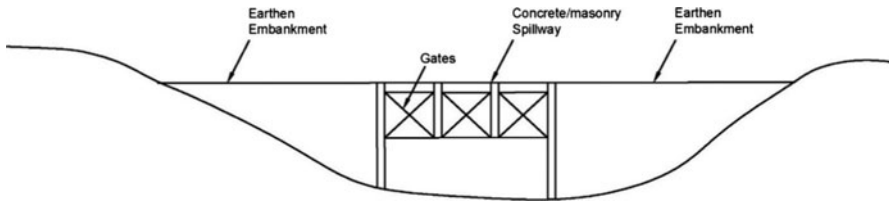
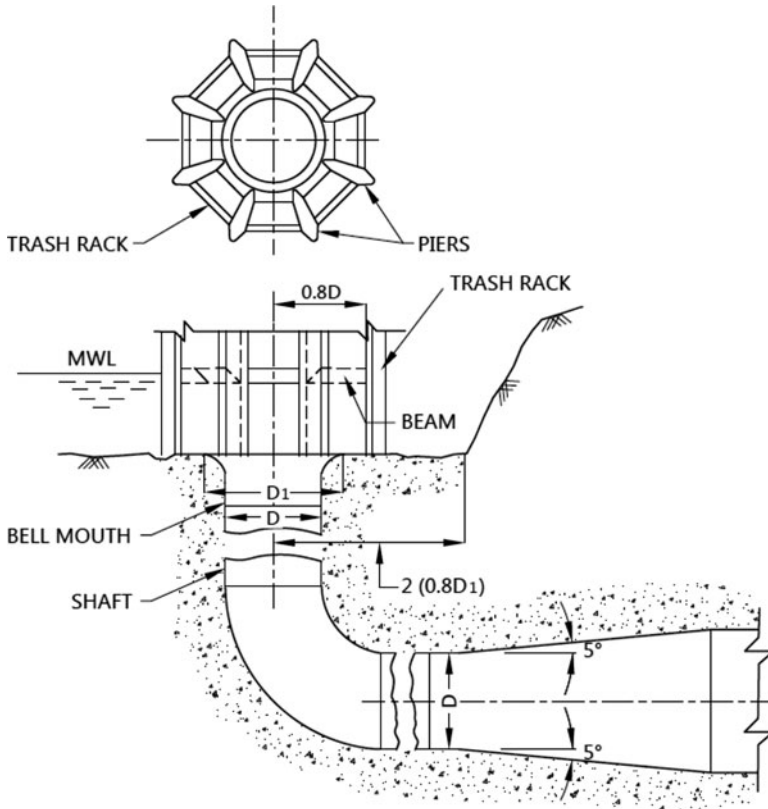


Fig. 5.7 Conversion of diversion tunnel into morning-glory spillway. Paradel dam (Portugal).



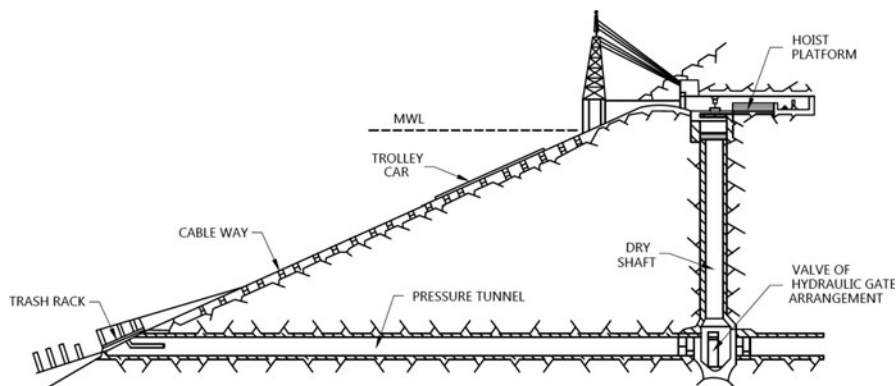
**Fig. 5.8** Typical composite dam.



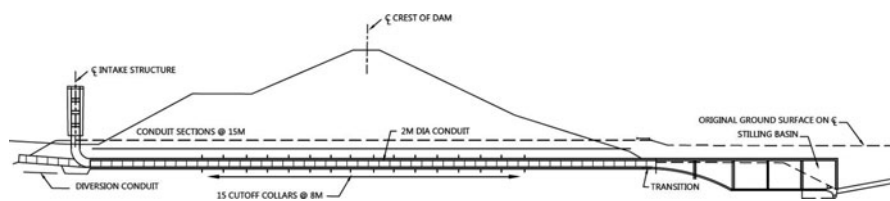
**Fig. 5.9** Tower type intake.

### 5.6.6 Outlet Works

Provision of outlet is required for the use of storage water. Generally outlets are not provided in the body of embankment dam because the improper interface bonding between concrete surfaces of outlet structure and the soil of embankment dam becomes the cause of piping which may result in the failure of dam. The outlet is therefore located away from the dam. A tower type intake (Fig. 5.9) or a sloping



**Fig. 5.10** Sloping intake for an earth dam.



**Fig. 5.11** Typical free-flow conduit outlet in earth dam.

intake structure (Fig. 5.10) is located at a suitable location in the reservoir near the dam and the water carrying conduit is made inside the abutment rock mass as a tunnel.

In small height dams the canal outlets are sometimes provided inside the body of earth-fill dam. A typical arrangement is shown in Fig. 5.11.

### 5.6.7 River Diversion

River diversion scheme at a site depends on the topography of river valley and the diversion discharge. This will affect design of the dam section. In narrow valley diversion tunnels through abutments are generally provided. The dam section will affect the length of tunnels. In large dams the U/S and D/S coffer dams are made part of dam section to affect economy. In wider valleys the embankments on both abutments are first constructed by isolating part of the river through ring bunds and river channel portion is made in the end as a closure section. River diversion is dealt in detail in Chapter 10.

### **5.6.8 Wave Action**

The wave action and wave height depend on wind velocity and the reservoir fetch. The freeboard and upstream slope protection depend on wave height.

### **5.6.9 Time Schedule for Construction**

Limited construction time needs certain adjustments in design. With limited construction time, material from foundation and spillway excavation may not be available for dam construction and so the material may have to be brought from distant borrow areas thereby affecting the economy.

If embankment construction is fast, construction pore pressure may be a problem. In such situations, either flatter slopes will be required from stability considerations or horizontal drainage may have to be provided.

Provision of seepage cut-off in case of pervious foundations may take longer time. If time is limited simpler measures allowing some seepage may be considered.

When

foundation grouting is a major item, saving in construction time may be affected by (i) adopting sloping impervious core or (ii) adopting concrete faced rockfill or (iii) providing a concrete gallery or tunnel in foundation for grouting of foundation independent of dam construction as is adopted in Tehri Dam.

### **5.6.10 Function of a Reservoir**

The function of dam/reservoir (storage or flood control) determines the quantity of seepage loss which may be permitted and that will affect the design of dam section i.e. the measures to be adopted for seepage control.

### **5.6.11 Seismicity**

The seismic activity of region governs the design of embankment section. In highly seismic zone flatter slope, wider core, extra freeboard etc. may be required.

### **5.6.12 Future Raising**

If an embankment dam is planned to be raised in future, the type should be such that it presents minimum problem. Raising is normally done during the operation of reservoir by placing material on downstream slope. The core of raised portion has to be tied to the toe of existing dam. It is easier with inclined core as compared with central core. CFRD is more suitable for raising because it needs adding rockfill on the top and the downstream slope of dam. But the toe slab and face slab shall initially be designed for the final height.

## **5.7 BASIC DESIGN REQUIREMENTS**

The basic principle of design is to produce a satisfactory functional structure at minimum cost. The following criteria have been set forth by the US Army Corps of Engineers (Manual 1968) as basic design requirements that should be satisfied in order to ensure a satisfactory structure.

- (i) The slopes of the embankment must be stable under all conditions of construction and operation.
- (ii) The embankment should not impose excessive stress on foundation.
- (iii) Seepage flow through embankment, foundation and abutment must be controlled so that piping, sloughing or removal of material by solution does not occur. In addition, the purpose of the project may impose a limitation on the quantity of seepage. The seepage through embankment and its control is discussed separately in paras 5.9 and 5.10.
- (iv) Freeboard should be sufficient to prevent overtopping by waves and include an allowance for settlement of the embankment and foundation. It is discussed in detail in para 5.8.4.
- (v) Spillway and outlet capacity must be sufficient to prevent overtopping of embankment.

## **5.8 GENERAL DESIGN FEATURES**

### **5.8.1 Side Slopes**

No specific rules can be given for selecting inclination of outside slopes but following are some useful guidelines.

- First estimate can be based on the experience of similar dams.
- These can be modified as required from stability analysis which depends on zoning of dam section and strength of foundation and embankment material. Slope stability analysis is discussed separately in para 5.11 and in Chapter 6.

- The average slopes range between 2:1 to 4:1 (H : V). Where foundations are weak, flatter slopes may be required and where foundations are strong slopes may be steeper (near 2:1).
- In rock and gravel-fill dams on strong foundations the slopes are steeper – near the angle of repose of fill material. It varies from 1.2 : 1 to 1.8 : 1 (H : V).
- For a homogeneous dam of fine grained soil, the higher the dam the flatter will be the slopes.
- For thin core dams with bulk of pervious granular soil, the allowable slopes are steeper and independent of height.
- It is economically advantageous to have variable slopes in dams above 30 m height. Steeper slopes at upper elevations and flatter at bottom are more desirable for high dams on weak foundations.
- In case of dams in narrow valleys, somewhat steeper slopes can be adopted from stability consideration.
- Flatter slopes at higher elevations may be required in high seismic areas.

### 5.8.2 Filter

#### 5.8.2.1 Filter of Natural Granular Material

In zoned embankment dams, filter is provided between the core of fine grained impervious soil (clay) and zone of coarse grained soil (sand and gravel) in shell zone. The purpose of the filter is to check the migration of fine particles under the water head which may result in internal piping in the core leading to the failure of the dam. The filter should also facilitate drainage of seepage flow. Some dams built earlier in USA by USBR without filter had failed. Now graded filters satisfying design criteria for filters are used. The two principal requirements for a satisfactory filter are as below:

- It must be more pervious than the protected soil.
- It must be fine enough to prevent particles of protected soil from washing into its voids.

Following rules are widely used for design of filters (IS 9429)

- (i)  $\frac{D_{15} \text{ (of filter)}}{D_{15} \text{ (of soil)}} > 5-20$  (permeability criterion). Sherard recommends this value as 4-5 [Permeability is approximately proportional to square of  $D_{15}$  size].
- (ii)  $\frac{D_{15} \text{ (of filter)}}{D_{85} \text{ (of soil)}} < 5-4$  (Piping ratio)
- (iii) Gradation curve of the filter material should practically have same shape as the protected soil.
- (iv) Filter material should not contain more than 5% of fines passing the No 200 sieve (0.074 mm) and the fines should be cohesionless. The maximum size is limited to 75 mm to avoid segregation during placement.

- (v) When protected soil contains large percentage of gravels, filter should be designed on the basis of gradation curve of the portion of material which is finer than 25 mm.

These rules are quite conservative particularly for silt and clay which can resist piping action because of cohesion. The above criteria may be somewhat deviated if thick filters are provided.

**Filter thickness:** Properly graded filter can be very thin. But from practical and construction considerations a minimum thickness of 150 and 300 mm is specified for sand and gravel layers in horizontal filters. The vertical or inclined filters, to protect earth core, are laid in a minimum horizontal width of 2 to 3 m and preferably in a width of 4 m.

D/S filter known as critical filter plays a vital role in the safety of dam against piping and seepage through core. U/S filter serves less vital role and is called non-critical. It prevents migration of core particles in U/S direction into shell in drawdown condition which is not a very adverse condition because of stiffness of clay and low amount of seepage. Often a coarse transition zone of graded 150 mm maximum size down may be adequate as U/S filter. Its thickness is less than D/S filter. If clay core contains cohesionless material, the conventional filter with  $D_{15}$  limited to 10 mm shall be provided.

### 5.8.2.2 Geotextile as Filter

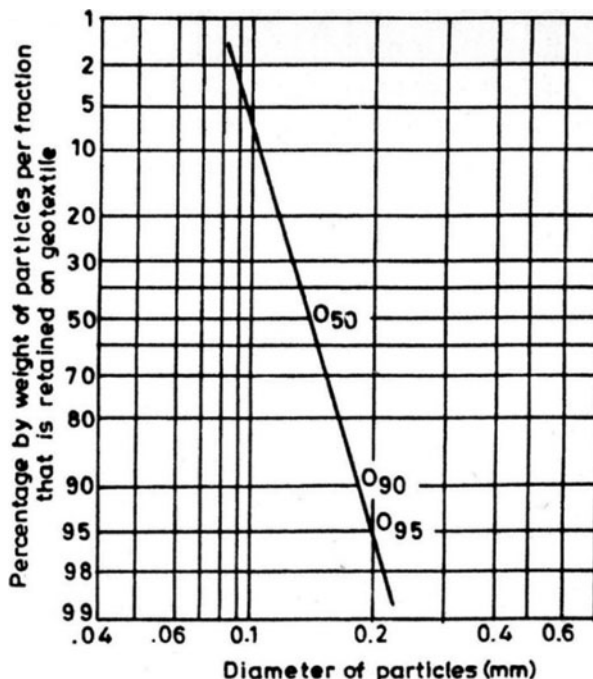
Geotextiles are fabric membrane with small pores. These are both woven and unwoven type. The mechanism of filtering action of geotextiles is similar to the granular filters. When geotextile is placed against the soil and water is passed through it, the soil particles smaller to the geotextile pores pass through it and the bigger particles are collected along the geotextiles and in the long run the amount of fine particles passing through geotextile is reduced. The two major properties which affect the suitability of the geotextiles as stable soil filter are its indicative pore size and permeability. The indicative pore size is determined by 'reverse sieving technique' which involves sieving soil of known particle size through geotextiles of unknown pore size. The weight of particles of each size retained or passing through the geotextile are recorded and plotted in a graph as shown in Fig. 5.12. For woven geotextiles  $O_{95}$  (size which is retained 95%) is taken as indicative pore size of geotextile. For non-woven geotextiles it is still a matter of research.

The criterion developed by US Army for both woven and unwoven geotextiles is piping requirement  $O_{i95} \leq D_{85}$  of soil, where  $O_i$  is indicative pore size of geotextile.

Permeability requirement  $O_i \geq D_{15}$  of soil

Use of geotextile has been quite limited so far in embankment dams. It has been used either to protect clay blanket or in coffer dam or in the toe drain of a dam. The examples are rare where geotextile has been used as a replacement for internal filters

**Fig. 5.12** Semi-logarithmic probability net for geotextile pore sizes.



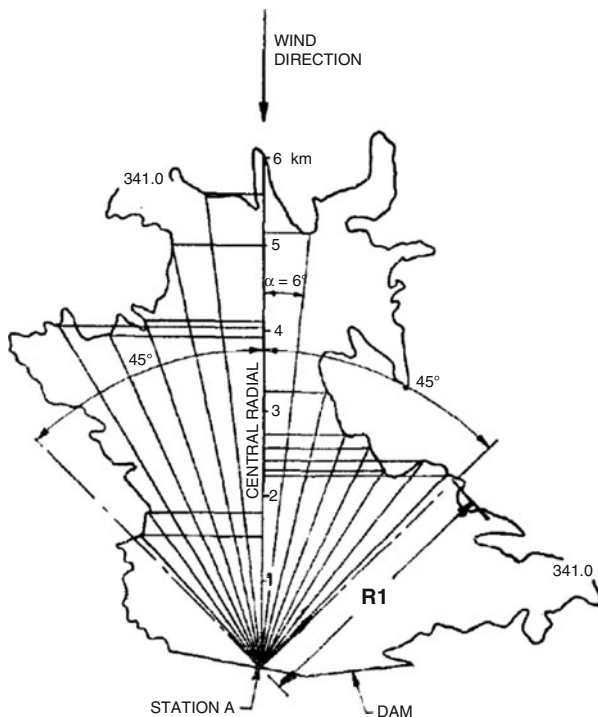
in high dams, where hydraulic head is high. The performance of geotextile as filter material need more research and field evaluation.

### 5.8.3 Crest Width and Camber

It has no appreciable influence on stability or volume of embankment material. It generally depends on requirement of working space and road width. It is provided between 6 and 12 m. A minimum of 3.0 m width shall be provided. For small height dams of 15 to 20 m height the formula suggested by USBR for crest width is  $(0.2H + 3)$  m. The crest should be provided with a pavement and a wearing surface conforming the highway practices. The crest should be drained with a slope towards the reservoir. Camber should be provided to compensate loss of freeboard due to settlement. It should be zero at abutment and maximum at the centre (0.2 to 0.4% of embankment height may be the settlement depending on type of soil).

In case of high dams in highly seismic areas a wider crest is provided. In Oroville dam (235 m) the crest is 18.3 m, in Neurek dam (300 m) it is 20 m and in case of Tehri dam (260 m) it is 25.5 m. The crest width should generally be slightly increased at the contact with abutment.

**Fig. 5.13** Fetch of a reservoir.



### 5.8.4 Freeboard

It is vertical distance between crest of dam and still water level in reservoir. It should be adequate to prevent overtopping in any case. It depends on wave characteristics which in turn depend on fetch (reservoir length) and the wind velocity.

Fetch is defined as the maximum straight line distance in the reservoir on which wind blows. Effective fetch (IS : 10635) is weighted average fetch of water spread covered by  $45^\circ$  angle on either side of trial fetch assuming wind to be completely ineffective beyond it.

$$f_e = \frac{\sum R_i \cos \alpha}{\sum \cos \alpha}$$

$R_i$  is length of  $i^{\text{th}}$  radial (refer Fig. 5.13)

Significant wave height  $H_s$  (m) and wave period  $T$  (sec) are worked out from the following expressions.

$$\frac{gH_s}{V^2} = 0.0026 \left( \frac{gf_e}{V^2} \right)^{0.47}$$

$$\frac{gT_s}{V} = 0.45 \left( \frac{gf_e}{V^2} \right)^{0.28}$$

Wave length  $L_s$  (m) =  $1.56 T_s^2$ .

where  $g = 9.81 \text{ m/sec}^2$ ,  $f_e$  the effective fetch in metres and  $V$  is wind velocity (m/sec) on water surface which is more than the wind velocity on land and the ratio is as below:

Fetch ( $f_e$ ) km	1	2	4	6	8	10
Ratio	1.1	1.16	1.24	1.27	1.3	1.31

Wind velocity for minimum freeboard is taken as half to  $2/3^{\text{rd}}$  of that adopted for normal freeboard. Usually assumed speed for normal freeboard is 160 km/hour.

$$\text{Design wave height } H_o = 1.67 H_s$$

**Wave run up:** It is maximum elevation attained by the wave on the slope above still water elevation. It depends on the roughness of slope. The wave run up  $R$  for smooth surface can be obtained from graph (Fig. 5.14) drawn between embankment slope and  $R/H_o$  for various values of  $H_o/L_s$ . The correction factor for roughness are as below:

Type of pitching	Correction factor
1. Cement concrete	1.00
2. Flexible brick pitching	0.8
3. Hand placed riprap	
(i) Laid flat	0.75
(ii) Laid with projection	0.60
4. Dumped rip rap	0.50

This correction factor is applied on  $(R - H_o)$ .

**Wind set up:** It is the result of piling up of water on one end of reservoir on account of horizontal driving force of wind. Rise in still water on this account is given by

$$S = \frac{V^2 F}{62000 D}$$

where  $S$  = wind setup in m,  $F$  – fetch in km,  $V$  – wind velocity on water surface (km/hr) and  $D$  – average depth of water in metres along fetch line within a few kilometres from dam.

Wind set up is generally not accounted for separately in deep reservoirs as  $D$  is large.

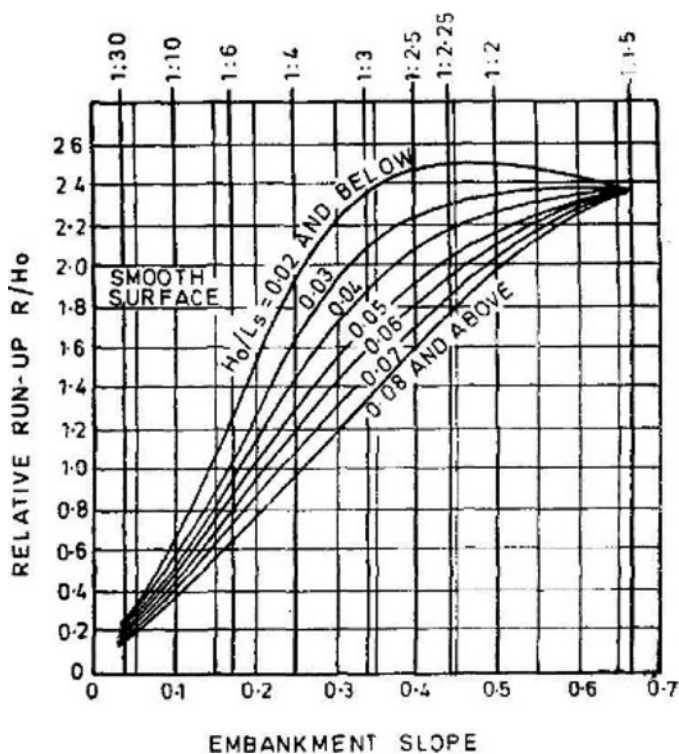


Fig. 5.14 Wave run-up ratio versus wave steepness and embankment slope.

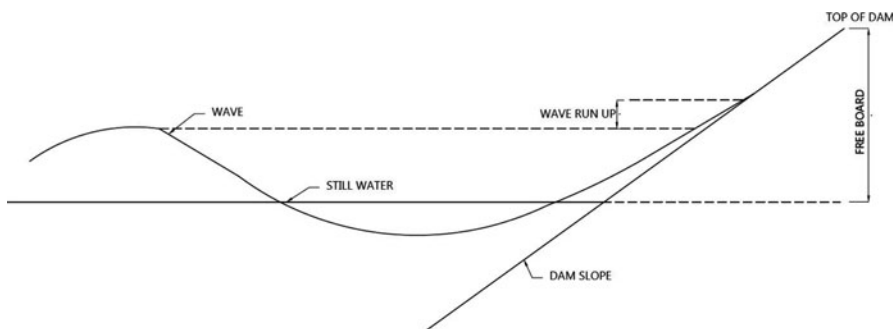


Fig. 5.15 Components of freeboard.

The freeboard is wave height (or wave run up) plus wind set up plus vertical settlement plus additional margin of safety. It is shown in Fig. 5.15.

It is worked out both for FRL and MWL and the maximum is adopted.

Sometimes about one metre high solid parapet is provided to reduce the height of dam from freeboard consideration.

**Table 5.3** Freeboard in some Indian dams

<i>Dam</i>	<i>Height of dam (m)</i>	<i>Free board (m)</i>
Hirakud	60	3.7
Beas (Pong)	134	3.0
Ramganga	125	2.7
Tehri	260	4.5
Ballimela	46	3.6

**Table 5.4** Size of riprap

<i>Max wave height (m)</i>	<i>Average rock size <math>D_{50}</math> (cm)</i>	<i>Layer thickness (cm)</i>	<i>Filter thickness (cm)</i>
0–0.6	25	30	15
0.6–1.2	30	45	15
1.2–1.8	38	60	22.5
1.8–2.4	45	75	20.5
2.4–3.0	60	90	30

A minimum freeboard of 2.0 m above FRL and 1.5 m above MWL is recommended. Actual freeboard provided in some dams are as shown in Table 5.3.

The USBR practice is to provide a minimum freeboard of 3.0 m above spillway gates for dams of height upto 60 m and 2.5 m for dams of higher height.

### 5.8.5 Slope Protection

The slope protection is required on the upstream slope to guard against damage by wave action and on the downstream by rain and wind. I.S. 8237 may be referred for working out slope protection.

#### (i) Upstream Slope

The protection of upstream slope depends on wave height. The types of protection used are dumped rock riprap and ‘hand placed’ stone pitching. In some cases concrete slabs/blocks are used. Soil cement slabs have also been experimented. There are a few examples of asphaltic concrete slope protection. The dumped rock rip rap is most commonly used type of U/S slope protection and is found least susceptible to damage. It is also found efficacious in destroying wave energy. It is generally laid over a layer or layers of filter. This filter shall prevent erosion of underlying embankment material. The rip rap shall be well graded and underlying filter shall not get washed through the voids of rip rap. The individual rock pieces should not move with wave action. The recommended rip rap design criteria depending on wave height for the embankment slopes between 2:1 to 4:1 (H:V) is shown in Table 5.4.

The rip rap shall be well graded. Half should be larger than recommended  $D_{50}$  and maximum size should be about 1.5 times of  $D_{50}$  and minimum down size of 2.5 cm. Spalls should be used to fill voids.

For filter it is recommended that  $D_{85}$  of filter should not be less than 5 cm and  $D_{15}$  of rip rap/ $D_{85}$  of filter to be not more than 10. If this criteria is fulfilled between the embankment material and rip rap, no filter is required. When filter is provided it should satisfy conventional filter criteria with respect to shell material.

Rip rap should extend from crest of dam to at least 2 to 2.5 m below lowest water level. A berm is usually provided at the lower end to prevent raveling of riprap. Usually, the rip rap is taken upto the bottom of the U/S slope to avoid erosion during first filling of reservoir. Experience has proved that dumped rip rap is better than other forms of protection but failure have occurred due to:

- (i) Segregation of small size during placement causing holes.
- (ii) Segregation of large size allowing bedding material to be washed.

Other causes of failure are underestimation of wave height and use of poor quality stone which disintegrated on prolonged exposure. Therefore, igneous and metamorphic rocks of good quality should be used. If good rock of required size is not available wedge shaped cement concrete blocks may be used. Concrete blocks have been used in Nanak Sagar and SardarSagar dams in U.P. USBR has used soil cement blocks placed in stair-step fashion with 0.6 m thickness normal to slope on large number of dams in USA. Reinforced concrete slabs have also been used but these, as a rule, are costlier. These slabs about 15 cm thick with water stops at joints form impervious membrane for sand gravel dams. It eliminates impervious core.

Hand placed rip rap is used where good rock is either expensive or not available in adequate quantity because its thickness is about half the thickness of dumped rip rap but the wave run up in this case is more. When placed in single layer is more vulnerable to displacement and being rigid is less able to adjust to settlement. It is used on small height dams.

#### (ii) *Downstream Slope*

These slopes are made of erodible material (sand, gravel etc.) and so these are to be protected from wind and rain erosion. Where climate and soil conditions are favourable vegetal cover of hardy grass offers adequate protection on medium and small height dams. Below turfing a 10 cm thick suitable soil (not part of embankment slope) for growing grass shall be provided (I.S. 8237). Bushes and trees should not be allowed to grow. Blanket of gravel / boulder, tunnel muck or broken rock is also used for slope protection. The part of downstream slope subject to high tail water level should be protected by rip rap dumped or hand placed properly designed for wave action. In the areas of contact with abutment which may be vulnerable for erosion well graded rip rap underlain with proper filter should be laid. In other areas filter may not be necessary.

## 5.9 SEEPAGE THROUGH DAM BODY

Embankment dam is made of soil and also of material such as sand, gravel, rock pieces etc. The voids in the soil mass provide the path of seepage and under the influence of water head a certain quantity of water will seep. It will be different for different types of soil depending on the soil characteristics. The soil property on which seepage is directly proportional is called permeability. The seepage flow through soil is defined by Darcy's law.

$$V = -Ki$$

where  $V$  is velocity of seepage flow across the soil cross section of area ( $A$ ) which includes solid plus voids and ' $i$ ' in the gradient or rate of loss of head denoted by  $h/L$  ( $h$  is head and  $L$  is length of seepage path) and ' $K$ ' is the permeability coefficient of soil and has the dimensions of velocity. The seepage discharge is given by

$$Q = KiA$$

So the seepage through dam will depend on head created by dam and the type of soil used. The impervious soil such as clay has a permeability coefficient of  $1 \times 10^{-5}$  to  $1 \times 10^{-7}$  cm/sec and the pervious soils such as sand, gravel, rock etc. has a permeability of  $10^{-3}$  to  $10^{-1}$  cm/sec. Thus seepage through impervious soil will be less than the impervious soil.

### 5.9.1 Phreatic Line

After the first filling of the reservoir and when steady state seepage condition has finally reached, the upper surface of seepage in dam is phreatic surface or zero pressure surface. Above this surface will be atmosphere pressure. In a dam cross section this is called phreatic line. Due to capillary action, the soil above this line gets saturated but seepage is limited through the portion below the phreatic line. The phreatic lines in a homogeneous and zoned embankment are described below.

In a homogeneous dam section made of sufficiently impervious soil with relatively flat slopes, it is inevitable that seepage will emerge on the downstream slope despite the impermeability of the soil. The seepage line will emerge approximately at a height of  $1/3^{\text{rd}}$  the water depth in reservoir. It is shown in Fig. 5.16. Hence homogeneous sections are seldom used for storing water. It is used only for small height flood control bunds. Homogeneous section are thus modified by providing some pervious material on the downstream to control seepage. The modified homogeneous sections with different types of drainage arrangements are shown in Fig. 5.17. These dam sections also show how the phreatic line is modified by the provision of different drainage arrangement. The drainage arrangement also makes it

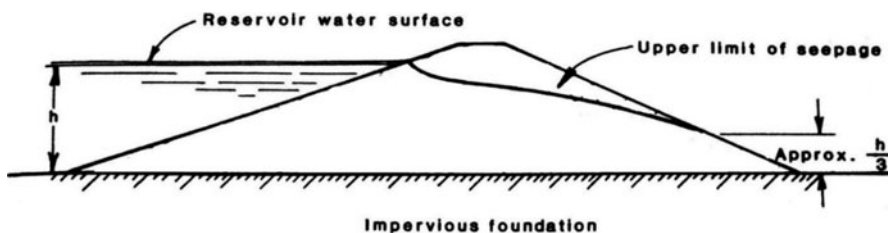


Fig. 5.16 Seepage through a completely homogeneous dam.

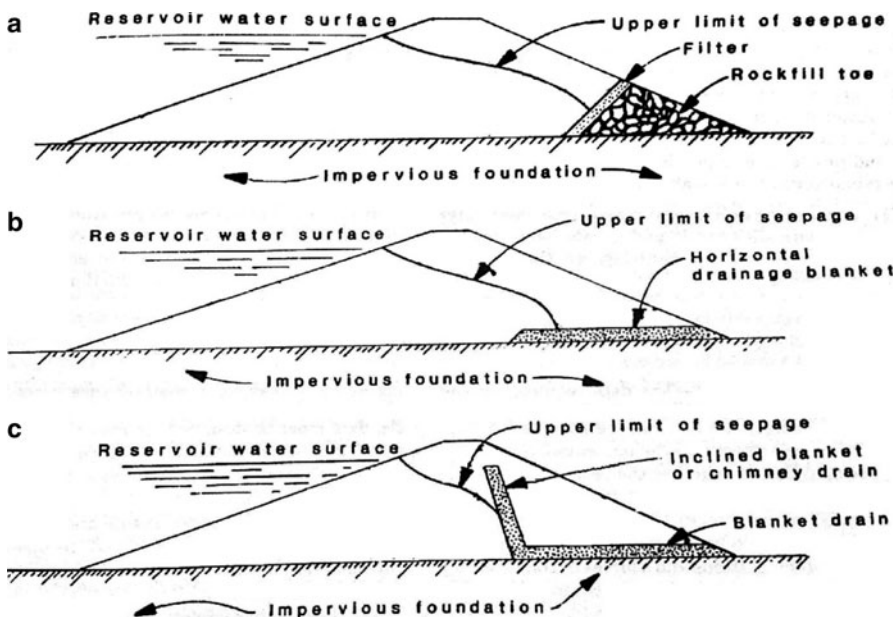


Fig. 5.17 Seepage through modified homogeneous dams.

possible to provide steeper downstream slope. The phreatic line in a typical zoned embankment is also shown in Fig. 5.18.

### 5.9.2 Flow Net

The flow of water through soil is made up of a number of stream-line flows. These can be represented by the Laplace equation which in two dimensions is given by

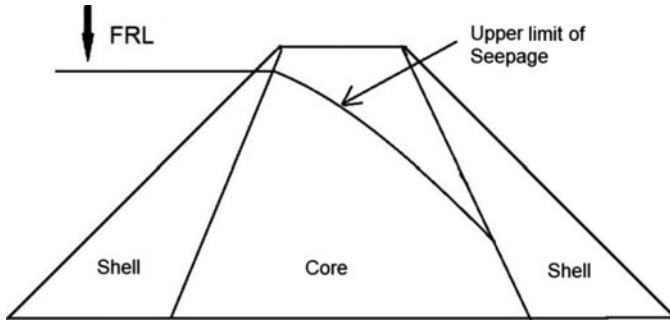


Fig. 5.18 Seepage line in a zoned embankment.

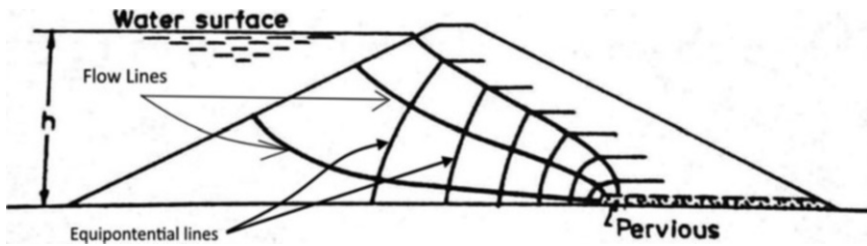


Fig. 5.19 Flownet in dam.

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial z^2} = 0$$

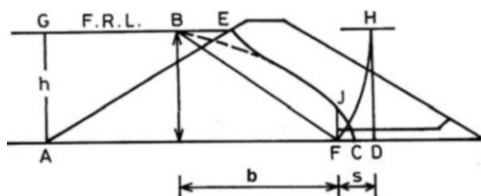
where  $\phi$  is flow potential. This equation gives two curves which intersect at right angles. One set of curves is known as flow line and the other equipotential lines. These are shown in a dam section in Fig. 5.19. The flow net can be drawn graphically. Numerical methods can also be used to solve the Laplace equation.

### 5.9.3 Determination of Phreatic Line in Dam Section

Casagrande suggested the method for drawing the phreatic line which is top flow line and it follows the base parabola in the central portion. This method to draw phreatic line for the modified homogeneous dam section with horizontal blanket for drainage is shown in Fig. 5.20.

BE = 0.3 GE. F is focus of parabola. FH is arc of radius BF. HD is directrix of base parabola. C is vertex of parabola and midpoint of FD. The base parabola is BC.

**Fig. 5.20** Casagrande solution for horizontal blanket.



Equation of parabola is

$$\sqrt{x^2 + y^2} = x + S \quad \text{or} \quad y^2 = 2Sx + S^2$$

On substituting point B ( $b, h$ ) we get

$$S = \sqrt{b^2 + h^2} - b$$

$Q$  (seepage discharge) =  $KiA$  (area  $A$  at any point is equal to  $y$  at that point)

$$i = \frac{dy}{dx} \quad \text{at point } J \quad \text{Thus } Q = K \cdot \frac{dy}{dx} \cdot FJ$$

$$\frac{dy}{dx} = \frac{d}{dx} (2xS + S^2)^{1/2} = \frac{S}{\sqrt{2xS + S^2}}$$

$$\frac{dy}{dx} \text{ (at } x = 0) = 1 \text{ and } y = S \text{ (at } x = 0)$$

$$y = FJ = S$$

$$\therefore Q = KS \text{ per m length of dam}$$

This expression can be used for approximate estimation of seepage discharge from rock toe section of dam. The phreatic line for homogeneous section and the modified homogeneous section with a rock toe can be similarly drawn as shown in Fig. 5.21 (a, b).

It can be seen that to draw phreatic line  $\Delta a$  shall be determined.  $\Delta a$  will depend on the value of  $\alpha$ . It can be determined from the following:

$\alpha$	30°	60°	90°	135°
$\frac{\Delta a}{a + \Delta a}$	0.36	0.32	0.26	0.13

It can be seen from Fig. 5.21 that  $a + \Delta a$  is the distance from the focus  $F$  to the point of intersection of base parabola with the downstream slope i.e.  $J$ . Thus  $a + \Delta a = FJ$ .  $FJ$  can be measured or computed from parabola equation. The above table will then be used to work out  $\Delta a$ .

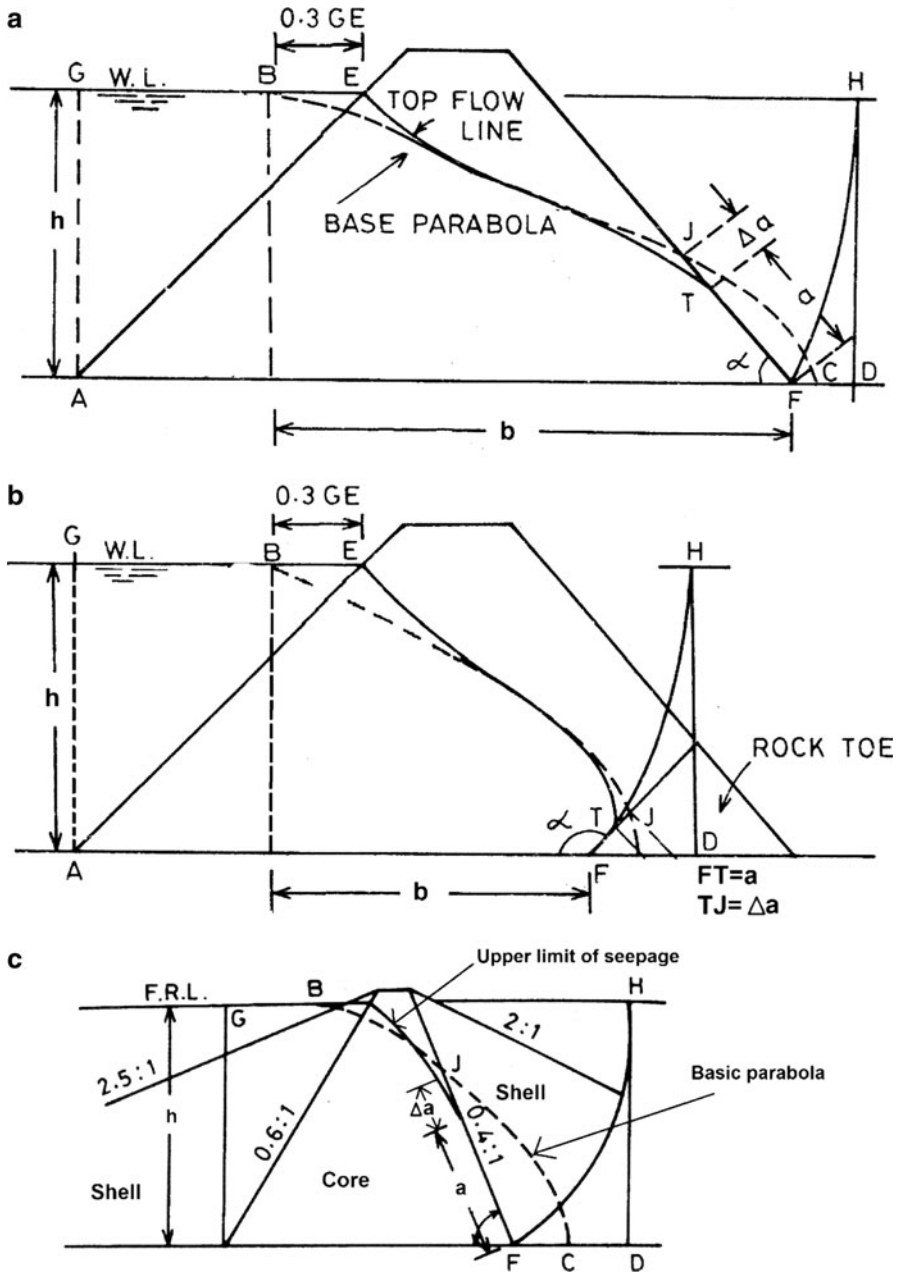


Fig. 5.21 Casagrande's method of drawing phreatic line.

The phreatic line for a zoned section can be drawn considering only the core portion which is like a homogeneous section and is shown in Fig. 5.21(c).

## 5.10 SEEPAGE CONTROL THROUGH DAM BODY

Seepage through body of dam causes loss of water during ponding in reservoir. Besides loss of water the movement of soil particles may cause piping. If it remains unchecked piping may result in the subsidence of dam body and finally in overtopping and failure of the dam. Seepage may also result in saturation of downstream slope causing its sloughing and the failure of dam.

Hence measures to control seepage are required for safe design and construction of embankment dams. These measures are generally in the form of rock toe or rock toe combined with horizontal blanket in homogenous dams and in zoned embankment it is the core to reduce seepage and filter between core and shell to check piping and to provide safe outlet to seepage. These are described separately in detail.

## 5.11 SLOPE STABILITY

### 5.11.1 *Approach for Stability Analysis*

There are two approaches to work out safe upstream and downstream slopes of an embankment dam.

One approach is stress method in which distribution of stresses and strains in the body of embankment and foundation are worked out and are compared with the shear strength of the material at that location to get the idea of the factor of safety. The development of FEM computer based packages has made it possible to carry out this kind of analysis. The accuracy of results of this analysis is based on the input data which is the geometry of section and the properties of different soils in embankment and the foundation.

The other approach is Limit-Equilibrium approach in which a sliding surface is assumed and the shear stress along the surface is balanced against the driving force which is due to the weight of the slipping mass and the seepage force. The ratio of the driving force and the resisting force is the factor of safety on that surface. By trial a surface which gives minimum factor of safety is determined and that is called critical surface. If factor of safety is unity or less, the surface is a failure surface.

The limit equilibrium approach being simple is widely preferred by the designers. In high dams of importance, the slopes are first finalized by limit-equilibrium approach and then analysed by stress approach method to modify the section if required.

### 5.11.2 Stages for Stability Analysis

*Construction Stage* – Since clay core in a zoned embankment is compacted at certain moisture content, at the end of construction maximum pore pressures normally develop. Hence both the upstream and downstream slopes or either of the two may be critical.

*Steady State Condition* – When reservoir is filled upto FRL for a considerably long period, the steady seepage condition develops. The soil mass below phreatic line becomes saturated and pore pressures develop. Since the pore pressures on the upstream slope are balanced with the weight of water, this condition becomes critical for downstream slope only. Hence a safe downstream slope shall be worked out for this steady state seepage condition.

*Sudden Drawdown Stage* – In this stage it is assumed that the reservoir level maintained during steady state condition is suddenly lowered. The soil remains saturated and the balancing weight of water on upstream slope is removed. Hence this condition is more critical for the stability of upstream slope.

### 5.11.3 Pore Pressure

Cohesive soils in dams are compacted with some specified moisture content. The water entrapped in the voids may have pressure higher than atmospheric. Such pressure is called the pore pressure. It depends on type of soil, and rate of construction and the possibility of drainage of moisture during construction. It is found that in soils with permeability of  $1 \times 10^{-5}$  cm/sec the development of pore pressures is small and in soils with permeability  $1 \times 10^{-4}$  cm/sec or more it is ignored. The pore pressures are more if compaction of clay soil is done at the moisture content of 2% more than optimal moisture content (OMC) with no drainage and are insignificant if compaction is done at the moisture content of 2% less than OMC. If the rate of construction is slow and the moisture can escape through the side filters the pore pressures will reduce. But if the construction rate is fast and there is no possibility of draining moisture then pore pressures will be high. IS Code (7894) recommends that if the rate of construction of raising the dam is less than 15 m per year then the end construction condition may not be critical as the residual pore pressures will be negligible.

The end of construction pore pressure in dams constructed at fast rate with no drainage is given by

$$u = \frac{p_a \Delta}{V_a + 0.02V_w - \Delta}$$

where  $p_a$  = atmospheric pressure,  $\Delta$  = embankment compression in % of original total volume of embankment,  $V_a$  = volume of free air in voids after compaction as % of total volume of embankment and  $V_w$  = volume of pore water as % of original volume of embankment.

If all air is forced out during compaction, then  $V_a = 0$  and  $V_w$  are determined in laboratory tests. When compression ratio  $\Delta$  is equal to volume of free air  $V_a$ , then

$$u = \frac{P_a V_a}{0.02 V_w}$$

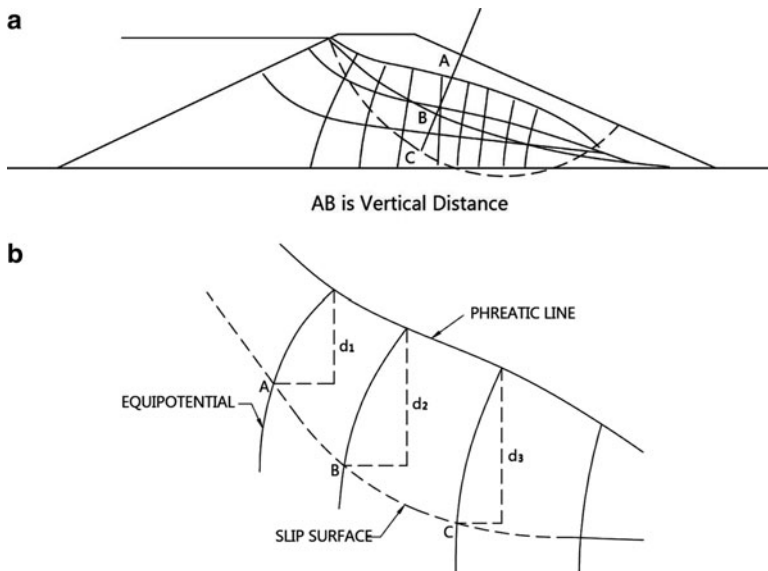
The above expression gives pore air pressure  $u_a$  whereas it is the pore water pressure  $u_w$  that determines the effective stress. When saturation is greater than 25%, capillary pressure due to surface tension on soil grain acts, and pore water pressure is given by

$$u_w = u_a + u_c$$

$$u_c = \frac{2T}{r}$$

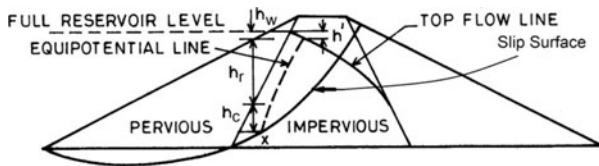
where  $T$  = surface tension of water = 0.076 gm/cm and  $r$  = radius of effective meniscus.

Pore pressure during steady stage condition can be worked out by the use of flow net. It will consist of equipotential lines drawn from the phreatic line. At phreatic line the pressure will be atmospheric. Each equipotential line will have equal pressure head at all points. These are illustrated in Fig. 5.22(a). At point A on phreatic line,



**Fig. 5.22** Steady seepage pore pressures from flow.

**Fig. 5.23** Bishop's solution for drawdown pore pressures in compressible soils.



equipotential line meets if the pressure is atmospheric and pressure head is zero. At point B, which is vertically at a distance  $Z$  from A, the pressure head will be equal to  $Z$ . The pore pressure on each point of the slip surface (shown in Fig. 5.22) where it will intersect a equipotential line will be equal to the vertical distance between the point of equipotential line on the phreatic line and the point of its intersection with slip surface. It is further explained in Fig. 5.22(b). The pore pressure at point A, B and C on slip surface will be distance  $d_1$ ,  $d_2$  and  $d_3$  respectively.

During sudden drawdown when reservoir level is suddenly lowered after steady state condition the pore pressure ' $u$ ' on any element in the core can be conservatively estimated by Bishops equation. Consider the element at X on the slip surface in the core as shown in Fig. 5.23.

In Fig. 5.23,  $h_c$  = vertical height of core material above X, the element under consideration,  $h_r$  = height of free draining material,  $h_w$  = height of water column above dam face = 0 (in sudden drawdown),  $h'$  = potential drop under steady state condition,  $N_s$  = specific drainability of shell material and  $r_w$  = unit weight of water.

The pore pressure at X is given by

$$u = r_w [h_c + h_r(1 - N_s) - h']$$

For homogeneous dams  $h_r = 0$

$$u = (h_c - h') r_w$$

$h'$  being small is ignored and taking  $r_w$  as unity

$$u = h_c$$

### 5.11.4 Shear Strength of Soil

The shear strength of clayey soils is made up of two components: cohesion and friction. According to Coulombs law the shear strength is expressed in the following linear equation.

$$S = C + \sigma \tan \phi$$

where  $S$  is shear strength,  $C$  is cohesion,  $\sigma$  is the normal stress and  $\phi$  is the angle of internal friction.  $C$  and  $\phi$  values are obtained under consolidated undrained condition. In case of pervious soils cohesion is absent. The frictional resistance is developed due to friction between soil particles under normal pressure. The presence of water between the soil particles in clayey soils cannot mobilise friction. The total stress minus pore pressure is called effective stress ( $\bar{\sigma}$ ). Thus

$$\bar{\sigma} = \sigma - u$$

The Coulombs equation in terms of effective stress is written as

$$\begin{aligned} S &= C' + (\sigma - u) \tan \phi' \\ &= C' + \bar{\sigma} \tan \phi' \end{aligned}$$

$C'$  and  $\phi'$  are the values of shear parameter under drained conditions tests.  $\phi'$  values are slightly higher than value of  $\phi$  under undrained condition but  $C'$  value is somewhat on lower side.

Based on the above total stress and effective stress concept, the slope stability computations for different stages of dam construction and operation are suggested as below by various organisations.

- (i) *End of construction stage:* Since pore pressures in this stage depend on a number of factors which need to be carefully controlled, the common practice is to adopt total stress approach using shear parameters determined under undrained condition.
- (ii) *Steady stage condition:* The flow net can give a reasonably accurate assessment of pore pressure in the dam section. Hence it is universally accepted practice to use effective stress approach.
- (iii) *Sudden drawdown condition:* In this condition, both the total stress and the effective stress approaches are used by the designers.

The total stress approach is commonly preferred because it is simple and eliminates determination of pore pressures under various conditions. It uses the parameter determined in laboratory under undrained condition.

### ***5.11.5 Slip Surface***

In fine grained soils actual slides are seen to be circular. Hence it is common practice in embankment dam design to assume a circular slip surface for testing stability of slope. In coarse material comprising gravel, cobbles and queried rock the failure may be a plane surface. Therefore, in case of rockfill the slip surface is considered to be made of two or more straight lines and the approach for slope stability is called wedge method.

### ***5.11.6 Computation Method for Stability of Slopes***

For the slippage plane to be circular, the method for slope stability is known as slip circle method. The assumed circular slip surface for computation of actuating and resisting forces is divided into slices. This method of slices was first introduced by Fellenius and is commonly used for slope stability analysis of embankment dams. It is described in detail in Chapter 6.

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# Chapter 6

## Earthfill Dams



### 6.1 GENERAL

Earthfill dams are made of natural soil. The soil shall be impervious or semi impervious and shall be available at or in the vicinity of the dam site. These are being constructed since early days of civilization to store water for irrigation and domestic use. Until modern times all earthfill dams were designed by empirical methods. Rapid advances in the science of soil mechanics has resulted in development of improved procedures of design and construction.

These are now being classified in two types: (i) homogeneous and (ii) zoned section. Small and medium height earthfill dams of the two types are being constructed generally on alluvial foundation. If the available soil is erodible such as silt and fine sand, a homogeneous section should never be used. The design aspects of these dams are discussed in this chapter.

### 6.2 HOMOGENEOUS DAM SECTION

The choice of a specific type of earthfill dam depends mainly on the availability of local material. A purely homogenous dam is made of only clayey soil. Slope protection material will be different. The soil used in this type of dam should be sufficiently impervious to prevent seepage. Generally soil with permeability of the order of  $1 \times 10^{-5}$  cm/sec is used. If the available soil is erodable such as silt and fine sand, a homogenous section is not to be adopted.

Homogeneous dam section has relatively flat slopes for stability and to avoid sloughing of the upstream slope during sudden drawdown and of downstream slope when saturation reaches to a high level. In a completely homogeneous dam, built for storage, it is inevitable that the seepage will emerge on the downstream slope regardless of the flatness of slope and the permeability of soil. The downstream

slope in homogeneous dam gets affected by seepage to a height approximately one-third of water head in reservoir (Fig. 5.16).

Earlier this type of dam was common for small height dams but because of seepage problem on downstream slope its use is now limited to small height flood protection works. For storage dams, completely homogeneous sections have been replaced by modified homogeneous sections in which pervious material is placed in the downstream to control seepage.

### 6.2.1 Modified Homogeneous Type Dam

This type of dam is basically a homogeneous section with drainage arrangements in the downstream. These are either a rock toe or a horizontal drainage or a combination of both provided in the downstream portion of dam section. A homogeneous dam section with a rock toe is shown in Fig. 6.1. The provision of drainage arrangement serves the following purposes:

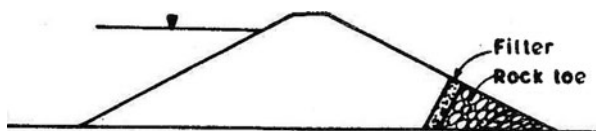
- Provides outlet for seepage.
- Lowers the phreatic line and avoids sloughing of downstream slope (refer Fig. 5.17).
- Prevents piping.

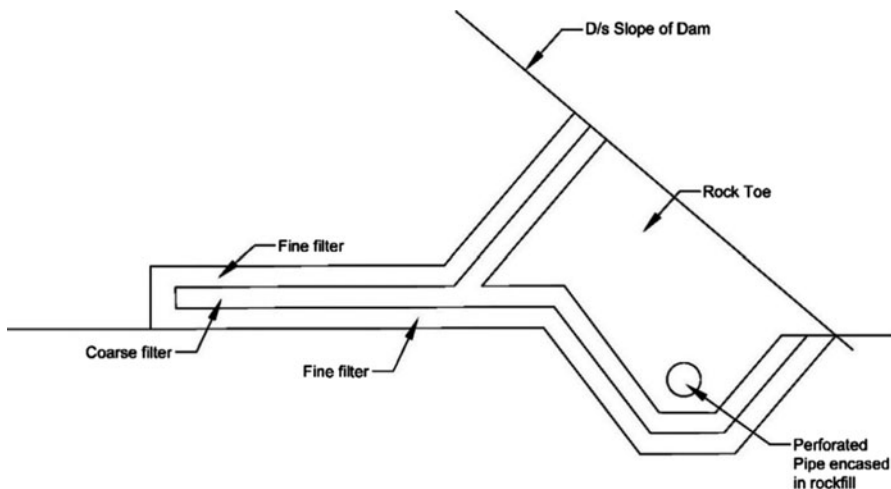
The provision for seepage control measures permits much steeper slopes as compared to a completely homogeneous dam.

#### 6.2.1.1 Rock Toe

For low height homogeneous dams, rock toe made of rock fragments is provided on downstream slope for a height equal to  $1/3$  to  $1/4$  of height of dam and the results have been found satisfactory. It requires large quantity of quarried rock, roughly about 10% of total quantity of dam fill. It may be expensive if rock is not available in the vicinity of dam site. The rock toe, as shown in Fig. 6.2 along with horizontal blanket, is protected by transition filler to prevent migration of fine material from dam fill and the foundation. The inner slope of rock toe is kept 1:1. A toe drain is combined with rock toe and is provided along the downstream toe of dam. The depth of toe drain is usually 1.5 m with minimum bottom width of 1.0 m and side slopes of 1:1. The drain has a suitable longitudinal slope to carry seepage water to a natural drainage. The toe drain is provided with a perforated and open jointed pipe of PVC

**Fig. 6.1** Homogeneous dam section with rock toe.





**Fig. 6.2** Rock toe with horizontal blanket and drain.

or concrete of a minimum diameter of 15 cm. The pipe shall be encased with suitably designed filter to prevent clogging by fines of foundation material.

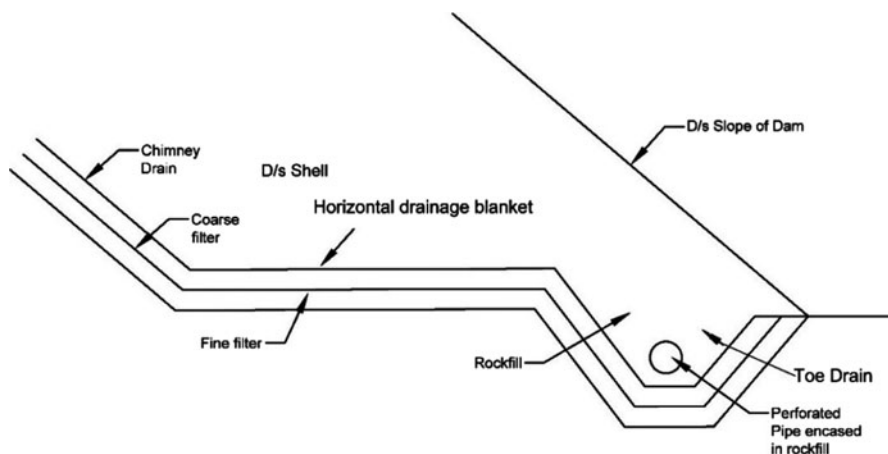
### 6.2.1.2 Horizontal Drainage

In low to medium height dams, horizontal drainage blankets are also provided to drain the seepage occurring through the dam as well as the foundation. It lowers the phreatic line and depending on the length it keeps the phreatic line sufficiently away from the downstream slope as compared to the rock toe. So the rock toe is generally combined with the horizontal blanket as shown in Fig. 6.2.

The determination of phreatic line and the minimum safe distance from D/S slope of dam considered necessary for safety can be a guide for length of blanket. Generally its length is provided upto mid of downstream slope from the toe of the dam. The blanket is made of layered filter material. The top and bottom layers are of fine filter material and the central layer is of coarse filter material. The gradation of fine and coarse filter should satisfy the filter design criteria. Each layer should be about 200 mm thick. The drainage blanket ends in a toe drain of dimensions same as given above with rock toe.

Horizontal drainage blankets are more often supplemented with inclined chimney drains/blanket also as shown in Fig. 6.3.

These drainage arrangements prevent emergence of seepage on downstream slope even if inadvertently layers of higher permeability are incorporated in dam



**Fig. 6.3** Horizontal drainage blanket with chimney drain.

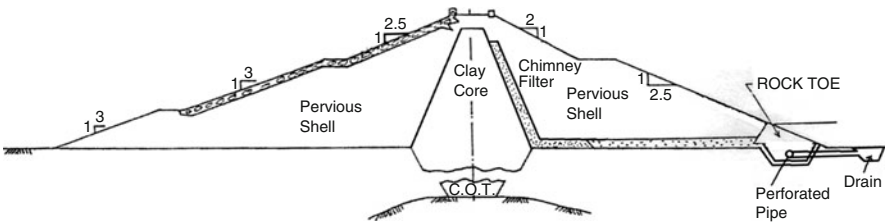
section. These also keep downstream portion free from seepage and reduce the construction pore pressures.

### 6.3 ZONED DAM SECTION

A zoned dam section is that in which a central impervious core is flanked by zones of pervious material called shells. These shells support and protect the impervious core. In many cases an inclined filter between the impervious core and the downstream shell and a drainage layer under the downstream shell are necessary. These filter and drainage layers must meet the filter criteria with the fill and foundation material. These are generally multilayered.

A zoned dam section is, however, preferred where different types of soils are available from borrow area because of inherent advantage of an economical construction. The pervious zones or shells may consist of sand or a mixtures of sand shingle or gravel. The excavations from foundations, approach channel and tail channel of the spillway can also be used in the shells, if it is not relatively impervious. The weaker material from excavations can be economically used in the upstream shell below minimum drawdown level.

The impervious core of clay should have a minimum width of 3 m at the top. The base width should be equal to the height of dam at all elevations or more depending on stability and seepage criteria and availability of material. Generally the zoned dam section has relatively steep outer slopes. A typical section of a zoned earthfill dam is shown in Fig. 6.4.



**Fig. 6.4** Typical section of zoned earthfill dam.

**Table 6.1** Suitable soils for construction of earth dams

Suitability	Homogeneous section	Zoned earth dams	
		Impervious core	Pervious shell
Very suitable	GC	GC	SW, GW
Suitable	CL, CI	CL, CI	GM
Fairly suitable	SP, SM, CH	GM, GCM, SM, SC, CH	SP, GP
Poor	-	ML, MI, MH	-

**6.4 CHOICE OF CONSTRUCTION MATERIAL**

The construction of an earthfill dam requires large quantities of material to be borrowed from areas near the site. Depending on type of material and quantity available a homogeneous or zoned section may be adopted. Excavations from the spillway foundations etc. should be used as best as possible. Attempt shall be made to use the material in the natural available form without processing.

The available soil and excavations shall be identified, tested and classified as per IS 1498. The suitability of soil for construction of earthfill dam as recommended in IS 1498 is given below and may be taken as a guide (Tables 6.1 and 6.2).

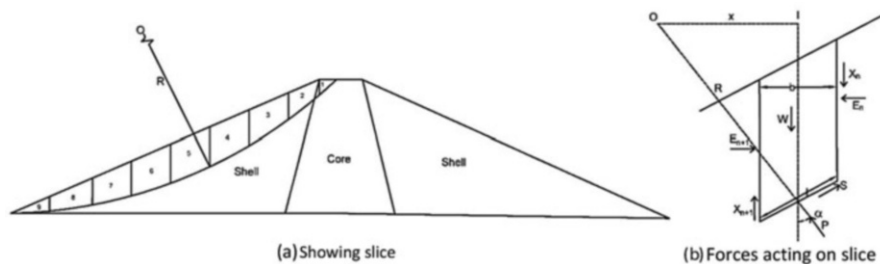
**6.5 SLOPE STABILITY**

As discussed in Chapter 5, the stability of slopes of an earth dam is computed commonly by the slip circle method introduced by Fellenius in which the slip circle is divided in slices.

The method of slices is illustrated in Fig. 6.5. A circular arc with centre at O is drawn. This arc is the potential surface for slide. If the moment of the resisting forces about O exceeds the moment of actuating forces, sliding will not take place. For computation the soil mass between the dam slope and the circular arc is divided into a number of slices by the imaginary vertical lines as shown in Fig. 6.5(a). These lines are usually made at equal spacing though not necessary. Generally six to nine slices are considered adequate for the analysis. The slice height and width should usually be 1 to 2.5 or maximum 3. The forces acting on a slice are shown in Fig. 6.5(b).

**Table 6.2** Suitability of soil for core material of earthfill dams in seismic zones

Relative suitability	Type of soil
Very good	Well graded mixture of sand, gravel and clay D <sub>85</sub> coarser than 50 mm D <sub>50</sub> coarser than 6 mm Fines (less than 75 micron) not more than 20%
Good	Well graded mixture of sand, gravel and fines D <sub>85</sub> coarser than 25 mm Fines (CL with plastic index greater than 12)
Fair	Fairly well graded mixture of sand gravel and fines D <sub>85</sub> coarser than 19 mm D <sub>50</sub> between 0.5 mm and 3.0 mm Fines (less than 75 micron) not more than 25%

**Fig. 6.5** Method of slices.

The slice is acted upon by its own weight  $W$  and by inter slice reactions which has tangential or shear component  $X$  and normal component  $E$ . The force acting at the base of the slice is shear resistance  $S$ . The stability of the soil mass requires that the mass as a whole must satisfy conditions of equilibrium i.e.

1. Sum of all vertical forces  $\sum F_V = 0$
2. Sum of all horizontal forces  $\sum F_H = 0$
3. Sum of moments of all forces about any point  $\sum M = 0$

In the Fellenius method, it is assumed that the reaction from the adjacent sides cancel each other and so the equilibrium condition satisfied in this method is  $\sum M = 0$ . It is experienced that this assumption leads to small error which is on safe side. With this assumption the forces left acting on the independent slices are  $W$ ,  $P$  and  $S$ . Applying equilibrium condition  $\sum M = 0$ , we take moments about  $O$ , the centre of slip circle.

$\sum W \cdot x = \sum SR$ , where  $\sum$  shows summation of terms for all slices,  $S = \tau \cdot l$  where  $\tau$  is shear stress intensity and ' $l$ ' is the length of the base of slice. The shear strength of soil at failure  $\tau_f$  according to Coulombs equation is

$\tau_f = C + \sigma \tan \phi$  and  $\tau = \frac{\tau_f}{F}$  where  $F$  is factor of safety

$$\tau = \frac{1}{F} \left[ C + \frac{P}{l} \tan \phi \right]$$

Putting  $x = R \sin \alpha$ , we get

$$\Sigma W R \sin \alpha = \Sigma SR$$

$$\text{or} \quad \Sigma W \sin \alpha = \frac{1}{F} \sum (Cl + P \tan \phi)$$

$$\text{or} \quad F = \frac{1}{\sum W \sin \alpha} \sum (Cl + P \tan \phi)$$

On putting  $P = W \cos \alpha$  we get

$$F = \frac{1}{\sum W \sin \alpha} \sum (Cl + W \cos \alpha \tan \phi)$$

If we use effective stress approach

$$F = \frac{1}{\sum W \sin \alpha} \sum [C'l + (W - ul) \cos \alpha \tan \phi]$$

where  $u$  is pore pressure and  $C'$ ,  $\phi'$  are soil parameter for drained condition tests.

There are several other methods which have attempted to include the soil reactions of the adjacent slices. But Fellenious method described above is easy to manually work out the slope stability. To determine the critical surface with minimum factor of safety several slip circles of varying radius will have to be analyzed. There are several limit equilibrium methods to assess the stability of the side slopes of the dam.

- (i) Fellenious method
- (ii) Bishop method
- (iii) Janbu method
- (iv) Morgenstern and Price method

The forces and moments considered by these methods are given below:

Sl. No.	Method	Force	Moment	Inter-slices forces considered
1	Fellenious	-	Yes	Ignore both H and V
2.	Bishop's simplified	-	Yes	V ignored, H considered
3.	Janbu's simplified	Yes	-	V ignored, H considered
4.	Morganstern-Price	Yes	Yes	Both H and V considered

Out of these, Morgenstern-Price method is considered better because it takes into account all the forces. The results of the analysis are used to assess the adequacy of

slopes, subject to the required factor of safety. Another method including the reactions from adjacent slides is Spencer's method. It has become easy to carry out slope stability analysis by using computer programs.

The Geo Studio software (Geo-Slope, 2004) is mostly used for analysis by considering different operation conditions. This software is based on Finite element method which uses a Mohr-Coulomb failure criterion. It can be employed for simulating seepage and stability analysis of the dam side slopes. Seep/W program for seepage analysis and Slope/W program for slope stability analysis are used for planned embankment dam section. Seepage analysis is intended for the purpose of quantifying various seepage phenomena such as hydraulic gradient, seepage vector and pore pressure distribution at the corresponding time of interest as the water level in the reservoir rises and recedes. Model set up is prepared in the Seep/W which defines the geometry of a model prior to consideration of the discretization or meshing.

The information required for numerical modelling and analysis of the cross sections of embankment dams are as under:

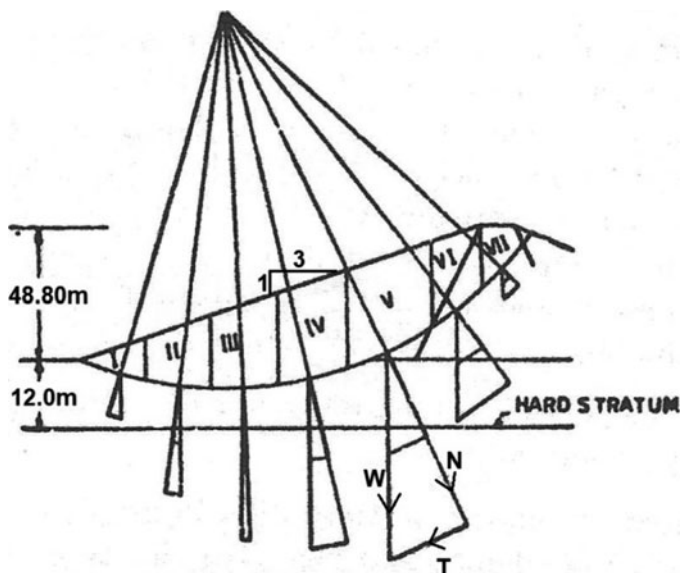
- (i) Layout map and cross section profile of dam section
- (ii) Permeability of all materials used in embankment dam
- (iii) Hydraulic conductivity of foundation strata under the dam
- (iv) Saturated and dry specific weight of materials
- (v) Soil shear strength parameters (coefficient of internal friction ( $c$ ) and friction angle ( $\phi$ )).

Boundary conditions (BC) are then assigned to the model in terms of reservoir water level on upstream face and tail water level on the downstream face treating seepage face condition of the dam. It varies for different cases of operation. The results of seepage analysis are used in slope stability analysis of the dam side slopes. However, to understand the fundamentals of the problem of slope stability manual computation should also be carried out. It is illustrated below.

### 6.5.1 Illustrative Example

An illustrative example of manual computation by Fellenious method of slices is given below (taken from Irrigation Engineering for B. Singh). It has made assumptions which has simplified the computation. The properties of foundation and shell material are same. The pore pressure is ignored and entire soil mass is assumed saturated. The arc is divided into seven slices. The areas and the length of each slice is worked out separately.

Figure 6.6 shows the upstream slope of a non-homogeneous dam section. The properties of shell and foundation material are: saturated unit weight  $1.92 \text{ gm/cc}$ ,  $\phi = 30^\circ$  and  $c = 500 \text{ kg/m}^2$ . The properties of the core material are saturated unit weight  $1.84 \text{ gms/cc}$ ,  $\phi = 10^\circ$  and  $c = 2,500 \text{ kg/m}^2$ .



**Fig. 6.6** Stability analysis by method of slices.

A trial arc has been drawn and the slip surface divided into seven slices. The slices have been so taken that except in the two uppermost slice, other slices have its base in one type of material only. To avoid complication the entire soil mass is assumed to be saturated though actually a small portion above the seepage line will not be saturated; this will involve a very slight error on the safe side. The stability has been computed for the condition of sudden drawdown in Table 6.3 for the slip circle shown in Fig. 6.6. It may be noted that submerged unit weight is equal to saturated unit weight minus the unit weight of water. The shell zone is taken as no free draining.

A more comprehensive illustrative example of slope stability is given in IS 7894.

### 6.5.2 Recommended Dam Slopes

For slope stability analysis a tentative section of dam with U/S and D/S slope has to be assumed. The slopes can be assumed on the basis of similar existing dams. The slopes recommended by Terzaghi are given in Table 6.4 which may be taken as a guide.

The slopes recommended by USBR for homogeneous and zoned section are also given in Tables 6.5 and 6.6 for guidance (Fig. 6.7).

The values of the factors of safety (FOS) which are generally used in design of the slopes of embankments under different conditions are given in Table 6.7.

Table 6.3 Stability computation

Slice No.	Area, m <sup>2</sup>		Weight, tonnes		N, tonnes, normal component of submerged wet	T, tonnes, tangential component of saturated wet.	N tan $\phi$ tonnes	cs tonnes
	Shell and foundation area	Core area	Saturated	Submerged				
I	190	-	364	174	174	-137	100	12.5
II	490	-	940	450	444	-136	256	12.5
III	700	-	1340	640	630	89	364	12.5
IV	800	-	1535	735	700	334	404	12.6
V	985	-	1890	905	818	772	468	15.7
VI	330	250	635 + 394	305 + 119	417	600	73	46.5
VII	28	130	54 + 250	26 + 120	137	224	24.2	52.5

Note: For any slice, for c is to be taken for that material through which the arc is passing.

$\Sigma T = 1746, \Sigma N \tan \phi = 1689.2 \Sigma c_{cs} = 164.3$

Factor of safety =  $(\Sigma N \tan \phi + \Sigma c_{cs}) / \Sigma T$   
=  $(1689.2 + 164.3) / 1746$   
=  $1854 / 1746 = 1.06$

**Table 6.4** Recommended dam slopes

Sl. No.	Type of material	Upstream slope	Downstream slope
1.	Homogeneous well grade material	2.5:1	2:1
2.	Homogenous coarse silt	3:1	2.5:1
3.	Homogeneous silty clay		
	(a) Height less than 15 m	2.5:1	2:1
	(b) Height more than 15 m	3:1	2.5:1
4.	Sand or sand and gravel		
	(a) With clay core	3:1	2.5:1
	(b) With RC core wall	2.5:1	2:1

**Table 6.5** Recommended slopes for small homogeneous earthfill dams on stable foundations

Case	Type	Purpose	Subject to rapid drawdown <sup>1</sup>	Soil classification <sup>2</sup>	Upstream slope	Downstream slope
				GW, GP, SW, SP	Pervious Unsuitable for dam	
A	Homogeneous or modified homogeneous	Detention or storage	No	GC, GM, SC, SM CL, ML CH, MH	2.5:1 3:1 3.5:1	2:1 2.5:1 2.5:1
B	Modified homogeneous	Storage	Yes	GC, GM, SC, SM CL, ML CH, MH	3:1 3.5:1 4:1	2:1 2.5:1 2.5:1

<sup>1</sup>Drawdown rates of 15 cm or more per day after prolonged storage at high reservoir levels.

<sup>2</sup>OL and OH soils are not recommended for major portions of homogeneous earthfill dams. Pt. soils are unsuitable.

The slopes provided in some major zoned type earthfill dams are given in Table 6.8 for guidance.

The typical sections of Nanaksagar and Obra dams are shown in Fig. 6.8.

### 6.5.3 Seismic Analysis for Slope Stability

There are two approaches, (i) pseudostatic and (ii) Dynamic response approach, which are commonly used.

In pseudostatic approach, the static inertial force is taken into account in analyzing the equilibrium of potential sliding mass and if a minimum factor of safety of 1.0 is achieved the slope is considered safe. In the Fellenious method of slices it is accounted for as follows.

Consider a slice of weight  $W$  and  $\theta$  is the angle between the radial and the vertical direction.  $N$  and  $T$  are obtained by resolving  $W$ . The slice is assumed to be subjected

**Table 6.6** Recommended slopes for small zoned earthfill dams on stable foundations

Type	Purpose	Subject to rapid drawdown <sup>2</sup>	Shell material classification	Core material classification <sup>3</sup>	Upstream slope	Downstream slope
Zoned with minimum core <sup>1</sup>	Any	Not critical <sup>4</sup>	Rockfill, GW, GP SW (gravelly), or SP (gravelly)	GC, GM, SC SM, CL, ML CH or MH	2:1	2:1
Zoned with maximum core <sup>1</sup>	Detention or storage	No	Rockfill, GW, GP SW (gravelly), or SP (gravelly)	GC, GM SC, SM CL, ML CH, MH	2:1 2.25:1 2.5:1 3:1	2:1 2.25:1 2.5:1 3:1
Zoned with maximum core <sup>1</sup>	Storage	Yes	Rockfill, GW, GP, SW (gravelly), or SP (gravelly)	GC, GM SC, SM CL, ML CH, MH	2.5:1 2.5:1 3:1 3.5:1	2:1 2.25:1 2.5:1 3:1

<sup>1</sup>Minimum and maximum size cores are as shown in Fig. 6.7.<sup>2</sup>Rapid drawdown is 15 cm or more per day after prolonged storage at high reservoir levels.<sup>3</sup>OL and OH soils are not recommended for major portions of the cores of earthfill dams. Pt. soils are unsuitable.<sup>4</sup>Rapid drawdown will not affect the upstream slope of a zoned embankment that has a large upstream pervious shell.

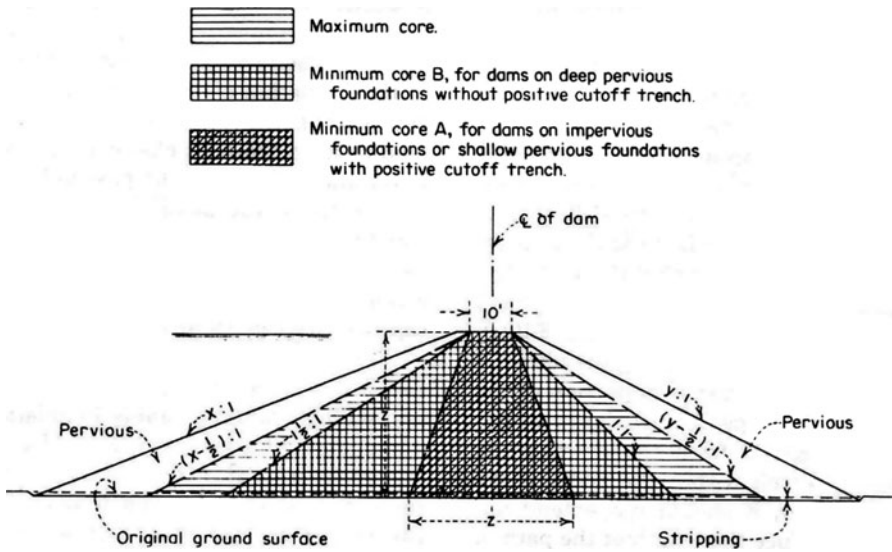


Fig. 6.7 Size range of impervious cores used in zoned embankments.

Table 6.7 Factor of safety

Sl. No.	Conditions of test	Factor of safety	
		Normal loads	With earthquake forces
1.	End of construction	1.25	1.0
2.	Steady state condition (D/S Slope)	1.5	1.25
3.	Sudden drawdown condition (U/S slope)	1.25	1.0

Table 6.8 Slope provided in some Indian dams

Dams	Height	Slopes provided ( H : V )	
		U/S	D/S
Nanaksagar	16.5 m	3:1	2.5:1
Panchet	56.5 m	3:1	2.5:1
Obra	30 m	2.26:1 in top portion 5.5:1 in lower portion	3:1
Tenughat	44 m	2.5:1 in top portion 3.75:1 in lower portion	2.5:1 in top portion 4:1 in lower portion

to an acceleration of  $\alpha g$  in the downstream slope direction. This gives an inertial force  $\alpha w$  which is assumed to be acting at the base of the slice. This force is also resolved in normal and tangential direction as shown in Fig. 6.9. The resultant force

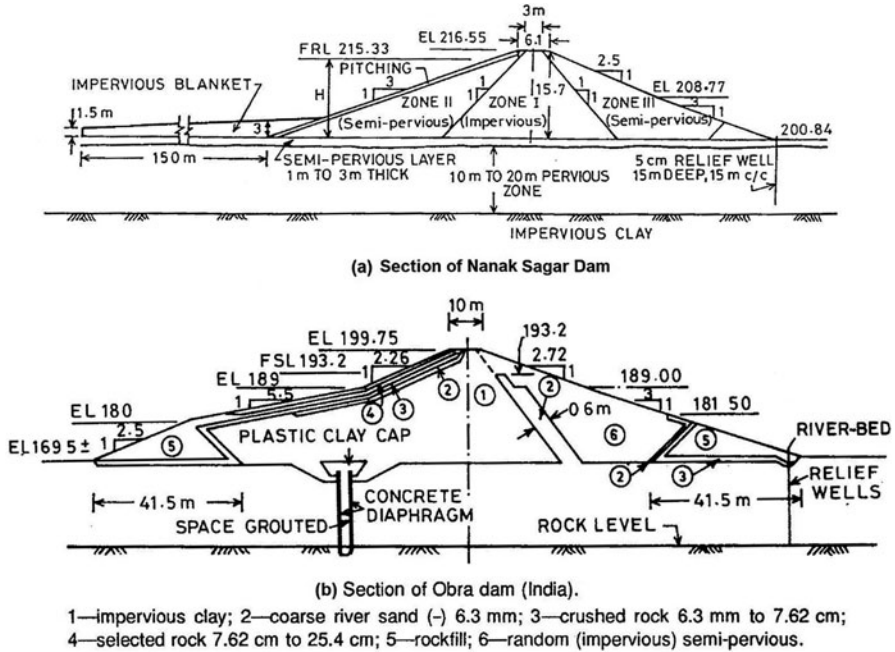


Fig. 6.8 Dam section of zoned embankments.

in radial direction is  $N - \alpha T$  and in the direction of slip surface it is  $T + \alpha N$ . Thus, the Fellenius expression for factor of safety is given by

$$F = \frac{\sum (N - \alpha T) \tan \phi + \sum C l}{\sum (T + \alpha N)}$$

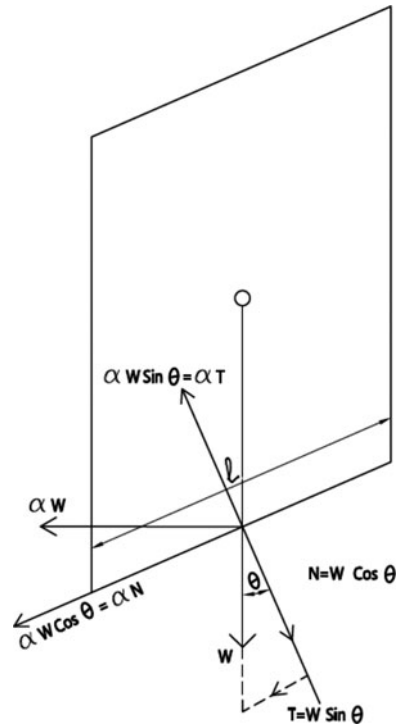
If effective stress approach with pore pressure is used, the expression will be

$$F = \frac{\sum (N - U - \alpha T) \tan \phi' + \sum C' l}{\sum (T + \alpha N)}$$

The value of  $\alpha g$ , the seismic horizontal acceleration appropriate for the use in the analysis of a specific project can be determined as per IS 1893-1975 depending on the zone in which the project falls and the importance factor.

The pseudostatic approach has its limitation as it does not include the change in the seismic acceleration in space and time and chooses a constant value of  $\alpha$ . The pseudostatic analysis is considered reasonably satisfactory for well built dams on stable foundations and where peak ground accelerations are not high (MCE not more

**Fig. 6.9** Seismic force acting on a slice.



than 0.2 g). The dynamic analysis shall be carried out for dams in highly seismic regions for both MCE and DBE or based on the actual seismic response spectrum. The FEM based computer program such as QUAKE/w from G-slope is being employed for such an analysis. The dynamic analysis is helpful in assessing (i) liquefaction potential of materials in dam and foundation, and (ii) permanent deformations.

In addition to the provisions required for the design based on dynamic response analysis of slopes, the following precautions shall be taken to prevent cracking and mitigate effect of cracking on the stability of dam.

- (i) The core should be moderately thick and core material should be highly erosion resistant and of good flexibility. If required it should be blended with pervious material.
- (ii) The filters should be sufficiently thick and of well graded material satisfying the filter criteria.
- (iii) The crest width should be more than usually provided.
- (iv) Excess settlement may occur during earthquake. Hence some additional provision should be made in the freeboard.

## 6.6 FILTER MATERIAL

Filter is an important component of zoned earthfill dam. It is generally made of natural material such as sand and shingle. The gradation of filter should satisfy the filter design criteria discussed in Chapter 5.

## 6.7 BERMS

The berms are provided in both upstream and downstream slopes of the dams. It breaks the continuity of the slopes and reduces surface erosion in downstream slope due to rains. A berm is also desirable at the top of the rock toe. It also prevents the damage to the lower end of rip-rap on upstream slope. These are also helpful during construction, operation and maintenance of a dam.

A minimum width of berm is recommended as 3.0 m. But wider berms are desirable from construction and maintenance considerations. One berm should be provided for every vertical elevation of about 10 to 15 m. The berm should slope towards the inner edge.

## 6.8 MEASURES AT INTERFACE WITH CONCRETE SURFACES

Sometime outlets are provided in the earthfill dams. In composite dams the spillway is provided in the central part of dam with earthen embankments on both sides. In both cases there is an interface between soil of earth dam and the concrete surface of outlet barrel/conduit and the abutments of the spillway.

In cases where proper compaction of soil at interface with concrete cannot be achieved, precautions and measures are required to be taken to reduce seepage and potential for piping along the interface of the outlet barrel, if the intake is provided in the body of dam, and the abutments of the spillway. Generally measures are designed to break the path of seepage along the interfaces. In the outlet barrels, RCC collars about 1.0 m high and 0.5 m wide are provided in the barrel at a spacing of about 3 m as shown in Fig. 6.10.

To check, seepage along the interface with the abutments of spillway one or two non-overflow blocks are embedded in the clay core of the dam body. In small height dams a concrete diaphragm wall 2 to 3 m wide and 15 to 20 m long is projected from the abutment which is embedded into the clay core. The typical arrangement is shown in Fig. 6.11. It is also essential that proper compaction should be ensured at the interface using small size hand operated compactors.

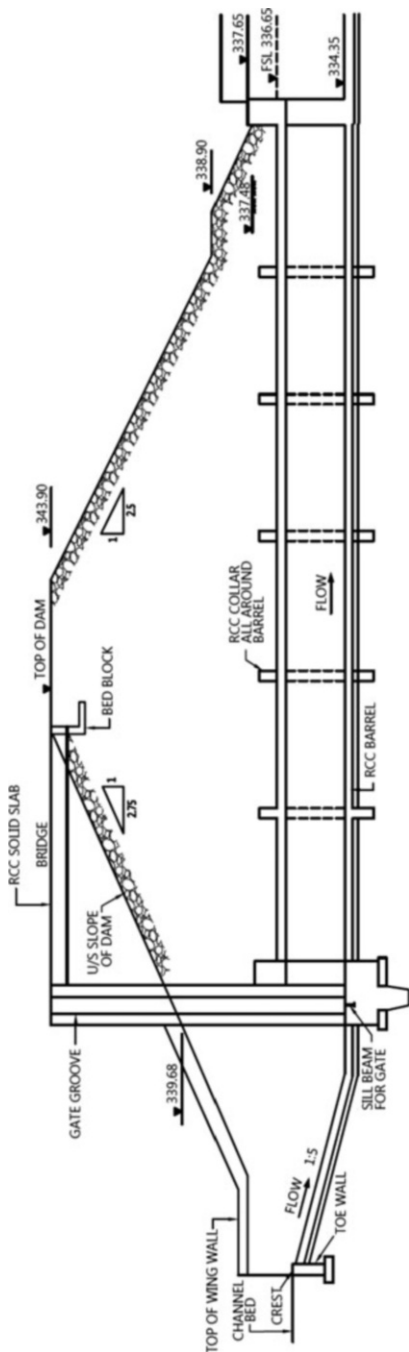


Fig. 6.10 Outlet barrel with collars in an earthen dam.

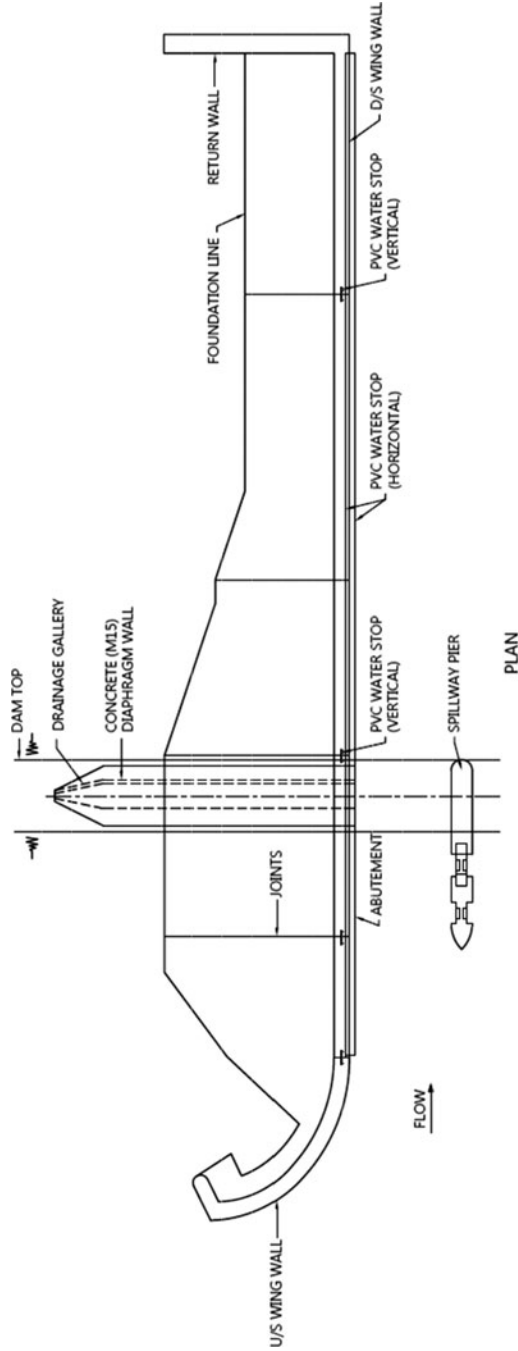


Fig. 6.11 Abutment and diaphragm wall.

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# Chapter 7

## Rockfill Dams

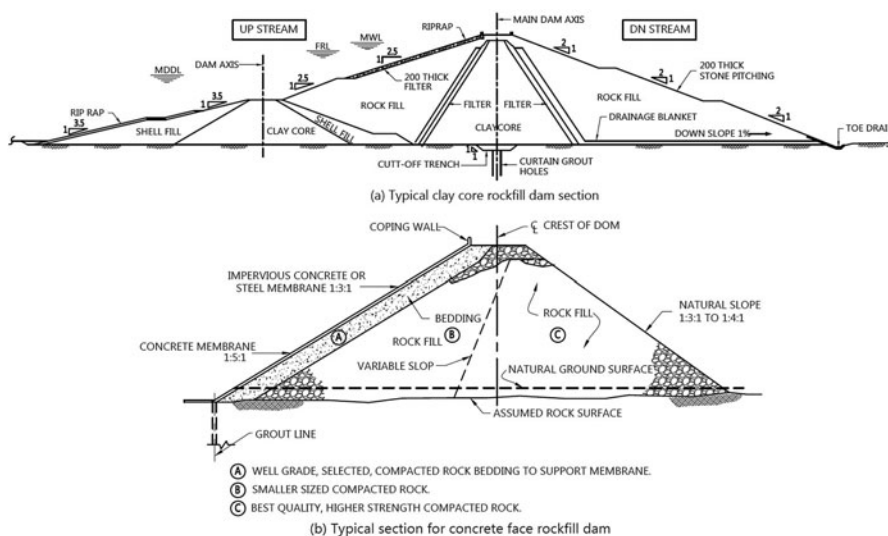


### 7.1 GENERAL

A rockfill dam is defined as ‘a dam that relies on rock either dumped in lifts or compacted in layers, as a major structural element’. In order to prevent seepage through rockfill, impervious membrane on the upstream face is provided. In small height rockfill dams constructed during nineteenth century, steel and timber were used on upstream face for water tightness. Around the turn of the century impervious membranes of cement concrete or asphaltic concrete began to replace timber or steel. The clay core rockfill dams were developed around 1940 and thereafter a large number of rockfill dams of both concrete faced and clay core type have been constructed all over the world with heights as much as over 300 m by the end of twentieth century.

The earlier concrete faced rockfill dams (CFRD) were dumped rockfill type. Typically the rockfill was dumped in thick lifts and were sluiced. This resulted in high deflection normal to the concrete face causing cracks in the concrete membrane and often in the damage of water seals at the joints of concrete membrane. The problem of leakage and maintenance became quite serious in such dams. The use of good quality rock without fines did not reduce the problem of extensive settlement and leakage. Later in about 1960's it was considered better to include fines and to compact the rockfill with the vibratory compactors in thin layers and the results were found extremely good. The deformations recorded were found about 1/10th of those found in dumped rockfill. This significant shift in construction technique paved the way for large height (ranging from 100 to 200 m) concrete faced rockfill dams. This reduced the requirement of good quality rock for rockfill.

The earth core rockfill dams (ECRD) are similar to zoned earthfill dams. In earth core rockfill dams the shell is made of rockfill. The construction of earth core rockfill dams of large heights started in USA in about 1940. Since 1960 ECRD progressed rapidly and the highest dam of this type (Nurek dam 300 m) was constructed in



**Fig. 7.1** Typical sections of rockfill dams.

Russia. Many dams of height between 200 to 300 m have been constructed. These have been provided with both inclined and central cores.

The compacted rockfill has been an essential part of both types of rockfill dams. The typical cross-sections of both CFRD and ECRD are shown in Fig. 7.1.

## 7.2 GENERAL FEATURES

The rockfill dams are basically embankment dams with highly pervious shell of rock or sand, gravel and boulder mix which can withstand much steeper slopes. The slopes are generally 1.3–1.4:1 (H:V) for CFRD and about 2:1 for ECRD. The water tightness is provided either by impervious membrane on U/S face or by clay core. These are founded on rock and can be constructed in narrow valleys. In these dams, spillway and outlets are not made part of main dam. When spillway is adjoining the main dam, the interface of rockfill with the concrete face of abutment wall needs special treatment. Flood diversion during construction is generally done for high frequency flood, say 1 in 100 years or more. It is done through the tunnels in the abutments. The U/S and D/S coffer dams made for flood diversion are generally made parts of main dam. The tunnels are later used either as spillway or as the outlet works.

The topographical, hydrological and geological investigations for rockfill dams are practically same as described for concrete dams.

### **7.3 MERITS AND DEMERITS OF CFRD AND ECRD**

- (i) The requirement of site conditions and foundation for the two types of rockfill dams are practically same.
- (ii) Both the types of rockfill dams require similar type of shell material. But in case suitable material for clay core is not available within reasonable distance, the choice is automatically for CFRD.
- (iii) The choice between the two types, if all other parameters permit both types, should be based on cost. Generally, ECRD appears to be economical but it is not always. The cost of ECRD depends on the availability of all types of material of quality and quantity and the distance of the borrow areas from the dam site.
- (iv) Since the material requirement and its processing is less in CFRD as compared to ECRD, the construction of CFRD is faster as far as the placement of rockfill is considered.
- (v) Future raising of CFRD is easier than ECRD.
- (vi) The clay core has unlimited life as it is not subject to deterioration as compared to concrete membrane which is subject to weathering.
- (vii) The CFRD is considered more seismic resistant than ECRD.

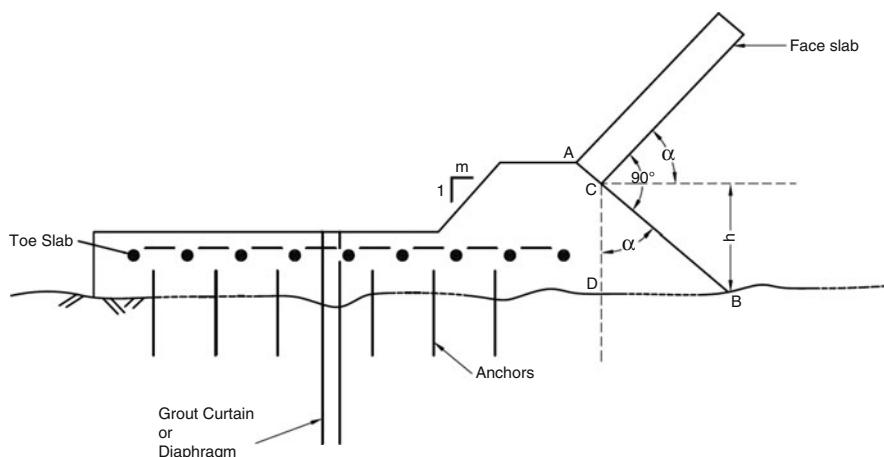
### **7.4 CONCRETE FACED ROCKFILL DAMS (CFRD)**

The main components of a CFRD are the following:

1. Toe slab (or plinth)
2. Face slab
3. Perimetric joint
4. Zoned rock fill

#### **7.4.1 Toe Slab**

The toe slab acts as the foundation support for face slab. Its dimensions depend on the stability, foundation conditions and the foundation treatment. The dimensions



**Fig. 7.2** General layout of toe slab.

generally vary with the reservoir head and foundation conditions. Excavation below the plinth depends on type of rock. Excessive excavation due to poor foundation should be backfilled with concrete.

The width of the toe slab generally varies from 3 to 10 m in good groutable rocks. It is also related with the head and is provided equal to  $0.04$  to  $0.05 H$  (when  $H$  is water depth). In poor foundation conditions, it may be  $0.3 H$ . If the rock is erodible, a shotcrete blanket 150 mm thick may be provided in the downstream of the toe slab in length equal to  $H/2$  to provide long seepage path.

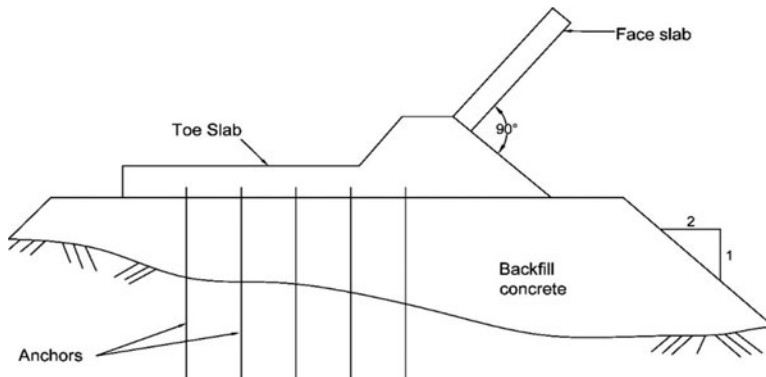
The thickness of the toe slab is generally equal to the thickness of face slab. A minimum thickness of 0.3 to 0.4 m is provided. According to reservoir head a thickness of  $0.3 + 0.003 H$  m may be provided. For construction convenience, a constant thickness is provided. Temperature reinforcement 0.3% each way at the top should be provided to minimize cracking. The reinforcement cover from top is recommended as 150 mm.

The toe slab is anchored to the foundation by anchor bars. These are arbitrarily provided to improve stability of the slab to act as grout cap. Anchors commonly used are 25 to 30 mm bars at a spacing of 1.5 m both ways with a depth of 3 to 5 m in the foundation rock.

The geometry of the horizontal toe slab (plinth) is shown in Fig. 7.2. Plane AB is kept normal to the face slab and point C is so located that its height above B i.e. ' $h$ ' is not less than 0.8 m. The value of  $m$  is taken as 1.3.

In case the foundation is required to be excavated deep below the toe slab the geometry of toe slab is shown in Fig. 7.3.

The stability of toe slab should be checked as an independent concrete block without taking support from the face slab and the rockfill. The stability under the water load, normal uplift must be checked against sliding and overturning. There



**Fig. 7.3** Toe slab on deep rock formation.

should be no tension on the upstream end of toe slab. At the downstream end of toe slab the uplift should be taken as zero.

The toe slab is also used as the platform for foundation grouting to provide a grout curtain as shown in Fig. 7.2. The anchors should resist the grout pressure.

At the junction of the face slab with the toe slab a proper joint with water stop is provided. It is known as perimetral joint.

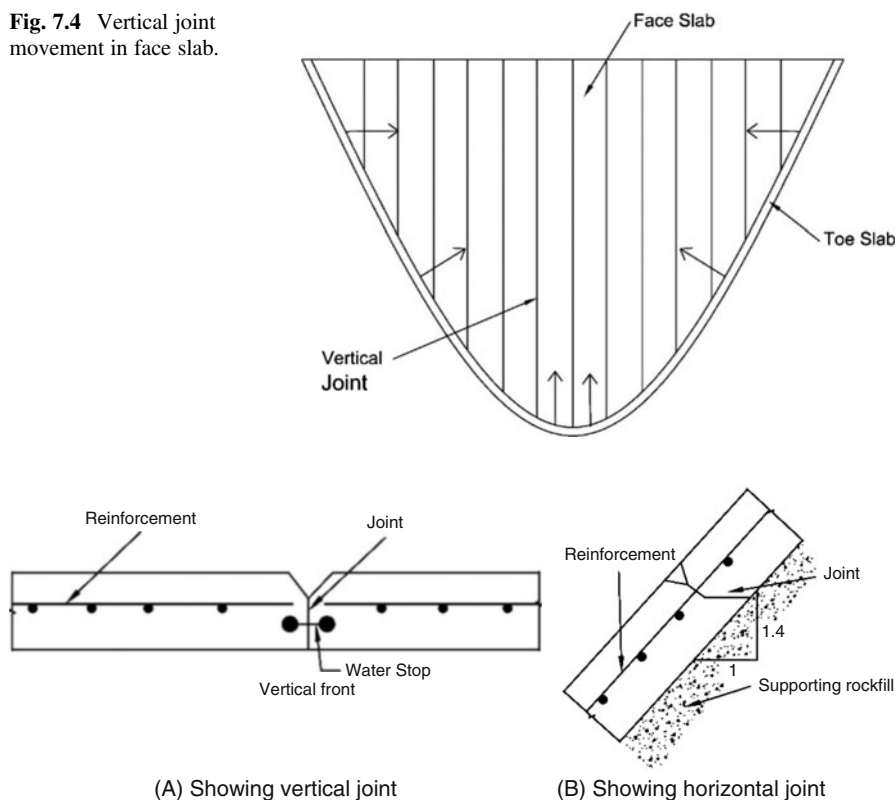
### 7.4.2 Face Slab

Face slab is a concrete slab laid on upstream face to provide water tightness to the CFRD. It is laid in panels of width ranging from 12 to 18 m but commonly a width of 15 m is used. The vertical joints between the concrete panels are provided with water stops.

In the CFRDs, face slabs are supported by a well compacted and well graded layer of crushed rock which provides continuous support under water load. This makes it possible to provide a thin slab. Current guidelines for the thickness of slabs for compacted rockfill CFRDs of height more than 100 m range from  $0.3 + 0.002 H$  to  $0.3 + 0.004 H$  (m) where  $H$  is head of water over the toe slab. For low height dams the thickness is around 0.3 m.

The face slab follows the deformation of the rockfill and it is observed that there is tendency for slabs to move towards the centre of the dam; the vertical joints would tend to close at the centre and open near the abutments as is shown in Fig. 7.4. The arrows in the figure show the direction of movement of the face slab. Vertical joints near abutment are designed as expansion joints just like perimetral joints with two water stops and in the centre as contraction joint with one water stop. There is discontinuity at the vertical joints.

**Fig. 7.4** Vertical joint movement in face slab.



**Fig. 7.5** Joints in face slab.

Horizontal joints are used to ease construction and when it is necessary to interrupt the construction. These are treated as construction joints with reinforcement crossing the joint. These are provided with or without water stops.

The typical vertical and horizontal joints are shown in Fig. 7.5.

The vertical and horizontal joint system provided at Cethana Dam in Australia is shown in Fig. 7.6. It shows continuous placement of vertical slabs. Generally no cracking is observed except temperature hairline cracks which are almost horizontal. These are nonstructural self-healing cracks and need no repair. Sometimes 10 to 15 mm wide cracks in high dams in the central part at the middle one third height or near the abutments are observed due to rockfill settlement. Such cracks are structural cracks and become the cause for excessive leakage and should be repaired by grouting, placing fine material, replacing concrete etc. Precautions should be taken during construction to avoid structural cracks. The rockfill base supporting the face slab should be well graded, well compacted and of adequate thickness. The thickness of the rockfill base should be increased near the abutment. The reinforcement in the face slab should be placed 100 to 150 mm from the top.

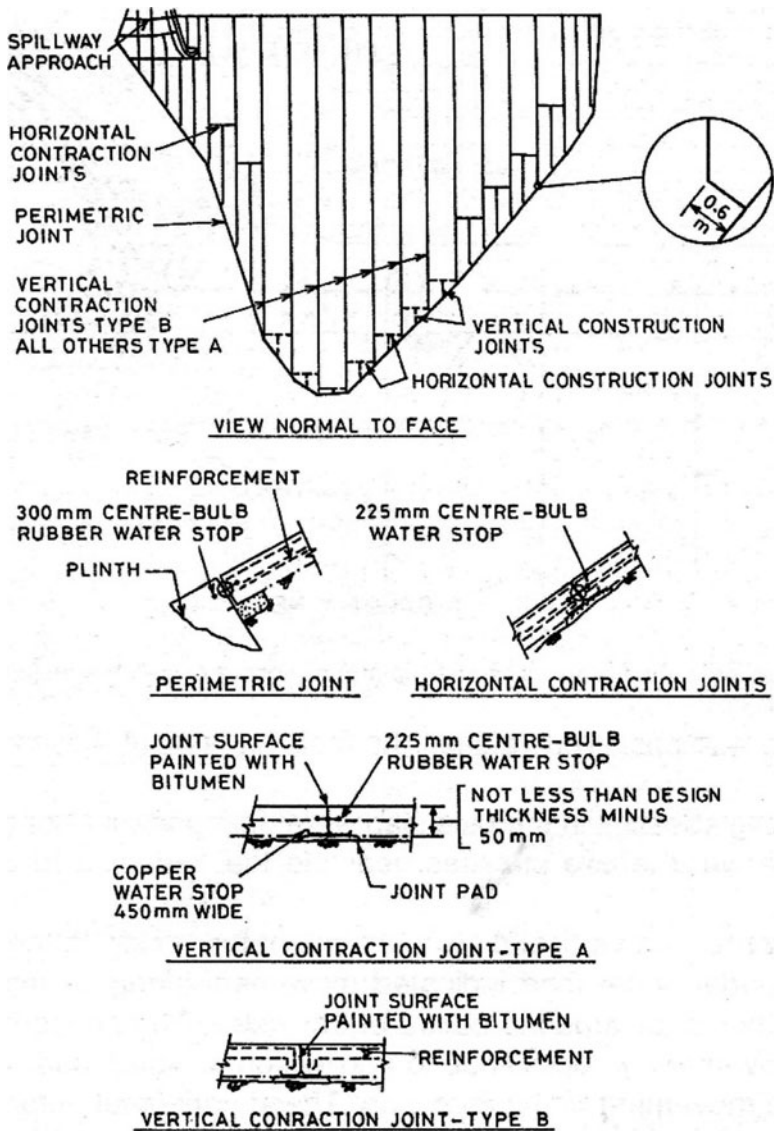
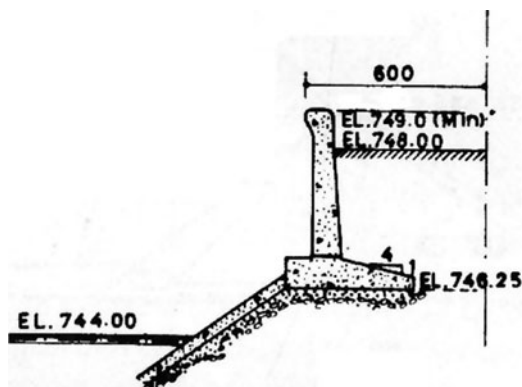


Fig. 7.6 Cethana Dam (Australia). Layout of joints and details. (Source: Singh and Varshney)

The reinforcement in the face slab is commonly provided equal to 0.4% of the concrete area in each direction. It may be reduced to 0.3% in low height dams. But it should be slightly more in the vicinity of the perimetral joint. Small diameter bars at closer spacing should be used to prevent cracking. The slab is strengthened near the joint to prevent crushing of corners by compression against adjacent slabs or perimetral joint. For strengthening, one or two additional layers of reinforcement

**Fig. 7.7** Parapet wall of Areia dam.



extending 2 m or so from the edge of the joint should be provided. This strengthening is generally not considered necessary in dams of height less than 50 m.

Concrete of grade  $M_{20}$  is considered adequate for the face slab. Richer mix is not considered desirable due to tendency for cracking. Maximum size aggregate to be used is generally limited to 40 mm. The aggregate should be tested for alkali aggregate reactivity. The aggregate to be used for face slab should be of the same quality as required for any concrete dam.

The concrete face slab being smooth surface does not offer resistance to the wave run-up. Hence a freeboard more than normally required should be provided in such dams. The required freeboard for the CFRD about MWL is thus measured upto the top of the parapet and so parapets as high as 3 to 4 m are generally provided in CFRD. The section of a parapet wall of Areia dam is shown in Fig. 7.7.

### 7.4.3 Perimetral Joint

The perimetral joint is between the toe slab and face slab. It provides a water tight seal against full reservoir head and allows the anticipated movement of the face slab due to the settlement of its supporting rockfill during and after the filling of reservoir. This movement of face slab tends to open the joint and it becomes a source for leakage. Hence this joint needs careful design and construction. It has been observed that the joint movement in dams upto a height of 100 m is under 30 mm and in higher dams it may go upto 100 mm.

In order to prevent leakage through the perimetral joint water stops usually two in number are provided: one of PVC and the other of copper at the bottom of joint. The joint is covered at the top by the mastic filler. The function of mastic filler is to prevent leakage when large joint movement takes place. The mastic filler is covered by a membrane which is bolted with concrete to make it a water tight seal and to transfer the water pressure to the mastic. Thus, there are three barriers for preventing leakage. This type of arrangement at the perimetral joint is now preferred

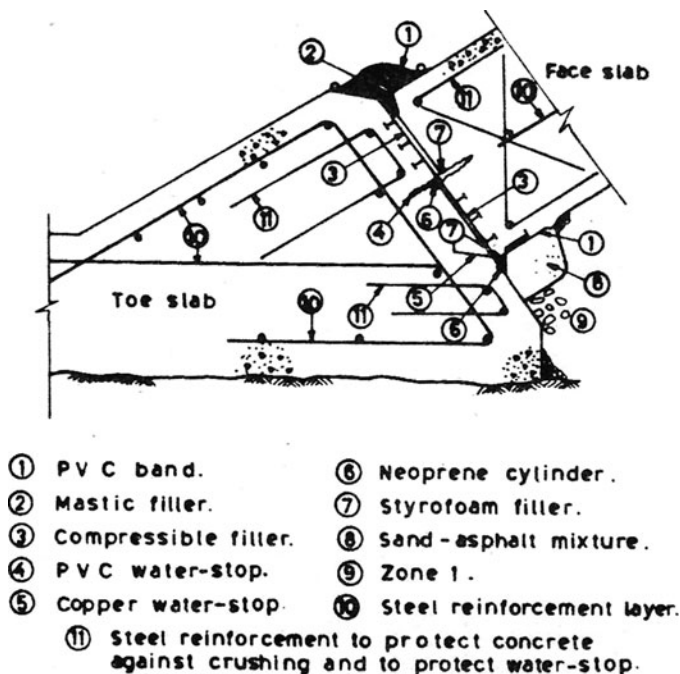


Fig. 7.8 Details of the perimetral joint at bottom of canyon.

irrespective of height of dam. The details of such a joint provided at Salvajina dam (148 m high in Colombia) are as shown in Fig. 7.8. In several small height dams the central PVC seal has been successfully eliminated. The copper or metal seal which is provided at the bottom is made of 0.8 to 1.2 mm thick sheets. The thicker sheet should be used for high dams. The middle water barrier is flat or dumbbell shape PVC water stop. The centre hollow bulb type PVC water stop is considered preferable due to the ability to undergo greater deformation. The timber board filler 12 to 20 mm thick is inserted between the toe slab and face slab (as shown in Fig. 7.8), to provide a cushion on which the face slab may rest during construction. This will also provide protection against spalling of concrete and damage to seals during deformation and movement of face slab.

In several high dams mastic filler water barrier at top of joint has been replaced by fine sand which may flow in the opening of joint and prevent leakage. In Areia dam (160 m high in Brazil), an additional precaution in the form of a cylindrical tube of neoprene of 5 cm diameter is placed at the bottom of the mastic filler to safeguard against the expected large movement. It is shown in Fig. 7.9. It is also the practice that the mastic filler in high dams is covered with impervious soil layer protected by less pervious soil as shown in Fig. 7.10.

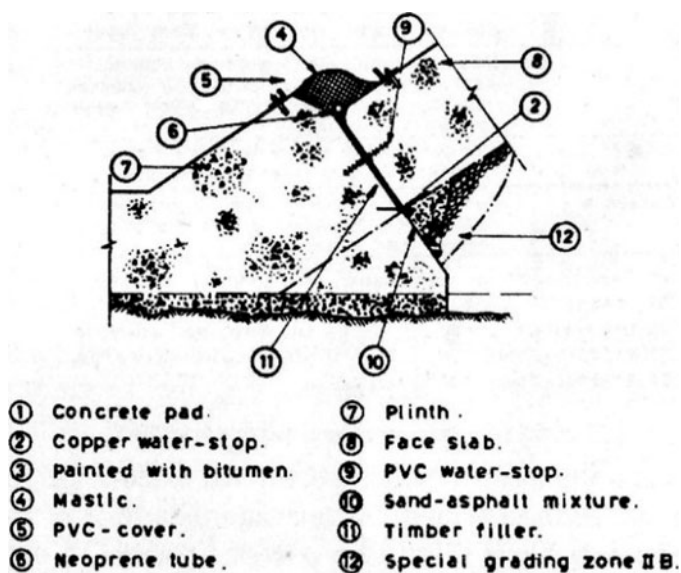


Fig. 7.9 Perimetral joint details of Areia dam.

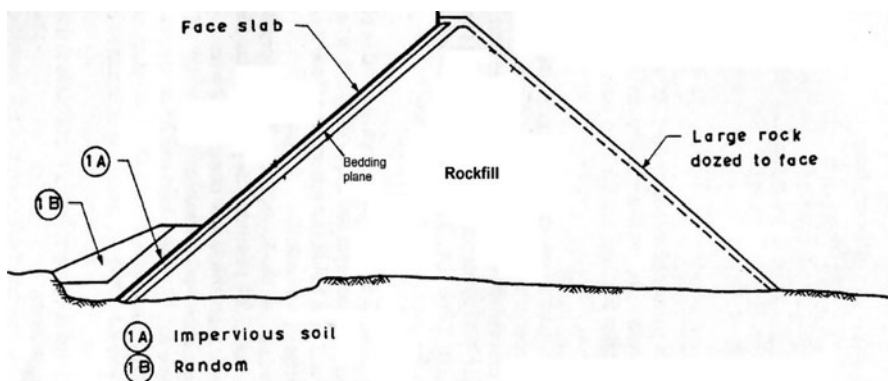


Fig. 7.10 Joint protection with impervious soil.

#### 7.4.4 Zoned Rockfill

A compacted well graded crushed rockfill is provided in the downstream of face slab which acts as watertight membrane. Hence, there is no pore pressure problem in the rockfill and fully drained shear parameters of the rockfill will provide stability to the downstream slope. Therefore, as a practice, no stability analysis is carried out and the

**Table 7.1** D/S slopes of CFR dams in seismic area

Earthquake magnitude	Peak base acceleration	Peak crest acceleration	Average D.S. slope for displacements 0.6 m or less	Average D.S. slope for displacement of 0.3 m or less	
6½	<0.1 g	<0.25 g	1.35	1.4	Areas of low to moderate seismicity
6½	0.15 g	0.45 g	1.4	1.4	
7½	0.15 g	0.45 g	1.4	1.4	
8½	0.15 g	0.45 g	1.45	1.45	
6½	0.3 g	0.75 g	1.5	1.5	
7½	0.3 g	0.75 g	1.55	1.6	Areas of high seismicity
8½	0.3 g	0.75 g	1.65	1.7	
6½	0.5 g	1.0 g	1.55	1.55	
7½	0.5 g	1.0 g	1.6	1.65	Areas of very high seismicity
8½	0.5 g	1.0 g	1.8	1.8	

slope is adopted on the experience of the existing dams. The slope commonly adopted is 1.3 H: 1 V and is found safe for well compacted rockfill of good quality rock with friction angles between 45° and 50°. In some dams even steeper slopes 1.2 H: 1 V have been used and in rockfill of weak rock for dams of height above 120 m a flatter slope of 1.4 H: 1 V are used. These slopes have resulted in a factor of safety of about 2.0 by conventional slip circle method of limiting equilibrium approach of stability analysis. Thus the criterion for selection of slopes shall be based on

- The height of dam: somewhat flatter slopes for dams of height above 120 m.
- The quality of rockfill: flatter slopes for poorer quality rockfill.
- The seismicity of the region: Flatter slopes shall be selected for dams in highly seismic zones.

The effect of seismic acceleration on CFRD dams is limited to the deformation of the embankment due to lateral spreading caused by shaking. On the basis of analysis, Seed et al. have given guidelines (Table 7.1) for the selection of downstream slopes for dams in highly seismic areas.

The displacements given in the table are estimated values and in actual movement in dams bulging of downstream slope and its contribution in flattening of upstream slope are observed. The significant movement is in upper parts of dam because of stronger motion at the top. Hence flatter slopes could be provided in top portion to reduce such movement.

#### A. Material for Rockfill

The rock to be used as rockfill should be of good quality but the specifications need not be as rigid as for concrete aggregate. It should be sound and should not be liable to weathering and should not get crushed under the load, to which it will be

subjected. Igneous and metamorphic rocks are generally suitable but shale, slates, schists and limestones should be avoided in medium and high dams. In several projects, river bed deposits of boulder, gravel and sand have been satisfactorily used. The rock should not disintegrate during alternate wetting and drying. The quarried rock supporting the construction equipment is supposed to be suitable. The maximum size to be used is limited to the lift size which varies from 1 to 1.5 m. Hence the maximum size should be 0.8 to 1.0 m. For well graded and well compacted rockfill the gradation should be such that not more than 50% is smaller than 2.5 cm and fines (silt and clay) are less than 6% but the material in between these sizes should be evenly graded.

The shear characteristic of rockfill which is important from design consideration is the angle of internal friction. It lies commonly between  $35^\circ$  and  $45^\circ$ . It should be tested for the design of high dams because the shear strength of rockfill reduces with the increase in confining pressure. The effect of confining pressure and maximum particle size on the angle of internal friction for crushed basalt rock on the basis of laboratory tests (Design of small dams) are shown in Fig. 7.11.

The USBR study reveals that:

- Rockfill material can be successfully modelled and strength characteristics can be obtained from small scale tests.
- At a specified confining pressure the angle of internal friction slightly reduces with the increase in size of particle.
- The angle of internal friction decreases with the increase in confining pressures for a given particle size.

This variation in angle of internal friction is of importance for the designer of a high dam where confining pressures are high. For design of small and medium height dams a fixed value of  $45^\circ$  is reasonable.

## B. Zoning of Rockfill

The section of a CFRD rockfill can be divided into three major zones as shown in Fig. 7.1.

Zone A:

A zone of fine well graded material to provide bedding for the face slab and to check seepage in case the face slab cracks.

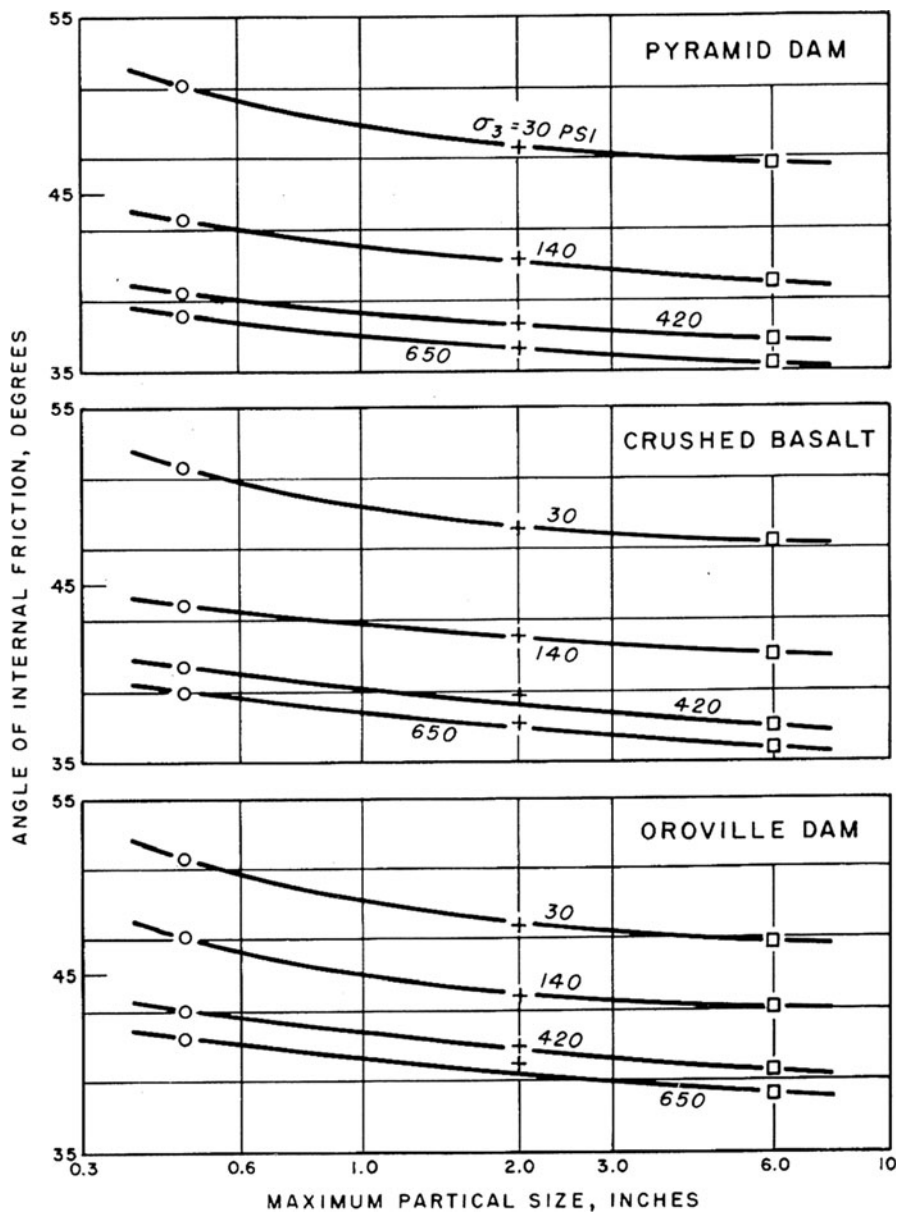
Zone B:

Transmits water load to foundation and should be of low compressibility and should be well graded and well compacted.

Zone C:

This is the major zone to give stability to the dam section but has little influence on the deformation of the face slab.

**Zone A:** Since it acts as a bedding for the face slab, it should give a smooth and uniform well compacted surface for the face slab. It commonly consists of fine well



**Fig. 7.11** Effect of maximum particle size and confining pressure on angle of internal friction. (Source: Design of Small Dam USBR)

1—Asphalt-concrete core wall; 2—Coarse rock waste; 3—Rockfill; 4—Impervious curtain; 5—Bedrock.

graded material of maximum size 7.5 cm to 3.5 cm range well graded down to sand size with 5 to 10% passing through No. 100 sieve (0.15 mm) and less than 5% passing through No. 200 sieve (0.074 mm). The material should be semi pervious with permeability in the range of  $10^{-3}$  to  $10^{-4}$  cu/sec. The width of this zone is usually 4.0 m and in high dams it is increased at the base. It is normally placed in layers of 0.5 m thickness.

**Zone B:** It should be well graded from the maximum size of 0.8 to 1.0 m depending on the lift (layer thickness) and should be well compacted and should have good permeability (more than Zone A and less than Zone C).

**Zone C:** This zone should grade from fine rock in the upstream to coarse rock in the downstream with largest and strongest rock pieces placed in the lower downstream portion of this zone. The rock is placed in layers of 1.5 to 2 m thickness and usually compacted by four passes of a vibratory roller.

### **C. Foundation for Rockfill**

The rockfill dams are normally founded on rock but the requirement is not as rigid as is for a concrete dam. The shell zone of rockfill can be placed after removing the weathered rock. The shell zone can also be placed on compacted river bed material if it is strong enough to support the load being transferred to foundation by the rockfill without excessive settlement. In such a case the slope stability should be checked inclusive of the foundation with its shear strength. The details of foundation treatment of rockfill dams is discussed in Chapter 9.

#### **7.4.5 Asphaltic Concrete Membrane/Central Core**

Many rockfill dams have been constructed with asphaltic concrete membrane on the upstream face in place of RCC slabs. It is more flexible and can tolerate larger settlements than concrete membrane. It has proved to be more economical and dependable alternative to concrete membrane. One advantage is that there are no joints to treat as is the case with concrete membrane but the exposed face of asphaltic membrane has to be protected in some cases depending on climatic conditions. The recommended upstream slope ranges between 1.7 H: 1 V and 1.75 H: 1 V for the compatibility with the construction equipment. The Zone A (Fig. 7.1) material should be well graded free draining rock material to eliminate uplift pressures during sudden drawdown. The gradation of Zone A material should be smaller than Zone B material. A well graded and well compacted base or leveling course with a minimum thickness of 150 mm should be provided below asphaltic concrete layer. The maximum size aggregate in the base course is 2.5 to 5 cm. It should be well graded down to 5 to 7 percent passing sieve No. 200 and well compacted.

The asphaltic concrete used is made of well graded aggregates from maximum size of 2.5 to 3.8 cm to fine sand and about 10% of fine rock dust passing sieve No. 200. The gradation used in Montgomery dam in USA is as below:

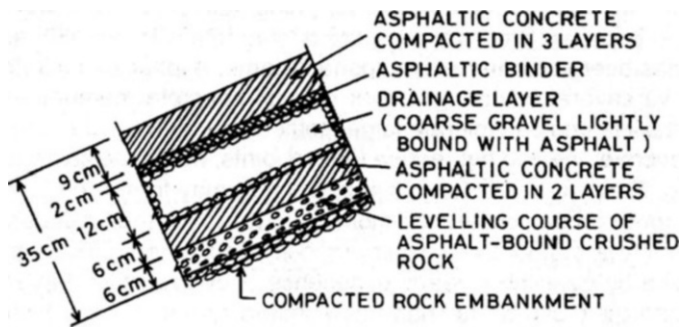


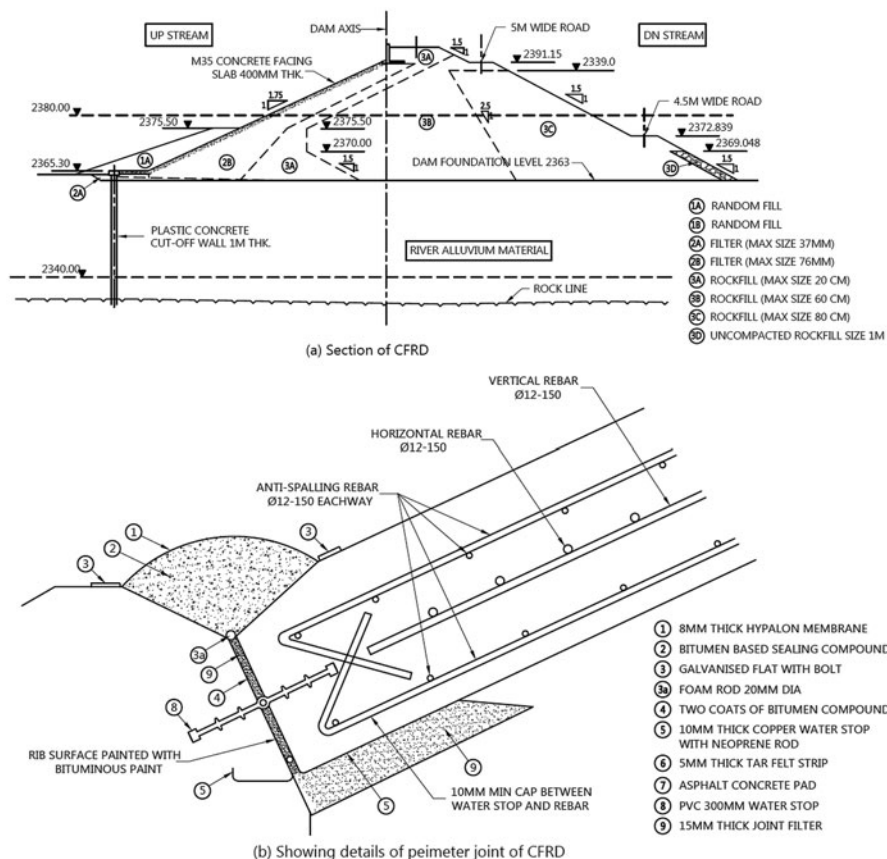
Fig. 7.12 Asphaltic concrete membrane for Genkel dam (Germany).

Fig. 7.13 Kolyma dam (CIS), 125 m high. Internal membrane.



Size	Percentage passing
38 mm	100
20 mm	86.4
12 mm	96.5
Sieve no 4 (4.75 mm)	58.7
Sieve no 40 (1.7 mm)	30.2
80	19.2
200	12.8

Pure asphaltic binder used is 8 to 10% by weight of aggregates. The aggregates and the binder are mixed at temperatures of 160–210 °C and placed hot to form the impermeable layers. The layers are placed in 2 to 3 m wide strips by carpet laying machines and compacted by smooth drum vibratory rollers shortly after placing. The required thickness is made by several layers of equal thickness. Details of an asphaltic membrane provided in 43 m high Genkel dam in Germany are shown in Fig. 7.12. It can be seen that a drainage layer is provided between two asphaltic concrete membranes. Later, after the experience of Genkel dam, the drainage layer was omitted in Grane Dam (67 m high) in Germany. The current practice in USA and elsewhere has been to favour one asphaltic concrete membrane comprising several layers laid over the surface of the dam carefully levelled and treated. It is found simpler in construction.



**Fig. 7.14** The dam section and perimetral joint of CFRD.

The asphaltic concrete membrane constructed on Iril-Emda dam (75 m high in Algeria) consisted of 2 layers each 12.5 cm thick resting on porous concrete and the top surface was protected by a layer of reinforced cement concrete.

Bituminous concrete has also been used for making central core in a rockfill dam. Some of the dams of height above 100 m have been constructed with bituminous concrete cores and have performed satisfactorily. The thickness of the core is generally varying from 50 cm to 150 cm. The thickness at the base is more than at the top. The mix is made of graded aggregates and asphalt. The aggregate is about 80% with half sand and half gravel of maximum size 20 mm and 10% is filler of minus 0.075 mm and 8 to 10% is asphalt. Transition zones are provided on either side of the bituminous concrete core to provide support and to act as filter in case of leakage. A section of Kolyma dam in USSR is shown with Central bituminous core in Fig. 7.13.

### 7.4.6 CFRD in India

CFRD has recently been adopted in India. One such dam is in operation in Sarda valley. One in Chenab valley has recently been commissioned and another is under construction in Testa valley. These are of height ranging around 50 m. The details showing the typical dam section and perimetral joint of a completed dam are shown in Fig. 7.14. The thickness of face slab is 40 cm. The rockfill section is divided into three zones following the criteria discussed in para 7.4.4(B). Since the dams are founded on alluvium and soft rock in lower Himalayas, deep (at deepest section) concrete membrane cutoffs have been provided under the toe slab varying from 40 to 70 m. The details of cutoff are also shown in dam section (Fig. 7.14).

## 7.5 EARTH CORE Rockfill DAMS (ECRD)

A large number of earth core rockfill dams have been constructed all over the world with height as large as 335 m (Rogun dam in CISR). The main components of ECRD are:

- Upstream and downstream rockfill shells.
- Central or sloping clay core as barrier against seepage.
- Transition filters on both sides of the core to check piping of core material.
- Drainage arrangement.
- Protection of upstream slope of rockfill shell from wave action.

The rockfill shell material requirement is same as for a CFRD and has been discussed above. Similarly the foundation requirement is also same as for a CFRD. The shape location, and material of clay core is same as in zoned earthfill dam section. Similarly the filter criterion has also been described in chapter of embankment dams. In many ECRD major dams well graded designed filters are provided such as Nurek, Ramganga and Tehri. But in several other important dams such as Ooville, Mica and Beas provision of a single transition zone of graded material is placed between core and the shell. The transition zone is made comparatively thick (about 10% of water depth). However, a layer of sand is placed by the side of core to provide a sealing for any potential crack in the core. The upstream slope is protected by adequately thick layer of dumped rock of good quality which should not disintegrate during alternate wetting and drying. Big size stones shall be dozed on the downstream slope. The downstream shell is self draining but arrangement should be made to collect and drain seepage flow. A typical section is shown in Fig. 7.1. The guidelines for design are continued in IS 8826.

### 7.5.1 Slope Stability Analysis

For earth core rockfill dams slope stability is determined as for other embankment dams using the slip circle method. The Wedge method of analysis is also adopted in which the slip plane instead of being circular is considered to be made of two or three straight lines. The results are generally found quite close in both the methods. Therefore generally slip circle approach being simple is followed for slope stability. It has been described in earthfill dam Chapter 6 (Refer IS Code 7894 for stability analysis).

In addition to limit-equilibrium analysis, in important high dams, a finite element analysis (FEM) is also carried out to determine stresses and deformations inside the dam mass (refer Chapter 6 also). This helps in finalizing the slopes and taking measures to prevent cracking and excessive settlements and deformations. In dams in narrow valley 3-D FEM analysis is more appropriate. In high dams in seismically active areas dynamic response analysis using FEM is also useful in refining the design. The slope stability analysis requires the slopes to be tested and the shear characteristics of material to be used in dam construction. The shear parameters of materials are obtained from field and laboratory tests. The slopes for stability analysis are assumed on the basis of existing similar height dams in same seismic zone. The slopes provided in some high earth core rockfill dams are given in Table 7.2 which may serve as a guide for the slopes to be adopted in a dam to be designed.

### 7.5.2 Clay Core

The analysis and experience have shown that differential movement at the interface of the core and shell results in cracking parallel to dam axis. Similarly, the fill adjacent to relatively vertical abutments or where there is abrupt change in slope, the differential settlement causes transverse cracks perpendicular to dam axis near abutment. It is experienced that additional moisture in clay core improves

**Table 7.2** Slope provided in some ECRD dams

Sl. No.	Dam	Location	Height (m)	Slopes (H:V)	
				Upstream	Downstream
1.	Nurek	Tadzbikistan	300	2.25:1	2.2:1
2.	Chicoasen	Mexico	261	2.1:1	2:1
3.	Mica	Canada	242	2.25:1	2:1
4.	Rogun	CIS	335	2.4:1	2:1
5.	Tehri	India	260	2.5:1	2:1
6.	Oroville	USA	230	2.6:1	2:1
7.	Ramganga	India	130	2.5:1	2.5:1
8.	Tarbela	Pakistan	143	1.81&2.65:1	2:1

deformability of the core. Hence it is better practice to use blended high moisture content clay for the core in the upper half height adjacent to the steep abutments to minimize cracking. The moisture content in such areas should be 2 to 3% over the optimum moisture content.

### **Design of Core**

Designing of core includes selection of material, thickness of core and location of core in dam section.

#### **1. Core Material**

The core shall be made of a material which is sufficiently impervious to check seepage, flexible enough to resist cracking under hydrostatic pressure because the reservoir head acts on core and shall be erosion resistant so that soil particles do not move under seepage pressure. It has been established by tests in laboratory and from the experience on other dams that well graded soils with about 20% fines (clay) with plasticity index (P.I.) around 15 are considered suitable for core. The suitable clayey soil has value of  $\phi$  varying from  $25^\circ$  to  $35^\circ$  and permeability of  $1 \times 10^{-5}$  cm/sec or less. However, soil of high compressibility shall be avoided.

Erosion resistance increases with compacted density but not the flexibility. The flexibility does not increase with increase in P.I. but can be improved with the presence of about 10% granular soil and if not available naturally then blending can be done. The recommendations of Bharat Singh regarding the core material are as below:

- (i) The rate of erosion decreases with increasing plasticity index (P.I.) upto a value of 15, after which the influence is small. High compacted density reduces rate of erosion.
- (ii) The addition of bentonite significantly improves erosion resistance, particularly in well graded soils. The addition or inclusion of stone chips to the extent of 10 to 20% also improves erosion resistance.
- (iii) An attempt should be made to select soil with a P.I. between 15 to 20%. Stone chips upto 10 to 20% should be included. Maximum size should be limited to compaction thickness. Soil should be compacted to high density.

For suitability of soil for construction of zoned dam section refer Tables 6.1 and 6.2. IS Code 12169 may also be referred.

#### **2. Core Thickness**

Core thickness varying from 1.0 H to 0.1 H (H is the head of water) have been used. A thickness consistent with the safety of dam should be adopted. The requirement of thickness is governed by the quantity and quality of the core material available within economic distance. The core material has less shear strength than previous shell. It is also subject to high pore pressures which further reduce the stability of core. If the available material is erosion resistant and is of good flexibility and low permeability, smaller core thickness can be used. Thin core should be provided with well-designed filters of adequate thickness on both sides. Thin core is more prone to cracking and high seepage gradient may cause erosion. A review of

**Table 7.3** Hydraulic gradients in the cores

Dam	Type of core	Country	Height (m)	Approximate hydraulic gradient (H/B)
Aswan	Vertical	Egypt	110	2
Blowering	Vertical	Australia	112	1
Blue Mesa	Vertical	U.S.A.	92	1
Chivor	Inclined	Colombia	238	3
Copeton	Vertical	Australia	109	1.2
Darmouth	Vertical	Australia	183	1.4
Hojjes	Inclined	Norway	81	2
Jindabyne	Inclined	Australia	72	3
Kajakai	Vertical	Afghanistan	100	2
Kennei	Inclined	U.S.A.	100	3
Llyn Brianne	Vertical	U.K.	90	3
Mica	Inclined	Canada	235	3
Mont Cenis	Inclined	France	81	2
Nantahala	Inclined	U.S.A.	80	9
New Don Pedro	Vertical	U.S.A.	180	2
Ord River	Vertical	Australia	98	2.5
Sayansk	Vertical	CIS	225	5
Talbingo	Slightly inclined	Australia	162	1.3

H/B (Maximum head/Core thickness)

experience on existing dam reveals that satisfactory thickness is 0.5 to 0.33 H with hydraulic gradient between 2 and 3. Hydraulic gradient provided in some dams is given in Table 7.3. Sherard recommends as below regarding core thickness.

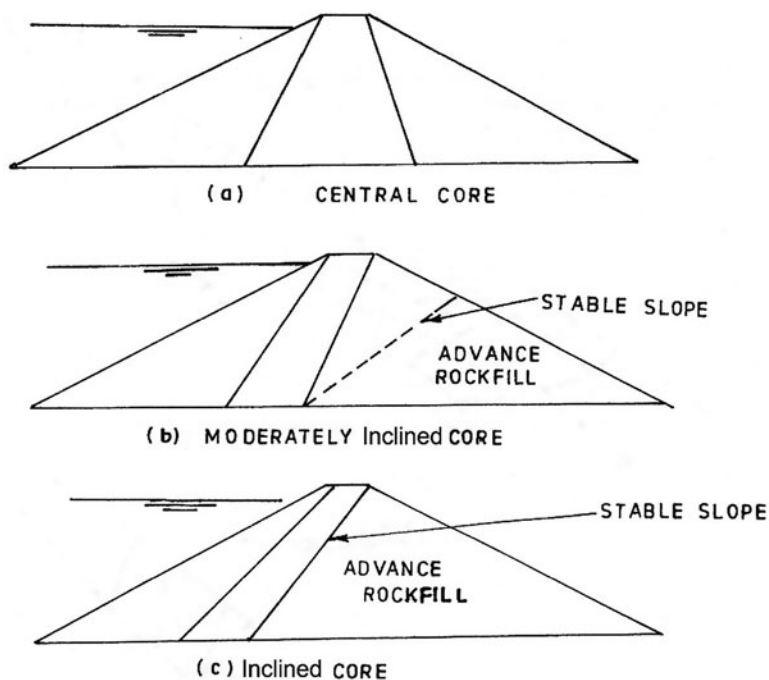
- (i) A core width of 30 to 50% of water head is satisfactory for any soil type and dam height.
- (ii) A core width 15 to 20% of water head is considered thin and is considered satisfactory if protected by properly designed filters.
- (iii) A core width less than 10% has not been widely used and is not considered satisfactory because large leaks may cause dam failure.

Generally a thick core is considered safe, more stable under high head and offers better contact with foundation. It is also advisable to increase the thickness of the core near the contact with abutments.

### 3. Location of Core

In various dams, cores centrally located, moderately inclined or inclined as shown in Fig. 7.15 have been used. Details of cores of some dams are given in Table 7.4.

The comparative merits and demerits of both central and inclined cores are as below:



**Fig. 7.15** Core location in a Rockfill Dam.

Sl. Central No.	Inclined core
1. It exerts higher pressure at contact with foundation, so possibility of seepage is reduced.	The downstream embankment can be placed independently and core can be placed afterwards.
2. Foundation treatment is usually required to be carried out before starting dam fill placement. In some cases, grouting of foundation and contact surface is done after construction of dams by boring vertical holes from a gallery in foundation	Foundation grouting can be done while downstream shell placement is in progress.
3. It will require slightly flatter downstream slope.	It is practically free from steady state pore pressures resulting in steep slope for downstream shell. The sudden drawdown pore pressures are also reduced but a large part of core is the part of slip circle as compared to central core resulting in flatter upstream slope.
4. It is prone to cracking	It can be more stable against cracking because of presence of large downstream shell.

The experience has shown that the total quantity of material in the embankment is practically same with both types of core. Some studies have shown that inclined core

**Table 7.4** Some important details of the core of some important embankment dams

No.	Dam	Country	Height (m)	Type of core material	Inclination	U/S slope (H:V)	D/S slope (H:V)
1	Nurek	Russia	300	Natural mixed of rocky clay	V	0.25:1	0.25:1
2	Chicoasen	Mexico	261	Clay-gravel mixtures	V	0.35:1	0.2:1 (vertical in nearly 60 m lower part)
3	Tehri	India	261	Blended material (clay with pebbles)	Vi	0.5, 0.4 & 0.3:1*	−0.1, 0.05 & 0.1:1*
4	Mica	Canada	244	Glacial till	Vi	0.4:1	−0.1:1
5	Oroville	U.S.A.	235	Cobble, gravel + clayey sand	Vi	0.9:1	−0.5:1
6	Keban	Turkey	207	Clay	V	0.16:1	0.16:1
7	Gepatch	Austria	153	Natural moraine clay (+1% bentonite in u/s portion)	V	0.25:1	0.25:1
8	Inferillo	Mexico	148	Sandy clayey silt medium plasticity	V	0.0891:1	0.0891:1
9	Tarbela	Pakistan	145	Gravel, sand silt mix	I	1.8:1	−1.22:1
10.	Ambuklao	Phillipines	131	Quarried loam	V	0.25:1	0.25:1
11	Miboro	Japan	130	Disintegrated granite + clayey soil	I	1.5:1	−0.85:1
12	Ramganga	India	126	Crushed clay shale	V	0.25:1	0.25:1

\*From upper part to lower part respectively; V = Vertical (central); Vi = Moderately inclined; I = Inclined and (−) = Sloping upstream or negative downstream.

is better for seismic shock resistance but these are not conclusive and opinions differ. Some details of dams having central core and inclined core are given in Tables 7.5 and 7.6, which show that both types of cores are being adopted in seismic regions.

**Table 7.5** Vertical core embankment dams

No.	Name of dam	Country	Height of dam (m)	L/H	Approx. maximum Hyd. Gradient	B/H	Blending/mixing/ processing condition	Remarks on seismic condition
1.	Nurek	USSR	300	3.0	2	0.5	Yes	Seismic shocks might be of magnitude as high as 8–9
2.	Chicoasen	Mexico	261	1.1	2.38	0.4	Yes	Highly seismic
3.	Darmouth	Australia	180	4	1.4	0.7	Yes	Within seismic area
4.	Trinity	USA	164	4.56	0.55	1.8	Yes	The dam is in zone 3 (1973 seismic zone map)
5.	Palo Quemado	South Africa	160	2.31	1.52	0.7	No.	High seismic condition
6.	Swift	USA	156	5	2.11	0.5	Yes	Maximum earthquake of magnitude 5.5 happened
7.	Geoscheneralp	Switzerland	15	2.32	3.08	0.3	Yes	–
8.	Gepatsch	Austria	153	3.95	3.7	0.3	Yes	–
9.	Infiemillo	Mexico	148	2.32	4.93	0.2	No.	Frequent and intensive earthquakes, and high seismic zone condition
10.	Netzahuaocoyotl	Mexico	137.5	2	2	0.5	No.	Within seismic zone
11.	Beas	India	132.6	14.5	0.7	1.4	Yes	Highly seismic zone region
12.	Ambuklao	Philippines	131	2.85	2	0.5	No	Safety factor due to earthquake has been considered
13.	Mornos	Greece	130	4	1.37	0.7	No.	The tectonic structure has an intensive plate characteristic with strongly folded cyclone and anticlyne

L = Length of dam along the axis at the top; B = Base width of core and H = Height of dam.

**Table 7.6 (a)** Moderately inclined core embankment dams

No.	Name of dam	Country	Height of dam (m)	L/H	Approx. maximum Hyd. Gradient	B/H	Blending/mixing/ processing condition	Remarks on seismic condition
1.	Rogun	CISR	335	1.53	2	0.5	Yes	High seismic and tectonic activity, intensity grade IX
2.	Bonucca	Costarica	302	2.99	2	0.5	Yes	Velocities vary; 1800–2800 m/sec
3.	Guavio	Colombia	245	1.6	2.94	0.3	Yes	–
4.	Mica	Canada	244	4	2.3	0.5	Yes	Core slightly inclined to counter earthquake
5.	Chivor	Colombia	237	1.3	3	0.3	Yes	High seismicity
6.	Croville	USA	235	8.99	2.94	0.3	Yes	A 0.1 g horizontal seismic was included in analysis.
7.	W.A.C. Bennet	Canada	183	11.2	1.1	0.9	Yes	–
8.	Cougar	USA	158	3.09	3.7	0.3	Yes	Seismic stability of the embankment was evaluated
9.	Round-Butte	USA	134	3.3	2	0.5	Yes	–
10.	Puebloviejo	South Africa	130	1.9	2.6	0.4	No	Very high seismicity

L = Length of dam along the axis at the top; B = Base width of core; and H = Height of dam.

**Table 7.6 (b)** Inclined core embankment dams

No.	Name of dam	Country	Height of dam (m)	L/H	Approx. maximum Hyd. Gradient	B/H	Blending/mixing/processing condition	Remarks on seismic condition
1	Tarbela	Pakistan	145	10	4	0.3	Yes	High seismicity and active seismic area
2	Miboro	Japan	130	3.3	1.33	0.8	Yes	High seismicity

L = Length of dam along the axis at the top; B = Base width of core; and H = Height of dam.

## 7.6 EXAMPLE OF DESIGN OF AN ECRD

The highest ECRD constructed recently in India is Tehri dam (260 m high). The design details of Tehri dam are described in subsequent paras.

### 7.6.1 Design Aspects of Tehri Dam Section

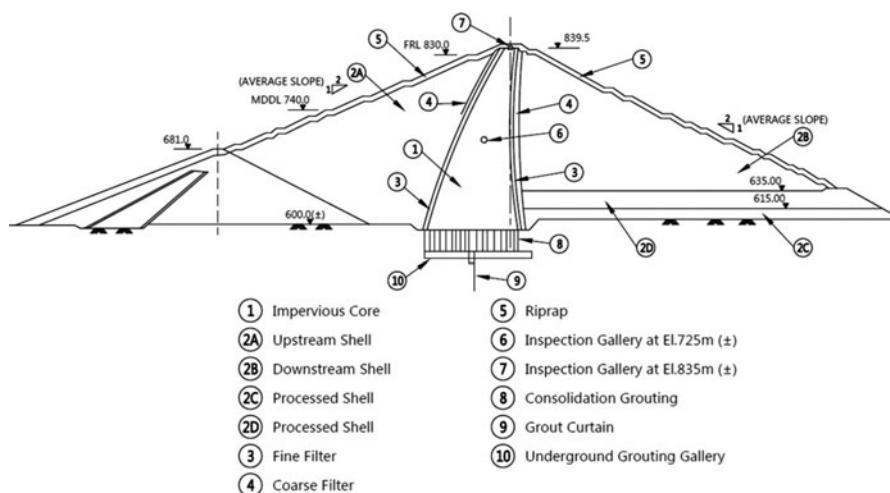
Tehri dam is 260.5 m high earth core rockfill dam (ECRD) located across river Bhagirathi at about 1.5 km downstream of its confluence with river Bhilangana in district Tehri of Uttarakhand (India).

The dam site is located in the Lesser Himalayas which lies tectonically between Main Central Thrust in the north and main boundary fault in the south. The project area falls in Zone IV of the seismic zone map of India and is highly seismic. Considering the history of earthquakes in the region, it was decided to design the dam for maximum credible earthquake of 8<sup>+</sup>.

The rock formations at the dam site comprise phylletic quartzites and quartzitic phyllites with bands of argillaceous materials. The rocks exposed in the gorge are sheared and closely jointed.

Initially the dam section was made of central impervious core, filters on both U/S and D/S of core and pervious shell zones and riprap to protect u/s slope. The average U/S and D/S slopes were designed as 2.5H : 1 V and 2.0H : 1 V respectively.

Locally available materials such as clay on river terraces and sand, gravel, boulder in the river bed have been used to build the dam. The clay available was CL soil and was found suitable for impervious core. However, in order to improve its shear properties and compressibility, clay was blended with gravel pebbles (60:40) by volume with maximum size of 200 mm in central portion of dam and 80 mm near abutments. The clay was placed in the vicinity of abutment at moisture content 2 to 3% above OMC to improve dam abutment interaction. Dry density achieved after compaction was 1.9 T/m<sup>3</sup>.



**Fig. 7.16** Typical section of Tehri dam.

The shape of the core was determined after carrying out stress strain analysis so that no plastic zones are formed and there is no excessive transfer of stresses between core and shell. The core width is  $0.4H + 10$  m at different levels where  $H$  is height of dam at that level. At top the core width is 10 m. It was increased to 15 m near abutments for better contact.

The maximum particle size in shell material is 600 mm boulder. Silt content which is upto 5% and fines ( $<4.75$  mm) are limited to 35%. Instead of horizontal filter drainage blanket below downstream shell to drain the seepage flow a more permeable shell material with fines limited to 10–20% is placed in thick layers. The shell is compacted to a high density of  $2.36 \text{ T/m}^3$ .

A two-layer filter is provided on both sides of core and designed to satisfy filter criteria. The filter material is obtained by processing river bed material.

A 10 m thick riprap of well graded (5 mm – 1200 mm) hard blasted rock has been provided on both faces of dam. The percentage of fines is limited to 3%.

### 7.6.2 Seismic Analysis

Initially the dam slopes were designed by pseudo static analysis using seismic coefficient of 0.12 g. Later considering the seismotectonic set up and the height of dam, the dynamic response stability analysis was independently carried out by Department of Earthquake Engineering (DEE), IIT Roorkee and Hydro Project Institute (HPI), Moscow.

The DEE, IIT (Roorkee) carried analysis by FEM for effective horizontal peak ground acceleration (PGA) of 0.25 g and vertical PGA of 0.125 g. The HPI

(Moscow) carried out non-linear (Static and Dynamic) stress analysis by FEM using Elasto-Plastic model for two accelerograms with worst response spectra, one with PGA of 0.5 g and other with PGA 0.4 g and optimized the section with several shapes of core. Both the studies established the safety of initially designed dam slopes under severe seismic conditions and finalized the shape of core. The typical dam section is shown in Fig. 7.16.

Later on, the suggestions and observations of various experts and organizations like HPI (Moscow) checked the dam section for the real observed accelerogram of Gazli earthquake (Russia) with vertical PGA of 1.36 g and horizontal PGA of 0.72 g acting simultaneously and found the section safe.

To accommodate some other experts observations, Department of Earthquake Engineering, IIT (Roorkee) carried linear dynamics stress-strain analysis taking seismic parameters upto about 1 g and confirmed the safety of dam section.

The studies have shown that the adopted section is safe under severe and extremely conservative seismic assumptions.

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# Chapter 8

## Failure of Embankment Dams



### 8.1 GENERAL

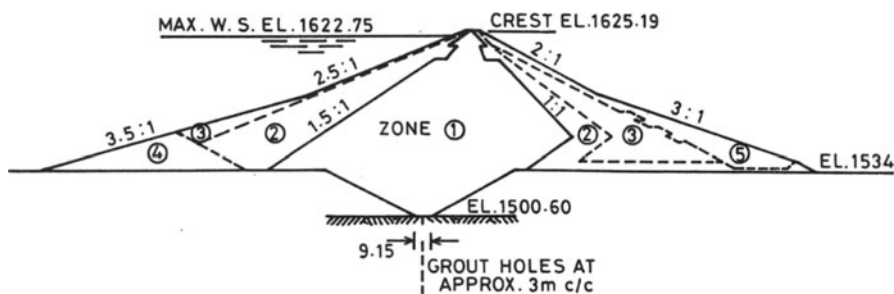
The safety of embankment dams depends on the adequacy of the measures taken to check seepage and piping through the body of dam and the foundation. These measures also include the drainage arrangement adequate enough for the safe exit of the seepage which takes place inspite of the provision of well-designed measures. It has been the experience that despite such measures some failures of embankment dams have taken place. The enquiries and investigations after the failures have revealed that the discrepancies in design and negligence in operation and maintenance of such dams are generally responsible for the failure. Some examples are briefly described in subsequent paras with the probable causes of failure. This may serve as the guidelines for design and maintenance of future projects.

### 8.2 EXAMPLES OF DAM FAILURE

#### 1. Teton Dam Failure

Teton dam (Idaho, USA), a 93 m high zoned embankment dam designed by the USBR, failed in 1976 on 5<sup>th</sup> June during first filling of reservoir. It was a significant event in the history of dam engineering as no dam of such height had failed earlier. The failure was extensively investigated by the US Department of Interior and the State of Idaho.

The foundation rock was highly jointed, joint width varied from 5 to 75 mm and occasionally as wide as 300 mm. The permeability was high. In river bed the bed rock was overlain by 33 m deep alluvium. A positive cut-off upto the bed rock by excavating and back filling with clay core material was provided in the central river section and a trench cutoff upto the rock and grouting was done in the abutment to check seepage through foundation.



**Fig. 8.1** Teton dam section through central portion of embankment founded on alluvium. 1—Silt with some clay, sand and gravel; 2—Selected sand, gravel and cobbles; 3—Miscellaneous fill; 4—Selected silt, sand, gravel and cobbles; 5—Rockfill.

In the dam section, a thick core with flat slope in upstream was provided. The zoned section in the river bed with material used in different zones is shown in Fig. 8.1.

It will be seen that there was no transition zone or filter between the core and the shell material. Similarly there was no filter by the downstream side of core trench in foundation.

A single row of grout curtain was provided with primary holes 3 m apart along the entire length of dam.

No instruments except settlement monuments were provided in the dam. Reservoir filling started in Nov. 1975. The rate of filling was 0.3 m per day. The unexpected runoff during the spring of 1976 and the delay in construction of outlet works resulted in a filling rate of 1.2 m/day in May 1976 and by 5<sup>th</sup> June the water level in the reservoir reached 0.9 m below spillway crest and 8 m below dam top.

Two days before failure, small water springs were observed in the river bed about 450 m downstream of dam. On 4<sup>th</sup> June water springs were also seen near the dam. On the fateful day of 5<sup>th</sup> June, the seepage through the downstream slope at two locations near the junction of dam with right abutment was observed and the seepage water was muddy. After a few hours the seepage discharge increased to about 500 lps (litre per second). Attempts to close the seepage holes on the dam face failed and the dam failure took place.

After detailed enquiries and investigations, the following were considered the principal causes of failure which were mainly responsible for internal erosion and piping.

- Inadequate treatment of open joints of bed rock at the contact with the impervious core.
- Unsuitable material for core. Silt in the core material was highly erodible.
- Absence of filter material between core and shell and between core trench and foundation material.
- Absence of instrumentation for monitoring the performance.

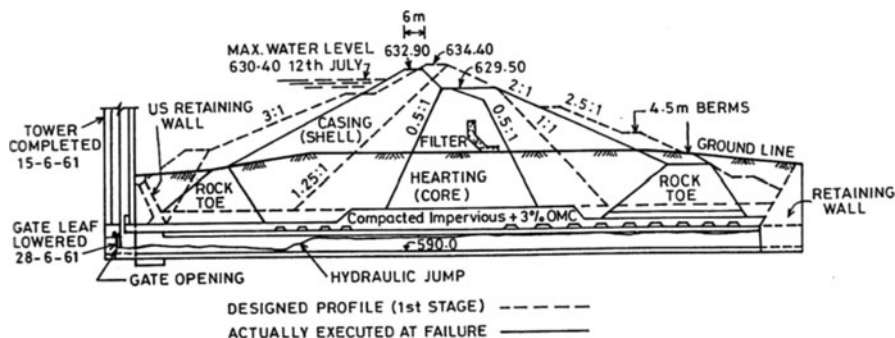


Fig. 8.2 Section of Panshet dam.

- Non-availability of water outlet to control reservoir filling.
- Provision of single row of grout curtain in highly jointed rock.
- Inadequate treatment at the junction of core with abutment.

## 2. Panshet Dam

Panshet Dam was a zoned embankment with thick clay core. It was under construction in stages in Maharashtra (India). Final height above river bed was 56.5 m. The first stage height of 51.3 m was near completion when the dam failed in July 1961. The flood wave due to Panshet dam failure caused a breach in the existing masonry dam built at Khadakwasla in its downstream and the combined flood wave caused lot of damage upto the town of Pune located about 20 km in the downstream.

An outlet conduit was provided in the dam with a tower intake. The outlet conduit was made of masonry with an arch roof and it was to be fitted with a steel pipe at a later date for power generation. The dam section is shown in Fig. 8.2.

For the first stage of Panshet dam, a masonry ungated spillway was provided at the saddle which was to be gated later in the second stage. Reservoir was to be connected through a channel with the spillway at the saddle. The first stage works were scheduled to be completed in January 1962 but it was rescheduled to be completed before monsoon of 1961 to take benefits of the storage of monsoon flows.

The conduit could be completed in April 1961. The clay was compacted around the conduit by May 18<sup>th</sup> and thereafter the 34 m deep gap in the embankment in the reach of the conduit was quickly completed between 18<sup>th</sup> May and 10<sup>th</sup> July but the shell portion remained 1.5 m and the core 5.0 m below the designed level. The upstream slope protection also could not be placed. The conduit entry was partially closed by gate. An opening of 0.6 m was left at the bottom. Spillway approach channel was not excavated in full width.

Heavy rainfall occurred in the catchment, 325 mm on 26/6, 210 mm on 9<sup>th</sup> July and 119 mm on 10<sup>th</sup> July. From 1<sup>st</sup> July to 12<sup>th</sup> July the reservoir had risen by about 24 m. The spillway due to inadequate approach channel could not pass the discharge equal to the design flood. Wave action caused the erosion of unprotected upstream

slope of conduit gap. Subsidence of shell portion and transverse cracks were observed in the dam. Due to rising water level overtopping of dam in the conduit gap portion took place resulting in failure of dam on 12<sup>th</sup> July.

The enquiries and investigations later revealed that the apparent cause of failure was subsidence of fill and overtopping of conduit gap portion of dam. The lessons which could be learnt from this failure are:

- The reservoir filling should not be started unless all works are complete and arrangement to control the rate of filling is ready.
- Avoid provision of an outlet conduit in an embankment dam.
- Adequate precautionary measures should be taken in design and construction of gap closure.

### 3. Nanak Sagar Dam

The dam is located on river Deoha in Nainital District (Uttarakhand), India. It was completed in 1962. It is a thick core zoned embankment dam about 18.5 km in length including both sides of centrally located masonry spillway. The maximum height of embankment is 16.5 m.

The foundation comprises an impervious stratum of thickness ranging from 1 to 3 m with 10 to 20 m deep pervious stratum underneath. Below pervious layer is again an impervious layer. Foundation treatment comprised of:

- (1) Upstream clay blanket – It is 150 m long from the upstream end of dam with thickness of 3 m at the junction with dam and 1.5 m at the upstream end. However, it was neither carried out under upstream pervious zone nor connected with the core.
- (2) Relief wells 5 cm diameter at 15 m spacing were provided at the toe of the dam. These were made 15 m deep to penetrate about 3/4<sup>th</sup> of the depth of pervious layer. No gravel filter was provided inside the relief well.
- (3) The above measures alongwith the dam section are shown in Fig. 8.3.

The dam was filled for the first time in 1962 upto RL 211.3 against the designed FRL of 215.33 m. Till 1966 the reservoir was not filled upto FRL. Since 1964 onwards boiling in the downstream of dam was observed at many places in long reaches on either side of the masonry spillway. Cracks in the downstream slope at several places alongwith settlement of rock toe were also observed. Repair measures were taken by loading the boils with filter material and reconstructing the drains. Bigger size (10 cm diameter) relief wells were constructed. But boiling conditions were observed every year after 1964.

The reservoir was filled upto the FRL for the first time in 1967. On 26 August 1967, boiling started at km 1.5 on the western flank. Loading of the boil resulted in the boiling from other nearby locations. The process continued till 7<sup>th</sup> September. The discharge from boils increased and two transverse cracks at the dam top 9 m apart had developed. Filter loading could not control the situation. Cracks suddenly widened and the dam top settled and a 180 m portion of dam was washed away.

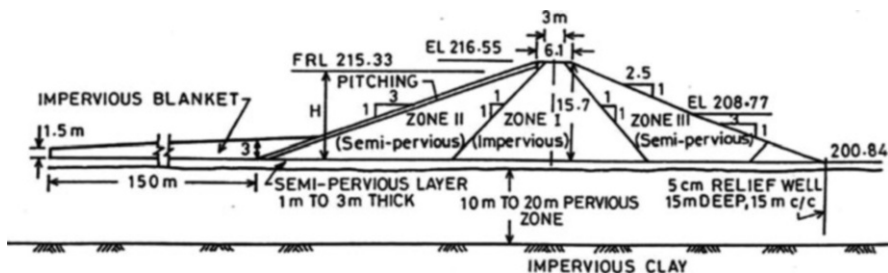


Fig. 8.3 Typical zoned section of Nanak Sagar dam.

The cause of the failure was found to be the piping through foundation resulting in settlement and overtopping of dam. The foundation treatment provided was not effective. Repairs were later carried out with modified design and the dam is since then functioning without trouble.

#### 4. Ahroua Dam

Ahroua dam was constructed as a homogeneous section of ML-CL soil of maximum height of 22.9 m in Mirzapur (U.P., India). It had 167.5 m long spillway on left abutment due to presence of good rock. The foundation for earthen embankment in the river bed comprised of sand and gravel layer upto a depth of 3.0 m over thick clay layer. The pervious material was removed before placing the dam fill. A masonry sluice with arch roof was provided in the dam body with gate hoist at the dam crest.

The dam was completed in June '53 and the filling was started. On 5<sup>th</sup> July heavy rainfall occurred in the catchment and the reservoir rose by 9 m. Leakage on downstream face near the sluice was observed in early morning. There was a whirlpool at the entrance of sluice. By 9 AM the dam breached in a length of 30 m and the sluice downstream of gate was damaged.

The cause of failure was clearly evident as the seepage and piping along the contact of sluice wall and the earth work of embankment. Due to piping, settlement took place resulting in overtopping and the failure of dam.

During repair gate shaft was located at the entry of sluice and the barrel was provided with collars and cutoffs. Thereafter the dam functioned satisfactorily. The sketch showing typical dam section before and after damage are shown in Fig. 8.4.

### 8.3 MACHHU DAM-II

Machhu Dam-II of 20 m height in Gujarat (India) was constructed in 1972 as a zoned embankment on the alluvial foundation with a positive trench cutoff upto the rock foundation on either side of a centrally placed masonry spillway of a capacity of

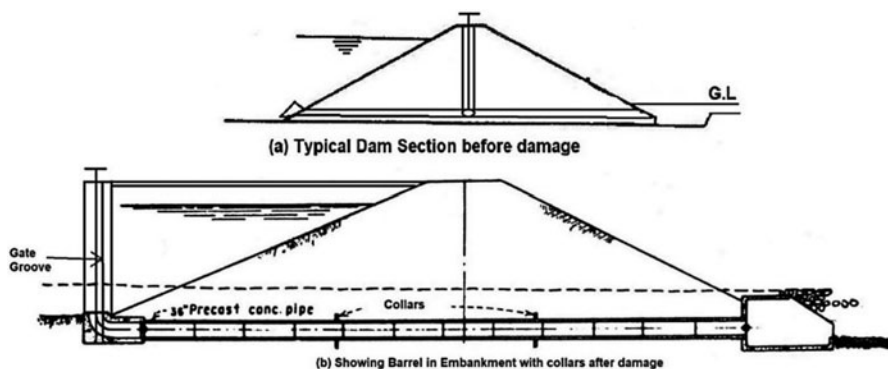


Fig. 8.4 Ahrouda dam breach.

$5550 \text{ m}^3/\text{sec}$ . Dam section is shown in Fig. 8.5. It was located 48 km downstream of a 30.5 m high masonry dam named Machhu-I.

Very heavy rainfall occurred in the entire catchment of river both above Machhu-I and in between the two dams during August 11 & 12, 1979. The average rainfall was about 530 mm in 21 hours.

The flood passed overflowing the crest of Machhu-I in the noon of 11<sup>th</sup> August and though the dam remained safe, the abutment was bypassed. The flood wave reached the Machhu Dam-II by 1.30 pm on 11 August and water level crossed the design high flood level and overtopped the dam and within two hours long stretches of earthen embankment on both sides of the spillway were washed away causing extensive damage to property and human life in Morvi Town which is located about 8 kms in the downstream. It is reported that the flood exceeded  $14,000 \text{ m}^3/\text{sec}$ .

Overtopping of dam embankment due to heavy flood is the apparent cause of damage.

## 8.4 LESSONS FOR FUTURE

The above are a few examples but many small height old embankment dams have failed in India. A list compiled by CWC of dam failures since independence is enclosed as Table 8.1. These examples of dam failure have shown that the main cause of failure is piping either through body of dam or foundation resulting in settlement of dam and its overtopping thereafter. Different modes of piping are conceptually shown in Fig. 8.6. In some cases overtopping of dam and its failure took place due to excessive flood and inadequate spillway capacity also.

These examples of failure of embankment dams highlight some aspects for careful consideration while designing, constructing and maintaining the dam.

- Conservative estimation of design flood and adequate provision for the same shall be made in the waterway of the spillway and the free board.

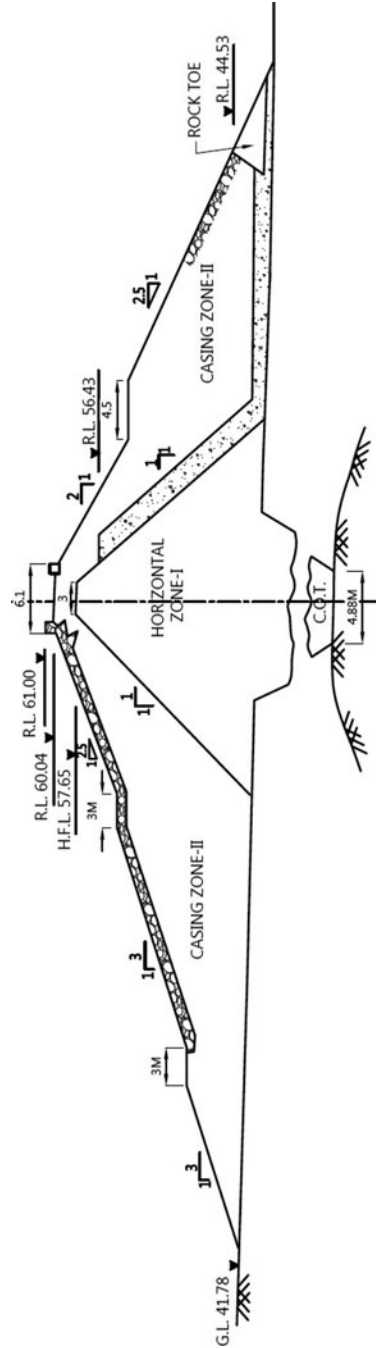


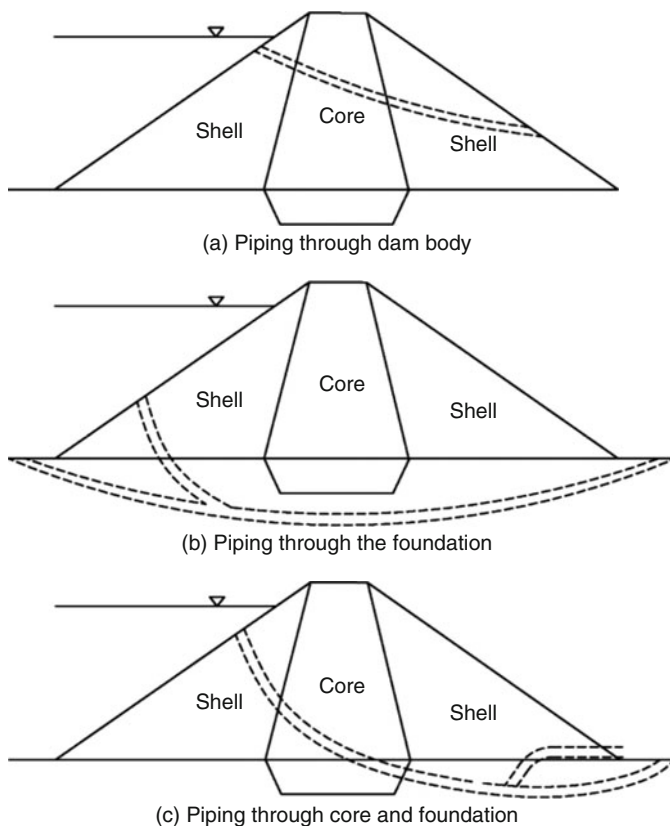
Fig. 8.5 Section of Machhu earth dam.

**Table 8.1** Examples of dam failure in India

Sl. No.	State	Name of project	Type	Max. height (m)	Year of completion	Year of failure	Cause of failure
1.	Madhya Pradesh	Tigra	Masonry	24.03	1914	1917	Overtopping followed by slide
2.	Maharashtra	Ashiti	Earth	17.70	1883	1933	Slope failure
3.	Madhya Pradesh	Pagara	Composite	27.03	1910	1943	Overtopping followed by breach
1951-1960							
4.	Madhya Pradesh	Palakmati	Earth	14.60	1942	1953	Sliding failure
5.	Rajasthan	Dakhya	Earth	NA	1953	1953	Breaching
6.	Uttar Pradesh	Ahrora	Earth	22.80	1953	1953	Breaching
7.	Rajasthan	Girinanda	Earth	12.20	1954	1955	Overtopping followed by breaching
8.	Rajasthan	Anwar	Earth	12.50	1956	1957	Breaching
9.	Rajasthan	Gudah	Earth	28.30	1956	1957	Breached due to bad workmanship
10.	Rajasthan	Sukri	Earth	NA	NA	1958	Breached by leakage through foundation
11.	Madhya Pradesh	Nawagaon	Earth	16.00	1958	1959	Overtopping leading to breach
12.	Rajasthan	Dervakheda	Earth	NA	NA	1959	Breaching
13.	Gujarat	Kaila	Earth	25.08	1955	1959	Embankment collapsed due to weak foundation
1961-1970							
14.	Maharashtra	Panshet	Earth	53.80	1961	1961	Piping failure leading to breach
15.	Maharashtra	Khadakwasla	Masonry	60.00	1875	1961	Overtopping
16.	Rajasthan	Galwania	Earth	NA	1960	1961	Breaching
17.	Rajasthan	Nawagara	Earth	NA	1955	1961	Breaching
18.	Madhya Pradesh	Sampana	Earth	21.30	1956	1964	Slope failure on account of inappropriate materials
19.	Madhya Pradesh	Kedamala	Earth	20.00	1964	1964	Breaching

20.	Utarakhand 1971-1980	Nanaksgar	Earth	16.00	1962	1967	Breached due to foundation piping
21.	Gujarat	Dantiwada	Earth	60.96	1965	1973	Breach on account of floods.
22.	Tamil Nadu	Kodaganur	Earth	12.75	1977	1977	Breached on account of floods
23.	Gujarat 1981-1990	Machhu-II	Composite	20.00	1972	1979	Overtopping due to floods
24.	Gujarat 1991-2000	Mitti	Earth	16.02	1982	1988	Overtopping leading to breach
25.	Madhya Pradesh	Chandora	Earth	27.30	1986	1991	Breached
26.	Andhra Pradesh	Kadam	Composite	22.50	1958	1995	Over topping leading to breach
27.	Rajasthan 2001-2010	Bhimlot	Masonry	17.00	1958	-	Breached due to inadequate spillway capacity
28.	Gujarat	Pratappur	Earth	10.67	1891	2001	Breached on account of floods
29.	Madhya Pradesh	Jamunia	Earth	15.40	1921	2002	Piping leading to breaching
30.	Orissa	Gurijoremip	Earth	12.19	1954-55	2004	The abutment structure along with wing and return walls got undermined with foundation scouring
31.	Maharashtra	Nandgavan	Earth	22.51	1998	2005	Excessive rain causing water flow over the waste weir to a depth beyond the design flood lift.
32.	Madhya Pradesh	Piplai	Earth	16.73	1998	2005	Breached
33.	Rajasthan	Jaswant Sagar	Earth	43.38	1889	2007	Piping leading to breaching
34.	Andhra Pradesh	Palemagu Dam	Earth	13.00	U/C	2008	Flash flood resulting in overtopping of the earth dam
35.	Madhya Pradesh	Chandiya	Earth	22.50	1926	2008	Breached

Source: CWC



**Fig. 8.6** Different modes of piping in dam.

- Avoid providing masonry sluice in the dam body. If it is to be provided in small dams, adequate collars in the barrel may be provided if adequate compaction cannot be achieved. Compaction of fill with hand compactors during construction should be done along the barrel to safeguard against seepage and piping. A graded filter may be provided on embankment slopes on both sides of the exit end of barrel.
- Adequate geotechnical investigations should be carried out for foundation and available construction material.
- Suitable foundation treatment should be planned, designed and constructed so that seepage is reduced and piping is safeguarded.
- Homogeneous dam section should normally not be adopted. Zoned section should be designed with adequate factor of safety with filter between core and shell zone.
- Adequate downstream drainage arrangement in all cases should be provided for the safe exit of seepage, both from the dam body and foundation.

- Reservoir filling at a slow rate should be planned. The filling should not be started unless flow controlling arrangement to restrict rate of rise due to sudden increase in river flow are not operational.
- Adequate instrumentation of dam is necessary.
- The contact of core with the jointed foundation rock and abutment should be properly designed and protected with filter against seepage and piping.
- Regular monitoring of instruments and inspection of the dam during first filling is very important.
- Flood forecasting, warning system and other dam safety measures should be planned and provided before starting the filling of reservoir.

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2. 'Design of Small Dams'. USBR, India edition 2011.

## Chapter 9

# Foundation Treatment of Dams



### 9.1 GENERAL

Dams are basically of two types: concrete dams and embankment dams. Concrete dams are concrete gravity dams, RCC dams, arch dams etc. Embankment dams are homogeneous earthfill dams, zoned type earthfill dams and rockfill type which are either CFRD or ECRD type. Dam foundations can be of varied type. Basically these are classified as rock type or alluvium type.

A particular type of dam may be suitable for a specific type of foundation, and the requirements for the foundation of a type of dam may also be varying for its structural stability.

The concrete dams require a strong and sound rock foundation capable of bearing the loads transferred by the dam on foundation. The joints in the rock should also be sufficiently watertight to prevent seepage of stored water. Depending on rock type and its properties and geological features the foundations, generally need treatment to meet the requirements of a concrete dam.

The rockfill type dams can be founded on any rock type foundation. The load transfer to the foundation is not the concern in this case. However, treatment is required to control seepage through foundation and to check the flow of fine particles of foundation material with the seepage flow.

The earthfill dams, homogeneous as well as zoned embankments are generally founded on alluvial foundations. Measures are, however, required to be taken to control seepage through foundation and to prevent flow of fine particles of foundation material which may otherwise result in piping, settlement, overtopping and finally the failure of the dam.

## **9.2 FOUNDATION TREATMENT FOR CONCRETE DAMS**

The foundation requirement for a concrete dam is for a strong and sound rock. It is very rare to get a rock foundation which is strong, watertight and free from geological weaknesses. It is an exception in Himalayas which is the youngest mountain range of sedimentary rocks. Generally rocks are jointed, highly permeable, stratified with weak zones like soft rock, shear zones, faults and fissures etc. Therefore, generally the concrete dam foundations need treatment to make them watertight, strong to bear the loads and safe in sliding. Weak zones often need special site specific treatment. The methods of foundation treatment generally adopted are discussed below.

### ***9.2.1 Normal Treatment***

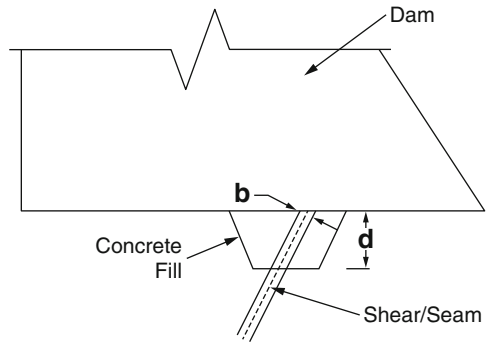
It is the treatment which is carried out in case of all the concrete dams founded on rock to improve the rock to meet the design requirements. It comprises shaping of foundation, dental treatment and grouting. Drainage is also important to control seepage and to reduce uplift.

#### **9.2.1.1 Shaping of Foundation**

The whole area of the dam is cleaned of vegetal cover and excavated to remove loose and weathered rock to reach the firm rock level over which the concrete shall be placed. The excavation near the level of firm rock should avoid blasting operation and instead jack hammers be used and cleaning should be carried out manually. The excavated surface at dam base should be made rough for better bonding with concrete. The abrupt change in elevation should be avoided. Some rocks disintegrate on exposure to air and water and in that case the excavated surface should be protected immediately after excavation either by concreting or by shotcreting.

#### **9.2.1.2 Dental Treatment**

It is generally experienced that after excavating the foundation upto dam base level thin seams or shears containing soft/crushed material upto considerable depth are exposed. These may not bear the load transferred by the dam. Hence these need treatment. Commonly the seam/shear is excavated out upto a certain depth and filled with concrete. It is known as dental treatment. The studies by USBR suggest the following for the depth to be excavated and backfilled with concrete.

**Fig. 9.1** Dental treatment.

$$d = 0.006bh + 1.5 \quad \text{for } h \geq 45 \text{ m}$$

$$d = 0.3b + 1.5 \quad \text{for } h < 45 \text{ m}$$

where  $b$  = width of seam/shear (m),  $h$  = height of dam above foundation (m) and  $d$  = depth to be excavated below rock foundation (m).

If the seam is a few centimetres wide the depth of the excavated will generally be 1.5 m. In clay gouge seams, the depth ' $d$ ' should not be less than 0.1  $h$ . The dental treatment is shown in Fig. 9.1. It is considered advisable to anchor the concrete back filled in the excavated area with the rock surface so that the backfilled concrete is monolithic with foundation rock. Alternatively the gap between rock and backfill concrete shall be grouted.

The above formula for depth is only a guideline and the final decision should be based on site specific conditions.

### 9.2.1.3 Grouting

Grouting of foundation rock with the concrete dam is done for two reasons, one is to create a barrier to seepage through the joints of the rock and the other is to consolidate the bed rock to improve its bearing capacity. The pattern of grout hole, the depth and grout pressure for both the purposes of grouting are different and depend on the quality of rock foundation. Grouting to create the seepage barrier is called curtain grouting and the one to consolidate the bed rock is called consolidation grouting.

#### (i) Curtain Grouting

Curtain grouting is generally carried out in case the water tests in the bore holes drilled during investigations show the permeability of rock more than 3 Lugeon. One Lugeon (Lu) is a water loss of one litre/minute/metre of hole at a pressure of 10 bars (1 MPa). It is also equivalent to a permeability of  $1.3 \times 10^{-5}$  cm/sec.

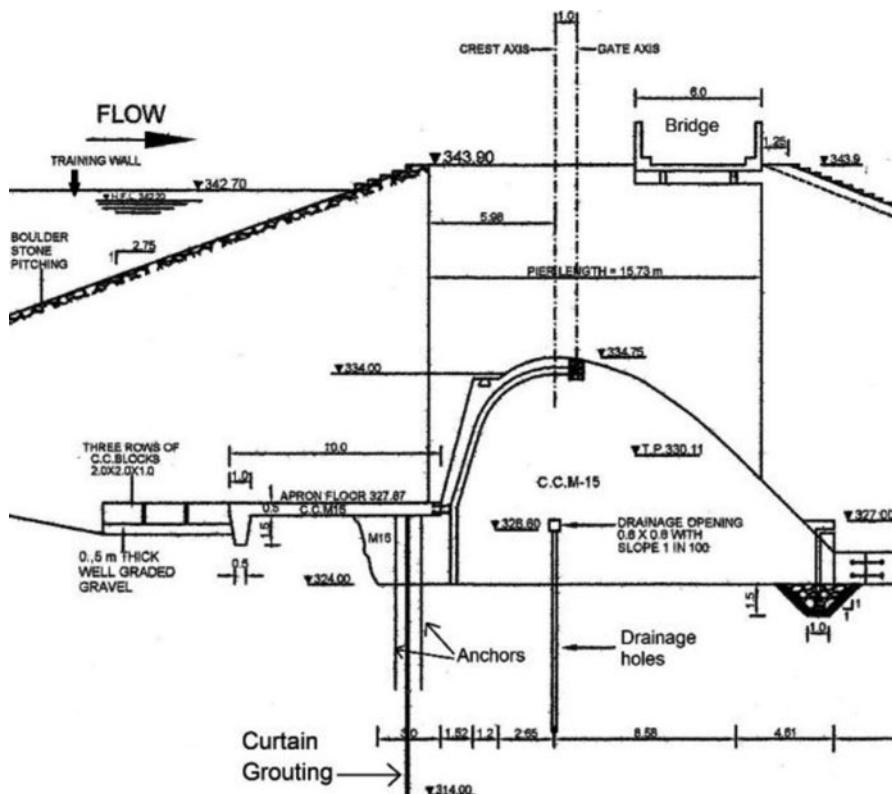


Fig. 9.2 Grouting through projected pad in a small dam.

Curtain grouting is carried out generally from the foundation gallery which runs along the longitudinal profile of the dam or parallel to the dam axis near the upstream face of dam. It may also be carried out from the grout cap or upstream fillet of the dam in case of a small gravity dam or arch dam. The typical arrangement in case of a small dam is shown in Fig. 9.2. In this arrangement the curtain grouting is done from a concrete pad projected from the upstream face of dam. The pad is anchored in the rock before grouting is done. The drainage of foundation is done through a small gallery in the dam body. The gallery runs along the length of dam and discharges in a sump at the end. This has been provided in several small and medium projects in Bundelkhand, Uttar Pradesh (India). In some cases, where abutments are steep, curtain grouting is done from the galleries constructed in the abutment at different elevations.

Generally a single row of grout curtain is provided. Multirow grout curtain is provided in Karstick rocks with clayey infilling. The grout holes are generally made vertical. These are made inclined as much as  $15^\circ$  upstream from the vertical in case these are drilled from the foundation gallery which is located at some distance from

the upstream face. The grout curtain is extended sufficiently inside the abutment by extending the gallery. It is sometimes turned in the abutments in the upstream side.

Grout holes may be of EX (37.5 mm) or NX (75 mm) size. Common practice is to use NX size holes for curtain grouting. These are made by using percussion or rotary drills. It is started with a spacing of 12 m. If the grout intake is more than 50 kg of cement, intermediate holes should be made in between and the minimum spacing of holes is generally 1.5 m. Some exploratory holes should also be made to explore the results of grouting. The water intake in the test results should be reduced to less than 3 lugeons in small height dams and 1 lugeon in high dams.

The depth of grout holes depends on the rock quality, head of water and the results of the water tests in the exploratory drill holes. The grout curtain holes should ordinarily be drilled upto a depth at which the water tests do not show intake of more than 10 lugeon. The general guidelines for the depth of grout curtain holes are as below:

- 30 to 40 percent of hydraulic height for good rock foundation.
- 70 percent of hydraulic height for poor rock foundation.

The USBR criteria for depth of grout curtain is

$$D = \frac{H}{3} + C$$

where  $D$  is depth of grout curtain,  $H$  is height of dam and  $C$  is a variable varying from 8 to 25 in metres.

The IS: 11293 suggests

$$\text{Depth of grout curtain} = \frac{2H}{3} + 8$$

where  $H$  is height of dam.

Considering the above guidelines, rock quality, and the water test results the decision on depth of grout curtain requires experience and judgement in each specific case.

Generally a water-cement mix (5:1) is used for grouting. Thick mixes say 1:1 are used for wide joints in the rock. Sand is mixed when cavities are to be filled. Grouting becomes difficult when the joints are less than 0.5 mm wide.

Grouting should be done at a pressure at which the foundation is not disturbed. Grout pressure should also be fixed for each depth zone depending on the geology. The thumb rule for determining grout pressure at the collar of the hole is that the maximum pressure should not exceed  $0.23 \text{ kg/cm}^2$  per metre depth of rock. For very sound rock, where joints are tight, the grout pressure might exceed the above value.

Generally two methods for grouting are in use depending on type of rock. One is descending stage and the other is ascending stage grouting.

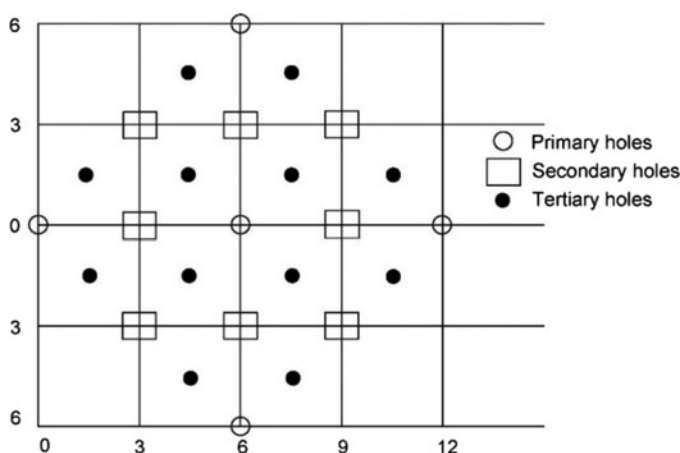
**Descending stage:** In this method, holes are drilled in two or three stages. The depth in each stage is kept practically same. In first stage the hole is drilled and

grouted. Thereafter the operation of next stage is taken-up. The grout pressure increases with the subsequent stages. In this method of grouting the drilling and grouting equipment is set on the hole again and again. The advantage is that the possibility of caving of hole is eliminated. Hence, it is suitable for weak rocks.

**Ascending stage:** In this method, hole is drilled to the required depth in one operation. Grouting is done in stages. It is started from the deepest portion of the hole. In each stage a depth of 3 to 6 m is grouted. The grouting of each stage is done by putting a packer. The packer in each stage starting from bottom is left in place. In subsequent stages moving upwards the grout pressure is reduced. This method is suitable in good rock where the possibility of caving of the hole does not exist.

## (ii) Consolidation Grouting

The consolidation grouting is done under the entire base of the dam to improve the foundation characteristics such as bearing pressure and rock modulus. It is a low pressure shallow grouting to fill the cracks and fissures at or below the surface caused due to excavation through blasting and heavy equipment. The holes are drilled vertically and the depth depending on rock and height of dam ranges from 6 to 15 m. Deeper holes are made for high dams. These holes are drilled in a grid of 3 to 6 m under the entire dam base area. The grouting is done at a low pressure ranging from 2 to 5 kg/cm<sup>2</sup>. The pressure should not cause rock displacement. Grouting is done in one stage in full depth. The consolidation grouting should be extended in a width of about 10% of the height of dams in the upstream of the upstream face to prevent leakage of grout to the river bed from the curtain grout holes. When intake of the grout is less than 2 l/minute averaged over a period of 10 minutes at the prescribed pressure, grouting shall be stopped. A pattern of holes is shown in Fig. 9.3.



**Fig. 9.3** Pattern of holes for consolidation grouting.

### 9.2.1.4 Drainage

Grout curtain is provided to prevent seepage and has to withstand the entire hydraulic head on the upstream. Thus theoretically no uplift should occur on dam base downstream of the grout curtain. But in practice often grout curtains are found leaking, because grout is not able to seal all the fine cracks in the rock mass. In order to arrest the seepage through and below the grout curtain, drainage holes are made in the downstream of the grout curtain. In general, these are made through the foundation gallery from which grout curtain is made.

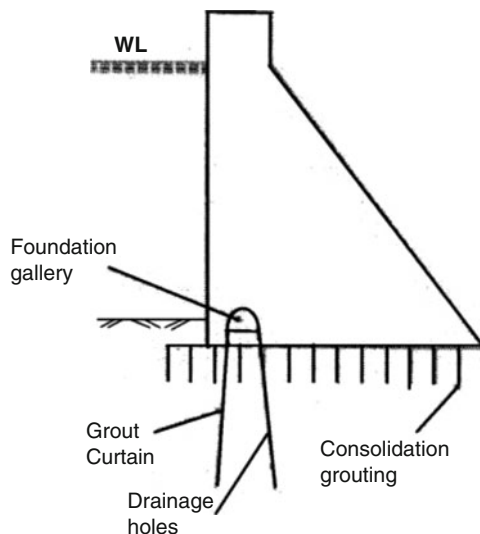
The NX (75 mm diameter) size holes are drilled generally at an angle of  $10^\circ$  with the vertical on the downstream side so that these are not shadowed by the grout curtain. The spacing of holes depends on types of rock. Generally the minimum spacing is limited to 1.5 m so that these may function effectively and the reduction in uplift on the dam base in the downstream can be ensured for stability considerations. The depth of holes depends on the depth of grout curtain and the type of rock. The guide lines for the depth of holes are that these should be 30 to 70% of the depth of grout curtain. It may be 20 to 40% of hydraulic height of dam. But the depth should be decided on the site specific considerations. For details refer IS Code 10135.

The seepage through drainage holes should be collected in a sump at the lowest point of the foundation gallery through a drain provided in the gallery. The sump is fitted with automatic pumps to pump water in the downstream of the dam.

A typical section of dam showing the grout curtain, drainage holes and consolidation grouting is shown in Fig. 9.4.

For details of drilling, grouting, water test etc., IS codes given in para 3.3.3 (Chapter 3) may be referred.

**Fig. 9.4** Grouting and drainage in a concrete dam.



### 9.2.1.5 Sequence of Foundation Treatment

The sequence of operations to be made to carry out normal treatment in concrete dams should be as follows:

- (i) All the bore holes, drifts and pits made during foundation investigations should be grouted or filled with concrete.
- (ii) Stripping and foundation excavation (blasting etc.) should be completed before grouting.
- (iii) First consolidation grouting is carried out.
- (iv) Curtain grouting is generally done when dam is built upto a height of 20 m or half the dam height.
- (v) Drainage holes not to be drilled till curtain grouting is completed within 30 m of the location of drainage hole.
- (vi) Curtain grouting should proceed from river bed towards abutments.

### 9.2.2 Site Specific Special Foundation Treatment

Special treatment of foundation is generally required for concrete dams because it is rare to get a geologically sound rock. Commonly the geological weaknesses encountered in most of the cases are of two types: (i) existence of weak rock bands of low rock modulus and strength between the strong rocks bands and (ii) the smooth bedding planes or thin clay seams having an orientation conducive to sliding failure. The treatment for these types of weaknesses in each case is site specific. Some examples are given below to give the idea of treatment provided in each type.

#### 9.2.2.1 Treating Weak Zones

Weak rock zones/bands are the pockets of weak material or gouge filled faults or shear zones. Generally the weak or soft rock is removed for sufficient depth and backfilled with concrete. In cases where it is not feasible, a mat or strut of reinforced concrete or roller compacted concrete is placed over the weak rock area to transfer the load to the adjoining competent rock. Some examples to illustrate this kind of treatment are given below.

##### (a) Bhakra Dam

In case of Bhakra dam which is 226 m high concrete gravity dam on river Sutlej in India, three clay-stone bands were found traversing the foundation area as shown in Fig. 9.5. The rock modulus of clay-stone bands was 0.2 to 0.3 of concrete. The downstream band of clay stone did not affect the stability of dam. But rest of the two had to be treated. A 22.5 m deep plug of concrete was found adequate for keeping stresses within permissible limit under the heel. Two longitudinal galleries were

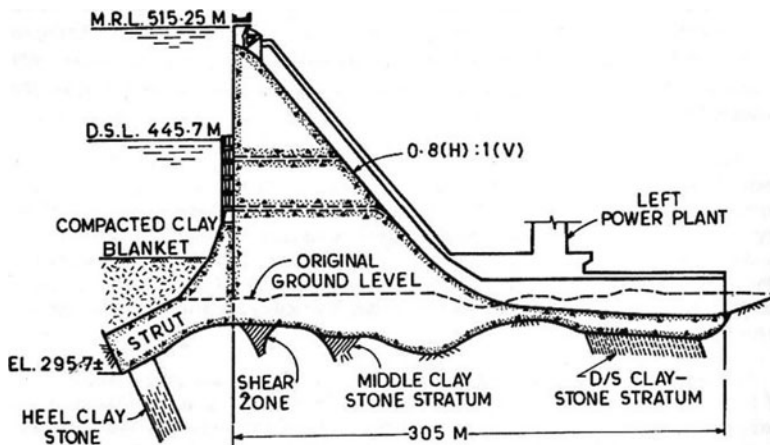


Fig. 9.5 Bhakra Dam – maximum spillway section showing treatment of heel claystone.

made in the plug for grouting. These were later plugged. The middle band which was 6 to 9 m wide was treated by excavating and backfilling with concrete.

#### (b) Supa Dam

Supa dam is a 101 m high concrete dam on river Kali in Karnataka (India). Like Bhakra dam weathered and fractured bands were found under toe and bed of dam in some blocks. In blocks 7 and 8 as shown in Fig. 9.6, highly disintegrated rock of low rock modulus was found. To transfer the load to the better rock which existed in upstream, a strut 9 m thick and 20 m long was provided as shown in figure. A joint was also provided between strut and the dam to avoid cracking due to tensile stresses in reservoir full loading conditions. Similarly a plug (or strut) was provided in the downstream in block no. 4.

#### (c) Sardar Sarovar Dam

It is a concrete gravity dam on river Narmada in Gujarat (India). A 8 metre wide fault was found in the river bed under the deepest spillway block. The fault zone material had rock modulus  $1/10^{\text{th}}$  of the basalt rock in foundation. It was treated by excavating 18.75 m deep trench along the fault line and backfilling it with concrete. Cut offs were provided at the two ends to prevent seepage. Contact grouting was done to make the backfill concrete monolithic with foundation. Arrangement for drainage was also made. The details are shown in Fig. 9.7.

#### (d) Lakhwar Dam

It is a 203 m high concrete gravity dam at Lakhwaron river Yamuna in Uttarakhand (India). During stripping of dam area and construction of the approach tunnel to the underground power house cavity on right bank a shear zone was found. Construction was stopped in about 1990 and further investigations were started. The

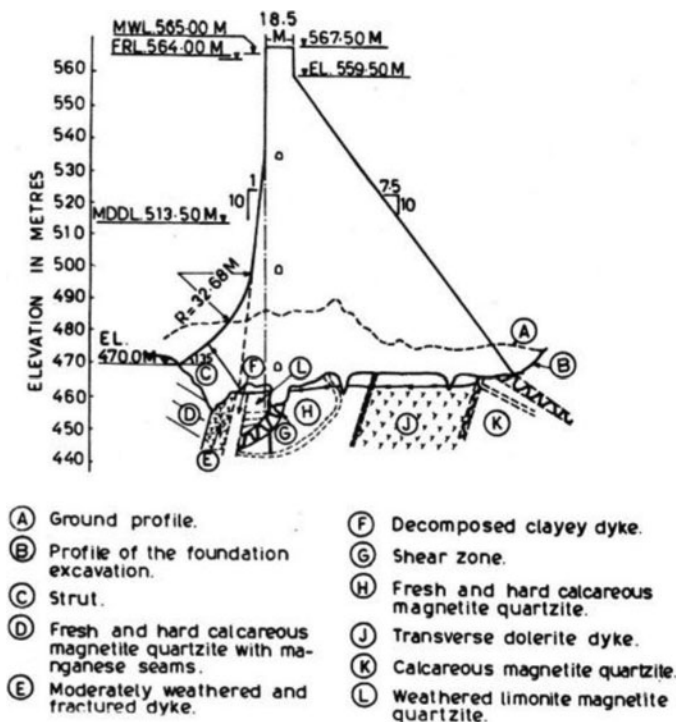


Fig. 9.6 Supa dam section (block 7) showing heel strut.

investigations revealed that the shear zone filled with gouge material crosses the river valley at the dam site and extends deep into foundation below the dam. It is shown in Fig. 9.8. The dam is placed on a narrow band of competent basalt rock which is flanked from both sides by weak rocks (slates in downstream and quartzitic slates in upstream side). Hence shifting of dam to avoid the shear zone was not feasible. Treating the shear zone was also not considered by experts feasible and dependable. Hence studies were started to find a suitable solution by changing the type of dam. Construction of dam is yet to be restarted.

#### 9.2.2.2 Treating Bedding Planes/Clay Seams

The existence of smooth bedding planes or thin clay seams in the foundation below the dam base parallel or slightly inclined to the base may cause sliding failure of dam. In such situations shear keys and anchors are commonly used to improve sliding resistance. If such planes or seams are located at shallow depth below dam base, shear keys cutting such planes or seams are made as part of dam. The optimal

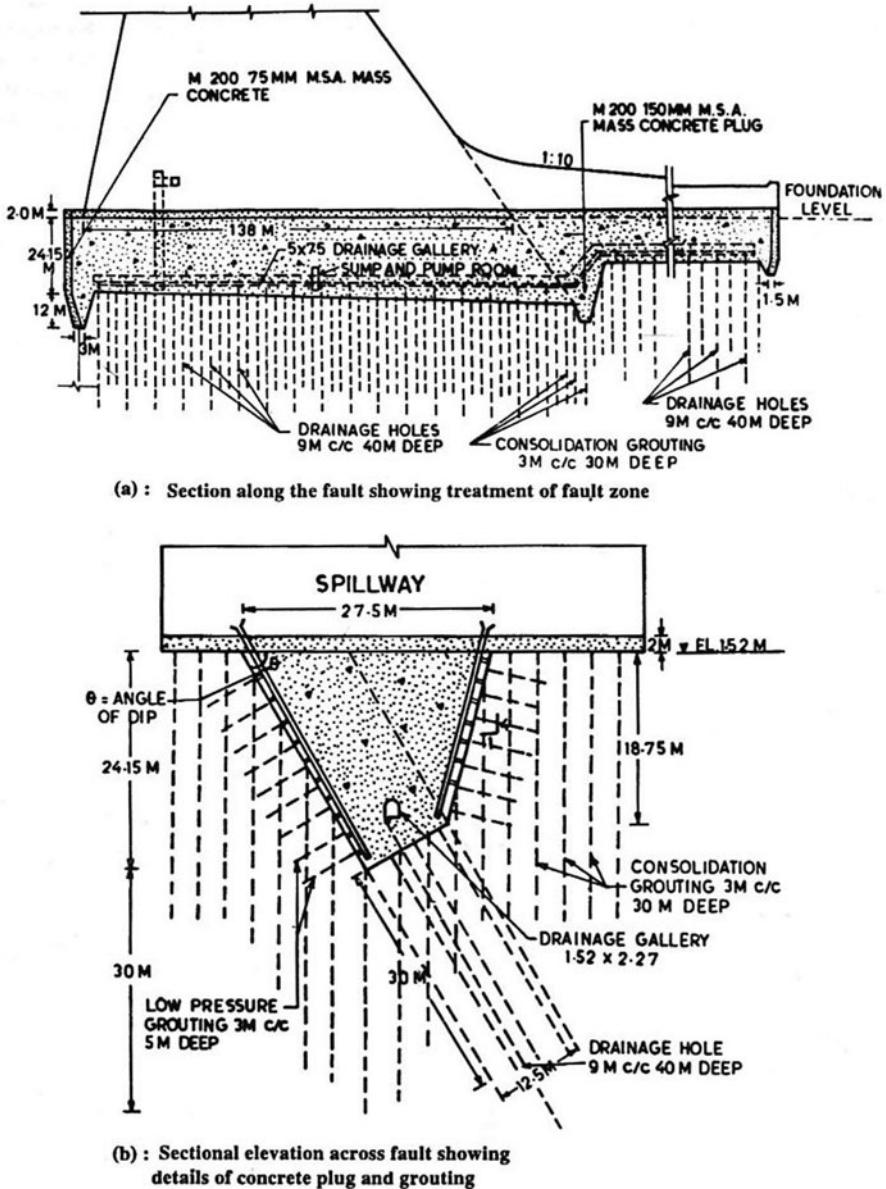
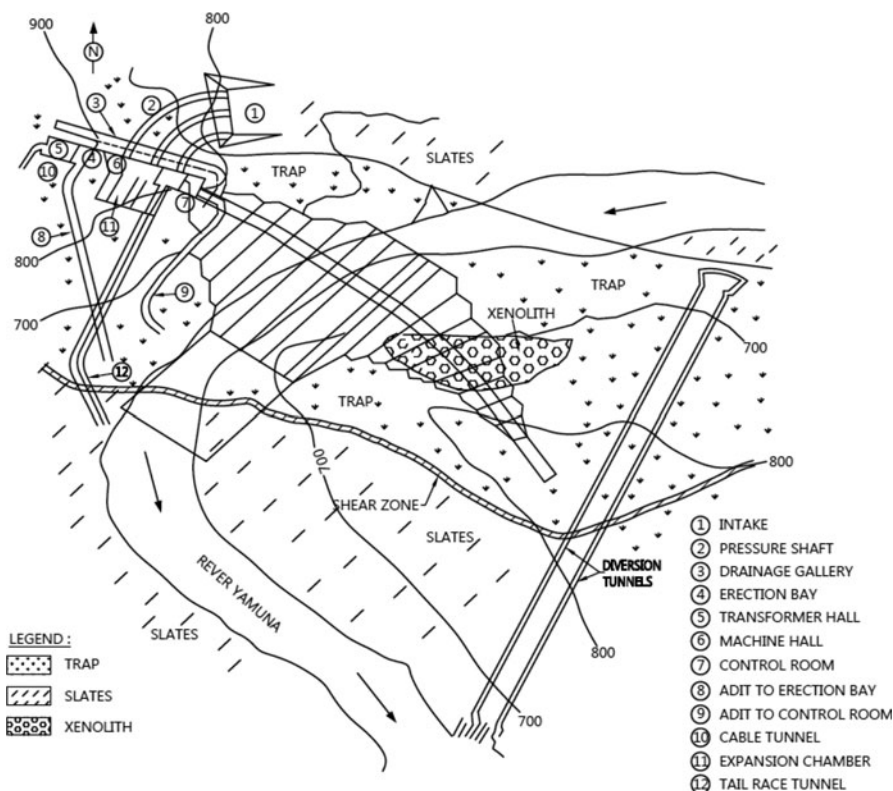


Fig. 9.7 Sardar Sarovar Dam – details of fault treatment.

position of such keys is in the downstream where normal vertical stresses are maximum. But the disadvantage is the increase in uplift in its upstream due to interception of seepage. Anchoring can also be done with anchors running across the seam. When the depth of bedding plane or seam is large then underground shear



**Fig. 9.8** (a) Layout plan of Lakhwar dam showing geology of area.

keys across the plane are constructed. Some examples to illustrate this type of treatment are given below.

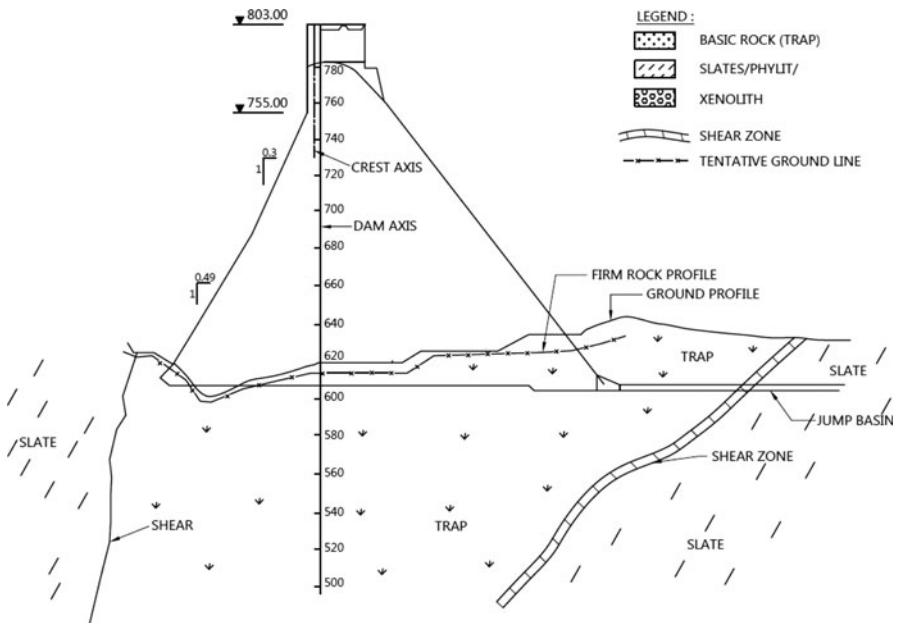
#### (a) **Dudhganga Dam**

It is a 75 m high concrete gravity dam. A shale band was found at a depth of about 8 m below the dam base. It was considered a potential plane for sliding failure. To improve sliding resistance, a 12 m deep shear key has been provided intercepting the shale band. In addition the dam downstream of shear key was anchored in rock mass below the shale band. The details are shown in Fig. 9.9.

#### (b) **Rana Pratap Sagar Dam**

It is a concrete gravity dam on river Chambal. It has a power house at the toe of dam. Two clay seams parallel to dam base at shallow depths in the foundation were found. To improve sliding stability of the dam, anchoring through cable anchors was done. The details are shown in Fig. 9.10.

#### (c) **Kadana Dam**



**Fig. 9.8 (b)** Geology section along deepest block of Lakhwar dam.

It is a composite dam with concrete gravity spillway portion on river Mahi in Gujarat. It is a 66 m high dam with power house at the toe of dam. The foundation comprises interbanded quartzites, quartz mica-schists and phyllites with thickness varying from 0.5 to 3 m. The rock is highly jointed.

The investigations indicated sliding along bedding planes. Minor reverse faults were found intersecting the rock formations. The low angle faults get exposed in the downstream in many of the blocks including that of the powerhouse. These faults in various blocks were very deep below dam base. Hence, the treatment to improve the shear resistance was done by providing shear keys by driving drifts below the dam. The details of this treatment under power house blocks is shown in Fig. 9.11. These shear keys filled with concrete were made 3 to 4.5 m wide running parallel to dam axis.

### 9.2.2.3 Treating Complex Foundation

In many cases the foundations are found quite complex in nature with weak zones, shear and faults, clay seams etc. with varying orientations and often these vary from one part of the dam to the other. Such cases require a combination of various kinds of treatments to make the foundation suitable for the dam. For example, in some deep river bed blocks of Srisaillam dam (144 m high masonry cum concrete gravity structure) weak zones in foundation were treated by deep excavation and backfilling

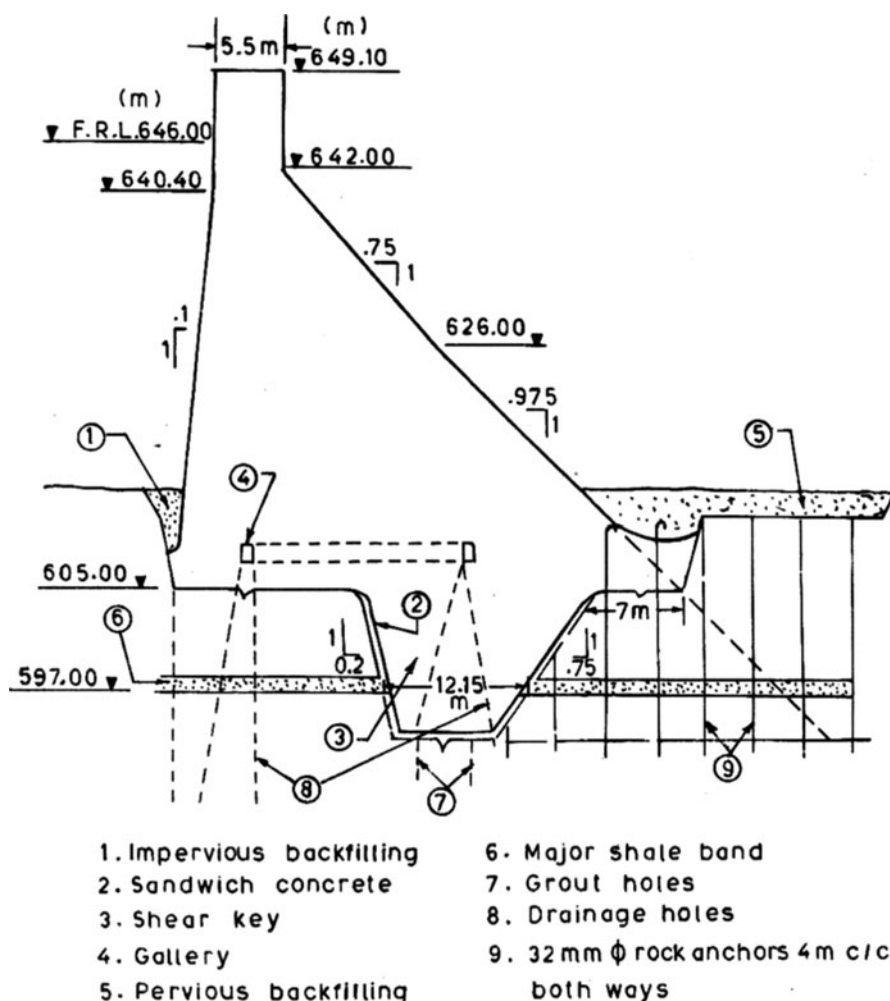


Fig. 9.9 Shear key at Dudhganga dam.

with concrete under the heel and the dam is made safe in sliding by providing a reinforced concrete block at the toe. In some other blocks the safety of dam in sliding was ensured, due to the presence of a horizontal shale seam at a depth of about 15 m below quartzitic formation, by providing several shear keys running parallel to dam axis constructed by driving vertical shafts as was done in Kadana dam. Similar was the situation in Sardar Sarovar dam where in some dam blocks shear keys were constructed. The parameters of such shear keys like the size, numbers, and location can be best determined by analyzing dam and foundation together using FEM analysis. Proper investigations for the properties of the foundation material is very important for such analysis.

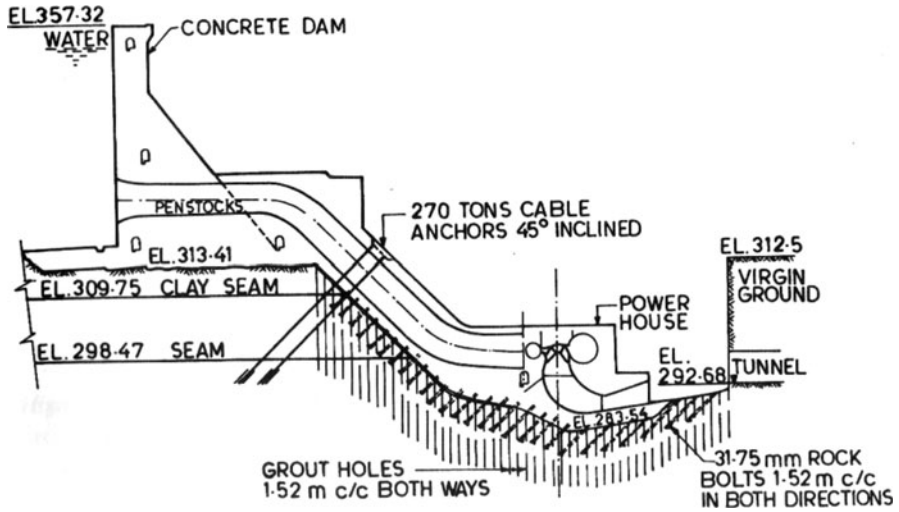
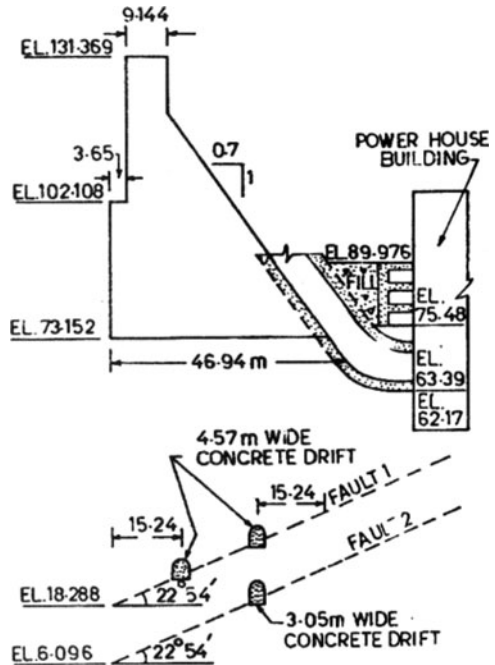


Fig. 9.10 Rana Pratap Sagar dam section showing cable anchors.

Fig. 9.11 Kadana Dam – shear keys below power dam blocks 3, 4 and 5.



The above description with a few examples gives a generalized view of various types of special foundation treatment. It needs no emphasis that in each case the selection of type of treatment and its design is site specific and requires sufficient geotechnical and geological investigations of the foundation. A study of the

foundation treatment carried out at other projects of similar nature is of great help in this regard. Scanning the proceedings of various ICOLDs specially of 17<sup>th</sup> ICOLD (1991) where case studies are reported from all over the world regarding foundation problems of large dams, is helpful in deciding the foundation treatment for complex foundation conditions.

### **9.3 FOUNDATION TREATMENT FOR ROCKFILL DAMS**

Rockfill dams are of two types, CFRD and ECRD, and are generally constructed on rock foundation which may or may not be as strong as for a concrete dam because bearing capacity and sliding requirements are not of much concern in a rockfill dam which has a wide base width. But foundation treatment is required to prevent seepage through the joints in the rock for which grouting of the foundation is done.

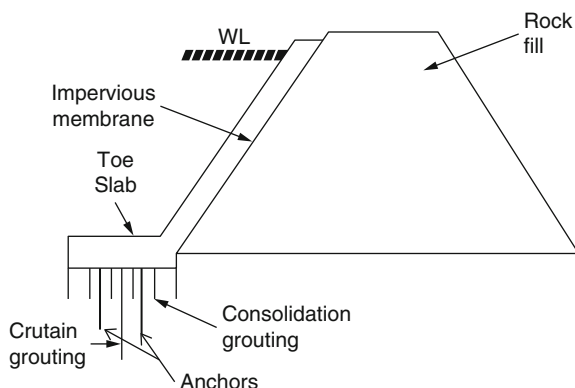
#### ***9.3.1 Concrete Faced Rockfil Dam (CFRD)***

In CFRD dam the rockfill area is striped and cleaned upto a level where the foundation is capable of taking the load. Cracks, pits and depression, if required, should be backfilled with concrete. Minor shears should be given dental treatment. The toe slab supporting the face slab is rested on competent rock. The toe slab is anchored to the rock. The grouting of foundation rock is done through the toe slab. Sometimes in dams on weak foundation strata concrete membrane can be provided instead of grout curtain. The area under the toe slab should be compacted through consolidation grouting. A typical section conceptually showing the location of grouting for the curtain grout and consolidation grouting is given in Fig. [9.12](#).

#### ***9.3.2 Earth Core Rockfill Dam (ECRD)***

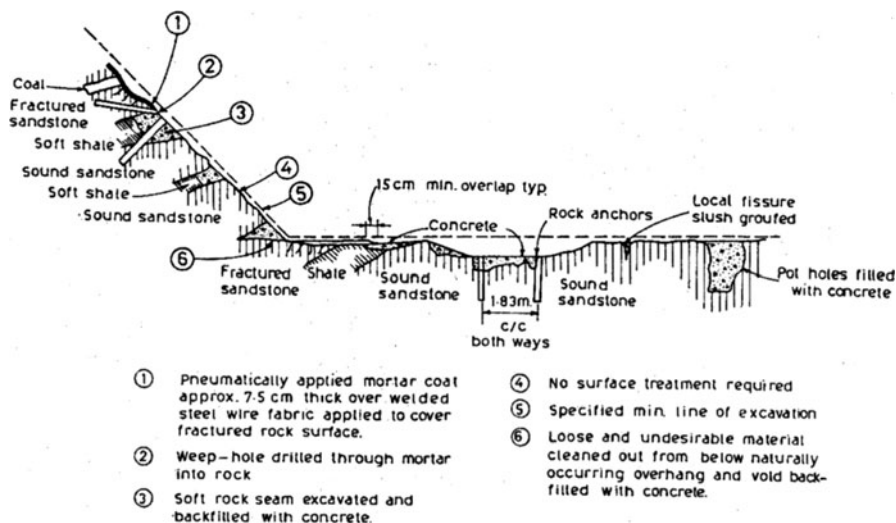
The ECRD has a clay core with shell zones on both sides. These are generally founded on rock foundation. These are also constructed where foundation consists of bed rock overlain with shallow river bed material (RBM) or of weathered rock. The dam area shall be cleaned of vegetal cover and overburden material. The shell zone can be placed on compact RBM or weathered rock if it is capable of taking the load without excessive settlement. The clay core is normally taken upto the bed rock. A grout curtain is provided in rock to prevent seepage.

**Fig. 9.12** Grouting at CFRD.



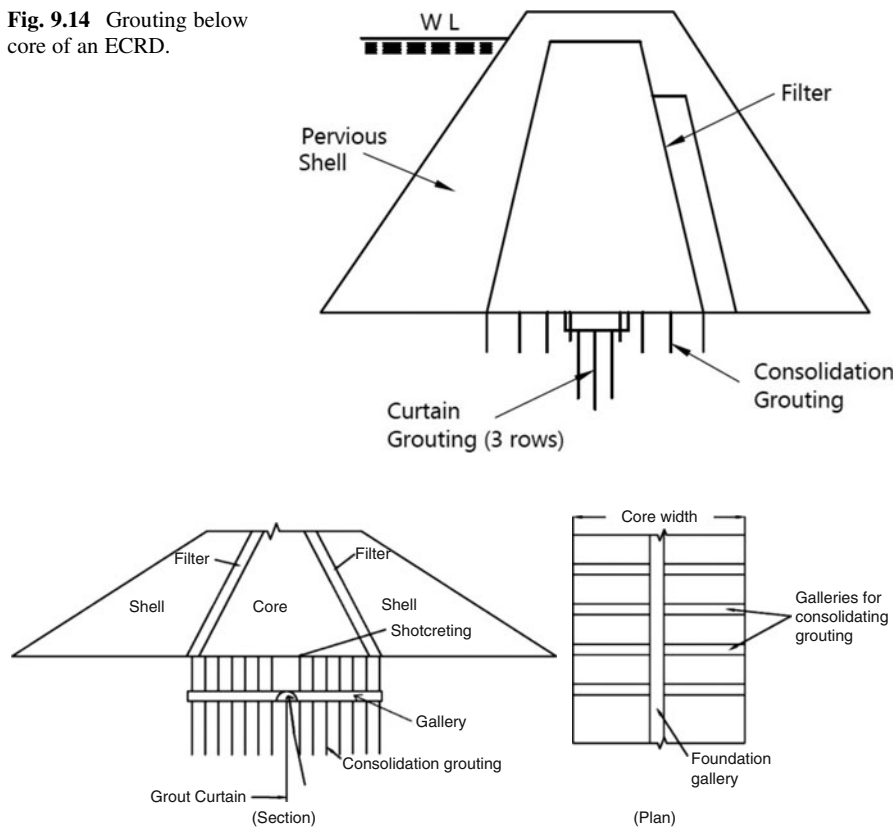
### 9.3.2.1 Treatment of Core Area

The core area after excavation should be treated for weak zones, minor shears and depressions. Dental treatment should be done for minor shears and weak zones and large depressions, pot holes, cavities etc. should be cleaned and filled with concrete. The core contact area treatment provided at Bennett dam (Fig. 9.13) illustrates the above described treatment. In case of Tehri dam the rock foundation was of highly jointed phyllites. In the core area dental treatment was done for wider joints and thereafter the whole core area was shotcreted.



**Fig. 9.13** Typical core-contact surface treatment details of WAC Bennett dam. *Source:* H.D. Sharma

**Fig. 9.14** Grouting below core of an ECRD.



**Fig. 9.15** Grouting of core area through foundation gallery in a high ECRD.

### 9.3.2.2 Grouting

Foundation grouting to prevent seepage through foundation is carried out from the core. A conceptual foundation grouting arrangement is shown in Fig. 9.14. It is generally done before placing the fill in the core area. A concrete pad, 4 to 5 m thick, is provided over the bed rock for the purpose of grouting. The concrete pad is anchored with bed rock. Through the pad first the consolidation grouting, 6 to 15 m deep, is done in the entire base area of core. After consolidation grouting, curtain grouting is done upto a depth of  $2/3 H$  to  $1 H$  ( $H$  is height of dam) depending on the type of rock. Grout curtain is made up of one row of holes or of multiple row of holes. Generally three rows of holes are adopted and rarely five rows are used as was done at Bennett dam. Grout rows are spaced 3 to 4.5 m apart and holes are staggered amongst the rows. Multiple rows are adopted in soft and highly jointed rock. These are made nearly parallel to dam axis and located in the centre or middle one third area of core. The holes are made vertical or inclined. Inclination depends on the orientation of joints and fissures. If three rows of grout holes are made, the downstream

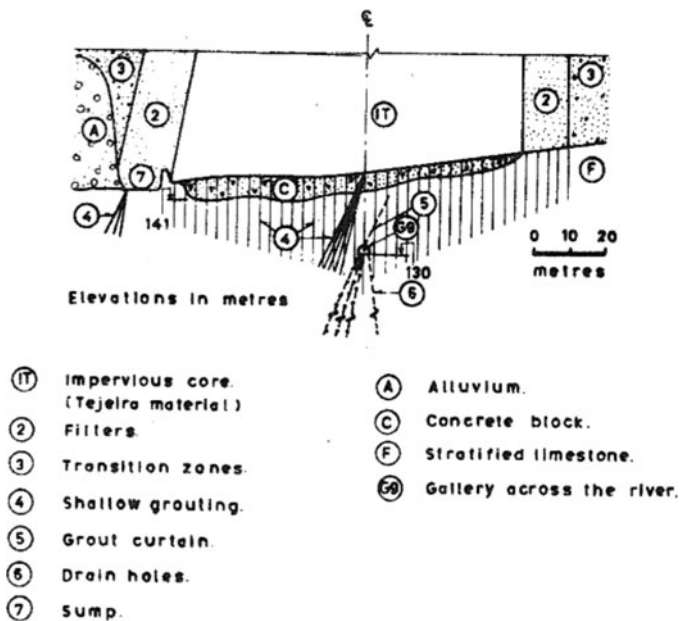
row is grouted first then the upstream row is grouted and the central row is done in the last.

In high dams the practice is to provide a gallery in the foundation below the core area. It is used for grouting and drainage. A conceptual sketch of this type of grouting is shown in Fig. 9.15. It has the following advantages:

- Foundation grouting can be done independently without interfering with the dam construction. It reduces total construction time of project.
- Drainage holes may be made in the downstream of grout curtain to reduce seepage gradient.
- Inspection and supplementary grouting is possible if required after filling of reservoir.

Such galleries have been provided in many dams such as Oroville, Nurek and Chicoasen. It has been provided in Tehri dam also. Foundation grouting of Chicoasen dam is shown in Fig. 9.16.

The specifications for curtain and consolidation grouting are practically same as in case of concrete. Some foundation grouting details of ECRD at Tehri are given below.



**Fig. 9.16** Location of consolidation and curtain grouting below the impervious core of the Chicoasen dam.

### 9.3.2.3 Foundation Treatment of ECRD at Tehri

The foundation investigations had revealed that there is an overburden of 10 to 15 m over the sheared and jointed phyletic quartzite and quartzitic phyllites which are permeable. After stripping, the insitu tests in core area revealed the permeability of 10 to 15 Lugeons which reduced to 1 to 3 Lugeon at a depth of 50 to 60 m. Thus, it was decided to carry out both surface and subsurface foundation treatment in the core area. For surface treatment the stripped core base area has been provided with 50 mm thick layer of shotcrete after carrying out the dental treatment of weak zones and shears.

The subsurface treatment comprised of consolidation and curtain grouting. Consolidation grouting in the entire core area was carried out upto a depth of 10 m. Curtain grouting was carried out in two rows, upstream row upto a depth of 30 m and downstream row upto a depth of 60 m along the centre line of the core.

Both consolidation and curtain grouting was mainly done through a network of underground galleries at four elevations at RL 572, 640, 700 and 760 m. Consolidation grouting above RL 700 was carried out from surface in conventional manner before fill placement. Grouting through underground galleries was carried out to reduce construction time as it could be done independent of dam fill placement and galleries could be used for post construction grouting if required after commissioning of project. The galleries and grouting details are shown in Fig. 9.17 (a,b). Figure 9.17(a) shows the details of curtain grouting and Fig. 9.17 (b) shows consolidation grouting details.

### 9.3.3 Abutment Contact

Core-abutment contact is important because the contact surface can be a potential plane for seepage and piping. Hence it should be suitably shaped and prepared for sealing. Overhang, rock projections and vertical surfaces should be trimmed and smoothened to provide continuous moderate slope. The abutment slopes provided in some dams have ranged from as steep as 0.1 H : 1.0 V to the moderate slope of 1 H : 1 V. The dam section including the core width at the junction is flared to provide longer seepage path. Near the abutment where cracks may occur more deformable soil should be used in the core.

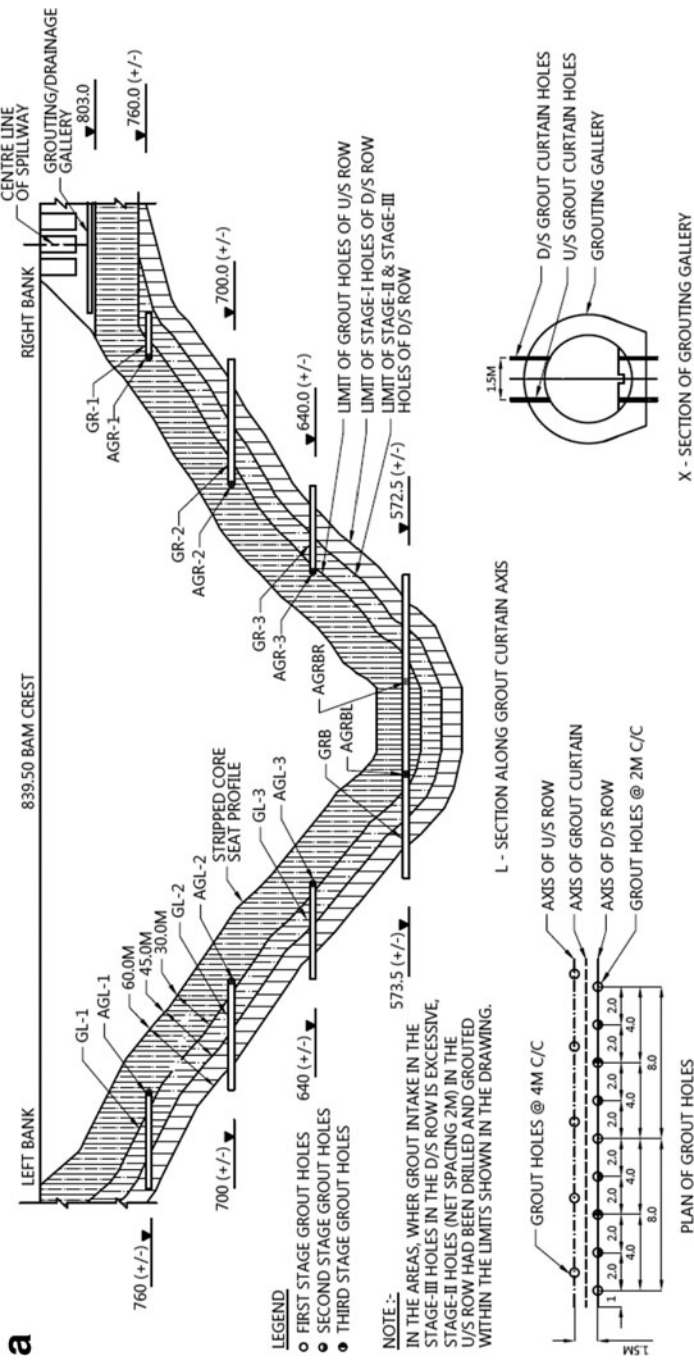


Fig. 9.17 (a) Curtain grouting at Tehri Dam.



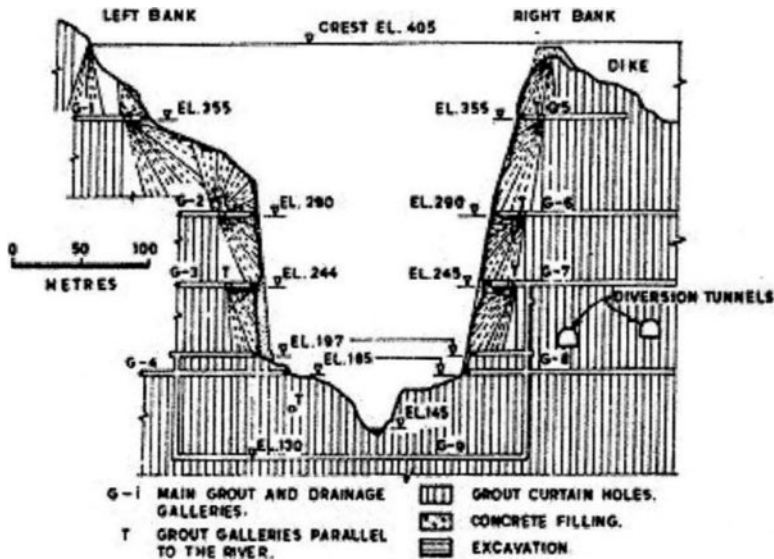


Fig. 9.18 Longitudinal section along the axis of Chicoasen Dam showing reshaping of canyon walls and grouting and drainage galleries.

### 9.3.4 Treatment of Abutment

The geotechnical and geological investigations of abutment rock in respect of type of rock jointing pattern, permeability, discontinuities etc. are essential to ensure watertightness. In case seepage is expected through abutments, treatment through grouting should be carried out. Galleries are commonly made in the abutment and grouting of abutment is done through the galleries at different elevations. Drainage holes are made at a spacing of 3 to 10 m in the galleries below the core to collect the seepage. Drainage holes are filled with filter material in case abutment rock is erodible. The abutment slopes and grouting through galleries carried out in Chicoasen and Nurek dams are shown in Fig. 9.18 and Fig. 9.19 respectively.

In case the abutment is made of material which is not economically groutable and seepage and piping is expected through the abutment, it is desirable that downstream face is covered with filter material. The upstream face of abutment shall be covered with impervious blanket to check seepage. A few examples of such treatment are given below.

In Ochoco dam in USA, where right bank abutment is made of landslide debris, a clay blanket 1.5 m thick was placed at a slope of 3:1 on the upstream slope of the right abutment in continuation of dam clay core covering a length of about 150 m in 15 m height (between river floor and FRL). The clay blanket was protected by 0.6 m thick riprap. It reduced the seepage significantly. A similar clay blanket has been placed on the upstream face of right abutment of Salma dam in Afghanistan. It is

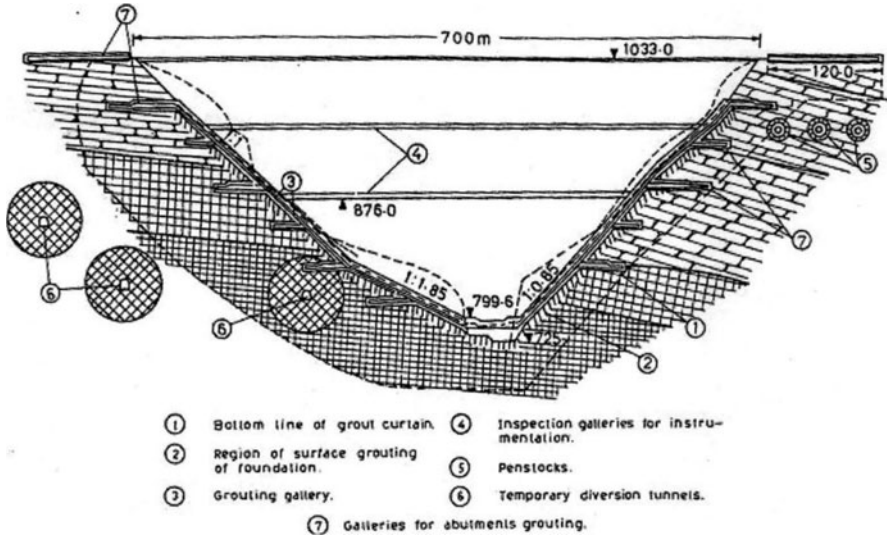


Fig. 9.19 Longitudinal section along the abutment grouting galleries of Nurek dam.

0.75 m thick. The abutment is a glacial deposit. Provision is also made to monitor the development of seepage line in the abutment downstream of dam axis.

## 9.4 FOUNDATION TREATMENT OF EARTHFILL DAMS

Earthfill dams are either homogenous type or zoned type. The foundation may be rock foundation or alluvial pervious foundation. Both type of foundations are required to be treated for seepage control. If seepage is not adequately controlled it will cause loss of storage water as well as removal of fine particles of foundation material due to erosion resulting in piping which in several cases has been found to be the cause of dam failure. The treatment of rock foundation is same as has been discussed above for ECRD. The commonly used methods to control seepage through pervious foundations of earthfill dams are as below:

- Seepage reducing methods: These are cutoffs which may be trench cut off, concrete cut-off or diaphragms, slurry trench cut offs, grout curtain and upstream impervious blanket.
- Drainage methods: The purpose of these is to offer safe exit to the seepage taking place through foundation. These are horizontal drainage blankets downstream of cut-offs, relief wells, drainage trenches etc.

The selection of the type of seepage control measure depends on foundation strata, depth of foundation, stratification and the extent of reduction required in seepage loss. Different types of seepage control measures are described below.

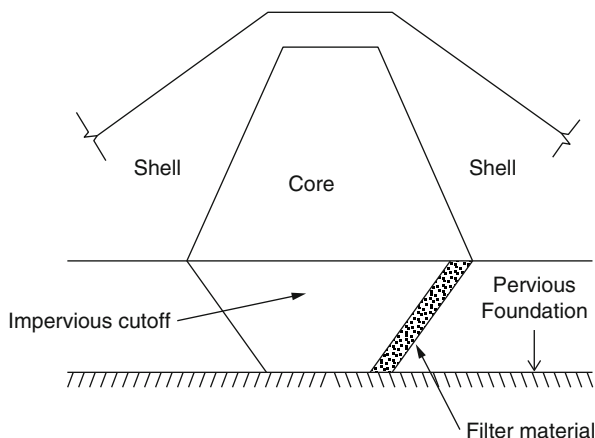
### 9.4.1 Seepage Reducing Methods

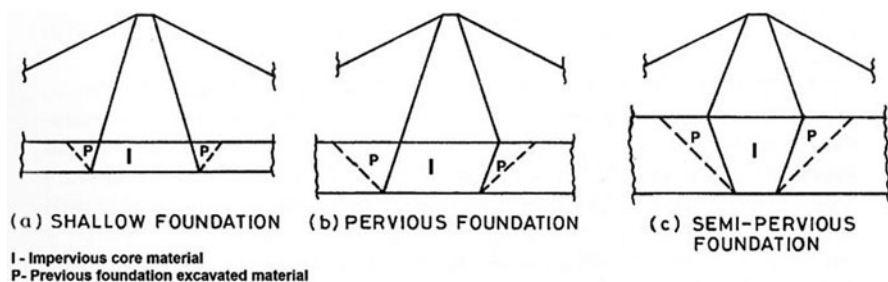
#### 9.4.1.1 Cut-off

These are generally of two types. One is sloping side trench and the other is vertical side. Since vertical side trench excavation is difficult, it is limited to small depths only and so generally sloping side trenches are made. These are located under the core. If the core is very wide it may be located either at the centre of core or a little upstream. These are generally made parallel to the dam axis and carried up the abutments. The side slopes depending on nature of foundation material should be stable. Generally a slope ranging between 1:1 to 1.5:1 (H : V) are found safe. The trench is excavated upto bed rock or impervious stratum. The bottom width should be adequate for the working of earth moving equipment and pumping arrangement of seepage water if excavation is to be done below water table. A minimum bottom width provided is about 4.0 to 5.0 m. The trench is backfilled with impervious material or core material and is compacted in the same manner as the core. When the trench is extending upto the rock it is known as positive cut off as it is the most positive means to control foundation seepage and is considered most reliable measure if economically feasible. The maximum depth upto which such cut-offs have been adopted is about 25 m but for many dams, it is preferred in smaller depths say 10 to 15 m. A filter shall be provided on the downstream face of cut-off as shown in Fig. 9.20 to check movement of fines into foundation.

In case the bed rock is highly jointed and fissured the full bottom width of cut-off should be either shotcreted or covered with concrete slab. Depending on the

**Fig. 9.20** Filter on D/S of cut off.





**Fig. 9.21** Open trench cut-off and excavation and backfilling.

permeability of rock and the head, curtain grouting through concrete slab may also be carried out.

In case the excavation depth of pervious strata required for the trench is large and relatively flat sides are required for stability of trench, the excavation of trench becomes costly due to huge quantity of excavation and the backfill. The backfill quantity of impervious material may be reduced as shown in Fig. 9.21.

#### 9.4.1.2 Partial Cut-off

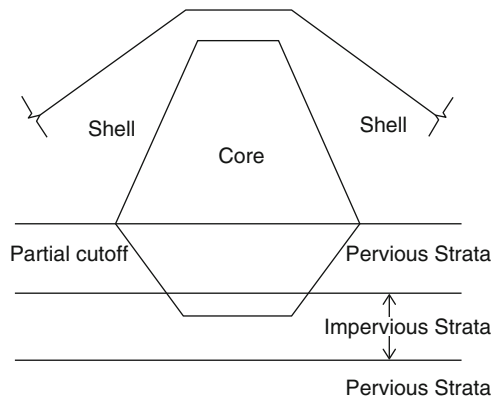
A trench cut-off not extending upto the bedrock or impervious stratum is called partial cut-off. It offers obstruction to the seepage flow but studies have shown that the reduction is not directly proportional to the depth. It is found that 80% cut off penetration reduces seepage by 50%. Even 10% opening would permit about 40% of seepage. Hence partial cut offs are of limited utility in homogeneous isotropic foundation material. It may be effective in a stratified foundation by intercepting more pervious upper layer and extending into impervious layer sandwiched between two pervious layers in case the impervious layer is continuous and thick enough to resist seepage pressure from the lower pervious layer. It is shown in Fig. 9.22. But it needs extensive subsurface exploration.

#### 9.4.1.3 Concrete Cut-off Walls or Diaphragm

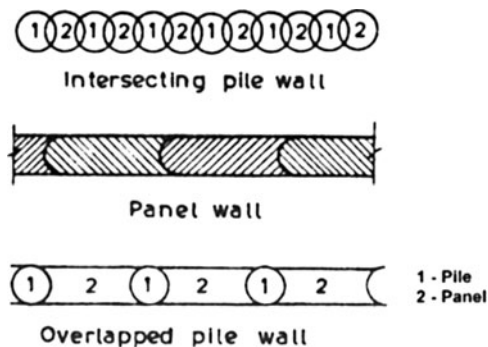
These are of three types: intersecting piles, panels and piles separated by panels. These are shown in Fig. 9.23.

The construction process is similar to all the three types. Holes are drilled and immediately filled with mud slurry consisting of a suspension of bentonite. The suspended fine clay particles form an impervious film on the sides and prevents the hole from collapsing. The composition and density of slurry is determined by experience and experiments. The practice is to initially use 5 to 7% by weight of

**Fig. 9.22** Partial cut-off in stratified foundation.



**Fig. 9.23** Different concrete diaphragm walls in plan.



bentonite in the slurry. After the holes are stabilized the slurry is replaced by concrete. In this way concrete piles are made into pervious soil foundation. These can be made intersecting type or the other type where piles are separated by panels by excavating soil between the piles by suitable equipment and pouring concrete. The diaphragm wall is made in panels which are 2 to 5 m long and 60 to 90 cm wide. The material between two holes is removed by a clam-shell. The space upto bottom of trench is cleaned and filled with bentonite slurry. The deposited mud of slurry is removed, then reinforcement cage made on the ground is lowered for the panel and thereafter tremie concrete is poured starting from the bottom. Grout pipes are embedded in concrete for subsequent grouting of joints. Alternate panels are constructed. These cut off walls are used where the depth of pervious strata is large and it is uneconomical for a cut off trench. These have been constructed as deep as 100 m and more. This construction of concrete diaphragms needs special equipment and construction technique which are patents of the construction firms. A typical construction process is shown in Fig. 9.24.

These diaphragm walls are generally placed centrally below the core of zoned section and are also used in ECRD. It has also been placed below the toe slab of the

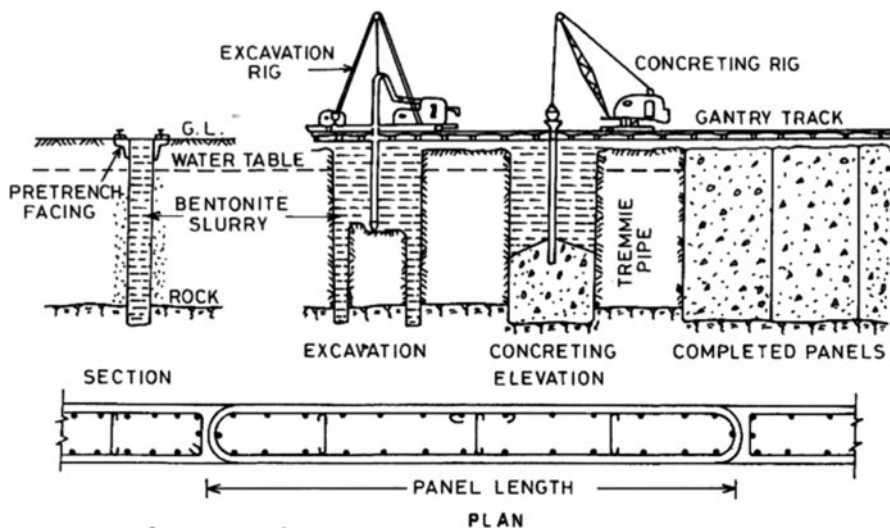


Fig. 9.24 Construction process of concrete diaphragm.

CFRD. Two types of diaphragm walls are constructed. One is of conventional concrete of grade  $M_{20}$  with reinforcing cage (usually 16  $\phi$  300 mm c/c on both sides and both direction) and the other is of plastic concrete which is unreinforced and can bend appreciably without cracking. Plastic concrete cut offs are preferred in highly seismic areas. Plastic concrete is made by mixing clay or bentonite in concrete in requisite proportion. A mix of 1 part cement, 0.85 part clay and 4 parts sand by weight was used at Ukai dam (80 m high) in Gujarat. The cut off was 20 m deep and 0.6 m thick. A 30 m deep, 1.0 m thick plastic concrete diaphragm wall type cutoff has been constructed under the toe slab of a CFRD constructed in Chenab valley in J & K. The concrete diaphragm wall type cut-off has been provided at Obra Dam (30 m high) in Mirzapur, Uttar Pradesh. It consisted of two 0.6 m thick concrete diaphragms 15 m deep separated by 3 m. The alluvium between the two walls was grouted. A similar cut off was provided at Tenukhat dam (India).

#### 9.4.1.4 Slurry Trench Cut-offs

The slurry trench cut off is made by excavating the trench which is first stabilized with bentonite slurry. It is later displaced by soil-bentonite mixture of consistency similar to high slump concrete. The soil should preferably have 10 to 30% particles finer than 75 micron and maximum size may be 20 to 75 mm. The soil backfill makes a low permeability cut off. The width ranges from 1.5 to 3 m and the depth may be as deep as 25 m. Instead of soil bentonite mixture the cement-bentonite mixture is also used to form slurry trench cut-off. The thickness of this type of cut-off ranges from

0.5 to 1.5 m and the depth may go as deep as 50 m. The permeability achieved is of the order of  $10^{-5}$  to  $10^{-6}$  cm/sec. The procedure of construction is similar to that of concrete diaphragm. Vertical sided trenches are excavated with either draglines, clam-shells, backhoes or trenching machine. Bentonite slurry is pumped to stabilize the trench avoiding the caving.

#### 9.4.1.5 Grouted Cutoff

These are made by injecting grout material to fill the voids of the pervious foundation material within cutoff zone. The grouting materials used are cement, clay, chemicals or a combination of these materials. Coarse sediment can generally be treated successfully by using cement or cement-clay grouts and permeability can be reduced in the range of  $10^{-4}$  to  $10^{-5}$  cm/sec. But these grouts cannot be successful in very fine granular materials. Generally chemical grouts which have viscosity same as water are used for fine grained soils. However, chemical grouts are too expensive to grout pervious foundations. Use of bentonite and chemical (sodium silicate) is more economical than pure chemical grout. Grouting reduces the permeability of soil in the grouted zone but is not dependable to provide a continuous curtain as a barrier to stop seepage flow even after providing several rows of closely spaced grout holes. Hence it shall be combined with other measures.

The selection of a grout for a particular type of soil should be done after field trial tests. The guidelines based on particle size and permeability, given in Table 9.1, may be used for initiating the trials.

#### 9.4.1.6 Upstream Impervious Blanket

The upstream blanket of impervious material is made connecting the clay core of the dam and extending it upstream of the toe. It increases the path of seepage in pervious foundation. Such blankets are generally used when positive cutoff upto the underlying bedrock or impervious layer are not practicable due to large depth. The length of the blanket may be made adequate to reduce the seepage to the extent required. But upstream blanket should not be relied upon to reduce seepage head enough to avoid the possibility of piping. Hence, to control exit gradient, drains or relief well and/or drainage blanket are provided in the downstream of dam alongwith the

**Table 9.1** Selection of suitable grout

$D_{15}$ of soil (mm)	1.0	0.6 – 0.8	0.3	0.1
Lower limit of permeability (cm/sec)	0.5–1.0	$0.5 \times 10^{-1} - 1.0 \times 10^{-1}$	$1 \times 10^{-2}$	$1 \times 10^{-3}$
Suitable type of grout	Cement	Cement bentonite	Clay-chemical	Chemical

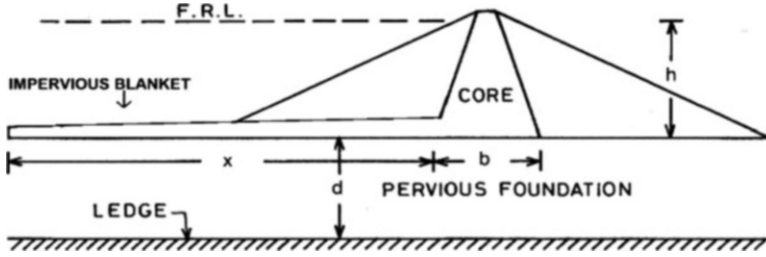


Fig. 9.25 Upstream impervious blanket.

upstream blanket. Partial cutoff trenches and grout curtains are sometimes provided in conjunction with the upstream blanket.

The length of the blanket is governed by the extent of reduction required in seepage loss. It depends on the purpose of the project. It can be worked out as below using Darcy's law and assuming that the blanket is impervious. Refer Fig. 9.25.

Seepage discharge without blanket:

$$q = K \frac{h}{b} d$$

Seepage discharge with blanket:

$$q_b = K \frac{h}{b+x} d$$

$$\frac{q_b}{q} = \frac{b}{b+x}$$

or

$$x = b \left( 1 - \frac{q_b}{q} \right) / \frac{q_b}{q}$$

Hence 'x' the length of blanket can be worked out for a predetermined value of  $q_b/q$ .

The optimal length beyond which the effectiveness of the blanket does not appreciably increase is given by

$$x = \frac{\sqrt{2}}{a}$$

where  $\frac{1}{a} = \sqrt{\frac{K_f}{K_b} Z_b Z_f}$ ,  $K_f$  = permeability of foundation,  $K_b$  = permeability of blanket,  $Z_b$  = thickness of blanket, and  $Z_f$  = thickness of pervious foundation.

Normally a suitable thickness of blanket is 10% of the depth of reservoir above the blanket with a minimum of 1.0 m. It shall be made of the same impervious material which is used in core of the dam. A length of 8 to 10 times the depth of reservoir is generally provided and is found adequate.

The blanket is normally laid on the floor of the river bed but is often continued on the abutments if required due to its soil characteristics and permeability.

Blanket should meet the filter criterion with the foundation material. If it does not, a filter shall be placed below the blanket. Generally filter is not provided. In that case the damage to the blanket may occur due to unequal settlement, cracks, sink holes and leaching of fine material etc. This happened in Tarbela Dam (145 m high) in Pakistan. Alluvial foundation was 220 m (maximum) deep. Positive cutoff was not considered feasible. An upstream impervious blanket 1400 m long and thickness varying 15.25 m to 1.5 m was provided which was found severally damaged in the first filling of reservoir necessitating heavy repairs.

### **9.4.2 Drainage Methods**

The downstream portion of dam on pervious foundation should be provided with adequate drainage methods when a positive cutoff is not constructed. These should permit safe exit of seepage water from the foundation preventing piping of fines. Generally horizontal drainage blanket, toe drain and relief wells are provided.

#### **9.4.2.1 Horizontal Drainage Blanket**

It is provided over the pervious foundation under the downstream shell generally in continuation of the filter laid on the downstream face of the core. The blanket must be pervious enough to act as a drainage and should be designed satisfying the filter criteria, to prevent the movement of fines from the embankment and the foundation. The horizontal drainage blanket carries the seepage water out of the dam through a drain provided at the downstream toe. The minimum thickness of drainage blanket should be 0.9 m and should be made of multi-layer filters (refer Figs 6.2 and 6.3).

#### **9.4.2.2 Toe Drains**

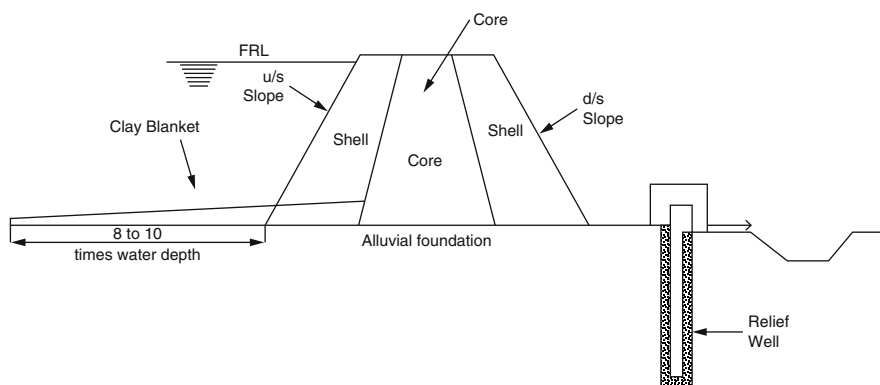
The toe drains are commonly provided along the downstream toe of the dam in continuation with the horizontal drainage blanket. The drain collects all the seepage from the dam and discharges either in a natural drain or main river. Hence its size should increase gradually starting from the abutments and moving towards the outfall. Sometime the toe drains are made of small size and these are at regular

intervals connected to another drain along the dam through small diameter pipes to drain small reaches of the dam say 15 m or so.

The toe drain has a minimum depth of 1.5 m and bottom wide of 1.0 m. It is made of multilayer filters which are continued from the drainage blanket. A slotted or perforated pipe of minimum size of 150 mm is placed in the toe drain. A section of toe drain along with the drainage blanket is shown in Fig. 6.3.

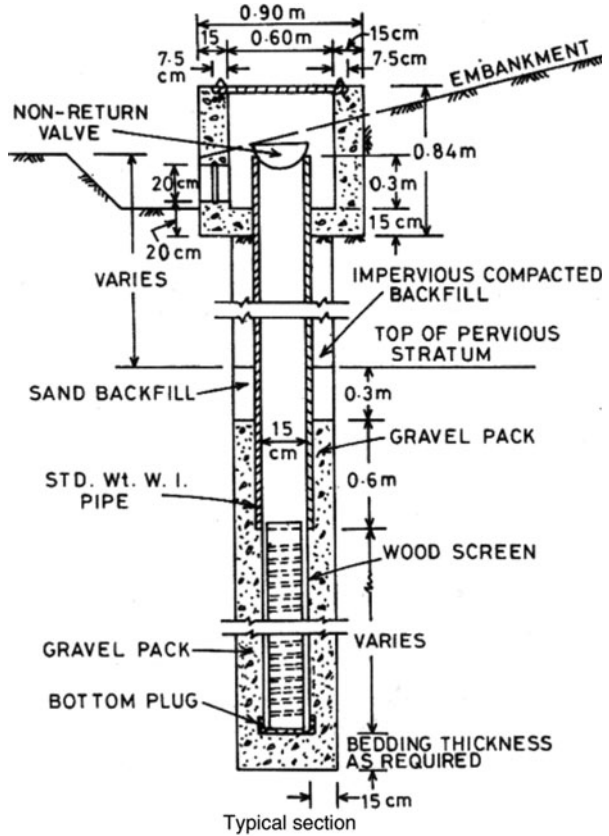
### 9.4.2.3 Relief Wells

Relief wells are usually provided in stratified foundations where pervious layers are overlain with relatively less pervious layers. It consists of a vertical hole in which a slotted pipe is centrally placed surrounded by suitably graded filter material. Studies have shown that the pipe diameter shall not be less than 15 cm and minimum thickness of filter around it should be 15 cm. The slotted pipe may be of timber, galvanized iron, cast iron, stainless steel, PVC or glass fibre. The typical slot size is 5 mm  $\times$  80 mm and total slot area should not be less than 10% of circumferential area of pipe.  $D_{15}$  size of filter material should be 1.2 times the width of the slot. The gravel size in filter is usually uniform and is about 9 to 12.5 times of aquifer particle size if it is uniform or 11 to 15.5 times if it is non-uniform or graded. The well should sufficiently penetrate in the pervious strata below the impervious one. The relief well spacing is usually 15 to 30 m. Closer wells can be made later if required to intercept seepage and to limit the hydrostatic head midway to a safe limit between the wells. This needs regular monitoring (periodical inspection) and maintenance of relief wells for the safety of dam. At the top of the well a non-return flap valve is provided along with a suitable cover and overflow arrangement. The well may be placed either at the toe of dam or a little away from it. A typical dam section with clay blanket and relief well is shown in Fig. 9.26(a) and details of the relief wells are shown in Fig. 9.26(b).



**Fig. 9.26 (a)** Clay blanket and relief well.

**Fig. 9.26 (b)** Details of a relief well.



### 9.4.3 Treating Clay Foundations

The clay foundations have low permeability and low shear strength and the pore pressures created due to load imposed by the dam do not dissipates quickly. Hence there is a possibility of failure through foundation. There is also a danger of bearing failure of foundations of saturated clays. It is thus essential to take precautions against such failures.

If the thickness of clay layer over the bed rock is less, it may be economical to excavate and refill the area. However, it should be investigated and evaluated for the cost as compared to other measures. In small height dams a shear key at the toe of

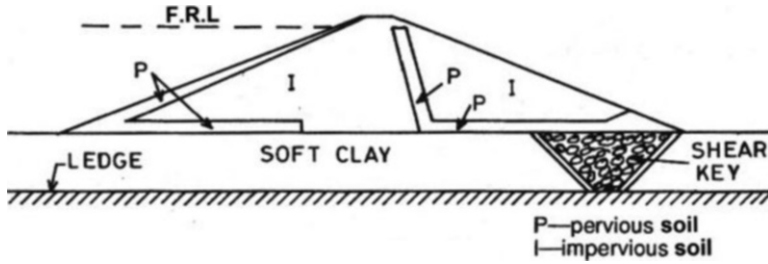


Fig. 9.27 Shear key.

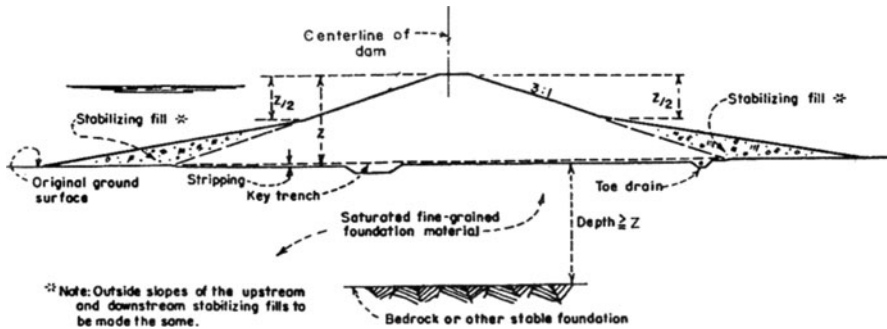


Fig. 9.28 Design of dam on saturated fine-grained foundation.

dam as shown in Fig. 9.27 may provide stability to the dam. The trench for the key is excavated upto the bottom of clay layer and backfilled with graded rockfill with filter between the clay and rockfill.

In several cases vertical drains have been used to improve shear strength and to reduce pore pressures at the end of construction. Horizontal drainage blanket should be provided and necessity of connecting the vertical drains to the blanket be examined. The drains are 15 to 20 cm in diameter and filled with clean coarse sand. Such sand drains of 25 cm in diameter spaced 4 to 5 m apart were used in river bed and left abutment of Tenughat dam (45 m high) in India. However, it is not recommended in case of small dams where practical solution is flattening of embankment slopes required by stability analysis with conservative factor of safety against sliding. But the slopes should not be steeper than 3:1 (H:V). The section of dam on clay foundation as proposed by USBR is shown in Fig. 9.28. It suggests that stabilizing fill (sand and gravel mix) shall be provided beyond the dam slopes from mid height of dam to provide extra stability. Slope of stabilizing fill as has been suggested by USBR is given in Table 9.2.

**Table 9.2** Recommended slopes for stabilizing fill for dams on clay foundation

Consistency	Average no. of blows per foot (0.3 m) within foundation depth equal to height of dam	Foundation soil group	Slopes of stabilizing fills for various heights of dams				
			50 ft (15 m)	40 ft (12.5 m)	30 ft (9.5 m)	20 ft (6.0 m)	10 ft (3.0 m)
Soft	<4	Special soils tests and analyses required					
Medium	4 to 10	SM	4.5:1	4:1	3:1	3:1	3:1
		SC	6:1	5:1	4:1	3:1	3:1
		ML	6:1	5:1	4:1	3:1	3:1
		CL	6.5:1	5:1	4:1	3:1	3:1
		MH	7:1	5.5:1	4.5:1	3.5:1	3:1
		CH	13:1	10:1	7:1	4:1	3:1
Stiff	11 to 20	SM	4:1	3.5:1	3:1	3:1	3:1
		SC	5.5:1	4.5:1	3.5:1	3:1	3:1
		ML	5.5:1	4.5:1	3.5:1	3:1	3:1
		CL	6:1	4.5:1	3.5:1	3:1	3:1
		MH	6.5:1	5:1	4:1	3:1	3:1
		CH	11:1	9:1	6:1		
Hard	>20	SM	3.5:1	3:1	3:1	3:1	3:1
		SC	5:1	4:1	3:1	3:1	3:1
		ML	5:1	4:1	3.5:1	3:1	3:1
		CL	5:1	4:1	3:1	3:1	3:1
		MH	5.5:1	4:1	3:1	3:1	3:1
		CH	10:1	8:1	5.5:1		

Note: Stabilizing fills are not needed when embankment slopes required are equal or flatter than the slope listed above.

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# Chapter 10

## River Diversion



### 10.1 GENERAL

River diversion is essential to isolate the construction area for foundation work of the dam and associated structures. As far as cost and time of constructing diversion works are concerned, these constitute a major item of the project and hence these require careful planning. The diversion works may differ from project to project. The choice of diversion scheme depends on following factors which require careful study as it is important for the economy of project:

- (i) Topography and geology
- (ii) Layout of associated works
- (iii) Type of dam
- (iv) Construction schedule
- (v) Diversion flood
- (vi) Cost and time required
- (vii) Risk evaluation

The topography and geology of site govern the choice of diversion scheme. It will not be same in both narrow and steep gorges and wide and flat river sections. The geologically sound rock abutments will generally favour diversion tunnels.

The magnitude and frequency of diversion flood depends on hydrological characteristics of river basin and type of dam. A concrete dam may be allowed to pass flood over incomplete dam but an embankment dam cannot be allowed to overtop. Therefore, diversion flood for a concrete dam may be either a low frequency flood or maximum non-monsoon flow but in case of embankment dam the diversion flood is a high frequency flood, say 1 in 100 years or more.

A diversion scheme for a higher flood will cost more but risk to damage will be less whereas the scheme for less discharge will be economical but risk to damage may be more. Hence, an assessment of risk potential to the damage of a dam is necessary to decide the diversion flood.

The time required to implement diversion scheme depends on its size and so it should be incorporated appropriately in the construction schedule after considering other factors.

## 10.2 DIVERSION METHODS

These are generally of two types: single stage and multi-stage diversion schemes.

### 10.2.1 *Single Stage Diversion*

In this type of diversion the structures for diversion scheme are planned to serve the purpose of river diversion for the entire construction period of the project. The structures of diversion scheme are to be completed before the start of construction of permanent works. This single stage diversion is used in narrow rivers valleys. The structures required for single stage diversion scheme are shown in Fig. 10.1.

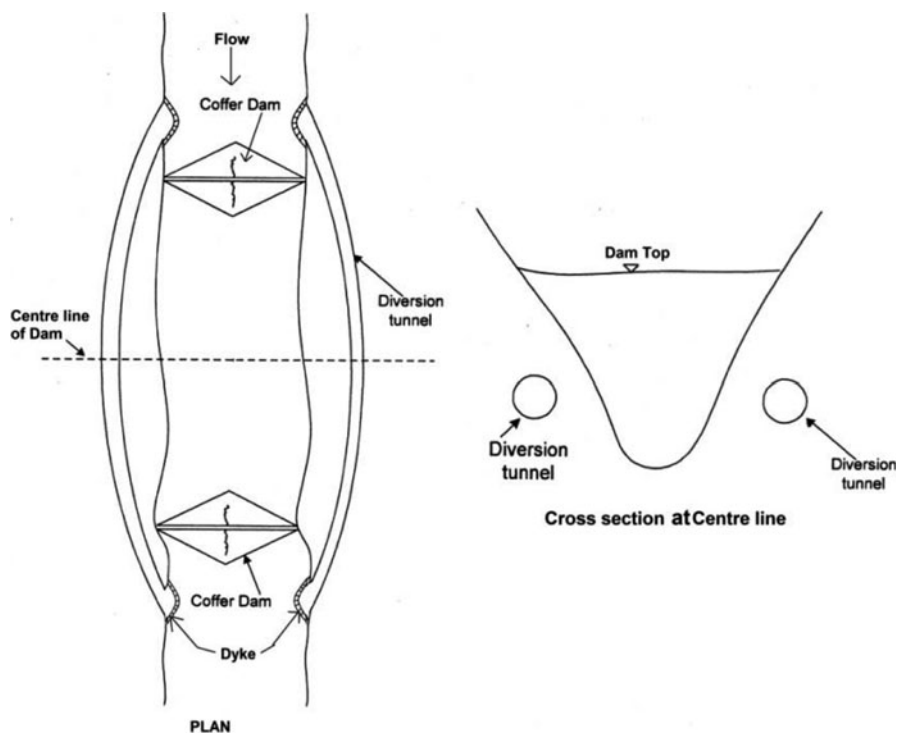
The sequence of construction and operation of the structures of this type of scheme till the construction of dam is as below:

- (a) Build small cofferdams/dykes to allow construction of diversion tunnels or culverts or flumes and the control works.
- (b) Complete the construction of tunnels/culverts and the control structures.
- (c) Divert flow through these passages, after construction of river closing structures.
- (d) Build cofferdam to the required height.
- (e) Keep diversion scheme in operation till the dam and other permanent structures are constructed.
- (f) Close diversion works and start filling the reservoir.

Tunnels are the choice for narrow valleys with good rock abutments, culverts and flumes used where river channel is relatively wide and abutment rock is not good for tunneling.

### 10.2.2 *Multi-stage Diversion*

The multi stage diversion is the most common arrangement on wide rivers. In first stage part of the river is blocked and the flow is allowed to pass through a small portion of river channel. The dam is constructed in the isolated part of river. In the second stage the river is allowed to pass through the constructed part of dam and the river is blocked in the rest portion for construction. A two-stage diversion sequence is illustrated in Fig. 10.2. The river blocking is done through cofferdams or embankments. The portion of cofferdams in contact with water is protected with heavy



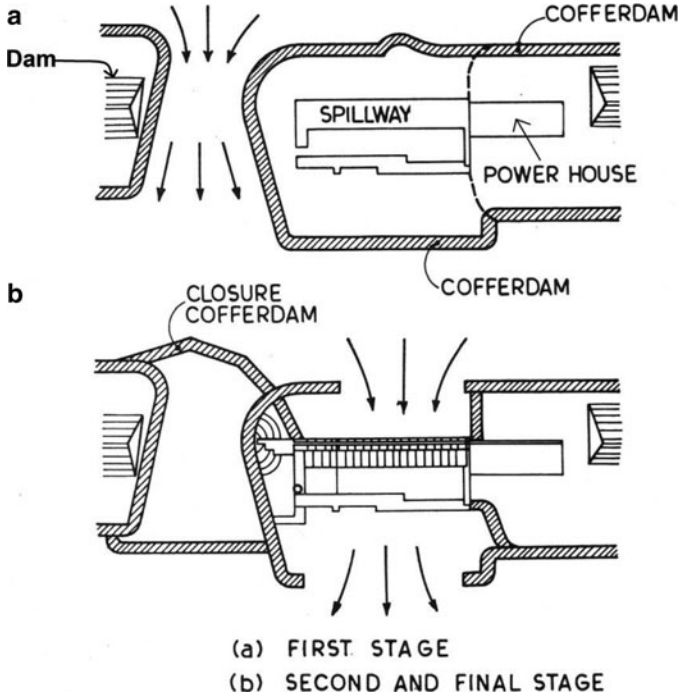
**Fig. 10.1** Single stage diversion.

rockfill, gabions etc. The flow velocity is kept to a reasonable value to avoid erosion. Depending on the river width two or more stages of diversion are adopted.

### 10.3 DIVERSION DISCHARGE

The selection of diversion discharge depends on river hydrology and on the risk, the project management is ready to take. The greater the diversion discharge more will be the cost but the risk to damage in case the river flood exceeds the diversion discharge is less. Smaller the diversion discharge lesser will be the diversion cost but risk to damage will be large. Hence an optimization between cost and risk shall be attempted to decide the diversion discharge.

Incomplete concrete dams can pass floods without appreciable damage. Hence generally diversion discharge for concrete dams vary from 1 in 10 to 1 in 25 year return period floods. In some cases the maximum non-monsoon flood from long-term discharge data is adopted as diversion discharge. Sometimes discharge corresponding to the non-monsoon peak flow of 1 in 25 year return period is adopted as diversion flow.



**Fig. 10.2** Schematic illustration of multi-stage diversion.

The risk evaluation of a diversion discharge of return period  $T$ -years being exceeded at least once in a construction period of  $L$ -year is given by

$$R = 1 - \left(1 - \frac{1}{T}\right)^L = 1 - e^{-L/T}$$

or approximately

$$R = \frac{L}{T + 0.5L}$$

If  $T = 10$  years and  $L = 3$  years

$R = 26\%$  i.e. risk of overtopping of the cofferdam.

If 5% risk is acceptable then  $T = 60$  years.

If  $T = 1000$  years and  $L = 10$  years, the risk  $R = 1\%$  i.e. the probability of flood exceeding 1000 year return period flood is once during the construction period of 10 years and this may be acceptable in case of major projects. Since the embankment

dams cannot be allowed to pass flood on incomplete dam, the diversion discharge of higher return period should be adopted. For dam projects on Ramganga, Beas and Tehri the diversion discharge is adopted corresponding to a higher return period flood. In case of Tehri dam, the diversion discharge was adopted equal to the river flood corresponding to a return period of 1000 years. This criteria is too expensive and is generally not adopted for small projects. In small projects on rivers which do not have history of sudden large floods a diversion discharge is adopted equal to a frequency of 5 to 10 times the construction period.

## 10.4 DIVERSION STRUCTURES

### 10.4.1 *Tunnels*

#### (i) **General**

Tunnels are the only feasible option in steep-sided rocky dam sites. Tunnels upto 16 m diameter for a capacity of 2500 m<sup>3</sup>/sec have been provided on some projects. The diversion tunnels should be of a length adequate to bye-pass the construction area as shown in Fig. 10.1. The size and number of the diversion tunnels depends on the diversion discharge and height of upstream cofferdam. High upstream cofferdam gives higher head which results in smaller size tunnel and lesser head results in bigger size tunnel for a given discharge. Hence an economic study of cofferdam height against tunnel size shall be made. Such an economic study can be made by working the cost of tunnel and the corresponding cofferdam for different diameters of tunnel. The minimum total cost will give the optimal size of tunnel. Besides economics, the tunnel size is also governed by the type of rock and geology of abutment. The height of cofferdam is also generally limited by the feasibility of its construction in one season which depends on the availability of resources. In many major projects two or more diversion tunnels have been made generally using both the banks. In Tehri dam four diversion tunnels of 12.5 m diameter are provided.

After using the tunnels for diversion, these are either permanently plugged or used as permanent structures such as shaft spillways or for power generation or emergency outlets. In all cases a concrete plug is placed in the upstream reach of the diversion tunnel. To construct the plug a one-time use stop log is placed at the mouth of tunnel for which arrangements are made at the inlet portal. Sometimes diversion tunnel is used to pass the river flow till the reservoir is filled upto the outlet/spillway crest level. In that case the regulation gate is placed at the mouth of tunnel. The plug is generally placed near the upstream face in line with the grout curtain of the dam. The plug with the use of diversion tunnel as a permanent structure (spillway) is illustrated in Fig. 10.3 (Glen-Canyon dam in USA). At Ramganga dam (India) two diversion tunnels each of 15 m diameter were constructed on right abutment and afterwards one tunnel was used to install a valve to be operated as emergency outlet

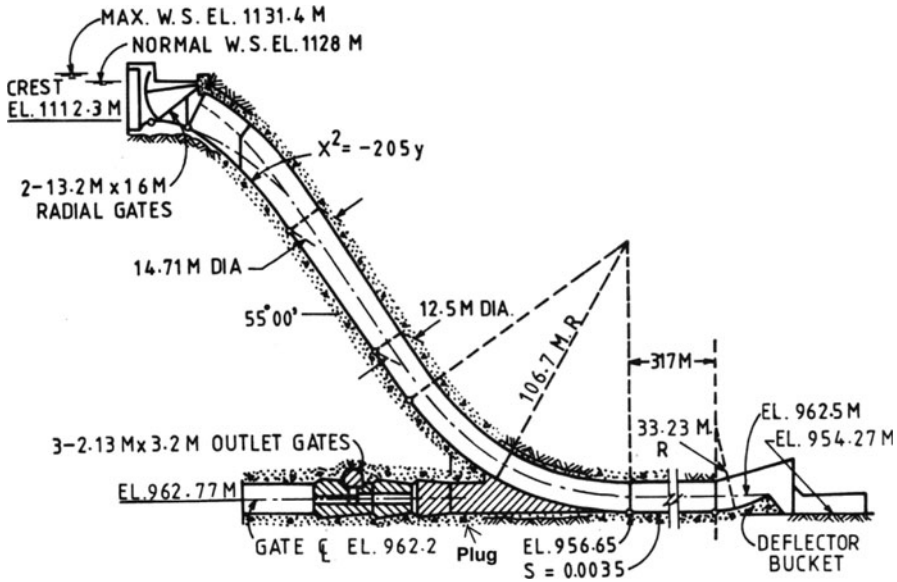


Fig. 10.3 Glen Canyon dam – Profile of left spillway and tunnel plug.

and the other tunnel was used as head race tunnel for the dam toe power house. At Tehri project there were four diversion tunnels each of 12.5 m diameter and these were later converted into shaft spillways. Such conversion of diversion tunnels into permanent structure is very common for embankment dams where diversion tunnels are made to cater for high discharge and are constructed at high cost.

## (ii) Lining

Generally lining is desirable in diversion tunnels, when these are to be used for a number of years, to avoid damage due to high velocity sediment laden flow. Even in tunnels in good rock provided to pass only non-monsoon flows, it is good practice to line the invert portion to prevent damage from rolling debris. Generally M20 grade CC lining is provided and its thickness may be based on the thumb rule of 6 cm for one metre of diameter of diversion tunnel. If the tunnels are to be used as permanent structures, the tunnels shall be designed as per IS 4880 Part IV.

## (iii) Velocities

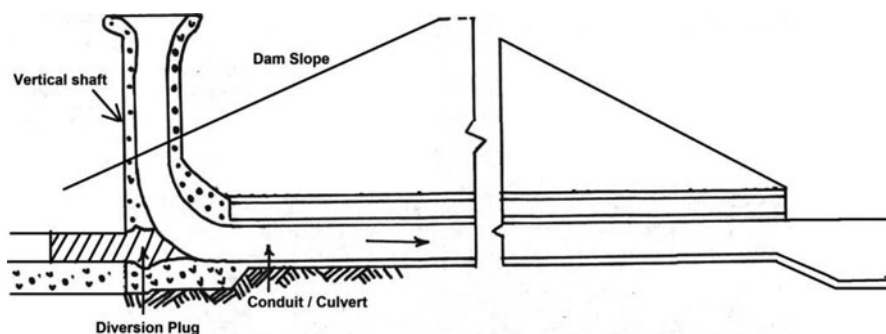
Concrete lined tunnels in most cases have satisfactorily withstood velocities upto 25 m/sec and even higher. Since diversion tunnels are designed for high velocities it is essential that concrete lining surface is smooth to avoid cavitation damage. A small off set of even 1 mm may cause cavitation.

**(iv) Hydraulics**

Diversion tunnels may be designed either as a pressure tunnel or as a free flow tunnel. A free flow tunnel is hydraulically designed as an open channel flow using continuity equation and Manning's equation for velocity. The required slope shall be provided. The flow depth for free flow in tunnel is limited to a maximum of 75% of the diameter. In case of pressure flow, tunnel runs full and the diameter is dependent on velocity permitted in the tunnel which is a function of the head available. There is no hydraulic requirement of slope. A nominal slope of 1 in 500 or less may be provided from construction consideration. Requirement of air vents shall also be examined in addition to free space. The required slope shall be ensured in the layout of tunnel. The designer must also ensure that change from free surface to pressure flow takes place smoothly inside the tunnel. Vortex formation at the inlet should be avoided. Some form of energy dissipation arrangement is generally required at the exit end. In case of a major project the hydraulic design of diversion tunnel shall be tested on a hydraulic model.

**10.4.2 Culverts**

When construction of tunnels is not feasible due to poor rock, concrete culverts in the dam foundation may be provided. These are constructed in dry behind the cofferdam. These are closed in the same way as tunnels after the use as diversion structure. These can also be used as permanent water carrier for a spillway after providing vertical shaft or tower intake. It is illustrated in Fig. 10.4.



**Fig. 10.4** River diversion through a conduit and vertical shaft for its use as spillway.

### 10.4.3 Channels

Diversion channels are generally used in wide river valleys where construction of tunnels or culverts is not feasible. The channel is located on the bank and is constructed in dry. These have been provided even for very large diversion discharges. For example a diversion channel was constructed at Ukai dam (India) for 49,500 cumec diversion discharge. The dam is an embankment dam. For the construction of Itaipu hydroelectric project in Brazil, diversion channel for 30,000 cumec diversion discharge was made. Diversion channels for post-monsoon flows have also been constructed to build concrete gravity dam for Sirisailam and Sardar Sarovar projects. These are shown in Figs. 10.5 and 10.6. In case of Sirisailam dam the diversion arrangement consists of both diversion tunnel for 566 cumec and channel on the other bank for the rest of diversion flow i.e. for 285 cumec. In Sardar

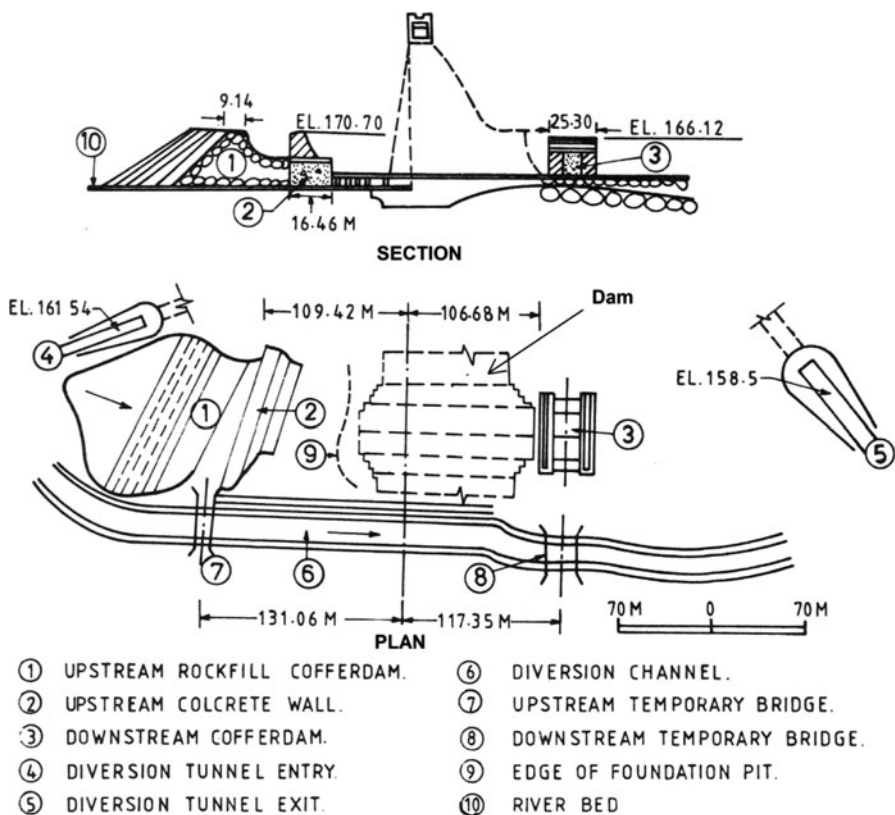
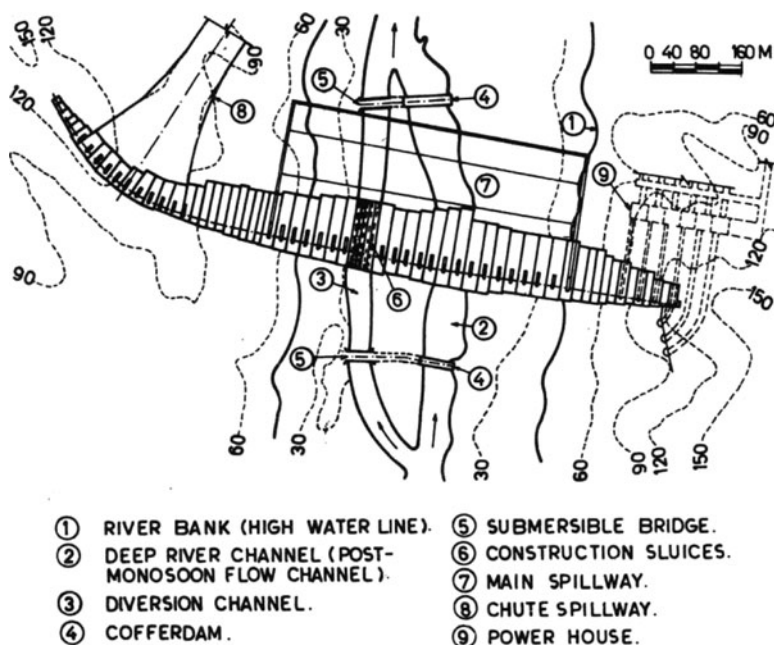


Fig. 10.5 Layout of diversion works at Srisailam dam.



**Fig. 10.6** Layout plan of diversion works at Sardar Sarovar dam.

Sarovar dam the river is very wide and diversion for construction of dam is done for non-monsoon flow through a channel. It was later closed through the dam blocks and construction sluices were left in dam blocks.

## 10.5 COFFERDAMS

The height of upstream cofferdam is related to the tunnel design. It has to provide required head for the designated flow through the tunnel. The height of downstream cofferdam is determined from stage discharge curve of the river for the designated flow, so that water after being discharged by tunnel or channel does not enter the construction area.

The cofferdams are made either as rockfill dams/dykes or as stone masonry or concrete dams. Recently roller compacted concrete dams of small height have been constructed as cofferdams. The choice depends on availability of material, foundation condition and time constraint. These shall be designed safe against seepage through the foundation and the body of cofferdam. A positive cut-off of concrete

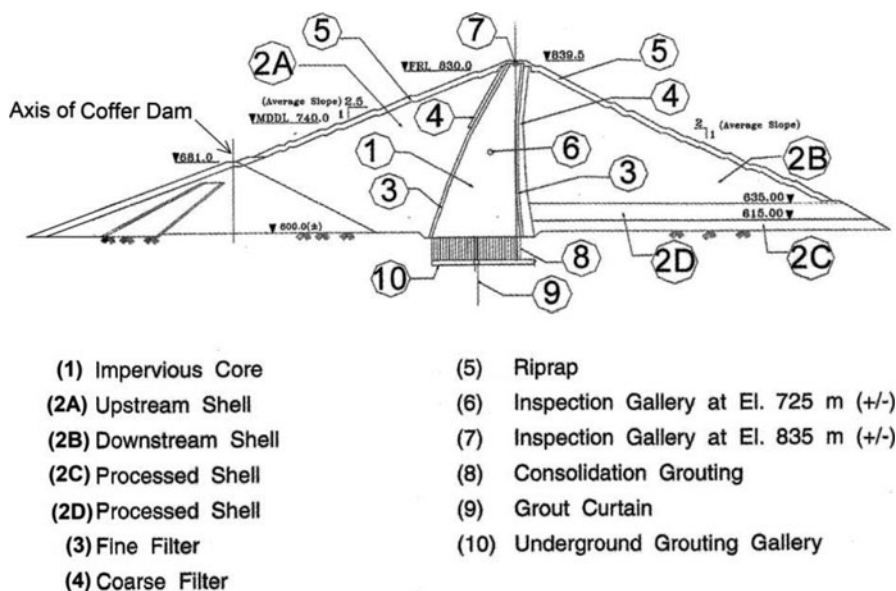


Fig. 10.7 Typical section of Tehri dam showing upstream cofferdam as part of main dam.

diaphragm, or sheet pile or grouting upto the rock foundation shall be provided in the foundation of cofferdam. In the body of rockfill cofferdams, either a clay core be provided or impervious membrane on U/S face be made. CFRD as cofferdam is usually not adopted as construction of impervious membrane takes a long time. The clay core type rockfill cofferdams are generally made which are later made the part of the main embankment dam. The cofferdams are, in such cases, made of the same material and specification as the main dam. This is done at Tehri dam where the upstream cofferdam is about 80 m high with an inclined clay core. It is shown in Fig. 10.7.

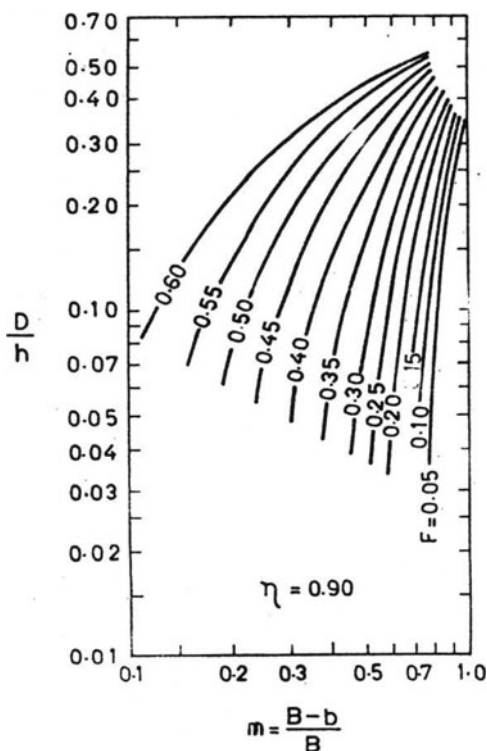
## 10.6 RIVER CLOSURE

River closure is required when river flow is first diverted into diversion tunnel in single stage scheme and at the end of first stage when multi stage diversion scheme in wide rivers is adopted. The river closure is usually done in low discharge period. There are two methods of river closure.

- (i) **End dumping method:** It involves advancing the embankment dyke above water level from one bank or both until the gap is closed. It requires dumpers and dozers to doze large quantity of material.
- (ii) **Frontal dumping method:** When closure material is placed uniformly across the whole width of the gap. It requires use of cable crane or a bridge to transport and dump the material.

Since large size dumpers and powerful dozers are now available, generally end dumping method is adopted. To operate these equipments the embankment top shall be atleast 15 m wide. Large size material (quarry run rock) of size 1–5 ton is required for closing the gap in final stage when gap width is small and flow velocity is large. Considerable loss of dumping material takes place at this stage. To reduce losses of rock material, big size concrete cubes or tetrahedrons may be used. These will be relatively costly. Guidelines based on laboratory studies for the size of material required are given in Fig. 10.8 and Table 10.1.  $D$  is size of material and  $h$  is depth of flow in Fig. 10.9.  $F$  is Froude number of flow. For a certain gap width and flow parameters the size  $D$  of the required material can be obtained from the curves.

**Fig. 10.8** Design curves showing plots of  $D/h$  versus  $m$  at various values of  $F$ , for a closure efficiency of 90%;  $B$  and  $b$  respectively denote stream width and gap width.



**Table 10.1** Material size for end-dumping method

Prevailing water depth (m)	First stage		Last stage	
	Deep water conditions (depth over 3 times the water head)	Shallow water (depth less than twice the water head)	Large loss of material accepted in fairly deep water	No loss of material accepted
0.5	2–10 kg		10 kg	100 kg
1	60 kg	0.5 – 1 t	120 kg	0.5 – 1 t
2	500 kg	5 – 10 t	1 t	5-10 t
3	2 t		4 t	20 t
4			8 t	50 t

## 10.7 OVERTOPPING OF COFFERDAMS

The overtopping of cofferdams should receive consideration because it may be cheaper to allow overtopping than to provide a high cofferdam. It may get damaged to some extent (10 to 50% of cost of cofferdam) during overtopping but initial cost of big size cofferdam may be larger. However, indirect cost of time required to repair the damage should also be considered.

It is generally advisable that in case of high diversion flows safe overtopping of cofferdam shall be examined. It is preferable to accept overtopping beyond 10-years return period flood and the damages in cofferdam rather than constructing a higher cofferdam for allowing overtopping by a flood exceeding 25-year return period discharge.

The protection of downstream slope of rockfill shell is required if overtopping is allowed. Excessive drops on unprotected slopes for flows in excess of  $1 \text{ m}^3/\text{sec}/\text{m}$  must be avoided but larger flows for drops less than 3 m may be permitted. Drops for different surfaces (depending on slope protection) which may be allowed are given in Table 10.2.

If downstream rock fill is protected by steel bars anchored into rockfill and the concrete blocks are placed, a discharge of 10 to  $20 \text{ m}^3/\text{sec}/\text{m}$  with a drop of 10–20 m may be permitted. In some cases bituminous concrete or RCC protection of D/S slope of rockfill has been successfully used. If a roller compacted concrete cofferdam is adopted overtopping may be allowed with little damage.

## 10.8 CLOSURE OF DIVERSION WORKS

The diversion works like tunnel/culvert are to be closed after completion of permanent work and before starting the filling of reservoir. The usual method is to close the diversion tunnel at the inlet by dropping a bulkhead gate of steel or concrete. Once the gate is placed, the permanent concrete plug can be constructed. After

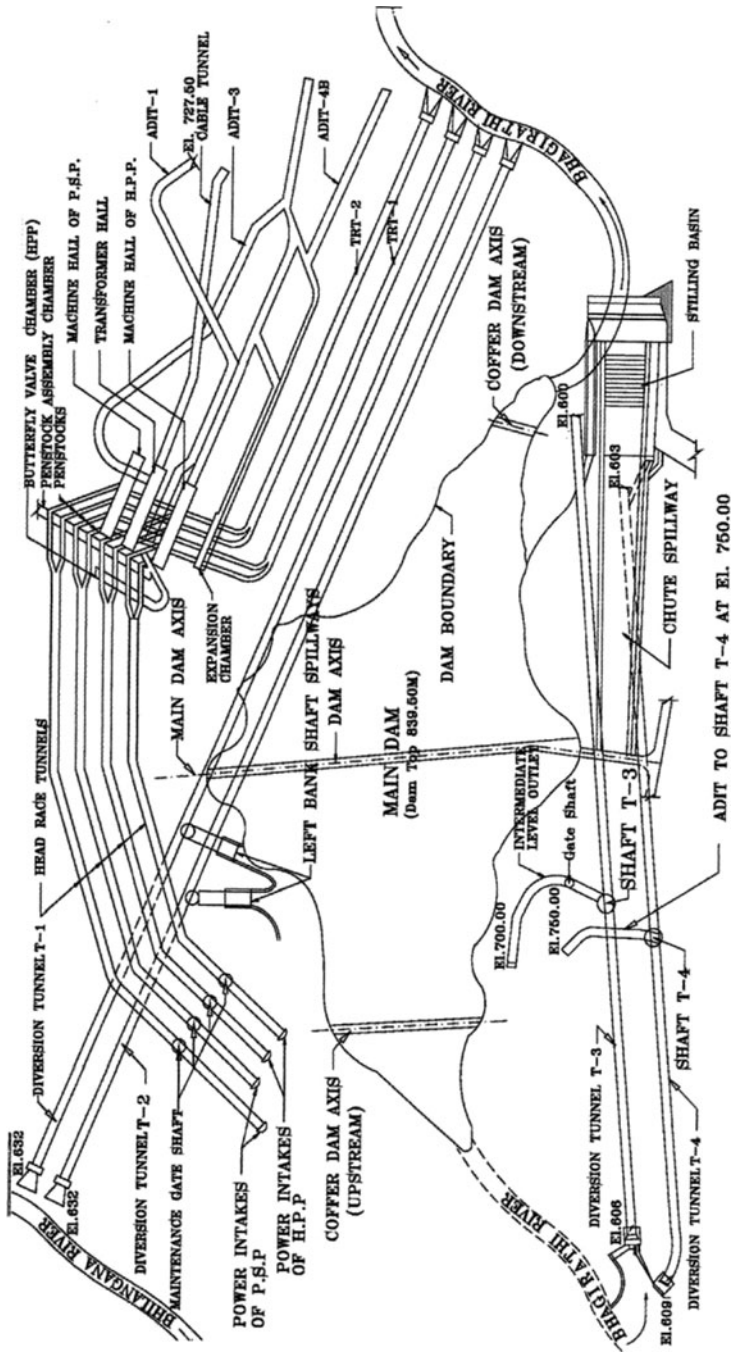


Fig. 10.9 General layout plan of Tehri dam.

**Table 10.2** Type of slope protection against the drop

Type of surface	Drop
Classified rockfill	Upto 3 m
Precast concrete blocks (20 t)	3 to 6 m
Reinforced rockfill	10 m
Reinforced concrete lining	30 m

constructing the plug, the gate may be taken out and can be used elsewhere if economical, otherwise it is left in place.

The length of concrete plug should be adequate to resist the water head through shearing strength of concrete along the cylindrical surface of plug. Normally a length equal to 1.5 times the diameter of tunnel is found adequate for the plug. However, it should be designed according to IS 11105.

If there are more than one tunnel/culvert these can be closed one after the other. If they are at different levels, higher level tunnel may be kept open to regulate the filling of reservoir.

If the tunnel is to be used as a permanent structure as spillway or intake, the plug is placed just upstream of the junction of shaft with the tunnel as is shown in Fig. 10.3.

## 10.9 EXAMPLES OF RIVER DIVERSION

### 1. Bhakra Dam

Bhakra dam is 226 m high concrete gravity dam on Sutlej river in India. It is located in a narrow gorge. Two diversion tunnels each 15.2 m in diameter nearly 800 m in length, one on each side, were provided. Upstream cofferdam was 65.5 m high and downstream was 40 m high. The tunnels were designed as free flow and laid at a slope of 2.68 m/km (same as river slope). Each tunnel had a capacity of 3228 m<sup>3</sup>/sec giving a maximum velocity of 21 m/km. The tunnels were lined with 1.3 m thick concrete. Diversion tunnels were closed at inlet through stop logs and these were plugged with concrete plugs.

### 2. Glen Canyon Dam

It is a 216 m high arch dam in USA. Two 12.5 m diameter concrete lined tunnels were provided for diversion, one in each abutment. Maximum discharge through each tunnel was 1857 m<sup>3</sup>/sec giving maximum velocity of 15 m/sec. The tunnels after diversion were converted into spillway tunnels (Fig. 10.3). The spillway tunnels were designed to flow partly full (0.7 times the diameter) with maximum discharge of 394.3 cumec per tunnel. Model studies indicated maximum velocity of 49.4 m/sec. Damage to Canyon walls was prevented by concrete walls at the exit end.

### 3. Srisailem Dam

Srisailem dam of maximum height of 144 m, is a masonry-cum-concrete dam of straight gravity type across river Krishna, with a maximum flood discharge of 30,000 m<sup>3</sup>/sec. The diversion arrangement were designed for 850 m<sup>3</sup>/sec, a capacity which would ensure working during non-monsoon period from November to June. It comprised: (i) a 9.14 m diameter tunnel of 566 m<sup>3</sup>/sec capacity through the left abutment, (ii) a diversion channel of 15.24 m bed width to carry the balance 285 m<sup>3</sup>/sec dry-weather discharge, and (iii) two semi-permanent concrete cofferdams, one on the upstream and the other on the downstream. The schematic layout is shown in Fig. 10.5.

The diversion tunnel is lined with concrete (minimum thickness 400 mm) and is fitted with regulating gates in a shaft located 32 m downstream of the entrance. Three sluices of size 1.22 × 1.52 m were provided on the left side wall of the diversion channel for admitting water (to serve as water cushion) into the main dam foundation area between the two cofferdams, at the end of the working season before the flood overtopped the upstream cofferdam.

The construction of the cofferdams was to be done under about 2 m depth of water due to backwater from Nagarjunasagar dam located further downstream. The concreting technique was adopted for construction of the cofferdams. The shape of the crest and of the downstream profile of the cofferdams were provided on basis of model studies. The foundation below the upstream cofferdam consisted of boulders and sands upto a depth more than 30 m, whereas the downstream cofferdam rested on rock muck deposited from earlier construction. The curtain grouting was done upto bed rock to prevent seepage through foundation. Protection against scour was also provided by placing gabions downstream of the cofferdams.

### 4. Sardar Sarovar Dam

Sardar Sarovar dam is a 125 m high concrete gravity dam with a crest length of about 1200 m across river Narmada. The river flows in a width of about 500 m during monsoons (June to September) and is confined to 60 m width during post-monsoon period. The river diversion scheme comprises diversion of the post-monsoon flow through an open cut channel excavated in the river bed on the left side of the deep river channel carrying post-monsoon flow. It is shown in Fig. 10.6. Two submersible cofferdams, one upstream and the other downstream of the dam-seat area, were constructed across the deep river channel. Two submersible bridges were constructed across the diversion channel to provide communication across the river during the post-monsoon period. Construction sluices at the foot of two spillway blocks located in the diversion-channel portion were constructed as part of the diversion scheme.

The cofferdams are designed to withstand submergence during floods to as high as 25 m above top of cofferdams corresponding to a maximum flood of 60,000 m<sup>3</sup>/sec. The layout of the cofferdams is so planned as to provide enough working space on both sides of the dam-seat area.

Each cofferdam comprises two rows, spaced 15 m apart, of 1220 mm hollow steel piles driven in contiguous fashion under flowing water condition with colgrouted rubble mass in between the two rows. The piles thus act as shuttering for colcreting as well as positive cut-off. The hollow piles were driven down to about 1-m depth inside the bed rock and filled with concrete using tremie after lowering reinforcement cages. Each cofferdam was divided into three compartments lengthwise by providing two cross rows of piles to facilitate colgrouting by stages. Reinforced-concrete coping was provided on top of each row of piles to achieve monolithic action. Wire-netted gabions were provided on both sides of the cofferdam as a protective measure against scour.

## 5. Tehri Dam

Tehri dam is a clay core rockfill dam of height 260.5 m above foundation constructed across river Bhagirathi in Uttarakhand (India). The river diversion scheme comprises four diversion tunnels each of 12.5 m diameter to pass a diversion discharge corresponding to a flood of 1 in 1000 years. The layout of tunnels is shown in Fig. 10.9. The inlet levels of left bank tunnels  $T_1$  and  $T_2$  was 632.0 m and outlet was at 596.0 m. The height of upstream cofferdam was worked out as 81 m above river bed at RL 600.0 m. For right bank tunnels  $T_3$  and  $T_4$  the inlet was at 606 m and 609 m and outlet was at 600 m and 603 m. For 1 in 100 years return period flood the height of cofferdam was worked out as 67 m. Construction of such a high cofferdam was not feasible in one season, so it was constructed in stages. When work on dam was started a dyke upto 625 m level was constructed. Dam fill was carried upto 615 level and the toe of downstream fill was protected by providing 20 m additional reinforced rockfill. The outer slope of this rockfill was protected by mutually tied CC blocks and a toe wall at the end, and top was protected by RCC slab. In a length of 50 m upstream of RCC slab an impervious blanket of PVC sheet with boulder protection on top was provided for sub-surface flow. Later the cofferdam was raised to 660.9 level in two seasons and to final level in the third stage. But the cofferdam at no stage was overtopped because of low floods in this period.

All the four tunnels were converted into shaft spillway by connecting them from the reservoir level through vertical shafts of about 200 m height. The shafts are connected with the tunnel eccentrically to develop swirling flow in tunnel for killing the energy and reducing exit velocity. It is shown in Fig. 10.10. The exit velocity of tunnel was reduced to about 20 m/sec. The design was based on hydraulic model studies.

The closure of these diversion tunnels was planned to meet the construction schedule of various works and the progress of construction activities. Tunnels  $T_3$  and  $T_4$  on right bank were closed first to isolate the area for the construction of spillway stilling basin. These were closed by dropping stop log gates at the intake and finally making a 35 m long concrete plug just upstream of junction of shaft and tunnel. After a couple of years first tunnel  $T_1$  which is at higher level of 632.0 m was closed and  $T_2$  tunnel was left open to pass the floods for a few more years. And lastly tunnel  $T_2$  was closed and impounding of reservoir was done in stages.

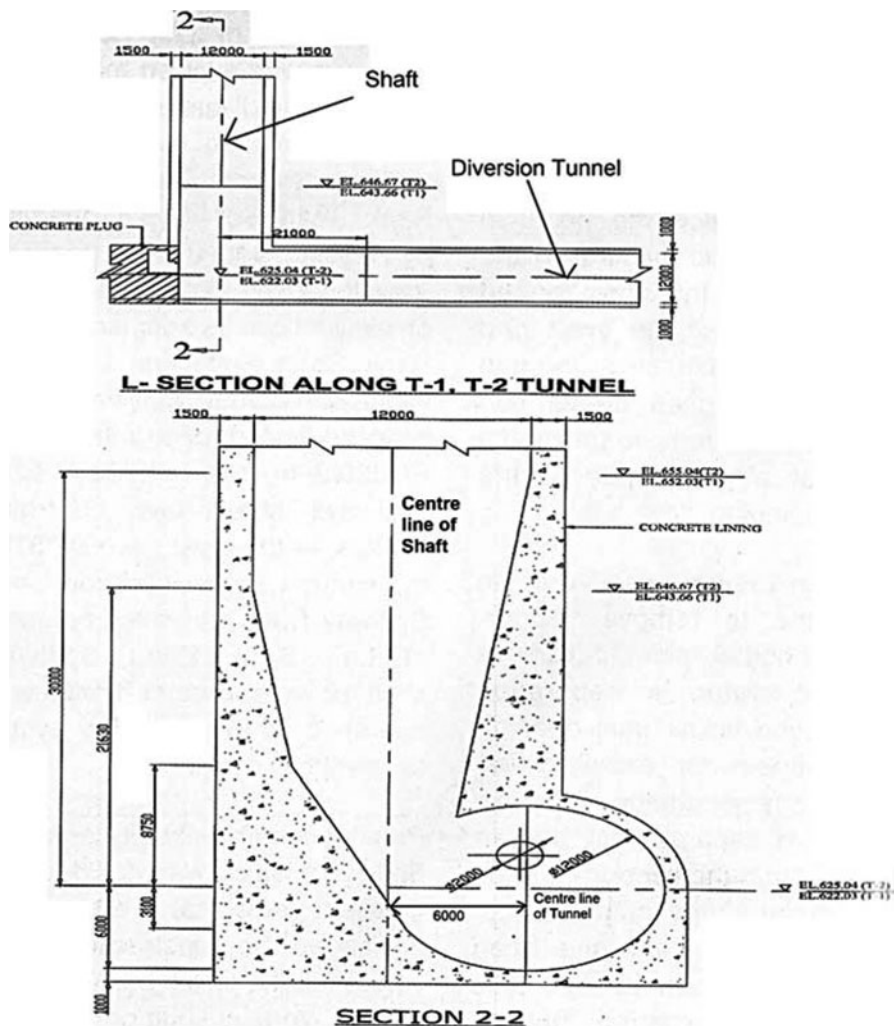


Fig. 10.10 Swirling device connection of shaft and diversion tunnel.

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# Chapter 11

## Hill Slope Stabilization at Dam and Power Projects in Himalayas



### 11.1 GENERAL

Himalayas are the youngest mountains in the world and its slopes specially in lesser Himalayas are covered with thick overburden and these slopes most of the time get destabilized due to natural causes and whenever some construction activity is carried out by cutting the slopes. Rock slides are very common on steep slopes. Stratification of rocks in Himalayas is characterized by weak rock layers sandwiched between two strong rock layers. Rocks are also highly sheared and fractured. The region is highly seismic and many high intensity earthquakes have shattered the region. Large temperature variation causes weathering of the rocks. Thus, slides and rock-falls in river valleys in Himalayas at any location depends on topography, geology and climate. These make it highly difficult to predict the slides or rock-falls. Besides topography and geology, climate is also a major factor for hill slides in the river valleys of Himalayas. Cloud bursts, which are unpredictable, frequently occur in different regions of Himalayas and cause landslides and rock-falls which often become catastrophic due to heavy loss to life and property and damage to existing projects. Such catastrophic events due to cloud burst occurred in recent years in 1970 and 2013 in Alaknanda Valley and in 1978 it occurred in Bhagirathi valley. The Alaknanda and Bhagirathi rivers are the tributaries of river Ganga.

The landslides caused by cloud burst block the river and a reservoir is created behind the block which after sometime is overtopped causing a highly sediment laden flood wave in the downstream. Such a flood wave created during 1978 in Bhagirathi damaged the incomplete structures at Maneri Hydro Project which was under construction. In August 2000 a flood wave with high sediment concentration occurred in Sutlej river causing serious damage to bridges, existing power plants and incomplete structures of Nathpa Jhakri Project. Similarly, the flood wave created in Alaknanda in 2013 overtopped and by-passed the diversion structure of Vishnuprayag Hydro Project which was in operation and the flood wave in Alaknanda's tributary Mandakini caused havoc in the vicinity of famous temple of

Kedarnath and the neighbouring settlements resulting in huge loss of life and property. It took more than a year to restore the operation of the power project. In 1970, the flood wave in Alaknanda caused silting practically upto the FSL of Upper Ganga Canal which takes off from river Ganga from Hardwar in a reach of about 12 km upto Pathri Power Station. In its travel upto Hardwar this flood wave caused huge damage to roads, bridges and the towns located along the river. About a decade ago a massive landslide on the right bank of river Bhagirathi practically destroyed the township of Uttarkashi.

Landslides from the slopes are generally activated either due to reduction in bond or friction along the surface of separation between two types of material as a result of entry of water during rainy season or probably due to being overloaded if it had withstood many rainy seasons without visible damage. The most common driving force to destroy the slope and cause landslide is gravity i.e. weight of slope material and that of super imposed load combined with the effect of entry of excess free water. Therefore, most of the landslides/rockslides occur during or soon after rainstorm or rainy spells. Dry season land-slides are a very rare exception.

Another cause for landslide or reason to destabilise the slope is cutting of the toe of the slope and removal of lateral support for certain constructional activity like road or dam and its associated structures like intakes, stilling basin, power house etc.

A large number of dam and hydro power projects which are located in Himalayas have been affected by mass movement processes. Some of these are listed below:

- (i) Loktak Hydel Project, Manipur
- (ii) Chineni Hydel Project, Distt. Udhampur, J & K
- (iii) Pair & Tanger slides around proposed reservoir of Sawalkot Project, J & K.
- (iv) Lower Jhelam hydro project, district Baramullah, J & K.
- (v) Baira Siul Hydel Project (H.P.)
- (vi) Pandoh Dam (H.P.)
- (vii) Giri Power House, District Sirmaur (H.P.)
- (viii) Pong Dam intake (H.P.)
- (ix) Rishikesh-Chilla Hydel Project (U.P.)
- (x) Ichari Dam on Tons (Uttarakhand)
- (xi) Tehri Dam on Bhagirathi (Uttarakhand).

On each of these project sites a number of slope stabilization measures have been implemented at substantial cost to the project.

## 11.2 STABILIZING MEASURES

A number of methods are used for stabilization of potentially unstable slope. Some methods which are commonly used are:

- Vegetal cover – to reduce seepage of rain water.
- Drainage improvement – to prevent entry of excess free water.
- Construction of retaining walls/breast walls – to support slipping mass.

- Piling and ground improvement – deep concrete piles going upto bed rock to act as shear keys, ground improvement is achieved by providing wide berms at intervals along the slope. These two are generally provided together. Sometimes only berms are provided.
- Protective facing – shotcreting of weathered rock surfaces. It is sometimes done with wire mesh.
- Anchoring – to anchor the slipping rock mass with strong rock strata. It is carried out by (i) rock bolting and (ii) pre-stressed anchors. At some locations both are provided.

## 11.3 ANCHORING

Anchoring is commonly used with other measures for stabilizing slopes at and around dam and power project and is discussed below.

### 11.3.1 *Rock Bolting*

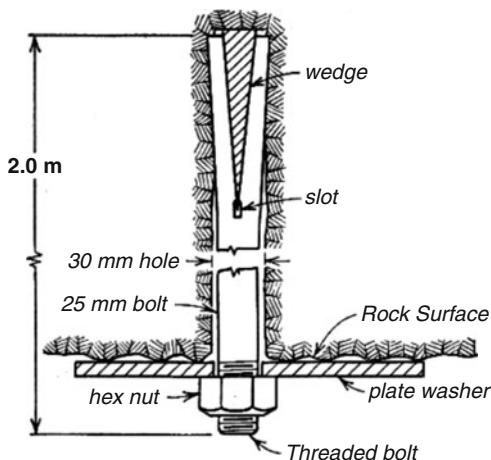
The rock bolts hold together the weathered potentially unstable rock mass with the firm stratum and prevents mutual sliding at the interface. Rock bolting is commonly used in all the works located in rocky environment and especially in underground works.

For rock bolting holes are drilled through the stratum and into the firm stratum across the interface plane and special bolts with a wedge at the end are inserted (Fig. 11.1) and tapped with a hammer. A small steel plate is placed at the outer end and a nut is screwed seated against the plate. Impact is used to drive the bolt into the rock. Because of wedge action the bolt ends spread apart and thus friction develops between the bolt and the rock. The length and spacing of the rock bolts has to be worked out depending on the rock mass to be supported. The diameter of the bolts normally used is 25 mm or sometimes more. Besides wedge type other type such as grouted rock bolt and resin filled type are also in use.

### 11.3.2 *Pre-Stressed Anchors*

The pre-stressed anchors for slope stabilization are used when anchorage forces are too large. The first application of pre-stressed anchor dates back to 1934 when a French Engineer Andre Coyne used these anchors to stabilize the Cheurfas Dam in Algeria. High capacity vertical holding down anchors of 1100 tons were used. Improvements in construction techniques particularly in drilling and pre-stressing and lot of research work during 1980s in load transfer mechanism and investigations

**Fig. 11.1** Typical rock bolt detail.



in performance of anchors increased confidence in the use of high capacity anchors for stabilizing weak rock slopes. These have been used on the projects constructed in Himalayas in the last three-four decades. The typical details of a pre-stressed anchor are shown in Fig. 11.2 (IS 10270).

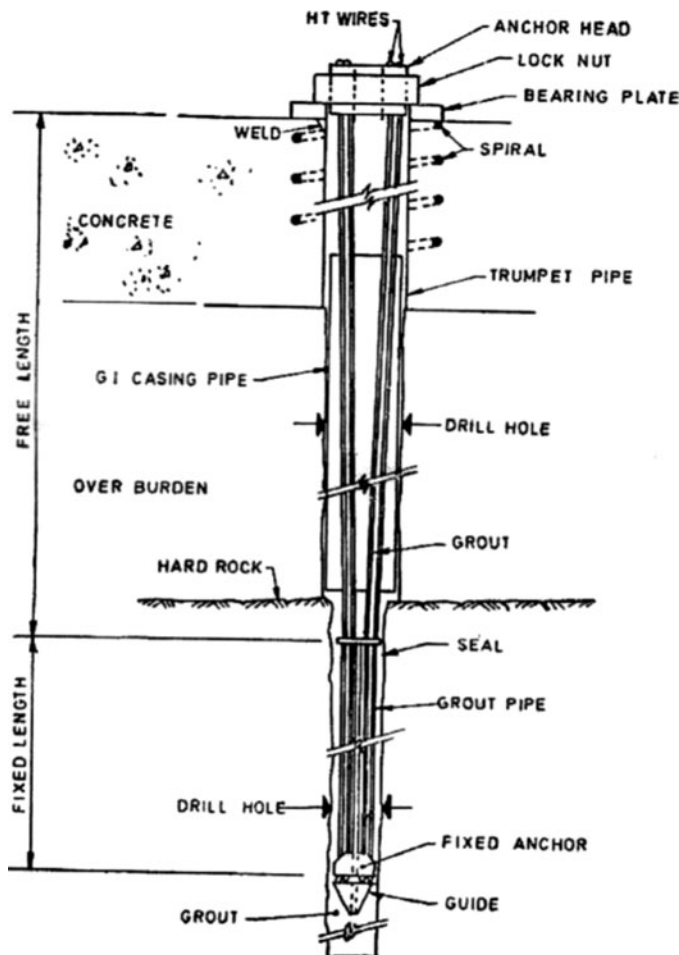
In slope stabilization, anchors are structural members which transmit tensile forces into the rock mass. The tensile force introduced is resisted by shear strength of the surrounding mass. They are inserted into boreholes and bonded to the rock by grout. Their action is two-fold. Firstly, on tensioning an anchor, the stress field gets modified in the vicinity of the anchor. Secondly, anchor also acts as a preventive measure against the further disintegration of the rock while holding a block of rock in its original position. The general arrangement of use of anchors for rock slope stabilization has been given in Fig. 11.3.

The design of anchors for any specific site should have the safeguards against the following modes of failure.

- (i) failure of the grout/tendon bond;
- (ii) failure of the ground/grout bond;
- (iii) failure within the ground mass;
- (iv) failure of the tendon steel or a component;
- (v) crushing or bursting of the grout column surrounding the tendon; and
- (vi) failure of a cluster of anchors.

**(a) Load Transference Mechanism from Tendon to Grout**

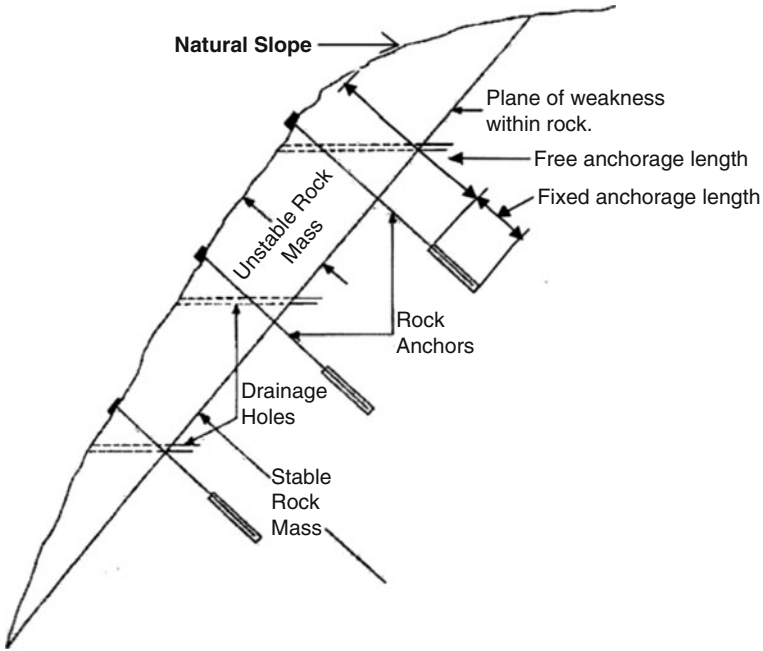
Initially, on loading of the tendon relative to the grout, bond (adhesion) is mobilized. Thereafter, further at finite relative displacements, the bond is effectively destroyed and resistance is developed by friction between the tendon and the confining grout. This frictional resistance, which is augmented by dilatancy of the grout, may be increased by irregularities of the tendon steel, which cause part of the shear strength of the grout to be developed.



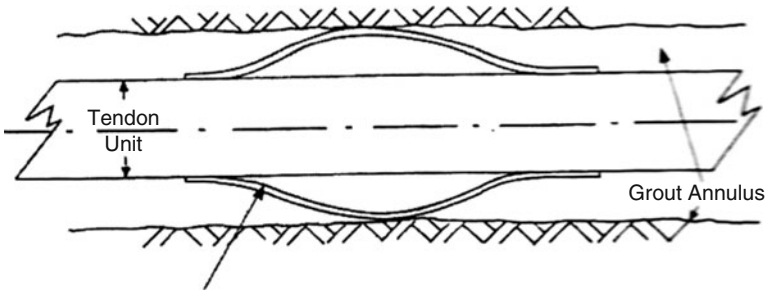
**Fig. 11.2** Pre-stressed rock anchor showing typical details (IS: 10270).

During the pulling of tendon, the reaction forces are taken by shear stresses within the grout. At the locations where principal tensile stress exceeds the tensile strength of the grout, cracks will develop. Hence control over the thickness of grout cover is necessary for a particular load. To minimize the effect of cracks minimum grout cover shall be maintained throughout the fixed length of anchor (usually  $>10$  mm) and provision of centralizer and spacer system shall be provided in the case of multi-strand tendon having longer lengths.

The main function of tendon centralizer is to maintain the steel centrally within the grout column and to ensure the minimum grout cover throughout the fixed length. The function of the spacers are to separate the individual units of the tendon e.g. strands shall be spaced in such a way that proper grout cover is maintained



**Fig. 11.3** Use of pre-stressed anchors for rock slope stabilization.



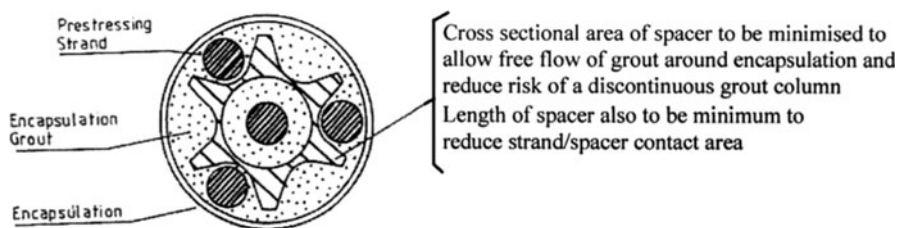
**Fig. 11.4** Detail of centralizer system.

among all strands. General arrangement of centralizer and spacer has been shown in Fig. 11.4 and Fig. 11.5 respectively.

**(a) Mechanics of Bond between Grout and Borehole Wall**

The anchor load  $P$  is given by

$$P = \pi \cdot d \cdot L \cdot \tau$$



**Fig. 11.5** Detail of a spacer system for a multi-strand system.

where  $L$  = fixed length considering a cylindrical borehole,  $d$  = diameter of borehole and  $\tau$  = average working bond stress.

In the above equation it is assumed that there is no local debonding at the grout/ground interface and failure takes place by shear at the grout/ground interface. Also over the whole of the fixed anchor interface, bond stress has been assumed to be distributed uniformly. In reality, such idealizations do not exist.

The formation of failure surface depends on roughness of the anchor hole, the strength of the rock and the effect of nearby construction works. On the loading of an anchor tendon, stresses are transmitted from the grout column to the rock in the form of radial stresses and shear stresses. The assumption is that failure may be at some distance from borehole wall or it may be at the interface. This depends on the relative strength of the interface and the adjacent rock. In case of hard rock, the failure may be at the grout/rock interface or even in the grout itself. Whereas for soft rock, failure may occur at short distance (probably a few mm or cm) into the rock.

The bond creates problems when failure occurs at an interface, because the interface is brittle in behaviour. Hence debonding is most likely to occur where the load levels approach the limiting value. The problem becomes more significant in case fixed anchor length is long. It is very important to note that with high capacity anchors, some debonding over the fixed anchor length is possible. As the load increases, the debonding progresses towards the lower end of the anchor. Therefore, in design, average bond values shall be used. IS 10270 recommends bond values for computing fixed length of anchors for various types of rocks which varies from 3 kg/cm<sup>2</sup> in sand stone to 5 to 7 kg/cm<sup>2</sup> in basalt. These values should be correlated with the rock quality at site and the results of pull out tests of test anchors.

#### (b) *Corrosion Protection in Anchors*

For the long life of the anchors, protection against corrosion and rusting is important. Rusting can be prevented by providing an alkaline environment with a pH value in the range 9 to 13. Hydrated cements have a pH value of about 12.5. the penetration of carbon dioxide and sulphur dioxide gases react with the alkali and hence alkalinity is significantly reduced. With the further permeation at the tendon

steel surface corrosion is likely, especially, if both oxygen and water are available. The penetration of air and water is usually small when the concrete is sound. There is difficulty in penetration, if concrete has good cover ( $>10$  to  $15$  mm).

The presence of chloride ions reduces the alkalinity which can create problem. Therefore, major precaution shall be taken to ensure that chlorides are not added to cement grouts.

### **Corrosion Risk near Anchor Head**

At the interface between the anchor head and start of anchor borehole, the potential hazard for corrosion is maximum due to following reasons:

- (i) In this area, there may be movements (in the form of thermal and/or rock rebounds) and settlement of the structure.
- (ii) In this region, groundwater seepage is comparatively high.
- (iii) Near to the ground surface, rock may be partly disturbed and aerated above the permanent groundwater table.

The tendon in this region is passed through metal or plastic pipe or casing through the structure and into the rock mass (Fig. 10.2). The tendon is passed through this pipe to the anchor head. The surrounding void shall be carefully filled with a cement grout or resin. Care has to be taken to vent off the air during the filling operation.

### **Anchor Head Protection**

The bare tendon steel at the anchor head is required to be protected against corrosion. Bearing plate also needs protection. The basic principle followed is to enclose the exposed head but simultaneously it should allow freedom to pre-stress the tendon and to allow load changes to occur in the anchor while in service.

Cement grouts are not used for inner head protection, because no flow of water is allowed in inner head area. The protection of outer parts can be done as per requirement of strands to be stressed i.e. restressable, or non-restressable. In the case of restressable multistrand, grease filler is normally used within plastic or steel caps and when restressing is not required then resin and other sealants can be used because there is no need to remove the cap.

### **(c) Stressing of Anchor**

Main purpose of anchor stressing is to load each anchor to a known load level in a standard manner. Anchors are usually stressed by direct pull using jacks capable of pulling all the strands or wires together.

To transfer the load completely in fixed anchor length, ensure that complete debonding of the tendon steel occurs along the free anchor length. After this it is possible to assess the credibility of the load/extension values obtained. The differences between gross and net extensions of the tendon shall be known. After locking the tendons into wedge grips there is small loss in the extended tendon length due to pull for maintaining equilibrium.

## 11.4 CASE STUDIES

Design of prestressed anchors and the measures for stabilization of unstable Himalayan slopes in the vicinity of dams and of power house of four hydro-electric projects is described below.

### 11.4.1 Nathpa Jhakri Hydro-Electric Project (1500 MW)

The salient features of the project are:

- A 62.50 m high concrete gravity dam on Satluj river.
- An underground desilting complex (4 nos. each 525 m long, 27 m deep and 16 m wide).
- A 10.15 m dia and 27.39 km long head race tunnel.
- 21.6 m/10.2 m dia and 301 m deep surge shaft.
- Three pressure shafts, each of 4.9 m dia and 571 m to 622 m in length.
- An underground power house with a cavern of size 222 m  $\times$  20 m  $\times$  49 m.
- A 10.15 m dia and 982 m long tail race tunnel.
- General layout of project has been shown in Fig. 11.6.

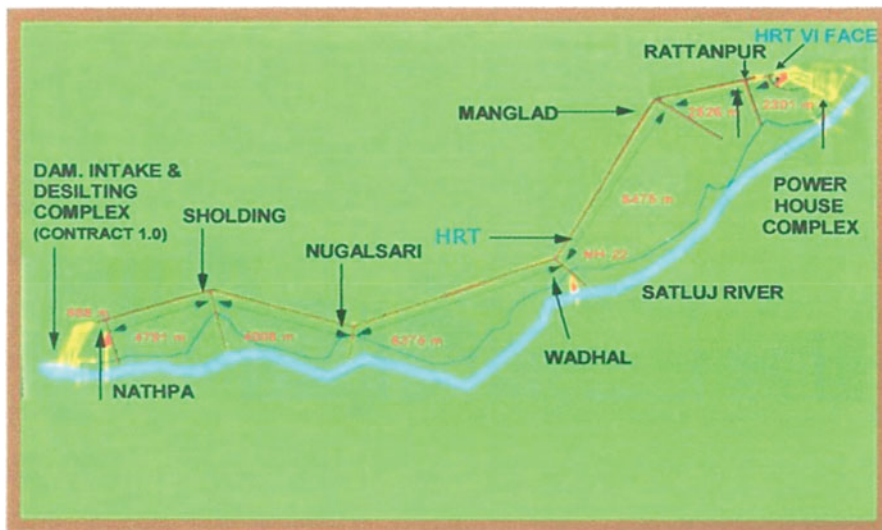


Fig. 11.6 General layout of Nathpa Jhakri hydroelectric project.

**(a) *Identification of Slope Instability Problems on the Banks of Nathpa Dam and Adjoining Areas***

A massive rock slide took place on 8th July, 1993 on the right bank of the river just about 100 m upstream of the dam site. The slide of about 10 lac cum of debris completely blocked the river resulting in formation of lake upstream of slide. On the left bank, slides had also been observed along the National Highway from time to time. Because of these slides further caution was necessary particularly in view of stabilization problems due to steep left bank slopes having foliation planes dipping towards the river bed. On left bank, construction of road at dam top level was unavoidable and minimum rock excavation was necessary for construction of dam and intake. Therefore, any rock cutting on left bank in the dam and intake area was expected to disturb the rock slope stability and could have resulted into a planar failure i.e. failure along the foliation planes.

Because of steep slopes having number of foliation planes dipping towards valley and biotite schist bands located about 15–25 m inside the hill almost parallel to exposed sloping surface dipping into valley, it was necessary to protect the slopes from sliding and necessary stabilization measures had to be taken accordingly.

The ski jump type energy dissipation arrangement has been provided for spillway in the dam. The ski jump jet is likely to develop deep scours in the river bed resulting in formation of a plunge pool. This scour, it was anticipated, will in all probability destabilize hill slopes on the bank.

**(b) *Topography and Geology in the Vicinity of Nathpa Dam***

On left bank, slopes rise for a height of more than 140 m from the river bed at El. 1445 m to National Highway No. 22 at El. 1585 m. It is covered by overburden from road at El. 1585 m to El. 1524 m. Below El. 1524 m, the rock comprises gneisses with pegmatite intrusions, bands of quartz mica schist, biotite schist, amphibolites etc. The bank slopes in the dam area and vicinity of dam varies from  $40^{\circ}$  to  $50^{\circ}$ . The dip of the foliation planes of gneisses varies from  $40^{\circ}$  to  $55^{\circ}$  towards river side having strike almost parallel to the river flow. The dam site is located mainly in the area underlain by a gneiss group named Jeori-Wangtoo complex aged Pre-Cambrian. The foundation rocks in the dam complex are comprised of gneiss with bands of quartz mica schist, biotite schist on the left bank and of augen-gneisses with the same bands on the right bank. The geological section at dam axis is shown in Fig. 11.7.

**(c) *Selection of Stabilization Measures***

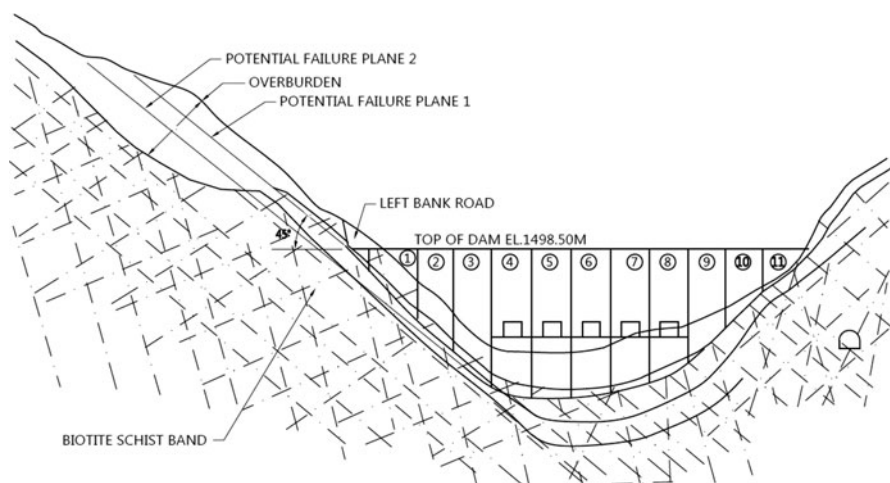
After identifying instability of existing slopes in the dam area, it was examined that what type of stabilization measures are required in order to line up with overall construction schedule and keeping in view importance of structures like Dam and Power Intake. Consequently, careful analysis and investigations were carried out and a comprehensive plan for stabilizing these slopes was worked out and implemented. This include drainage and dressing of overburden material, construction of retaining

walls and most important is anchoring of the potential slide rock mass with rock bolts and prestressed cable anchors of different capacity viz. 40, 100 and 200 tonnes at different locations. From the computations based on potential failure plane and rock mass to be stabilized, it was found that destabilizing force comes to be very high, which can be tackled only by providing prestressed cable anchors of higher capacities. It was found that by providing only rock bolts and dressing of slopes at feasible locations, it was rather difficult to stabilize the slopes having huge destabilizing rock mass with unfavourable dipping and orientation of joints. Therefore, it became necessary to provide prestressed cable anchors of higher capacities alongwith other measures.

**(d) Criteria for Deciding Prestressed Cable Anchors on Left Bank Dam Area**

On the left bank, dam area was analyzed for stabilization from 15 m u/s to 105 m d/s of dam axis. The proposed road cut alongwith proposed excavation profile of dam was marked on these geological sections. Geological section at dam axis showing position of road on left bank, assumed failure plane, position of biotite schist band etc. is shown in Fig. 11.7.

The angle of potential failure plane was taken for analysis as  $45^\circ$  from 15 m u/s to 60 m d/s of dam axis and thereafter  $40^\circ$  from 60 m d/s to 105 m d/s based on observed foliation angle. The location of failure plane was marked on each section based on deepest cut i.e. the location with respect to exposed surface. In some sections, road cut is governing the deepest cut and in some sections dam foundation cut is governing the deepest cut. The sliding mass based on failure plane was analyzed considering the effect of resisting forces, disturbing forces, earthquake forces, uplift pressure, angle of internal friction and cohesion along the failure plane. The anchorage forces at different section were computed for a factor of safety of 1.1



**Fig. 11.7** Geological section at dam axis.

for non-seismic case and 1.0 for seismic case. The critical of two cases were finally adopted in the design. The number of rows and spacing of anchors and their capacity were decided accordingly.

For purpose of design the values of angle of internal friction and cohesion for rock and overburden were adopted as  $41.9^\circ$  and  $1 \text{ t/m}^2$  and  $41^\circ$  and  $0 \text{ t/m}^2$  respectively on the basis of investigations and back analysis of stable slopes. The value of horizontal seismic co-efficient adopted was 0.08 (site is located at Zone IV of seismic map specified in IS: 1893) and vertical component was assumed to be 50% of the horizontal component. Unit weights of rock and overburden were taken as  $2.7 \text{ t/m}^3$  and  $2.3 \text{ t/m}^3$  respectively based on test results in the dam area.

On the above criteria 166 nos. of prestressed cable anchors of 200 tonnes capacity have been provided in left bank dam area. In the intake area, about 274 nos. of cable anchors of 200 T capacity have been installed. The cable anchors are about 20 m long with 10 m as fixed length. Figures 11.8 and 11.9 show the details of the scheme of stabilizing measures adopted on the left bank in a cross section and in elevation.

Following the above criteria 33 nos. of 100 tonnes capacity prestressed cable anchors and 37 nos. of 40 tonnes capacity cable anchors alongwith grouted rock bolts have been provided on the right bank. Similar treatment has been done in plough pool area on both banks.

## **11.4.2 Ichari Dam on River Tons**

In this case study, details of prestressed anchors to ensure stability of left training wall of spillway stilling basin comprising thin section and stabilization of Ichari Dam left bank slopes in the upstream of dam axis are discussed.

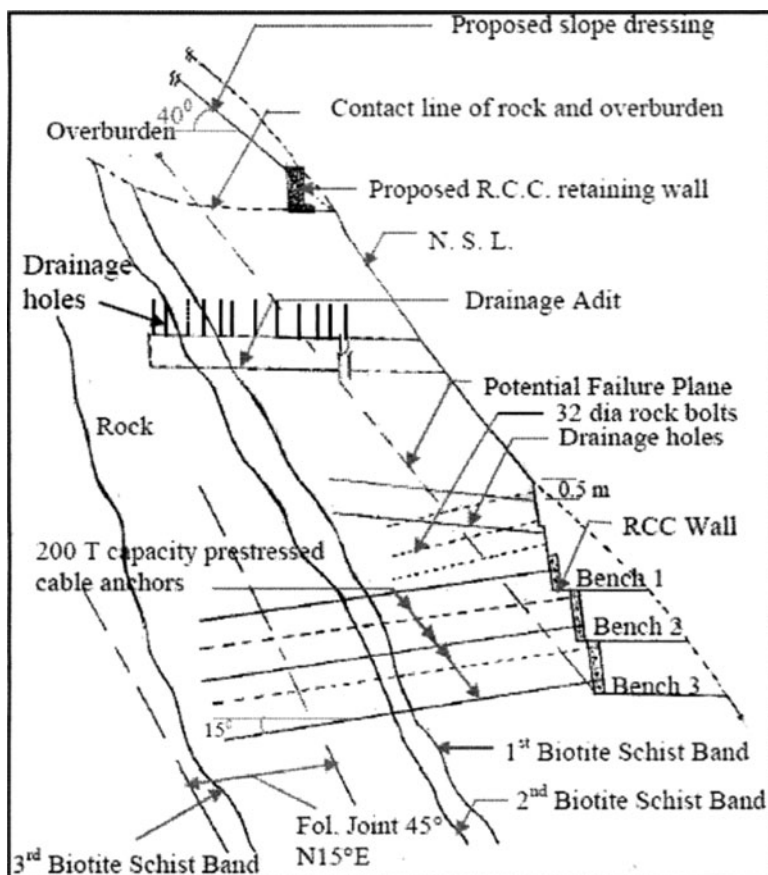
The concrete gravity diversion dam at Ichari on river Tons is 59.25 m high. The installed capacity of underground power house at Chibro is 240 MW. The layout of Yamuna Hydel Scheme is shown in Fig. 11.10. It shows various components of the scheme. The project was completed in 1975.

### **(a) Left Training Wall Downstream of Dam**

The training wall along the left flank of spillway of the dam is a concrete wall. The length of wall is 140 m extending 35 m downstream of the roller bucket. It was to be made safe against sliding and overturning.

#### **(i) Geology**

The bed and left bank rock consists of inter-bedded quartzitic slates and slates. The slates are soft and thinly bedded having the bedding parting at an interval of less than even 1 cm. The rocks are dipping at an angle of  $10^\circ$  to  $15^\circ$  towards the river. Above El. 618.00 (approx.) the rock consists of many shear zones and glide cracks dipping towards the river. It was decided to provide a retaining wall to stabilize the slope.

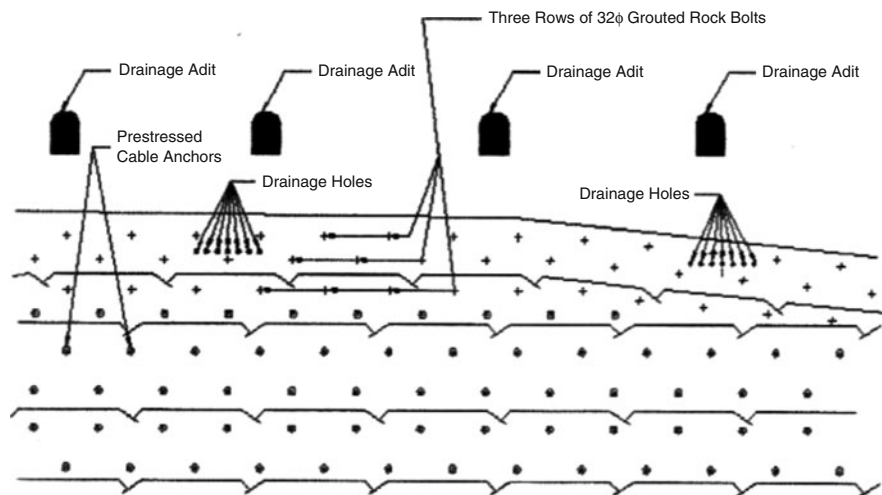


**Fig. 11.8** Section showing stabilization measures including installation of cable anchors on left bank.

### (ii) Proposal for Anchors

Due to adverse geological features, it was decided to provide a thin section of RCC retaining wall with anchors because the other alternative comprising full gravity section of wide base width needed lot of cutting at the toe of hill slope. It was considered that cutting the toe of the hill slope for gravity section would destabilize the slope. Hence, a proposal of a retaining wall with a bottom width of 4.0 metres anchored to the rock with pretensioned anchors of 50 T capacity was worked out. The length of anchors was 12.5 m out of which 5 m was to be grouted.

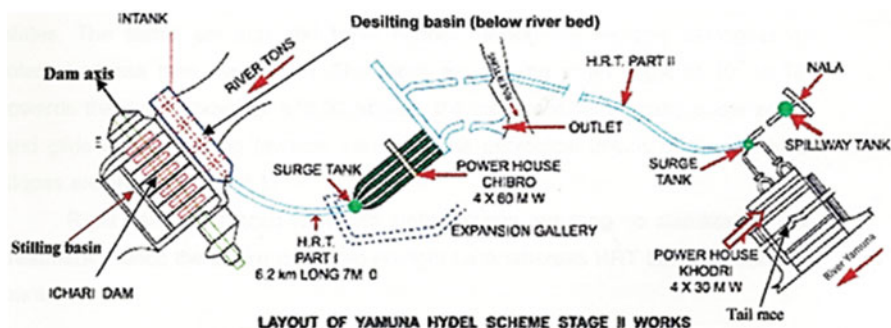
Later it was decided to use positive prestressed anchors similar to those installed in the underground power house and other cavities at Chibro. In positive prestressed anchors, both ends of anchors are accessible and stressing can be done from both ends (Fig. 11.11).



**LEGEND :**

- Prestressed Cable Anchors
- + 32φ Grouted Rock Bolts
- Drainage Holes

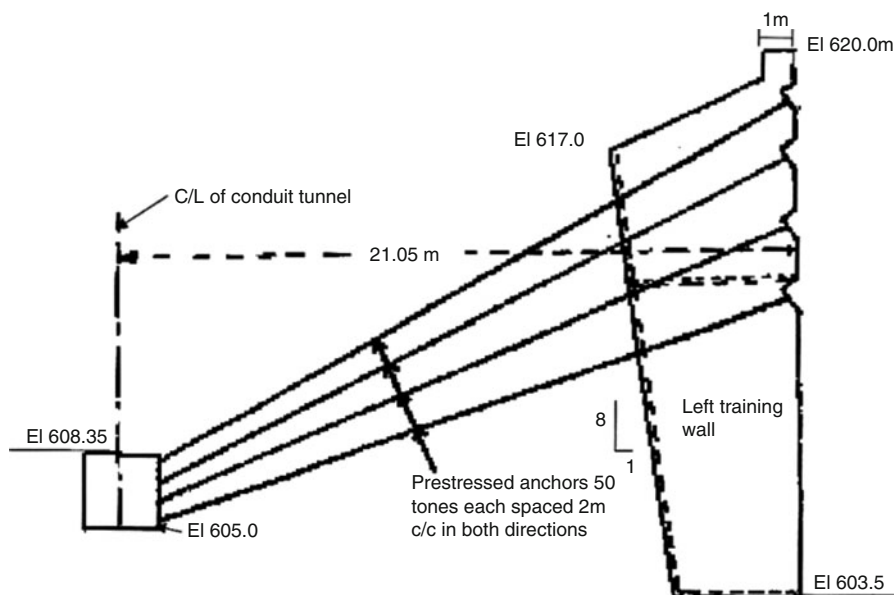
**Fig. 11.9** Elevation of prestressed cable anchors of 200 tonnes capacity including rock bolts and drainage arrangement (left bank dam area).



**Fig. 11.10** Yamuna Hydel Scheme stage II.

Considerations for providing positive prestressed anchors are as below:

- The tunnel for the third flushing conduit of desilting basin provided a face in rock mass for positive anchorage without any extra provision.
- By providing positive anchors, there is no element of uncertainty in getting good performance, as both the terminals of the anchors are approachable and anchor plates can be fixed as required. Whereas in the case of grouted anchors, better skill for installation is required.



**Fig. 11.11** Section showing positive pre-tensioned anchors.

**(b) Stabilization of Ichari Dam Left Bank Slopes – Upstream of Dam Axis**

The left bank slopes in a stretch of 140 m in the u/s of dam axis comprised of thick overburden of hill wash material and debris and the underlying rock had a number of shear zones and glide cracks dipping gently towards the river.

It was considered that these adverse features would create instability of slopes during the operational stage of project due to the impact of impounding of the reservoir and its daily drawdown because of peaking at Chibro Power House. Hence, this necessitated extensive investigations and planning for stabilization measures including anchors. The instability of slope was responsible for the shifting of the location of intake structure from left bank to the right bank.

For slope stabilization of left bank, measures comprising easing of slopes and drainage improvement were first taken to reduce the actuating force. Further it was considered essential to increase the resisting force by following one of the two alternatives. One alternative was thought to provide prestressed anchors and another, which was finally adopted, consisted of concrete anchors/piles.

The following were components of concrete anchors:

- (i) A concrete toe wall based on a rock ledge at El 636.00 and varying in thickness of 10 metres at the bottom to 3.0 metres at top.
- (ii) Reinforced cement concrete anchors were constructed in situ. Plan and typical section showing this arrangement are shown in Fig. 11.12 and Fig. 11.13 respectively.

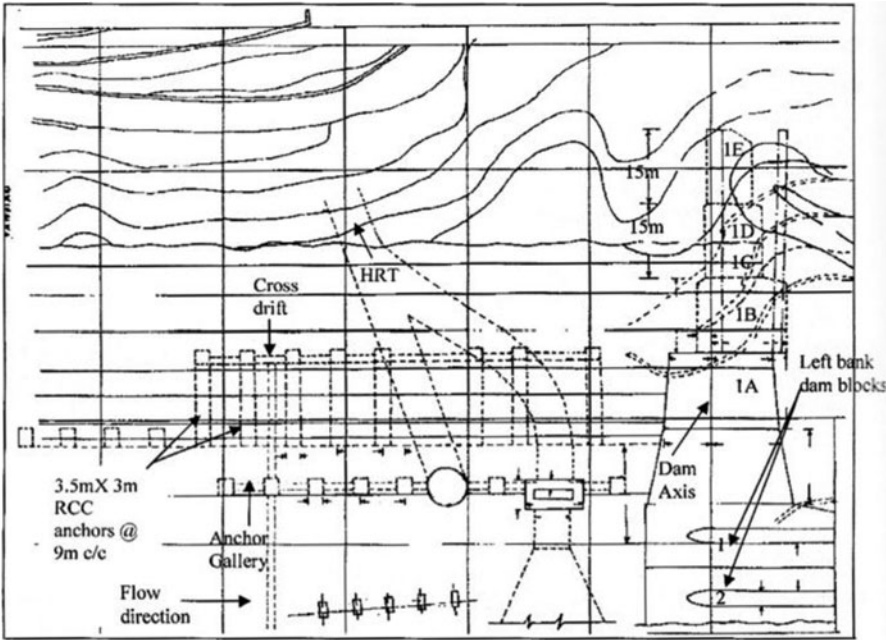


Fig. 11.12 Plan showing concrete anchors (Ichari dam area).

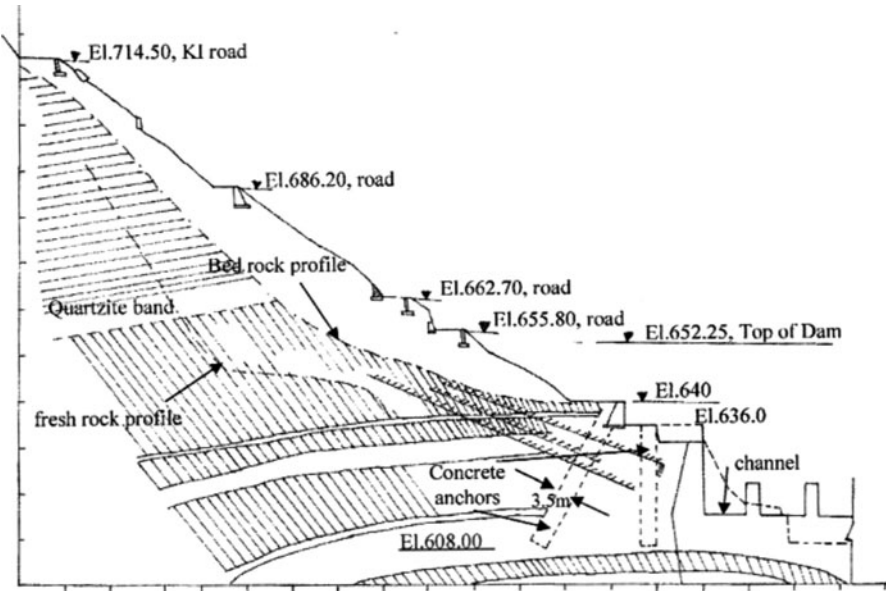


Fig. 11.13 Typical section showing geological features and concrete anchors.

The concrete anchors are closely spaced so that the uncertain effect of variable yield of rock in between the anchors is minimized or eliminated. The concrete anchors are needed in the region of the toe of the rock formation and at the same time so disposed that sufficient rock cover is available behind the anchors towards the river side. Keeping these factors in view a system of anchors in tandem has been proposed in which one anchor was inclined at an angle of  $30^\circ$  with the vertical while the other anchor kept vertical.

In addition, provision of a mechanical anchorage had also been made by a cross drift (Fig. 11.12) filled with reinforced concrete. This would also help in a more uniform distribution of stress near the toe of anchors. The drift was very useful during construction for drainage and speedy excavation of the shafts for the anchors.

The size of the shafts had been kept as  $3 \text{ m} \times 3.5 \text{ m}$  each and they are provided at 9 metre centre to centre staggered. This arrangement had been made keeping in view the feasibility of construction, adequate rock cover in between the adjacent shafts and providing maximum coverage along the flank to resist the sliding forces as uniformly as possible.

#### (c) *Behaviour of Slopes*

The geologically adverse slopes in the vicinity of Ichari dam have been treated with anchors in conjunction with other stabilization measures (shown in Fig. 11.13). The thin section of training wall in the downstream of dam axis has been stabilized with positive prestressed anchors and portion upstream of dam axis with concrete anchors. Till to date, the hill slopes are functioning well and structures adjoining to the slopes are safe.

### 11.4.3 *Dharasu Power House*

Maneri Bhali Hydro Electric Project Stage-II is a run-of river scheme (304 MW) for generating 1642 MU during average year by harnessing a head of 285 metres in river Bhagirath (a tributary of Ganga) between Uttarakashi and back water of Tehri dam at a power house near Dharasu.

Dharasu surface power house is situated near the river bank by cutting the rock slope. The rock slope levels were between El. 860 to 892 m and the deepest foundation level of power house was 809 m. Phyllite and greywacke are the main rock types in the power house area upto El 855 m in unit 4 and units 1, 2 & 3 respectively and above that is river-borne material. During the excavation work for Dharasu power house and appurtenant works in its first stage consisted of removal of river-borne material (RBM) from El. 892.00 to El 855.00. Thereafter, the rock excavation in the power house pit was further done upto El 815.00. At this stage, it was decided to shift upstream boundary of the power house towards the hill side. This shift made the upstream excavated slopes upto El 855.00 m very steep. Further, at this stage of work, the dressing of the slopes was not possible for improving the stability of slopes. Before any treatment of slopes could be taken

up, heavy rock slide occurred on u/s side of unit 4 as well as on gable end side in July, 1985 after heavy rains. Initially, stone pitching in a sufficient length in RBM portion was done along with 4–5 m deep 2–3 m c/c grouted anchors and the surface was shotcreted with chain link fabric. But the same was not found adequate. It was later strengthened.

Thereafter on the u/s side of power house pit prestressed anchors were provided to improve the stability of slopes. Plan (Fig. 11.14) shows different units of power house. Figure 11.15 shows the power house and adjoining hill slopes with treatment.

(a) *Design Parameters and Criteria for Prestressed Anchors in Phyllite Rock*

(i) **Phyllite Rock**

In unit 4, mainly phyllite rocks existed. These consisted of thinly foliated rock mass of poor grade quality, puckered, weak and totally shattered and their properties matched with soil mass. The stability was checked by slip circle criterion which is normally adopted for soil mass. The excavation in this zone was not possible, as overlying river-borne material was already treated by dressing the slopes and covering top portion with boulder pitching (shown in Fig. 11.16). In view of this, it was decided to provide prestressed anchors covering the fixed length zone beyond

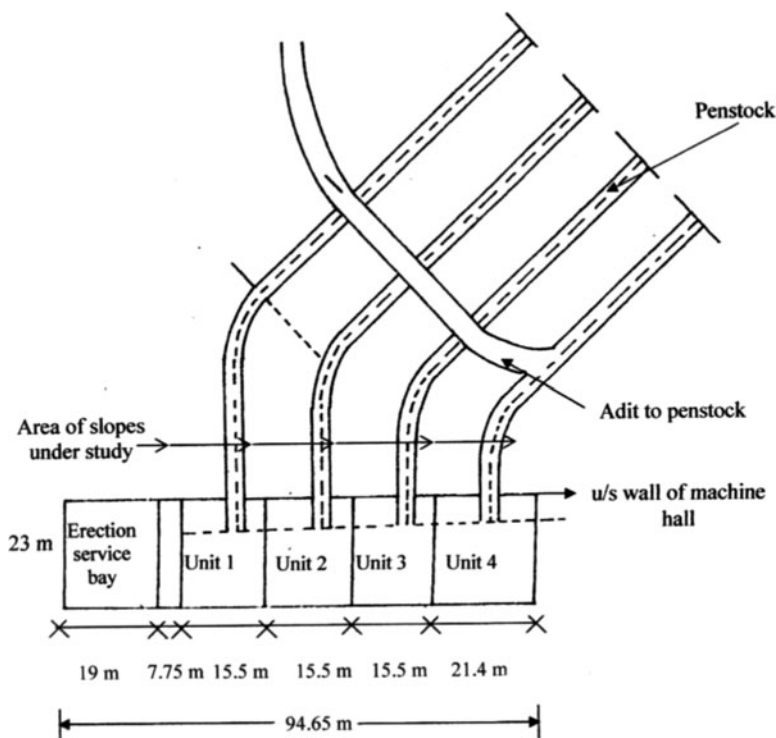


Fig. 11.14 Plan showing different units of Dharasu Power House.

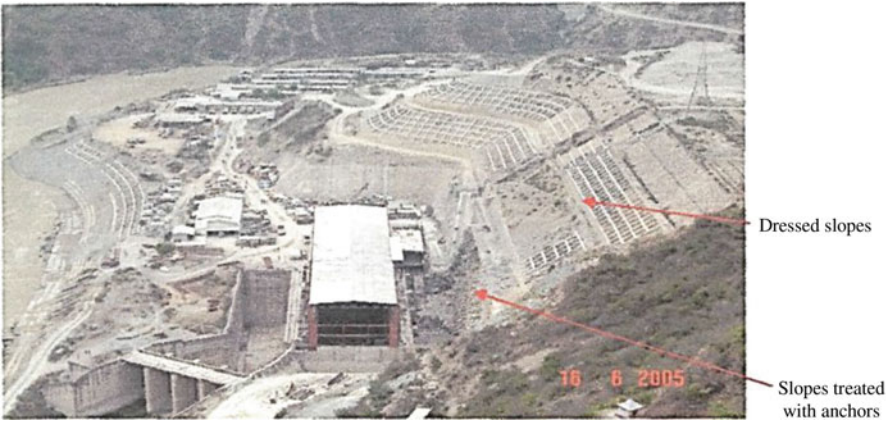


Fig. 11.15 View of Dharasu Power House and adjoining hill slopes with treatment.

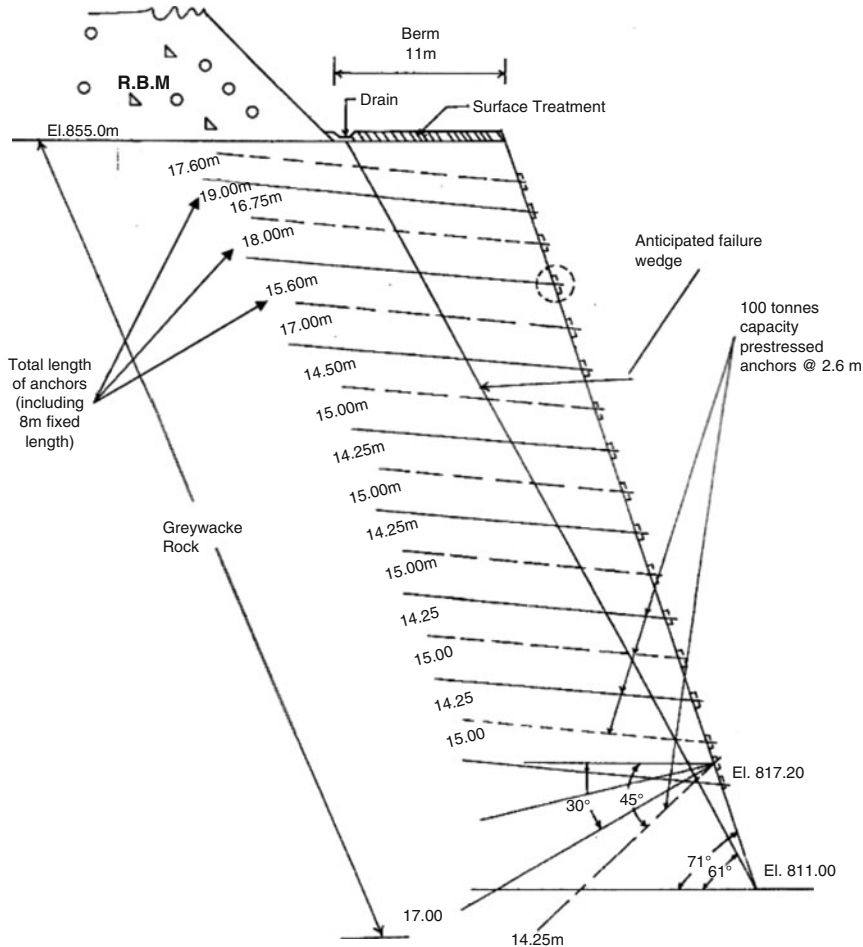


Fig. 11.16 Section showing prestressed anchors (wedge failure case).

the critical slip circle established after different trials. For trial analysis the values of cohesion ( $c$ ) and angle of internal friction ( $\phi$ ) of material were adopted as  $0.60 \text{ kg/cm}^2$  and  $36^\circ$  which were found by conducting in-situ test under saturated conditions in unit 4. The bond strength between steel and concrete was assumed as  $6 \text{ kg/cm}^2$  and bond strength between phyllite rocks and concrete was taken as  $2.75 \text{ kg/cm}^2$ .

Various trials were made with different slope angles and the above values of shear parameters and finally at a slope angle of  $29^\circ$  with phyllite and R.B.M. the safety factors were found out to be 1.61 in normal condition and 1.04 in seismic condition towards u/s side slope. The safety factor with slope angle of  $27^\circ$  with phyllite rock only towards gable end worked out to be 1.51 in normal condition and 1.06 in seismic condition. These values were considered adequate for slope stability.

## (ii) **Greywacke Rock**

The  $c$  and  $\phi$  values of greywacke rock in units 1 to 3 were tested under saturated condition in the field and were found as  $1.30 \text{ kg/cm}^2$  and  $41^\circ$  respectively. The slopes were tested by wedge method. The factor of safety of wedge is worked out assuming that sliding is resisted by friction only. A factor of safety of 1.5 was adopted.

## (b) *Slope Stabilization Measures*

In case of Dharasu power house, slope protection measures have been provided on the hill slope facing upstream longitudinal wall of the power house. The prestressed anchors are provided on the hill slope facing upstream wall of unit nos. 1 to 4 in the area where penstocks are approaching to machine hall (Fig. 11.15). Details are as below.

### (i) **Measures based on Slip Circle Failure**

Mainly in the area upstream of unit 4, there is predominance of phyllite rock which is thinly foliated with puckered, sheared and brecciated and of poorer grade. Phyllite has R.B.M. over it. The stability of slope was checked as mentioned above, by slip circle method. The slopes in this portion were stabilized by providing prestressed anchors of different capacities varying from 70 to 100 tonnes. The total depth of anchors having bore diameter of 100 mm varied from 39 m to 56 m at different locations and at different inclinations. The anchors have been provided at a spacing of 2.4 m c/c. The fixed length of anchors varies from 8.75 m to 16 m beyond the critical slip circle.

### (ii) **Measures based on Wedge Failure**

There is predominance of greywacke type of rock on the upstream side slopes of power house unit nos. 1 to 3. Fresh greywacke rock is hard and moderately to highly jointed. These joints are spaced from a few mm to 30 cm apart and are almost filled with clay gouge material. In this reach the steep rock portion had been stabilized by providing 100 tonnes capacity anchors, varying in length from 14 to 19 m including the fixed length of 8 m. The anchors are provided at a spacing of 2.6 m c/c both ways with upward slope of  $5^\circ$  with the horizontal. These are shown in Fig. 11.16.

### (c) *Behaviour of Slopes*

In case of Dharasu power house, slope protection measures have been taken on the hill facing upstream longitudinal wall of power house. The prestressed anchors have been provided (during 1987-91) on the hill slopes facing upstream wall of unit nos. 1 to 4 in the area where penstocks are approaching to machine hall. After taking adequate measures for slope stabilization, the area between hill face and upstream wall of power house has been utilized for keeping the transformers after filling the gap with lean concrete. The slopes are functioning well and structures adjoining to slopes are safe.

## **11.4.4 Slope Stabilization at Tehri Dam**

### (a) **Intake Area**

The intake area comprises hill slopes from El 705 to 910 m. The semicircular intakes for the four HRTs (2 for hydro power plant and 2 for pumped storage units) are located at El 720. The top of intake structure is at El 745 (5 m above MDDL) at which a platform is developed for hoisting of intake gates and trash rack. Another platform is made at El 840 (top of dam) to locate hoisting for operation and maintenance gates of HRT through a gate shaft.

The hill slopes from El 705 to 910 were geologically investigated and several vulnerable areas were identified. The rock mass in general was poor and unstable. A wide and extensive deformed zone was delineated which was confined between two major shears between El 910 and 835 m. It was also considered that the slopes below El 830 m (FRL) will be submerged and will be subjected to repeated reservoir fluctuation upto MDDL. Hence to ensure stability of slopes elaborate treatment was designed and adopted.

Above El 890 m rock bolts in poor rock failed. Then shotcrete with wire mesh in panels having space for drainage above and below this level was attempted. Slopes did not respond well to it and slope below El 890 m failed. Then slope of cut faces was flattened and retaining wall was erected. A wide bench was made at El 856 m and slope upto El 840 was stabilized by providing rock bolts, 8–10 m long with wire meshed shotcrete.

From the platform at El 840 to the intake bench at El 720, the hill slope was provided with benches at El 835, 818, 805 and 790 and the slopes in between these benches and below the El 720 was treated with concrete pile shafts driven from the bench upto competent rock at suitable intervals to function as shear keys and the slope was protected by thick reinforced concrete cladding anchored with rock through rock bolts and prestressed cable anchors going across the bedding plane. The typical sketch shown in Fig. 11.17 gives the idea of the slope treatment between benches. The platforms at El 840 and 720 m were provided with consolidation grouting and thick concrete floor. A view of the intake area after stabilization is shown in enclosed Photo 11.1.

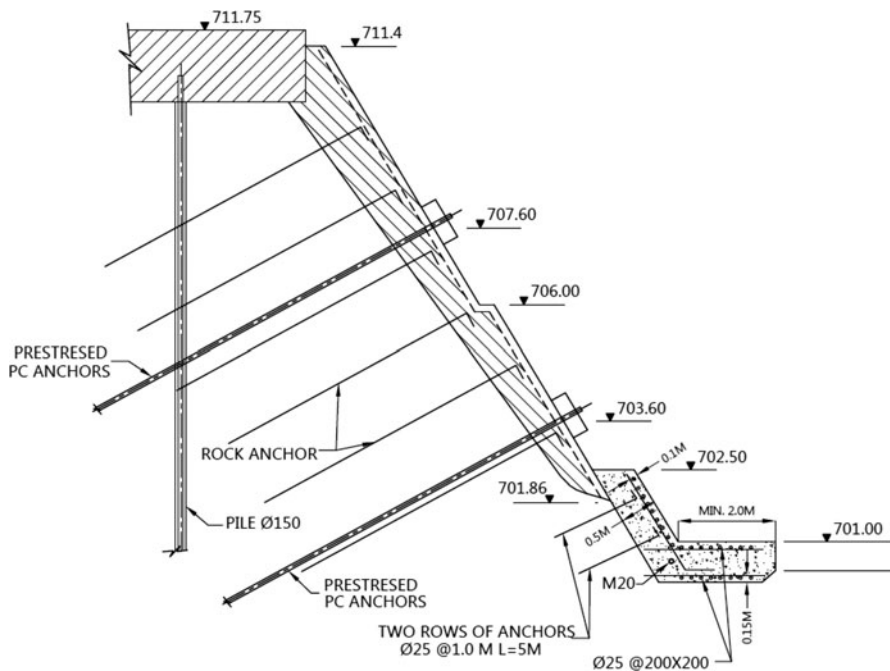


Fig. 11.17 Slope treatment of intake area (typical section between benches).

### (b) TRT Outlet Area

Two tail race tunnels (TRT-1 and TRT-2), each of 9 m finished diameter, carry discharges from four units of HPP, run parallel to the diversion tunnels and have the outlet portals on the left bank of river Bhagirathi with invert at El 598 m (river bed). These are shown in Fig. 11.18. The portal of cable tunnel from transformer hall and the switchyard are also located in this area.

The entire work area on the left bank was found geologically unstable and with the cutting of the slopes at the lower levels while making benches, movement of slope started. The nature of the slums mass was then geotechnically assessed and movement observations were made inside the adits and on the surface. On the basis of observations it was decided that there was no option except to stabilize the unstable mass to protect the proposed structures. The stability analysis of the sliding mass was made along two sections I & II (Fig. 11.18) by both the methods i.e. limit equilibrium and numerical analysis. The shear parameter as obtained from test were adopted as  $\phi = 29^\circ$  and  $C = 2 \text{ T/m}^2$  for analysis. The results revealed:

- Hill slopes were in unstable condition.
- Unloading of overburden mass above El 740 m did not improve factor of safety.
- The slope became stable with a toe at El 622.



Photo 11.1 View of intake area before reservoir impoundment.

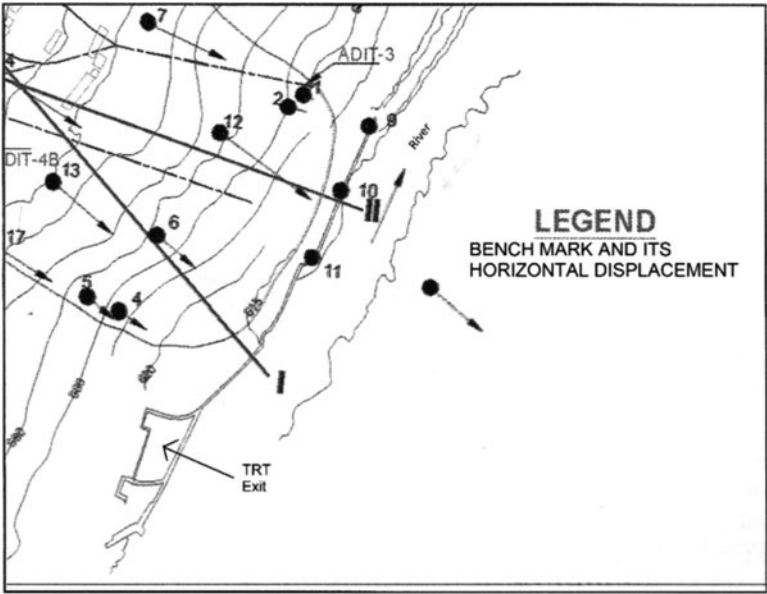
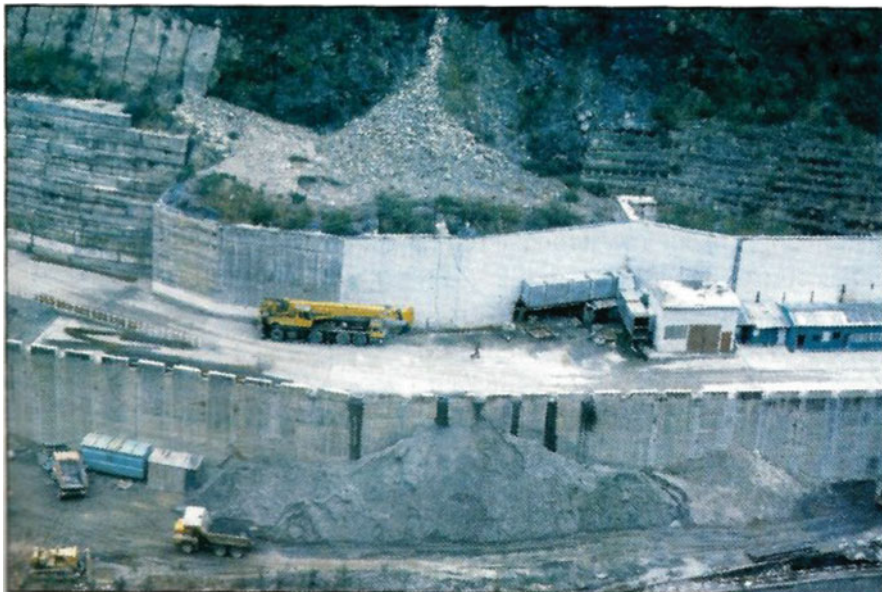


Fig. 11.18 Map of the landslide area near the exit of TRT.



**Photo 11.2** View after stabilization measures (2007).

On the basis of these investigations and analyses following stabilizing measure were adopted.

The river training wall with base at El 604.7 was raised to El 622 and the space between the wall and the road at 622 was filled with rock muck to build the toe to stabilize the slope.

Two-tier cellular walls from El 622 m to 643 m were made each 10 m high and earth filling between two walls was done as shown in Photo 11.2.

The slopes above El 643.0 m were eased flatter than 1:1 supported with toe retaining walls founded on rock. Drainage holes of 100 to 120 mm were drilled in the retaining walls. These measures were found effective in stabilizing an unstable rock mass in a large area.

#### ***11.4.5 A Review of Case Studies***

A review of the above described case studies of slope stabilizing reveals that for counteracting major destabilizing forces high capacity pre-stressed anchors or concrete anchors are required. These should generally be combined with usual protection measures such as placing wire crabs or stone facing or concrete facing on slopes and constructing retaining walls. Rock bolts are also required alongwith drainage arrangement. Hence, each specific slope stabilizing situation needs detailed investigation, analysis and generally a combination of several slope stabilizing measures.

## 11.5 RESERVOIR RIM SLOPE STABILITY

The reservoir rim slope stability study is as important as of the hill slopes in the vicinity of the dam. The stabilization of potentially unstable slopes in the vicinity of dam is necessary for the safety of the dam and its associated structures. Similarly if the hill slopes along the rim of reservoir are unstable or become unstable due to submergence and the fluctuation of reservoir level which cause drying and wetting, the slides may cause additional reservoir siltation and may also damage the villages located on such slopes. If a major slide takes place it may create high waves in the reservoir which may overtop the dam and cause damage to the dam. At Vajont arch dam (262 m) in Italy a major slide into the reservoir caused by an earthquake created a wave which overtopped the dam resulting in its failure. Hence detail investigations of slopes along the reservoir rim and adoption of necessary slope stabilization measures are necessary. Such a study was carried out for Tehri dam reservoir rim. It is briefly described below.

The construction of 260.5 m high Tehri dam has created a vast reservoir in the valleys of river Bhagirathi and Bhilangana. These valleys have the history of small as well as deep slides in the past. The reservoir extends upto the town of Uttarkashi. The hill slopes in general along the rim of reservoir appear to be stable with bedding planes and joints dipping towards hill side. But on the surface the phyllite rocks are highly fractured and jointed. Hence investigations were taken to examine the impact of interaction of reservoir water with the valley slopes and the effect of seismicity as the area is highly prone to strong earthquakes.

For these investigations and study the Earthquake Engineering Department, IIT Roorkee was engaged. The study was based on aerial photograph, topographical maps of Survey of India and geological maps of Geological Survey of India. Laboratory tests were carried out to determine the shear parameters of rocks. Cross sections were prepared. 55 cross sections were selected for stability analysis. The evaluation of over- all stability of hill sides from present river bed to hill top has been carried out under prevailing normal condition. Earthquake force and sudden draw down conditions were not considered to occur simultaneously in evaluation of reservoir slope stability below FRL. The critical circular failure surface is considered to pass through the toe of the hill to obtain minimum factor of safety by friction circle method. The effect of water load on stability was neglected. All the slopes tested were found to be stable with factor of safety above 1.2.

The displacement of the steepest hill side under MCE condition was found to be 8.7 cm and this was considered insignificant to initiate mass movement of the hill range. The surficial deposits lying from the river bed level to the elevation 1000 m are found in general to be stable and these deposits below FRL are expected to attain stability at new angle of repose in due course of time.

The villages within 500 m distance from the rim of reservoir at FRL are generally located near terrace cultivation developed on surficial deposits and these were in general found safe against damage from landslides under normal conditions and during MCE condition.

Thus, after detailed investigations and studies the hill slopes along the rim of the Tehri reservoir were considered stable. The reservoir is in operation for the last over more than ten years and no occurrence of any landslides is reported.

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## Chapter 12

# Reservoir Sedimentation



### 12.1 GENERAL

All the reservoirs, big or small, created by constructing a dam across a river are subjected to silting to some extent. This results in reduction of storage capacity affecting the useful life of the reservoir. The problem in India is more severe in the reservoirs which are located in Himalayas where rivers carry more sediment with water as compared to the reservoirs on southern rivers. The two basic parameters of reservoir sedimentation problems are the rate of silting and the pattern of deposition. The deposition of coarser sediment occurs in the upper reaches of reservoir whereas finer sediment reaches the dam and influences the design of outlet works. The weighted average rate of loss of gross storage in Indian reservoirs is 0.44 percent per year. In some reservoirs of run-of-river type schemes, the silting was observed at a very fast rate and these got silted upto the spillway crest in 2 to 7 years such as Maneri Stage-I, Ichari, and Pandoh dams. Nizamsagar and Dhukwan reservoirs have practically lost all its useful capacity. The world over many reservoirs have lost practically all the useful storage. Hence, there is a need to plan and implement the sediment control measures in all the reservoir projects. In this chapter three aspects of reservoir sedimentation i.e. rate of sediment, deposition pattern and sediment control measures are briefly discussed. For more details readers may refer (Asthana, Ref. [2].

### 12.2 SILTING RATE

The two dominant factors influencing the rate of silting in a reservoir are (i) sediment content in the flow entering the reservoir and (ii) the capacity-inflow ratio of reservoir.

### 12.2.1 Sediment Inflow

The source of sediment is the erosion in the catchment of the river. The erosion rate in the catchment and the transportation of sediment in the river network above dam site determine the sediment inflow in the reservoir. The erosion in the catchment depends on its characteristics which are as below:

- (i) Amount of rainfall and its intensity
- (ii) Topography
- (iii) Geological formation and soil type
- (iv) Land use

Sediment transported from catchment to the reservoir site depends on

- (i) Drainage network characteristics such as intensity of network, slope, shape, size, alignment etc.
- (ii) Discharge
- (iii) Sediment characteristics such as size, mineralogy.
- (iv) Channel hydraulic characteristics.

Some mathematical models like Universal Soil-Loss Equation (USLE) and weighting techniques have been developed in USA for working out the erosion rate and equations of sediment transport in channels have been used to work out sediment yield at a site but these are not considered as dependable as the results of direct measurement. The most dependable is the direct sediment sampling across a river section at or near the proposed dam site. Generally suspended load is measured and depending on sediment size the measured suspended load is increased for the bed load. The increase is generally 10 to 20%. In India the Central Water Commission (CWC) measures the sediment load of the rivers at many of its discharge sites. The data is published by CWC annually as Sediment Year Book for all river basins. When the data of direct measurement of river sediment load is not available the data of observation of reservoir sedimentation of adjoining reservoirs in similar conditions shall be used for planning purposes. A large number of reservoirs have been surveyed for sedimentation since 1960 by various agencies and the owner/operators of the reservoir. The method generally used for sediment surveys is hydrographic survey with ecosounder. Sometimes inflow-outflow method or remote sensing techniques are adopted. The observed data for large number of reservoirs has been compiled and published by CWC and lot of data is available in the CBI&P publications. On the basis of observed sediment data of reservoirs CWC has classified the country in seven regions (Fig. 12.1) and has specified range of sedimentation rate in each region as given in Table 12.1.

In view of the observations of various reservoirs and considering other local factors the Tehri project is designed for a sedimentation rate of 14.86 ham/100 km<sup>2</sup>/yr. Other dams on river Yamuna and its tributary Tons such as Lakhwar and Kishau

**Table 12.1** Sedimentation rate in different regions

Sl. No.	Region (shown in Fig. 12.1)	Average sedimentation rate (ha.m/100 km <sup>2</sup> /yr)	Medium values of sedimentation rate (ha m/100 km <sup>2</sup> /yr)
1.	Himalayan region (Indus, Ganga, Brahmaputra basins)	17.65	21.1
2.	Indo-Gangetic plain	10.45	8.9
3.	East flowing rivers (excluding Ganga) up to Godavari	6.35 (Hirakud)	6.35
4.	Deccan peninsular east flowing rivers including Godavari and South Indian rivers		
	(a) Excluding Western Ghat reservoir	7.43	4.65
	(b) Reservoirs in Western Ghats	135.3	-
5.	West flowing rivers upto Narmada	10.93	8.4
6.	Narmada Tapi basin	7.29	7.5
7.	West flowing rivers	35.33	17.9

are planned for a rate of 13 ham/100 km<sup>2</sup>/yr. However, the earlier dams such as Bhakra and Ramganga were designed for 4.25 to 4.3 ham/100 km<sup>2</sup>/yr but later the observations on these reservoirs revealed that the designed value was far less than the observed.

### 12.2.2 Capacity Inflow Ratio

The capacity inflow ratio ( $c/i$ ) is the ratio of the storage capacity of reservoir and the inflow to the reservoir. The  $c/i$  ratio influences the sediment trapping efficiency of the reservoir. Brune developed empirical relationship for estimating long-term trap efficiency in normally impounded reservoirs relating  $c/i$  ratio with observed trap efficiency of reservoirs of USA (mostly TVA). The relationship is shown in Fig. 12.2.

The curve of Brune shows that trap efficiency of reservoir is normally more than 90% if  $c/i$  ratio is 0.2 or more. Hence the quantity of sediment trapped in small capacity reservoirs on big rivers is less. The observed trap efficiency of Indian reservoirs given in Table 12.2 confirms the relationship of Brune.

The trap efficiency of a reservoir changes with time as the capacity reduces due to sedimentation. The observations have also shown effect of age on rate of silting. The siltation rate is observed comparatively high in the first 15 to 20 years of operation and thereafter reduces and ultimately becomes insignificant. This may be the reason why old reservoirs and tanks in Andhra Pradesh, Tamil Nadu, Maharashtra, Southern U.P. are still functioning. The data of reservoirs in India showing the effect of age on rate of siltation is given in Table 12.3. It shows the above trend.

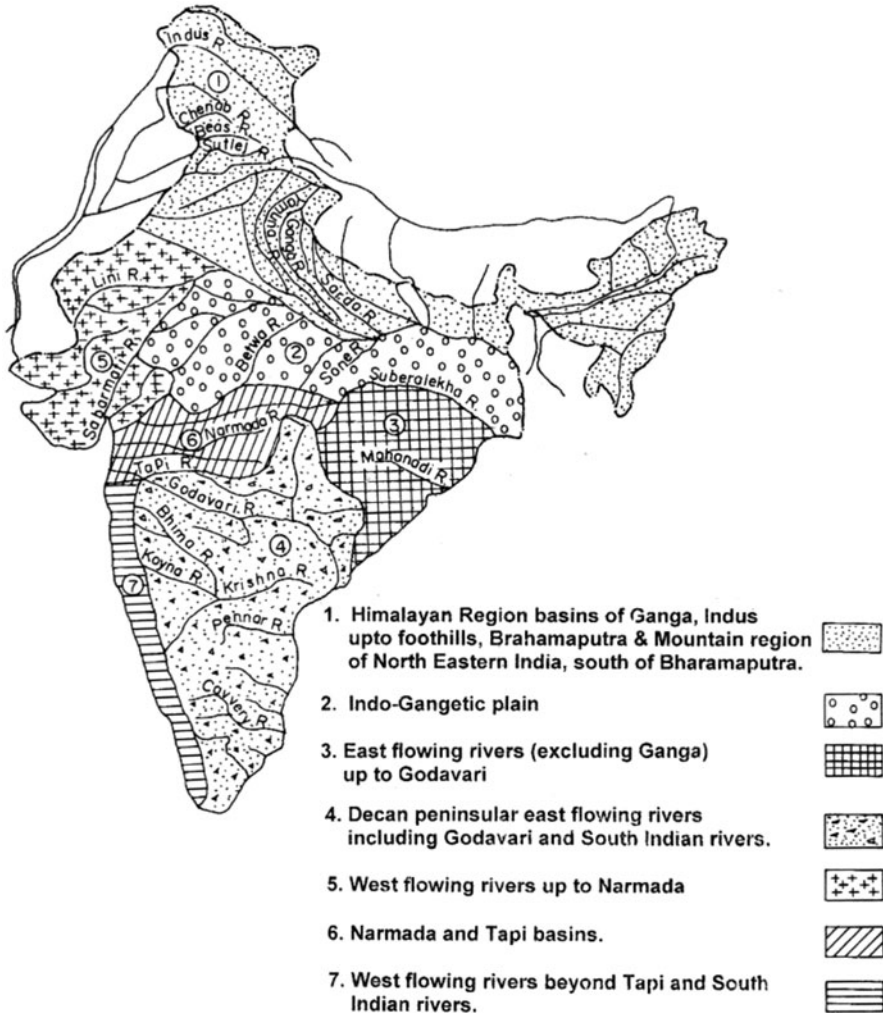


Fig. 12.1 Map showing zones for sedimentation rate.

### 12.2.3 Density of Sediment Deposits

The sedimentation rate and the loss of reservoir capacity due to sedimentation worked out by the reservoir surveys are expressed in volume. But the volume of deposited sediment changes with the change in density with age. Thus, the knowledge of density and determination of its change with age is of importance. Observations of the density of deposited sediment is generally an important part of reservoir survey. The density is the dry weight of unit volume of reservoir sediment in place. It depends on a large number of factors such as mineral composition, shape

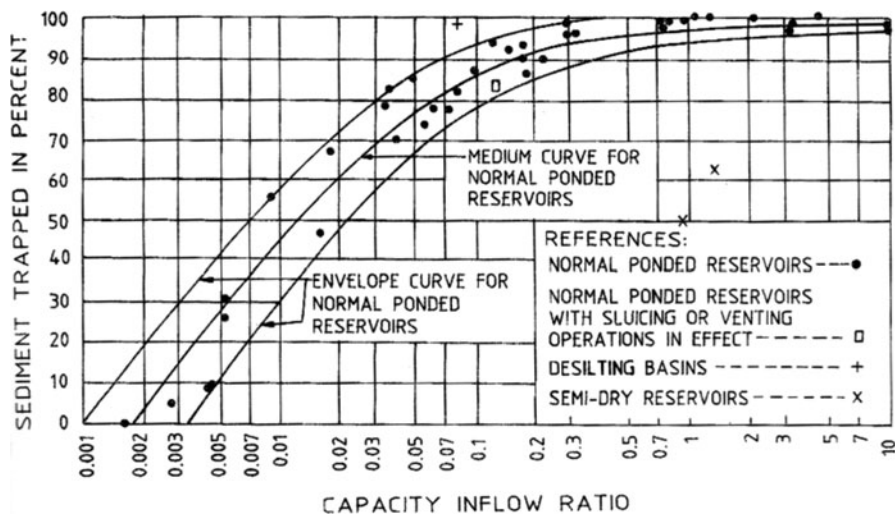


Fig. 12.2 Brune’s curve for trap efficiency.

Table 12.2 Trap efficiency of Indian reservoirs

Name of reservoir	Trap efficiency	c/i ratio
Matatila	66.5 to 89.2	0.19
Hirakud	64.3 to 91.3	0.20
Bhakra	98.4 to 99.8	1.0
Maithon	92 – 95	0.58
Lower Bhawani	79 – 89	0.49

Table 12.3 Effect of time on rate of sedimentation

Sl. No.	Name of reservoir (State)	Rate of sedimentation in thousand cu m/km <sup>2</sup> /year			
		First period of 10 years	Rate of siltation	Last period of 10 years	Rate of siltation
1.	Panchet hill (Bihar)	1956-66	0.973	1986-96	0.343
2.	Maithon (Bihar)	1955-65	1.170	1984-94	1.132
3.	Pong (HP)	1974-84	2.558	1988-98	1.350
4.	Tungabhadra (Karnataka)	1953-63	0.602	1983-93	0.226
5.	Hirakud (Orissa)	1967-77	0.657	1984-94	0.562
6.	Bhakra (Punjab)	1958-68	0.633	1988-98	0.663
7.	Lower Bhavani (TN)	1953-63	0.306	1973-83	0.246
8.	Vaigai (TN)	1958-68	0.409	1973-83	0.246
9.	Matatila (UP)	1956-66	0.849	1984-94	0.340
10.	Dhukwan (UP)	1907-17	0.042	1970-80	0.012

Source: CWC Publication 2001

**Table 12.4** Value of coefficients for density of sediments

Operation type	Initial weight in lbs/cft (kg/m <sup>3</sup> )		
	W <sub>c</sub>	W <sub>m</sub>	W <sub>s</sub>
1	26(416)	70(1120)	97(1550)
2	35(561)	71(1140)	97(1550)
3	40(641)	72(1150)	97(1550)
4	60(961)	73(1170)	97(1550)

**Table 12.5** Value of constant K for type of operation

Operation type	K in Ft-lb units (metric unit)		
	Sand	Silt	Clay
1	0	5.7 (91)	16 (256)
2	0	1.8 (29)	8.4 (135)
3	0	0 (0)	0 (0)

and size, reservoir operation, extent of exposure to atmosphere etc. The density of deposited sediment near the dam is generally less than that observed in the upstream reach or regions of reservoir which are exposed to atmosphere due to depletion of water level. The values of density observed in Indian reservoirs vary from 0.7 to 1.2 gm/cc.

Lane and Koelzer (1943) developed relation to estimate density of sediment which is a mix of different sizes using observed data of large number of reservoirs. This relation was improved upon by Lara and Pemberton (1963) and is given below:

$$W = W_c P_c + W_m P_m + W_s P_s$$

where  $W$  = unit weight in lbs/cft (kg/cum),  $P_c$ ,  $P_m$ ,  $P_s$  = percentage of clay, silt and sand respectively in incoming sediment and  $W_c$ ,  $W_m$ ,  $W_s$  = coefficients of clay, silt and sand respectively which can be obtained from Table 12.4. These are specified for different types of reservoir operation which are defined as:

- Operation type 1: Sediment always submerged or nearly submerged
- Operation type 2: Normally moderate to considerable reservoir drawdown
- Operation type 3: Reservoir normally empty
- Operation type 4: River bed sediment

Miller (1953) developed the following relation to work out average density of the sediment deposited in  $T$  years.

$$W_T = W_1 + 0.4343 K \left[ \frac{T}{T-1} \log_e(T) - 1 \right]$$

where  $W_T$  = average density after  $T$  years of reservoir operation,  $W_1$  = initial unit weight worked from Lara and Pemberton equation and  $K$  = constant based on type of operation and size of sediment and can be obtained from Table 12.5.

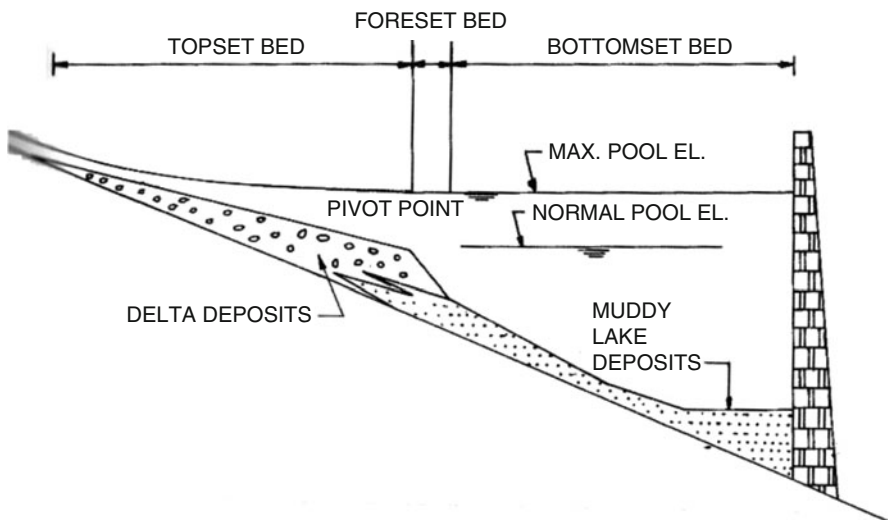
## 12.3 PATTERN OF SEDIMENT DEPOSITION IN RESERVOIRS

When the river flow enters the reservoir, the flow velocity decreases and the sediment starts depositing on the river bed. The sediment comprising bed load and coarse particles deposits in the head reaches forming delta deposits and fine sediment in suspension is carried into the reservoir towards the dam and deposits near the dam. A generalized longitudinal deposition profile is shown in Fig. 12.3 where delta deposits are made of coarse sediment and lake deposits are of fine sediment. Sometimes lake deposits have layers of coarse sediment either because of tributaries meeting the reservoir, or hill slides or sudden drawdown or extreme flood.

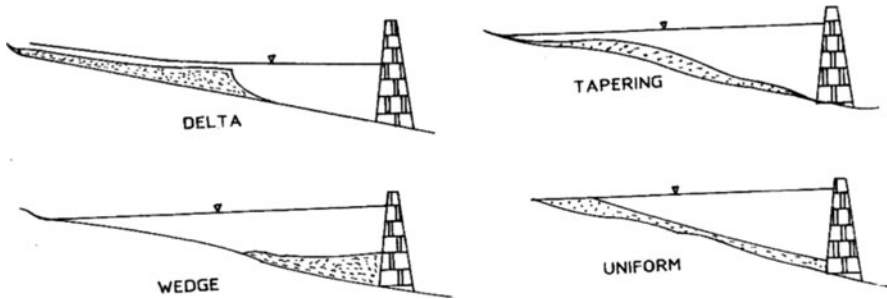
The surveys have shown that the actual patterns of deposition are far from the generalized deposition profile shown in Fig. 12.3 because the deposition is controlled by a large number of factors such as:

- Reservoir shape
- Reservoir operation
- Sediment characteristics such as size, intensity, bed load.
- River valley slopes
- Location and size of outlets
- Vegetation at the head of reservoir.

Depending on these factors the following four basic types of deposition pattern are identified. These are as shown in Fig. 12.4.



**Fig. 12.3** Generalised sediment deposition zones in a reservoir. *Source:* Morris et al.



**Fig. 12.4** Longitudinal patterns of sediment deposition in reservoir. *Source:* Morris et al.

- (i) Delta deposits: These are formed in head reach when sediment is predominantly coarse.
- (ii) Wedge shaped deposits: These are formed near the dam when sediment is predominantly fine and when reservoirs are operated at low water level to pass floods.
- (iii) Tapering deposits: These deposits progressively become thinner towards the dam. These occur in long reservoirs operated normally at high pool elevation with fine sediment moving towards the dam.
- (iv) Uniform deposits: These may occur in narrow reservoirs with fluctuating operating level and with small proportion of fine sediment.

However, observations and survey of reservoirs exhibit complex patterns which are the combination of above basic types. The observed longitudinal profiles of sediment deposition of a few Indian reservoirs (Matatila, Maithon, Panchel, Nizam Sagar) are shown in Fig. 12.5. These patterns exhibit delta formation at the minimum drawdown level and it is seen slowly progressing towards the dam. The patterns are generally a combination of tapering and uniform types.

The observed depth-wise distribution of some of the Indian reservoirs is shown in Fig. 12.6. It is seen that in some reservoirs large proportion of total incoming sediment deposits in higher elevations resulting in loss of live storage which affects the life of reservoir. Hence it is very essential to have the knowledge of depth-wise distribution and pattern of sediment deposition at the planning stage of reservoir. Besides the effect on life of reservoir, sediment deposition pattern affects the fixing of sill level of outlets and penstocks as well as in locating tourist spots and water games.

### 12.3.1 Predicting Sediment Distribution

Several empirical and semi-empirical methods have been developed for predicting sediment distribution in reservoirs by various agencies such as USBR, US Corps of Engineers and other researchers working in this field.

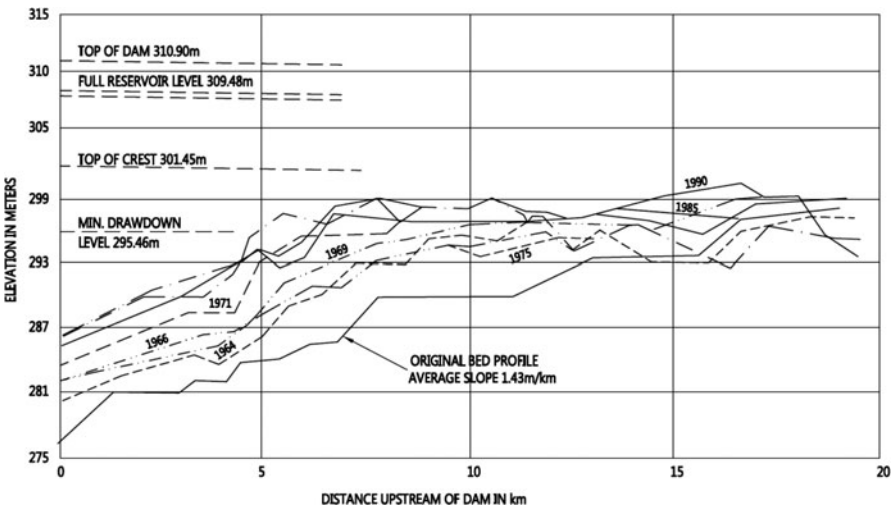


Fig. 12.5a Longitudinal bed profile of Matatila reservoir. *Source:* Asthana

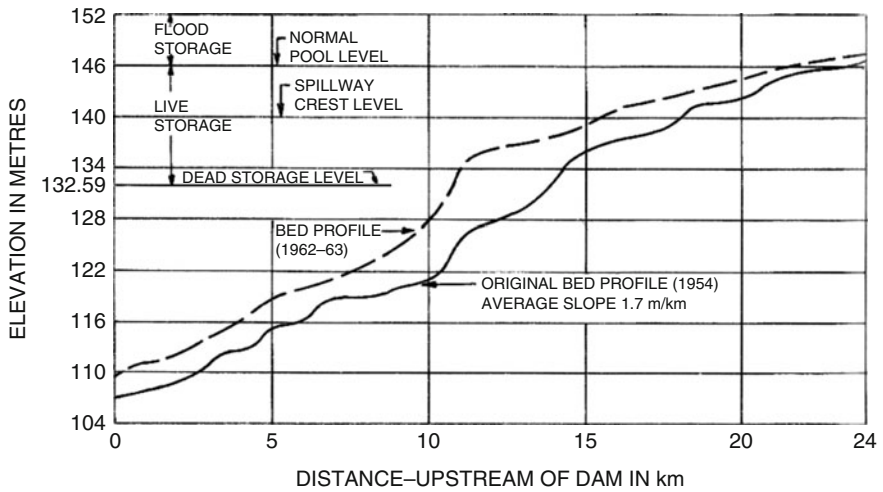


Fig. 12.5b Longitudinal bed profile of Maithon reservoir. *Source:* Asthana

12.3.1.1 Empirical Area Reduction Method

The most widely used method is known as Empirical Area Reduction method developed by Borland and Miller of USBR. It is based on the observed data of 30 reservoirs in USA.

In this method the reservoirs are classified into four standard types according to the reservoir geometry. The depth-wise distribution of these four types is shown in Fig. 12.7.

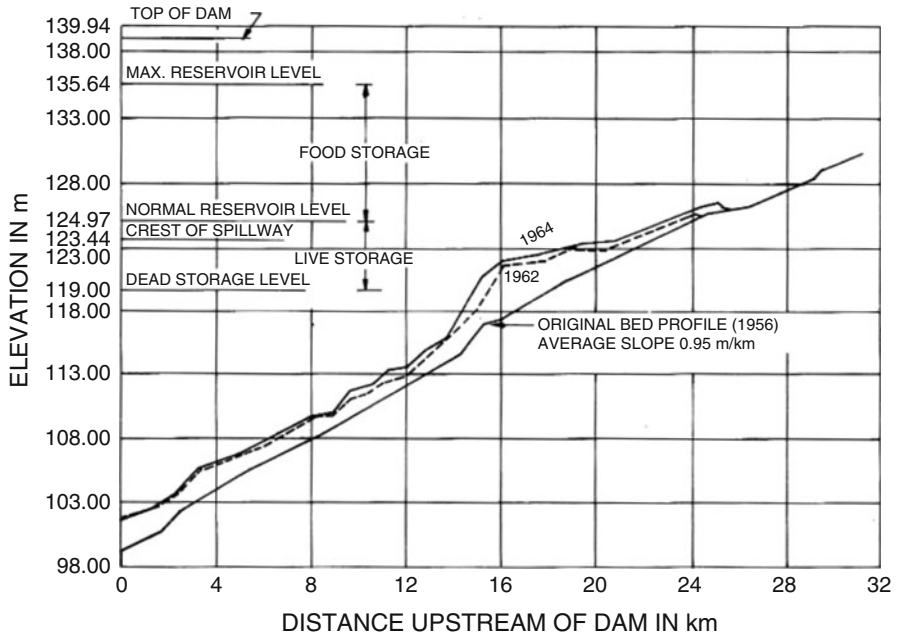


Fig. 12.5c Longitudinal bed profile of Panchet reservoir. *Source:* Asthana

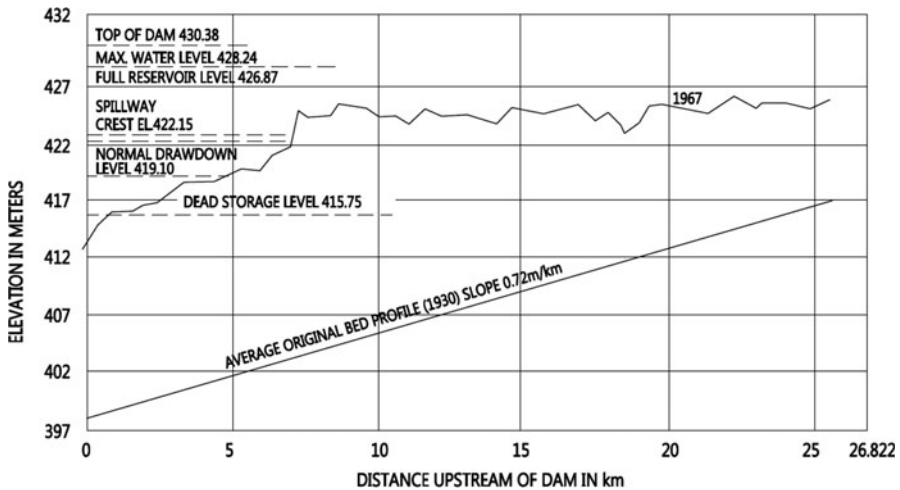


Fig. 12.5d Longitudinal bed profile of Nizamsagar reservoir. *Source:* Asthana

The type of a reservoir can be determined by plotting the reservoir depth and capacity on a log-log paper as shown in Fig. 12.8. From the plot determine the value of  $M$  which is the reciprocal of the slope of the line. Depending on the value of  $M$ , the

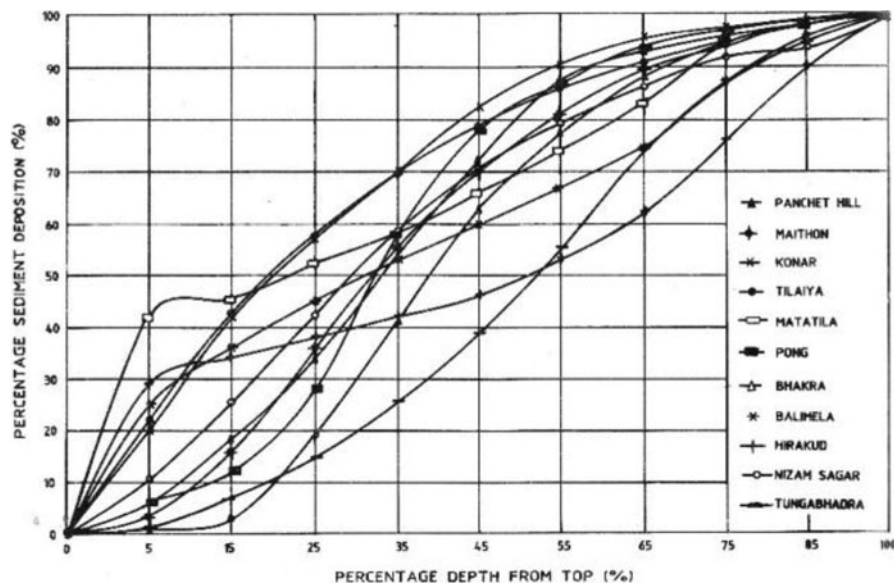
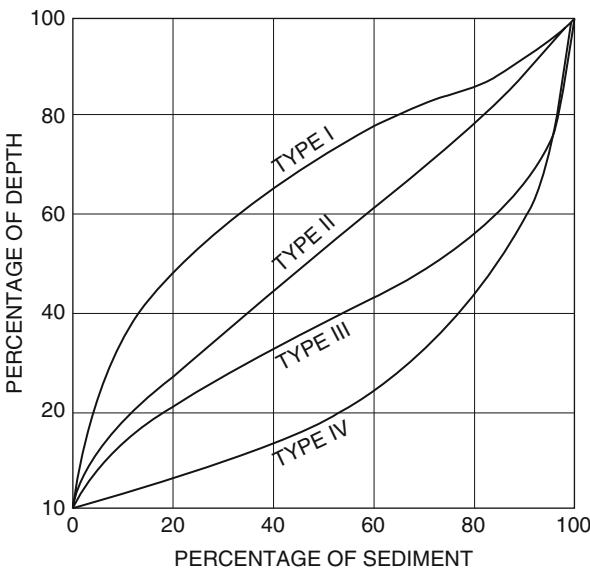
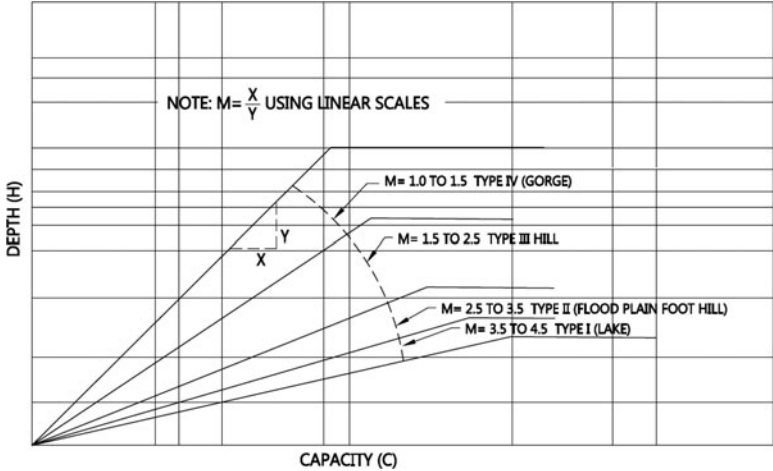


Fig. 12.6 Depth wise sediment distribution in some Indian reservoirs. *Source:* Data CWC

Fig. 12.7 Type of reservoir curve based on observations. *Source:* Borland et al.





**Fig. 12.8** Type of reservoir based on shape of reservoir. *Source:* Borland et al.

**Table 12.6** Type of reservoir – M value

Value of M	Reservoir type	Standard classification
1.0 – 1.5	Gorge	IV
1.5 – 2.5	Hill	III
2.5 – 3.5	Flood plain-foot hill	II
3.5 – 4.5	Lake	I

type of reservoir can be assigned the standard type on the basis of Table 12.6 for further computations.

The four standard type sediment versus depth curves have been converted into area design curves for computational purposes using the equation.

$$A_p = cp^m(1 - p)^n$$

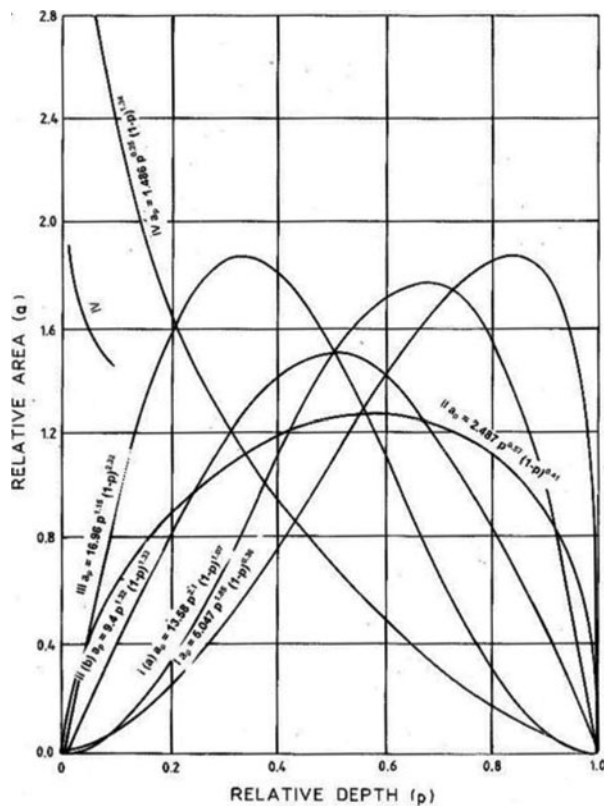
where  $A_p$  is dimensionless relative area at a relative depth ‘ $p$ ’ above the river bed and  $c$ ,  $m$  and  $n$  are dimensionless constants.

The numerical values of the constants are computed by least square method for each type satisfying the condition that the total area under the curve is unity. The area design curves are shown in Fig. 12.9.

The stepwise procedure for computation is as below:

- (i) Determine type of reservoir on the basis of value of  $M$ .
- (ii) Determine relative depth for each increment. This is the ratio of incremental depth to the total depth (FSL above river bed).
- (iii) Select new zero elevation to start first trial. New zero elevation is the elevation upto which reservoir is expected to be silted-up. The sediment areas at and below this level will be original reservoir areas. Sediment areas at each depth

**Fig. 12.9** Area design curves. *Source:* Borland et al.



- increment above the assumed new zero elevation would be obtained by dividing the original reservoir area at new zero elevation by corresponding  $A_p$  value and multiplying this ratio by the  $A_p$  values at each succeeding depth increment.
- (iv) After computing sediment areas the incremental sediment volumes can be worked out by the average end area formula.
  - (v) If the total of the computed sediment volume at different depths does not tally with the total sediment to be distributed, the next approximation for the new zero elevation would have to be made.

The application of this method requires the original area-capacity curve of the reservoir and the total quantity of sediment expected to be deposited in the reservoir. The procedure as explained above requires several trials to arrive at correct new zero elevation. Later USBR developed procedure to select new zero elevation avoiding trials. The procedure is illustrated in CBI&P Publication 89. It is also detailed in IS 12182. Now computer programs are available for these computations which are time consuming.

The studies later revealed that selection of the standard type of a reservoir only on the basis of shape (value of  $M$ ) is not adequate for predicting the sediment

**Table 12.7** Selection of type based on reservoir operation

Reservoir operation		Shape		Weighted type
Class	Type	Class	Type	
Sediment submerged	I	Lake	I	I
		Flood plain foothill	II	I or II
		Hill and Gorge	III	II
Moderate drawdown	II	Lake	I	I or II
		Flood plain-foothill	II	II
		Hill and Gorge	III	II or III
Considerable drawdown	III	Lake	I	II
		Flood plain – foothill	II	II or III
		Hill and Gorge	III	III
Normally	IV	All shapes		IV

**Table 12.8** Selection of reservoir for sediment type

Prominent size of sediment	Type of reservoir
Sand or coarser sediment	I
Silt	II
Clay	III

distribution in a reservoir. To give equal weightage to reservoir operation and shape, the selection of type of reservoir may be made as given in Table 12.7.

In cases where the weighted type is the choice of two types, then a judicious judgement should be made on the basis of which of the two i.e. the reservoir operation and the shape is more governing. The texture and out size of sediment could be considered in this judgemental analysis from the guidelines in Table 12.8.

### 12.3.1.2 Area-Increment Method

This method is also developed by USBR which is purely mathematical. It is based on the assumption that reservoir area at each elevation is reduced by a constant value which is termed as area correction factor and is equal to the original area at the elevation upto which the reservoir is completely filled with sediment. The basic equation is:

$$V_s = A_o(H - h_o) + V_o$$

where  $V_s$  = total volume of sediment to be distributed,  $A_o$  = reservoir area at new zero elevation,  $H$  = reservoir depth at dam (FRL – river bed level),  $h_o$  = depth of new zero elevation (level upto which reservoir will be filled-up) and  $V_o$  = volume below new zero elevation.

$h_o$  is determined by trial satisfying the above equation. Then  $A_o$  (the area correction factor) is worked out. The reservoir capacity at other elevations is then computed reducing surface areas by  $A_o$  and using end area formula.

The distribution pattern by this method is found almost identical to the type-II design curve of empirical area reduction method and is often used to estimate new zero elevation for the trials in empirical area reduction method. It is also seen that applicability of this method is limited to the cases where sediment expected to be deposited in the reservoir is of small quantity. Borland and Miller have limited this sediment volume to 15% of the reservoir capacity.

### 12.3.1.3 *Mathematical Models*

Mathematical models with computer programs have been developed after 1970s, to predict sediment distribution in reservoirs at different times. It gives sediment deposition contours in the reservoir showing aggradation for the quantity of incoming sediment. The loss in storage capacity is also evaluated and degradation below dam can also be predicted. The models which are widely used are HEC-6, Mike-11, Mike-21 and RASEED. These models use techniques of finite difference or finite element to solve momentum and continuity equations. These have the limitations as these are generally one dimensional, and have limitation in correctly modeling the reservoir topography. Mike 2I-C of DHI is used for two dimensional studies. The two dimensional models, due to longer processing time are currently being used in coordination with one dimensional model for short time frames in limited locations requiring more details. Mike 2I-C takes into account the vertical distribution of flow (both main flow and helical flow) as well as the vertical distribution of suspended sediment. This model thus gives lateral variation of bed elevation.

There are practical difficulties in obtaining required data for discharges and quantity and characteristics incoming sediment, and in making a reasonably correct choice for sediment transport equation for the movement of sediment in reservoir. The mathematical model studies are being carried out for planning every major reservoir and are being found useful in locating the outlets and sediment flushing conduits in a dam.

## 12.4 SEDIMENT CONTROL MEASURES (REF. 2, 12, 14)

The average annual loss of capacity of reservoirs in India varies from 0.2 to 0.9% of the storage capacity. The rate of siltation can be reduced by taking measures which aim at reducing/minimizing the following:

- (i) Sediment yield from catchment
- (ii) Sediment influx in reservoir and
- (iii) Sediment deposition in reservoir.

### ***12.4.1 Reducing Sediment Yield from Catchment***

Sediment yield from river catchment can be reduced by adopting soil conservation measures which are generally classified as below:

- (i) Structural measures such as check dams, contour bunding, gully plugging, bank protection etc. These are expensive and need maintenance for over long period of time.
- (ii) Non-structural measures are of two type: (a) Agronomic and vegetative measures which include afforestation, suitable cultivation practices, vegetative screening etc. and (b) Operational measures include a planned schedule of construction and timber harvesting activities in the areas to minimize exposure of soil to erosion.

The vegetative measures are usually adopted because these are effective, inexpensive and easy to implement along with structural measures. The experience with soil conservation practices throughout the world indicates that (i) these are advantageous when implemented for over a long period of time, (ii) these are not very effective under geologically unstable conditions and in regions of extreme climate, (iii) these are more effective in smaller catchments where reduction in sediment yield may range from 30 to 70 percent, and (iv) in large catchments their effectiveness is less and these are cost prohibitive and results cannot be expected in short period.

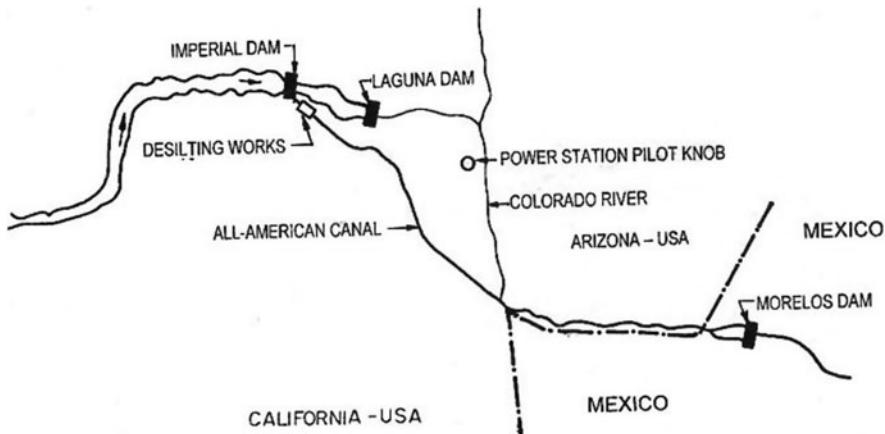
In order to make conservation measures economically viable these are integrated with watershed management plan of the area through which other benefits such as employment opportunities, income generation, stable ecology etc. are achieved.

In view of the experience of these conservation measures in various projects in India (CBI&P Pub.89), the opinions greatly differ regarding their effectiveness. But in some cases such as Tungabhadra reservoir where long term both structural and non-structural measures are adopted, these are found quite effective. Hence in all projects such as Ranjit, Largi and Tehri such conservation measures are being adopted to treat highly degraded areas as part of watershed management plan.

### ***12.4.2 Reducing Sediment Influx in Reservoir***

One way of reducing sediment inflow in the reservoir is to adopt bypassing of the sediment laden flows. A diversion dam or weir is constructed in the upstream of the reservoir and the sediment laden flows are carried around the reservoir through a canal or tunnel to meet the main river channel in the downstream of the dam. The configuration of by-passing scheme depends on factors such as reservoir length, river configuration, river slope, geology, river hydrology, economics, maintenance of structure and environment problems.

One of the biggest open channel bypassing example is All-American canal on river Colorado in USA (Fig. 12.10). Bypassing technique has been successfully,



**Fig. 12.10** The All-American canal from Colorado river in USA. *Source:* Batuca et al.

adopted in other countries and on several reservoirs in China. An example of Nagla reservoir in South Africa is shown in Fig. 12.11 where a maximum discharge of  $2000 \text{ m}^3/\text{sec}$  is bypassed through the channel.

Sediment bypassing through tunnels/galleries/conduits etc. has also been successfully attempted in small reservoirs in several European countries, Japan and South Africa. An example of sediment bypassing through a gallery at Amsteg reservoir in Switzerland is shown in Fig. 12.12. The gallery is 305 m long.

A variation of above concept of sediment bypassing is to build off stream reservoir. The possible configurations are shown in Fig. 12.13. It needs proper space and topography and is expensive also and so rarely adopted. Several such reservoirs are built in Taiwan.

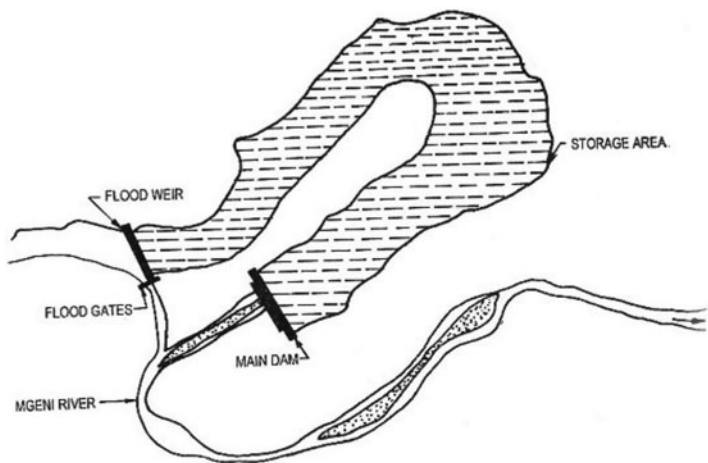
Sediment bypassing is techno-economically feasible in small reservoirs with a small capacity inflow ratio.

### 12.4.3 Reducing Sediment Deposition in Reservoir

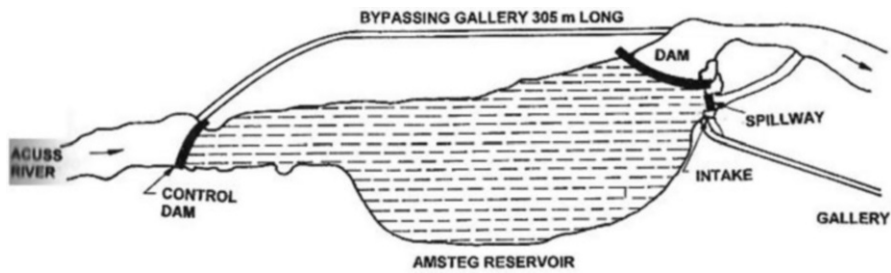
Sediment deposition in a reservoir can be effectively reduced or minimized by providing low level sluices and adopting well planned operational techniques of sediment sluicing and venting density currents. These have been followed with varying degree of effectiveness in USA, China, Egypt, India and other countries. In both these operational techniques some flow has to be wasted.

#### (i) Sediment sluicing

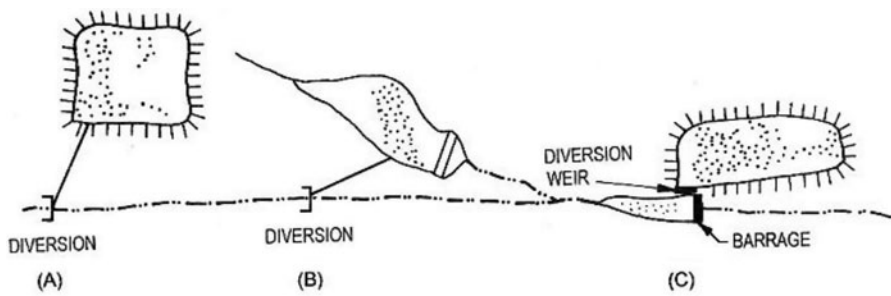
It is achieved in two ways: (a) free flow sluicing which involves emptying the reservoir to outlet level and (b) pressure flushing which requires lesser draw-down. The effect of reservoir lowering on flushing of sediment is shown in Fig. 12.14. The



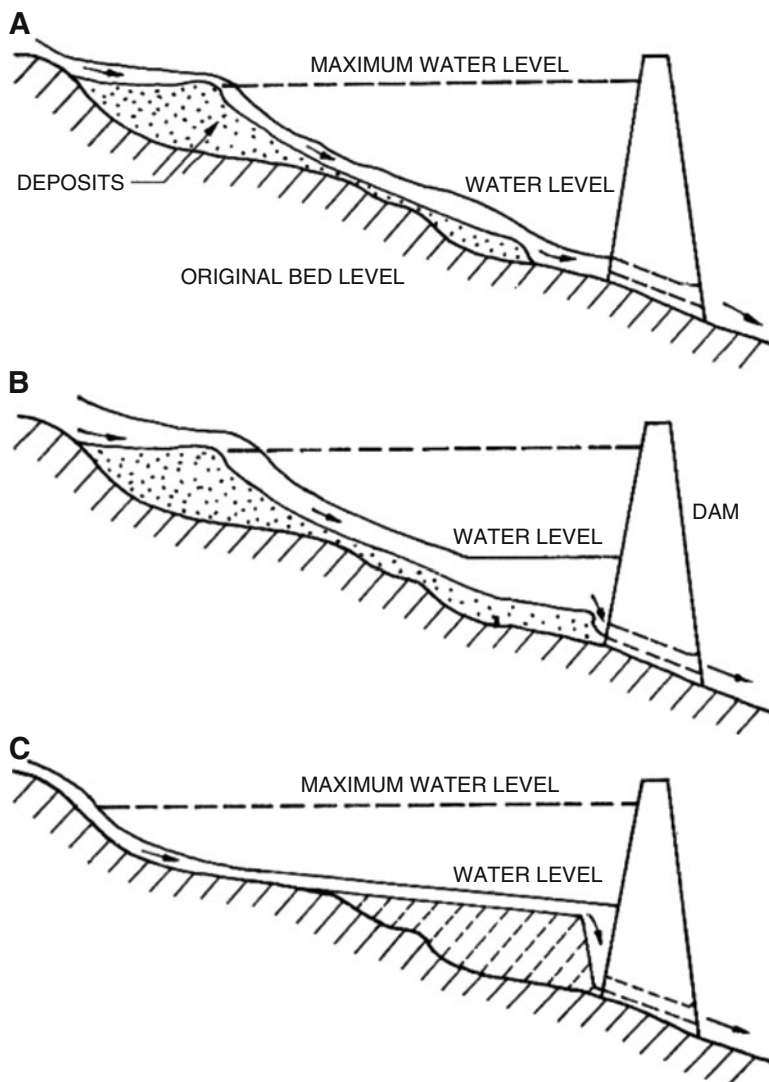
**Fig. 12.11** Sediment bypassing configuration at Nagda dam South Africa. *Source:* Battuca et al.



**Fig. 12.12** Bypassing of Amsteg reservoir in Switzerland. *Source:* Battuca et al.



**Fig. 12.13** Alternative configuration for off-stream reservoirs: (A) Excavated for Dyked impoundment; (B) Reservoir on a small tributary; (C) A barrage to control flow in off stream reservoir. *Source:* Morris et al.



**Fig. 12.14** Showing conditions after (a) flushing with full drawdown, (b) flushing with insufficient drawdown and (c) long period flushing with insufficient drawdown.

free flow flushing has been found more effective and is generally adopted. The following conditions favour successful sluicing operation:

- Reservoir should be narrow and elongated and of small and medium capacity.
- It should have sufficient outlets of adequate size close to river bed level. ICOLD (Bull. No. 67) recommends that outlets should be able to pass a discharge of 1 in 5 years return period. These should be designed to produce free flow at full drawdown.

- Run-off should be large compared to reservoir capacity.
- Sediment should predominantly be of fine to medium size and in suspension.

In view of the effectiveness of free flow sluicing, large size low level sluices have been provided in the diversion dams of run-of-river power projects in India such as Salal, Chamera II and Nathpa Jhakri. In Baira-Siul project flushing is being done through the diversion tunnel. After emptying the reservoir, flushing is done for 24 hours to flush the deposited silt. This is done atleast once in a year. During flushing the power house is closed.

The experience has shown that for successful and efficient flushing operation, emptying of reservoir is necessary. Therefore, its application is limited to small reservoirs where  $c/i$  ratio is less than 0.3 because large quantity of water is wasted in flushing the sediment. Several relations based on field data have been developed to relate the quantity of sediment flushed and the quantity of water released during flushing. One simple relation is as below:

$$V_s = 0.048 V_w^{0.687}$$

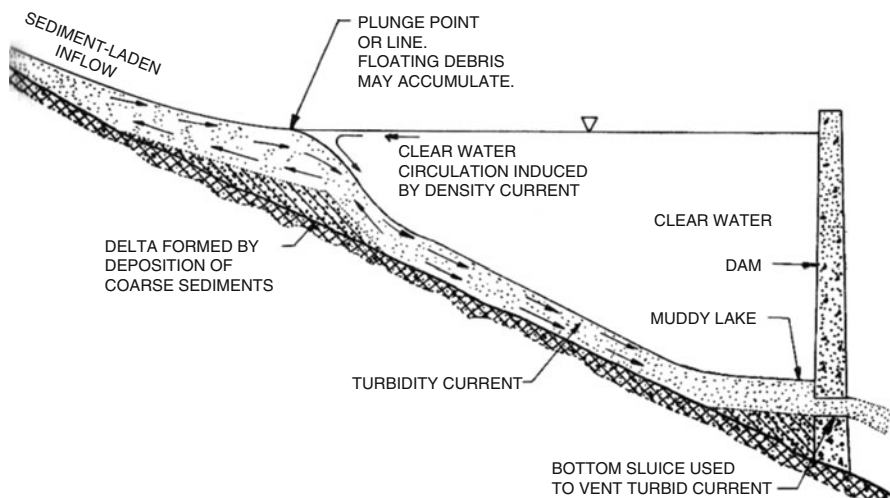
where  $V_s$  and  $V_w$  are the volumes of sediment flushed and volume of water used, both in  $Mm^3$ .

#### (ii) Venting density currents

Density differences at various depth in a reservoir are caused either due to temperature change or turbidity variation due to sediment in suspension or both. Turbidity density current are useful in reducing reservoir sedimentation if they reach upto the dam and are vented through low level outlets. A conceptual sketch showing travel of turbidity density current is shown in Fig. 12.15. If these are not vented, they form a muddy pool near the dam. If the duration of turbid inflow is less than travel time to the dam, the current dissipates in the area near the dam. In Lake Mead (USA) these density currents are vented out after travelling a distance of 129 km. Successful venting of density currents depends both on the size and location of low level outlets and the timely opening and closing of outlet gates. Observations in Indian reservoirs have not shown development of strong density currents.

## 12.5 REMOVAL OF DEPOSITED SEDIMENT

Another method to restore the silted capacity of reservoir for future utilization is removal of silt deposits. Removal to deposited sediment from a reservoir shall be resorted to when the above methods to reducing reservoir sedimentation are not techno-economically feasible. The techniques used are: (i) sediment siphoning, (ii) excavation and (iii) dredging. These are briefly explained below:



**Fig. 12.15** Schematic diagram of the passage of a turbid density current through a reservoir and being vented out through a low level outlet. *Source:* Morris et al.

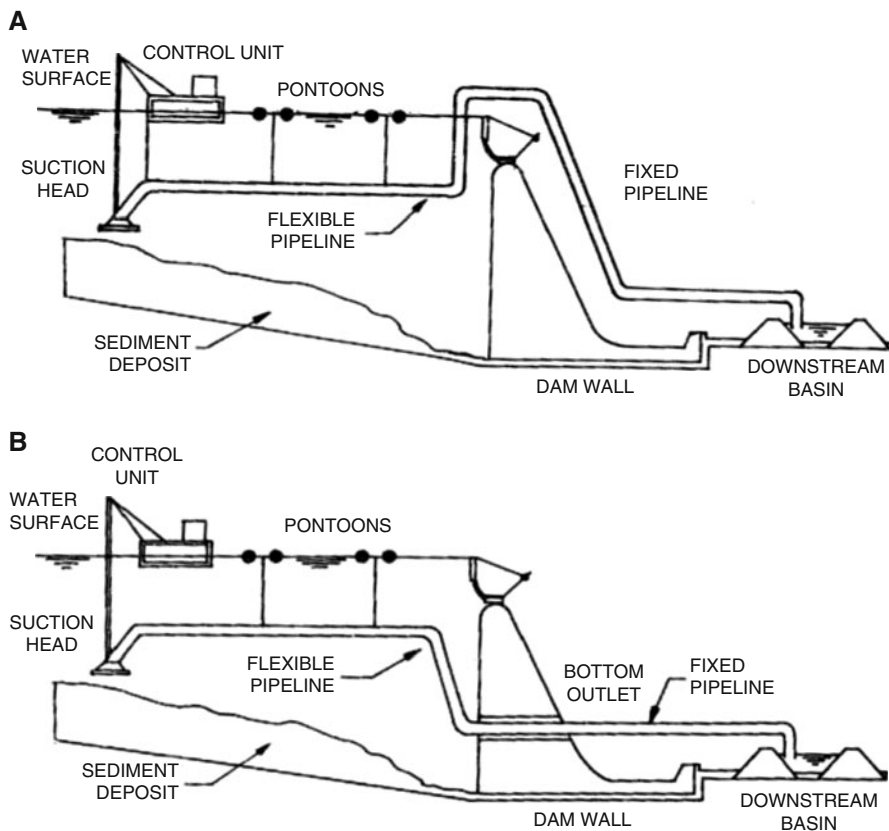
### 12.5.1 Sediment Syphoning

It is another hydraulic process of removing deposited sediment from a reservoir by accelerated pressurized flow in a pipe crossing the dam using hydro aspirator. The syphon works under upstream and downstream head difference. The working principle of two types of syphoning schemes is shown in Fig. 12.16. In the first case pipe goes to the downstream over the dam crest. In this case a vacuum pump is required to initiate syphonic action. In the second case, the pipe line goes through a bottom outlet or intake in the dam body. The hydraulic head upto which the syphon acts is limited to 10.0 m. Hence, it has been used in small dams in USA, Italy, Nepal and other countries with success. It is recommended in reservoirs where density of sediment deposits is of order of  $1.3 \text{ T/m}^3$ . The length and diameter of pipe should be so adjusted that velocity ' $V$ ' in pipe is greater than 1.5 m/sec. The sediment carrying capacity  $Q_s$  (kg/sec) is given by the following relation based on experimental and field observations.

$$Q_s = 0.272 V^{2.73}$$

### 12.5.2 Excavation

It involves emptying the reservoir and removing deposited sediment manually or mechanically by using conventional earth moving equipment. Excavation is



**Fig. 12.16** Working principle of sediment syphoning: (a) Syphoning over the dam and (b) Syphoning through the dam. *Source:* Batuca et al.

frequently done to help flushing operation. Excavation and flushing operations were used to remove deposited sediment in Kundah Palam reservoir in Tamil Nadu. Manual excavation is carried out in small reservoirs in India, China, Indonesia etc. and, mechanical equipments are used in small reservoirs in USA, Japan etc. This can be done in upper reaches of reservoirs where coarse sediment gets deposited and the reach gets emptied every year for a considerable period.

### 12.5.3 Dredging

It is the process of removing sediment deposits from the bottom of the lake/reservoir with the help of a dredger which is kept afloat on the water surface. Dredgers are of two types: (i) mechanical and (ii) hydraulic. Mechanical dredgers use buckets to dig and lift sediment to the surface with minimum water content and lifted material is

placed into conveyance system for transportation and disposal. It is not practicable to use these dredgers in big reservoirs.

In hydraulic dredging sediment is loosened, mixed with water and transported from point of excavation to the disposal point in the form of sediment water slurry. A rotating head of dredger loosens the deposit under water and the slurry is sucked through a suction pipe. Then it goes to disposal site through a pipe line. The advantages of hydraulic dredging include low production cost, high dredging rate, working without interfering with the functioning of reservoir and convenient mode of transporting through pipe line. Long pipelines usually need intermediate booster pumps to maintain proper velocity in the pipeline. It can handle coarse to fine sediment. The major drawback of hydraulic dredging is loss of large quantity of water. The slurry is 80–90% water and 10 to 20% sediment. Hydraulic dredging has been adopted in the balancing reservoir of Beas-Sutlej Link Project. It is operative since the commissioning of the project around 1975. However, there has been environmental problems in disposal of sediment.

## 12.6 DECOMMISSIONING OF DAM

The dam and its reservoir due to siltation needs decommissioning when it has outlived its utility. It can be done in two ways: (i) The dam may be left with sediment if it is safe and environmentally acceptable and (ii) remove the dam but before that the sediment management measures would have to be taken. A large number of small (height around 7.5 m) dams have been removed in USA. However, this stage has not reached in India. The process of the decommissioning (ASCE, 2000) is in development stage.

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# Chapter 13

## Instrumentation



### 13.1 GENERAL

Instrumentation in a dam is done to monitor its structural behaviour and of the foundation during construction and operation. The objectives of instrumentation in dams are the following:

- To provide information on a continuous basis for assessing the performance of dam and to control through remedial measures any adverse condition detrimental to the safety of dam.
- To verify design assumptions through observations from the instruments. It also gives confidence to the designer if the dam is performing as intended.
- To improve design techniques.

### 13.2 CONCRETE DAMS

#### 13.2.1 *Types of Measurements*

The measurements for which instruments should be installed in a concrete dam are grouped into two categories (IS 7436-Part-2), one is obligatory and the other is optional.

##### ***Obligatory Measurements***

The following measurements shall be made on all dams.

- (a) Uplift pressure at the base of dam at sufficient number of transverse sections (atleast 3).
- (b) Seepage through dam and foundation
- (c) Temperature of the interior of dam

- (d) Displacement measurement – except for dams of height 20 m or less with no foundation problems, the displacement measurement may be done by one or more of the following methods:
- By suspended plumb bob
  - By geodetic measurements
  - By joint meters embedded at the contraction joints where grouting is required to be done.

### ***Optional Measurements***

These measurements are warranted for high dams or under special circumstances such as unusual design of structures or doubtful foundation conditions.

- (a) Stress
- (b) Strain
- (c) Pore pressure and
- (d) Seismicity

The above measurements are considered obligatory for all high dams in Himalayan region which is highly seismic and has geologically weak rock formations.

## ***13.2.2 Purpose of Measurements***

The measurements grouped above serve specific purpose and have importance in assessing the behaviour and performance of dam which is briefly given below.

### ***Uplift Pressure***

It gives hydraulic pressure at any point at the base. The effect of grout curtain on reduction of uplift is required to confirm the design assumption and to assess the efficiency of grout curtain in reducing uplift.

### ***Seepage***

The seepage is customarily collected from foundation through seepage drains located downstream of grout curtain and form drains in the body of dam which discharge in foundation gallery. Seepage measurement at certain locations is an indicator of the overall performance of dam. Any sudden change in quantity or quality of seepage water at certain location indicates some problem either with the grout curtain or with the seals at transverse joints or with both. This may warrant remedial measures. The seepage appearing at the abutments or downstream of dam also indicates a problem to be investigated and remedied.

### ***Temperature***

Temperature of concrete in dam is measured by embedding thermometers inside the body of dam and also on or near the surface. Temperature observations during construction, if found abnormal, may lead to change the lift height or the treatment of aggregate. Temperature observations during operation lead to ascertain nature and extent of thermal stresses and the structural behaviour of dam. It also helps

during construction to ascertain when to undertake grouting of contraction joints if it is proposed in design.

### ***Displacement***

The dams are generally built in blocks separated by transverse joints. The plumb bob is generally placed in the deepest block to measure the tilt or deflection of the block with reservoir full condition. The regular observations are used to study the elastic behaviour of the dam. The foundation displacement is also measured by using inverted plumb bob.

The geodetic measurements of the surface markers on top and downstream face of dam indicate the displacement of dam with respect to surrounding area and is direct indication of the structural behaviour of dam.

### ***Stress Measurement***

The stress measurement inside the mass concrete helps in monitoring the structural behaviour of dam. A comparison of computed stresses and actual stresses can indicate whether there is any distress condition warranting the remedial measures.

### ***Strain Measurement***

Factors like temperature and stress in the concrete mass result in volume changes causing strain. Hence strain measurement is necessary. It is also necessary to compute tensile stresses which cannot be measured by stress measuring instruments as these can measure only compressive stress.

### ***Pore Pressure***

Measurement and record of pore pressure inside the concrete mass would indicate the effectiveness of formed drains and unusual reduction of pore-pressure from normal would be indicative of crack formation.

### ***Seismicity***

Seismological laboratory may be established near the dam site to record changes in seismicity pattern due to creation of reservoir and to study behaviour of dam during earthquake.

## ***13.2.3 Measuring Instruments – Type, Choice, Location etc.***

### ***General***

Measuring instruments are grouped in two classes, embedded and external.

- (i) Embedded instruments – These are instruments placed inside the concrete mass to measure stress, strain, joint opening, temperatures, pore pressure, uplift, foundation deformation etc.
- (ii) External instruments – These are for measuring displacement and tilt of dam and are placed on the surface of dam.

The measuring instruments available are mechanical, hydraulic, pneumatic and electrical types. The electrical type instruments facilitate easy remote reading, use of data logger and computers. These have high resolutions and accuracy. Hence electrical type instruments are being widely used due to their long-term stability, though hydraulic type are easy to handle and to install and can be read remotely but they have problems with connecting tubes getting blocked or broken.

The electrical type instruments are of two types: (i) resistance wire type and (ii) vibrating wire type. In resistance wire type the change in the resistance of the conductor due to variation in temperature and stress is measured and calibrated. These measure strain which can be converted into stress as elastic constant  $E = \text{stress/strain}$ . These suffer from the influence of change in cable conductor resistance and leakage resistance between conductor due to temperature changes and moisture ingress respectively. The effect can be minimized and temperature corrections can be applied.

In vibrating wire type instruments the increase in tension increases natural frequency of wire which is measured by the use of magnetic circuit. These are not sensitive to cable resistance and moisture movement. The influence of temperature changes is less significant. The correction factor supplied by the manufacturer can be applied to compensate the effect. Hence the vibrating wire type instruments are generally preferred over resistance type.

### ***Uplift Measuring Instruments***

It consists of a pipe installed at the point where uplift is to be measured and terminates in a gallery directly above the measuring point which is generally 1 m below dam base. The upstream face of pipe is fitted with a tee section and a Bourden type pressure gauge for observing water pressure. In the pipes placed downstream of grout curtain the pressure reading can be taken either by sounding or with pressure gauge.

The uplift measurements should be done at three dam sections in case of small dams (less than 30 m height) and at five sections in large dams out of which one each in deepest over flow and non-overflow blocks and rest near abutments. At every section uplift should be measured at following locations:

- (a) Upstream of grout curtain
- (b) Downstream of grout curtain before drainage hole.
- (c) Immediately downstream of drainage hole
- (d) At the end of cross gallery
- (e) Space between (c) and (d) equally divided.

Instead of pressure pipe, pressure cell of electrical type can be used.

### ***Seepage Measuring***

The drain in the foundation gallery carrying seepage water leads to the drainage sump from where the water is pumped out. The total seepage from dam can be estimated in the sump. Some V-notches can be installed at different locations in the drain to locate and monitor the reach of abnormal seepage in the dam.

***Temperature Measuring Instrument***

Temperature measurement in mass concrete can be done either through Carlson type resistance thermometers (thermistors) or vibrating wire type thermometers. In resistance type thermometer the resistance of electrical wire is function of temperature and in vibrating wire type the length of wire changes with temperature resulting in change in frequency. Resistance type are found more accurate and are extensively used.

These are placed atleast in one of the deepest section and in long dams may be placed in additional sections in a  $15\text{ m} \times 15\text{ m}$  grid horizontally and vertically. Thermometers should normally be placed both at upstream and downstream faces or near the faces and along the centre line of the block.

At the locations of resistance type stress, strain and joint meters, the use of thermometers is not necessary as these measure the temperature accurately.

***Displacement Measuring Instruments***

Electrical type joint meters either resistance type or vibrating wire type are placed at the transverse and or longitudinal joints spaced vertically at about 15 m in the entire height of joints for measuring joint movement. Movement of joints can also be measured through calibrated tapes by fixing reference points on either side of joints at locations accessible from galleries

Displacement of dam is measured through plumb bob which is suspended through a shaft in the dam. The upper end of wire is anchored slightly below the top with the dam body and the bob is immersed in oil to damp oscillations. The deflection of the wire is measured with reference to a reference point by means of a microscope and micrometer slide or any other simple means. Generally one plumb bob is placed in the deepest block.

The horizontal movement of foundation is measured by inverted plumb bob. The lower end of the wire is anchored in the foundation at the bottom of the borehole and the upper end is attached to a float buoyed up by water in a covered tank. It is generally placed in the same block in which normally plumb bob is installed.

The vertical displacement of foundation can be measured by installing a joint meter at the contact of dam base and foundation. The end fixed with foundation can be mounted on a pipe with lower end anchored at the bottom of hole.

For the movement of dam with reference to abutments geodetic methods are used. The targets are fixed on the two abutments and the downstream face of dam and the top. Measurement of movement of the targets on dam with respect to these placed on abutment is made by using precision theodolites or electronic distance measuring equipment.

***Stress – Strain Meters***

The vibrating wire type strain meters are generally used and are embedded in the block of maximum height near the foundation where the stresses are maximum. These can be placed at mid height if height of dam is large. The strain observations

are converted into stress. The stress meter measures only compressive stress. Hence both strain meter and stress meters are placed at same location.

Strain meters are generally installed in clusters of 5 or 7 in three-dimensional configuration for determination of principal strain and stress. The cluster of stress-strain meters should be placed at least at 5 locations in a straight line from upstream to downstream face in a horizontal plane. By the side of the cluster of strain meters a 'no stress' strain meter is installed to determine correction to be applied due to thermal expansion of mass concrete. It is an ordinary strain meter and is preferred to be installed at a location where load deformation is absent but temperature change is affecting the concrete.

The stress-strain meters shall also be installed at specific locations of high stress whose observations may be of interest.

#### ***Pore Pressure Measuring Instruments***

The pore pressure measurement is generally done by installing vibrating wire type piezometers. Porous tube piezometers installed in the holes drilled in concrete can also be used. The piezometers are normally installed at a spacing of 10 to 15 m along the width of dam and the bottom row is located either at the contact plane of dam and foundation or just above the foundation or in the foundation by drilling holes to meet design requirement. The other row may be at  $1/3^{\text{rd}}$  or mid-height of dam.

#### ***Seismic Measurement***

Seismometers are used to monitor the effect of earthquake on the dam. These shall be installed at the dam base in the gallery and also at the dam top. It may be installed at mid height if the dam height is more than 100 m. A seismological laboratory is established near the dam at the abutment at a location which is not affected by vibrations.

#### ***Other Measurements***

A water level recorder preferably an automatic type should be installed to record the water level changes in the reservoir. A meteorological laboratory should also be established at site to record rainfall, temperature, wind velocity, humidity etc.

### ***13.2.4 Installation of Instruments***

The installation of instruments is a specialized job and is done by the experts of the supplier. The details about installation of various types of instruments mentioned above can be found in relevant IS Codes such as IS 13232 for strain meters, IS 8282 for pore pressure measuring instrument, IS 13073 for plumb bob, and IS 6524 for thermometers. The 'Guidelines for Instrumentation of Large Dams' published by CWC shall also be referred for details on instrumentation of dams.

### ***13.2.5 Frequency of Measurement***

During construction, observations of temperature and joint opening at regular interval should be recorded to review concrete production, placement and cooling process.

The crucial period is the first filling of reservoir. Hence all the observations should be recorded before the filling of reservoir starts. The next set of observations may be made when reservoir is filled upto  $1/4^{\text{th}}$  height and the second set when reservoir is filled upto mid height and thereafter the observations may be made at every tenth of total height. The interval between two sets of observations should not be more than a month.

During the reservoir filling visual inspection of dam faces and abutments and study of leakage measurement should be carried out every day.

The following schedule for observations may be adopted after first filling during the reservoir operation.

1. Geodetic – four surveys in a year.
2. Plumb bob – weekly.
3. Strain gauges and thermometers – twice weekly.
4. Piezometers and uplift measurements – weekly.
5. Seepage – weekly.

These are as guidelines and may be altered as per site requirements.

### ***13.2.6 Example***

Figure 13.1 illustrates the provision of various measuring instruments at different elevations at a section of a concrete gravity dam which is 80 m high. The instruments are placed at the foundation level as well as at four elevations. The instruments installed are:

1. Uplift measuring pipes at foundation
2. Piezometers to measure pore pressure inside dam
3. Strain meters
4. Stress meters
5. No stress strain meters
6. Thermometers
7. Plumb bob to measure dam deflection
8. Invented plumb bob to measure foundation movement
9. Three-point bore hole extensometer in foundation
10. Survey targets at dam top
11. Strong motion accelerograph at top of dam

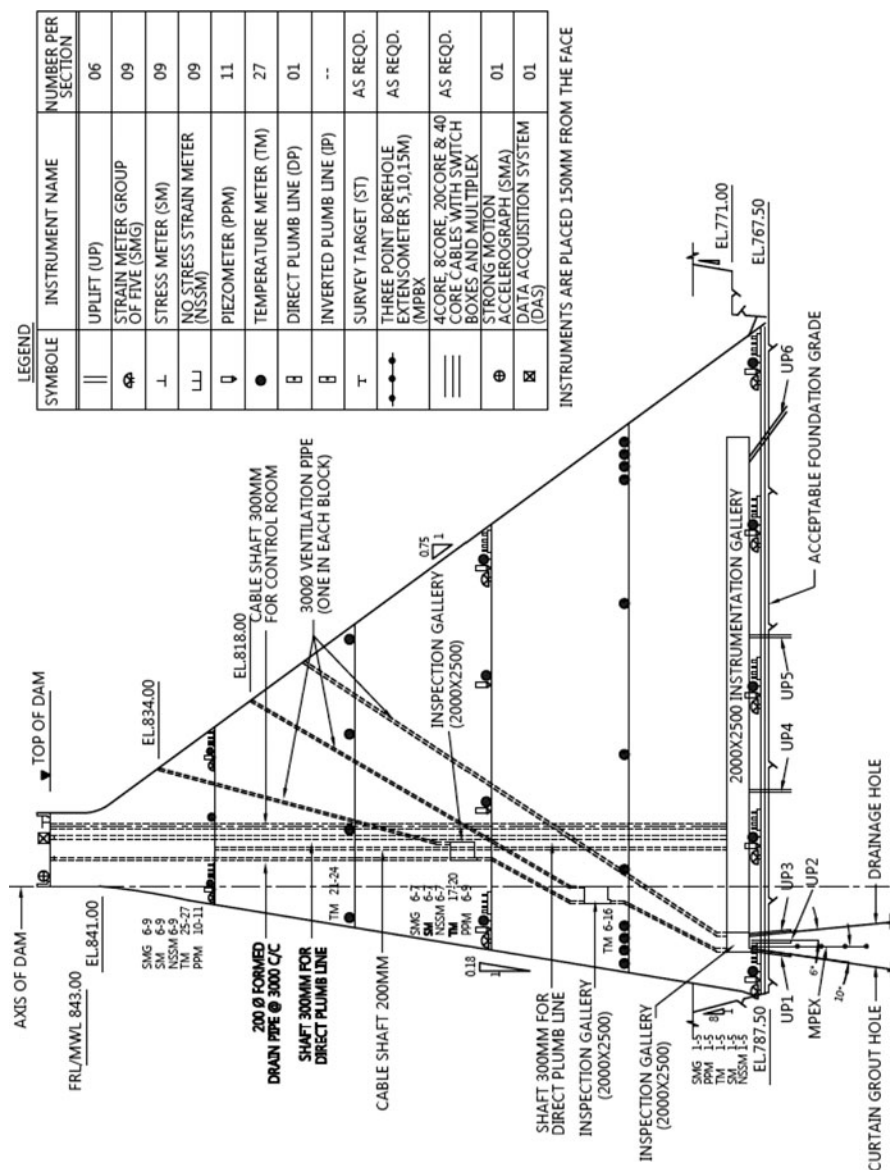


Fig. 13.1 Concrete Dam instrumentation (non-over flow section) of 80 m high dam.

In general stress meter, strain meter, no stress strain meter, piezometer and thermometer are placed at one place.

As a principle, instruments are provided in sufficient number because a large number of instruments become inoperative during construction.

### ***13.2.7 Recording, Monitoring and Evaluation***

The central data recording station for all instruments shall be established at a location which is easily accessible either in the inspection gallery or at the dam top. The data of instruments shall be recorded and monitored and evaluated for the performance of dam. The automated data acquisition and modelling is increasingly being used for the evaluation so that timely action, if required, may be taken.

### ***13.2.8 Instrumentation in RCC Dams***

The design and the behaviour of RCC dams is similar to the concrete gravity dam. The type of concrete and the construction technique of the two dams are different. The thermal stresses in RCC dam are generally insignificant. Therefore, generally thermometers are not to be installed in RCC dam as extensively as in a conventional concrete dam. The contraction joint off-sets or cracking is not associated with temperature variation but can be direct evidence of foundation instability; hence joint meters could be used to detect the movement. Piezometers shall be installed to record pore pressure in dam body and uplift in foundation. Plumb bob should be installed to record the movement of dam. Seepage measuring devices should also be installed. Surface targets on top and downstream slope to measure movement of dam with respect to abutments should be established. Strong motion accelerographs should also be installed to record the shaking during an earthquake.

## **13.3 EMBANKMENT DAMS (I.S. 7436 – PART-1)**

### ***13.3.1 Types of Measurements***

1. Pore pressure measurement – The pore pressure in foundation and clay core is measured by piezometers.
2. Internal instruments to measure horizontal movement, settlement and embankment compression.
3. External measurement of horizontal movement and settlement.

### ***13.3.2 Types of Instruments***

#### ***Piezometers***

Different types of piezometers are in use. These are hydraulic, pneumatic and electrical type. The electrical type are of two categories (i) resistance wire type and (ii) vibrating wire type. Generally hydraulic type and vibrating wire type piezometers are used.

#### ***Pressure Cells***

The electric type (vibrating wire) pressure cells are installed inside the embankment to measure the stress. These are generally placed in the core at the base as well as at different elevations of dam.

#### ***Measuring Horizontal and Vertical Movements***

For these movements inclinometers and horizontal movement gauges are installed inside the body of dam.

#### ***Horizontal Extensometers***

These are placed in decompressed zones near the top of dam to monitor the cracks.

#### ***Surface Settlement Points***

These surface targets are established externally at the dam top and slope to monitor through geodetic survey the movement of dam with respect to abutment and the settlement.

#### ***Seepage Measurement***

The seepage is measured through V-notches installed in the toe drain located in the downstream of dam. Along with the quantity the quality of seepage water should also be monitored.

#### ***Other Instruments***

These are same as in concrete dam stated in Section [13.2.3](#).

### ***13.3.3 Example***

The earth core rockfill dam which has recently been constructed (about a decade ago) at Tehri is the highest (260 m) such type of dam in Himalayas which are highly earthquake prone. The dam has been intensively instrumented to monitor the behaviour of dam and the foundation. Measuring instruments of the following type have been installed in the dam and its foundation.

1. Twin type hydraulic and vibrating wire type piezometers in dam foundation and the clay core to measure hydrostatic pressure at various points. The two types of piezometers are placed together in the foundation and at same locations in core.

2. Earth pressure cells to measure free field stress within the embankment.
3. Full profile gauge to monitor continuous internal vertical movement of d/s shell and in higher reaches of core.
4. Deflection and settlement measurement by inclinometers and horizontal movement gauges to measure internal movements.
5. Horizontal soil extensometers for monitoring the cracks especially in decompression zones of dam near the crest.
6. Vibrating wire temperature gauges to monitor temperature variation in dam body as well as foundation.
7. Surface settlement points to determine settlement and horizontal movement of both the dam faces.
8. Other devices for measuring seepage, seismicity, reservoir and tail water levels etc.

The instruments are provided at three sections of dam in several rows vertically spaced at 20 to 30 m. In all 353 nos. of measuring instruments including seven seepage measuring devices besides 152 nos. surface settlement points have been installed. The details of the instruments installed at the deepest section are as below:

• Hydraulic piezometers (H)	23
• Vibrating wire type piezometers (V)	57
• Vibrating wire thermometer (T)	03
• Vibrating wire type pressure cell (P)	48
• Inclinometers (S)	03
• Horizontal movement gauge (HMG)	02
• Full profile gauge (FPG)	02
Total	138

The location of these instruments in the deepest dam section is shown in Figs. 13.2 and 13.3. Besides instrumentation, provision has been made for two galleries in the core so that physical inspection of dam behaviour during construction as well as during operation could be done. Strong motion accelerograph have also been accommodated in the galleries. One gallery is provided at the top just below the crest and the other at the mid-height. These are shown in Fig. 13.3. The gallery at the top is 2.5 m (W) and 2.4 m (H) and it is open at base for visual inspection of tension cracks in the core if any. The mid height gallery is 2.5 m diameter circular made in segments with gap joints and the elevation in the centre point of gallery was kept 5.2 m above the elevation of the ends at abutments. A maximum settlement in the mid height gallery is observed as 2.178 m at the time of reservoir filling.

An automatic data acquisition system has been installed to acquire data from instruments. A common central recording station for all instruments (except hydraulic piezometers) has been constructed at top of dam.

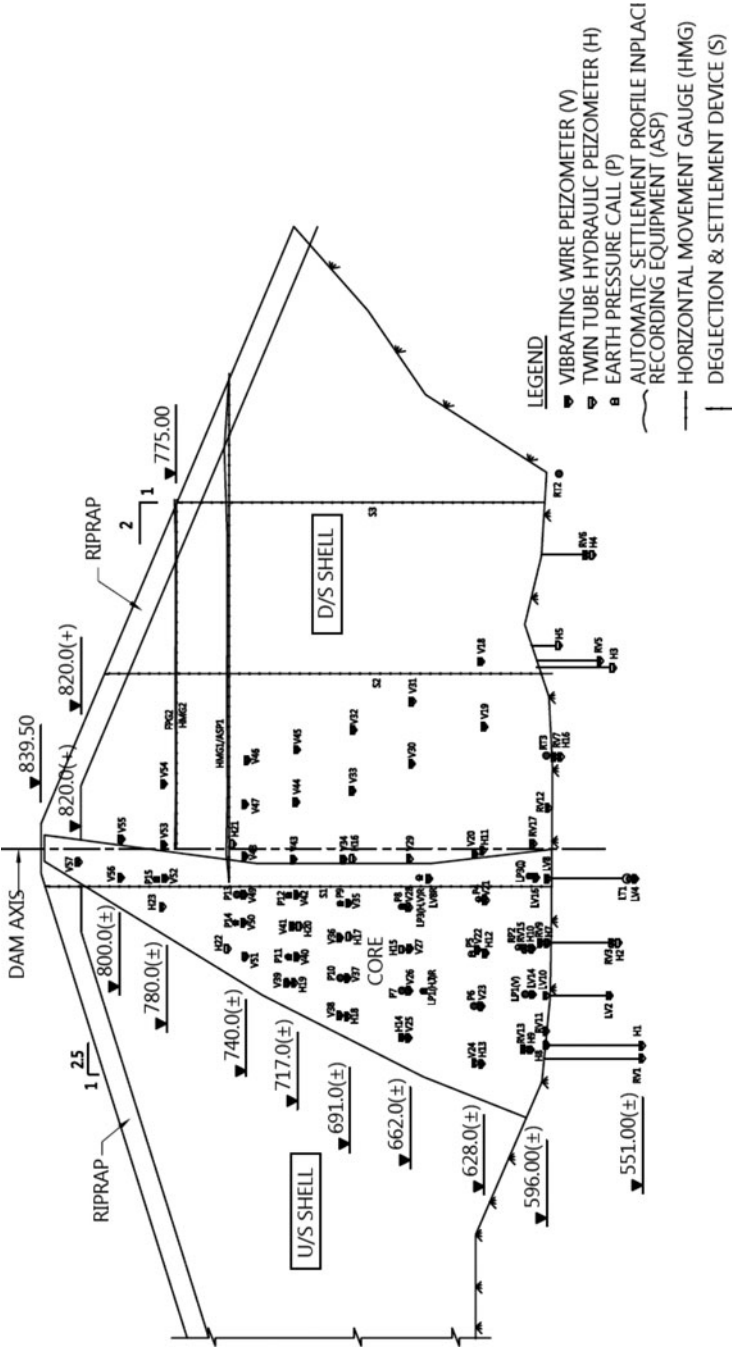


Fig. 13.2 Instruments at deepest section of dam.

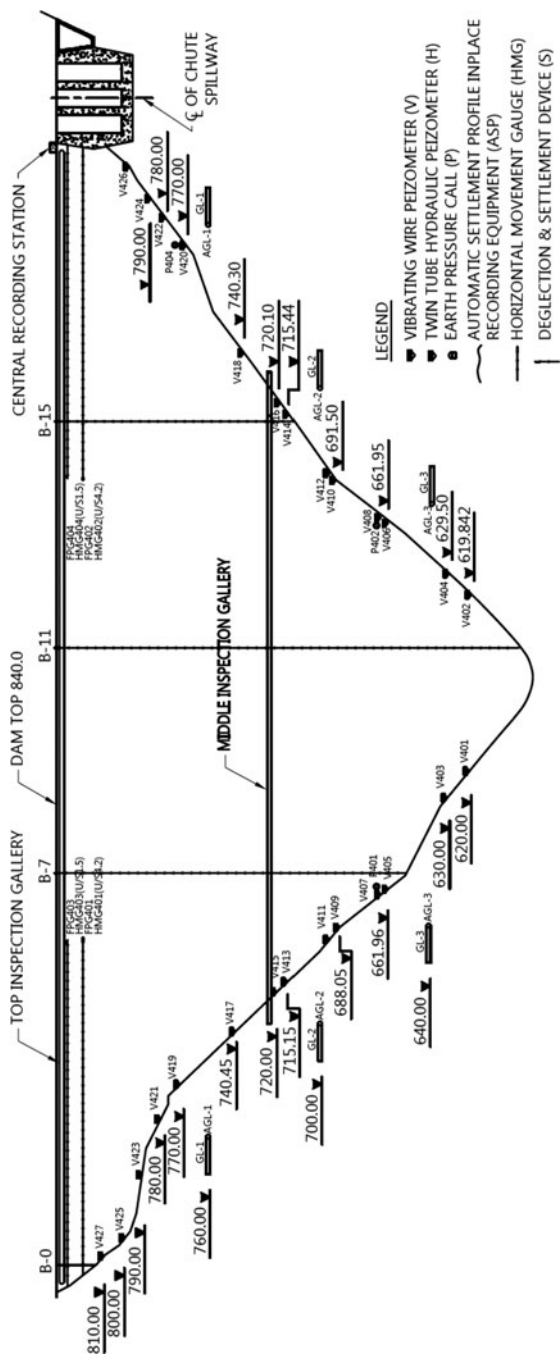


Fig. 13.3 Instruments in valley section along centre line of core trench.

The frequency of recording observations of instruments installed at Tehri dam is as below:

Sl. No.	Instruments	Frequency during reservoir filling	Frequency during operation
1.	Vibrating wire piezometers, pressure cells, thermometers, horizontal extensometers and seepage devices.	Once in a day	Once in a fortnight
2.	Hydraulic piezometers	Once in two days	Once in a fortnight
3.	Deflection and settlement devices and gauges	Once in two days	Once in a fortnight
4.	Surface settlement points	Once in two days	Once in a fortnight
5.	Joints of inspection galleries and observation of cracks	Once in two days	Once in a fortnight

The dam is in operation since 2006 and the monitoring and evaluation of observations of instruments and visual inspection have not indicated any abnormal behaviour of the dam.

## 13.4 CONCRETE FACED ROCKFILL DAMS (CFRD)

### 13.4.1 *Types of Instruments*

CFRD is a dam made of rockfill with a concrete slab on upstream face to act as impervious membrane. It is different than earth core rockfill dam because it does not have a clay core. Hence the instrumentation is slightly different than the one described for embankment dams because the instrumentation is required separately for observing the behaviour of the concrete slab and the rockfill. Instruments commonly installed in a CFRD are:

- Piezometers – open stand pipe and the vibrating wire type to measure water pressure.
- Inclinometers, electro – levels, joint meters, settlement cells, extensometers to measure settlement, deformation and movement.
- Seepage measuring device.
- Surface targets – to observe surface movement.
- Seismographs.

### **13.4.2 Instruments**

#### ***Piezometers***

Piezometers are installed to measure groundwater level and water pressure. These are generally installed in abutments and the foundation.

#### ***Settlement Cells***

Generally electric type (vibrating wire) settlement cells are placed to monitor settlement at various locations and levels in the embankment during construction, reservoir filling and operation of reservoir. This data can be used to evaluate to deformation of the face slab.

#### ***Electro-levels***

In addition to inclinometers the electro-levels are currently being used to monitor the deformation of face slab. It is installed and read as soon as concrete is poured. These are placed at a spacing of about 10 to 15 m at the maximum section and at two abutment sections.

#### ***Joint Meters***

Commonly vibrating wire type joint meters are used. One dimensional joint meters are used to monitor joint opening between concrete face slabs across vertical joints. Three dimensional joint meters are normally used in perimetral joints to monitor joint opening in all the three directions.

#### ***Horizontal Extensometer***

These are installed within the embankment to monitor horizontal movement. These are generally placed at the level of settlement cells.

#### ***Pressure Cells***

Electrical type pressure cells are generally installed at maximum cross section to monitor internal stresses within embankment at various levels in both directions.

#### ***Other Instruments***

Commonly placed devices are water level recorder, seepage measuring devices, surface targets on top and slope of dam, seismograph etc.

### **13.4.3 Example**

Instruments placed in a 178 m high CFRD (TSQI) constructed in China is given below to illustrate instrument in a CFRD.

Location	Instrument type	Numbers
Rockfill embankment	Settlement cells	50
	Horizontal extensometer	31
	Pressure cells	28
	Single joint meter	27
	Piezometers	34
	Seepage measuring	3
	Stand pipe water meter	19
Concrete face	Electro level	64
	3-D joint meter	36
	Single joint meter	26
	Strain meter	84
	No. stress strain meter	15
	Steel bar stress meter	76
	Thermometer	27
	Stainless steel pins (joint movement)	33
Crest and slope	Surface monuments	67

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# Chapter 14

## Construction Aspects of Dams



### 14.1 GENERAL

The dams are basically of two types: (i) concrete dams and (ii) embankment dams. The concrete dams are made of concrete and embankment dams are made of local material like soil, sand & gravel and quarried rock. The construction technique and the equipment to be used in construction for the two types of dams are different. The construction of dams in twentieth century has undergone lot of changes due to advances in concrete technology, scientific development of soil mechanics and rock mechanics and development of construction equipment. These developments changed the construction of dams from labour intensive activity to mechanization. This has also made it possible to construct dams as high as 300 m or more.

The cost of project largely depends on men, material and machines to be used. The project engineer-in-charge and his team is, thus, required to have a reasonable knowledge regarding the constructions methods/techniques involved in the construction of a dam and the management skill in handling men, material and machines to deliver a techno-economic project. It is a vast subject and lot of literature is available. In this chapter some aspects regarding material, construction processes/techniques involved in construction and the suitability and choice of equipment for concrete and embankment dams are described to give basic information about the subject to the readers.

### 14.2 CONCRETE DAMS

#### 14.2.1 Concrete

The concrete dams are made of concrete which is basically a mixture of aggregates coarse and fine, cement and water. Admixtures and pozzolones are added for

purposes such as improving workability, accelerating or retarding setting time and reducing heat of hydration. The proportion in which each ingredient should be mixed is basically based on the strength required, the aggregate to be used and the workability required for the placement.

### ***14.2.2 The Construction Processes/Activities***

The construction processes/activities and the stages involved in the construction of a concrete dam are as below:

- River diversion structures – This requires construction of cofferdams and tunnel or channel to pass the diversion discharge so that construction area for dam is isolated.
- Stripping of foundations – It involves removal of vegetation and overburden/ loose material and excavation upto fresh rock on which dam base can be placed.
- Foundation treatment – It comprises normal treatment which is consolidation grouting of dam base area and construction of grout curtain. Special treatment in some case is also required to be carried out where there are geological discontinuities, faults, shear etc.
- Quarrying, processing and stalking of material in separate piles of different sizes.
- Production of concrete – For producing concrete of required quality batching of material before mixing is of importance. It shall be done by weight in case of major dam project. In small works, batching by volume may be done. After batching, the ingredients of concrete shall be mixed with required quantity of water in a mixing plant of adequate capacity to meet the construction requirement.

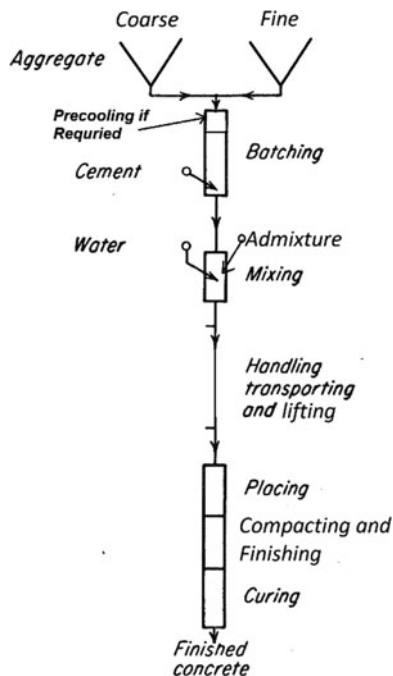
If precooling of aggregates is required the aggregate shall be cooled before it is brought to the batching and mixing plant.

The samples of concrete after mixing shall be collected and tested for strength and workability.

- Placing of concrete – It involves transportation of concrete from the mixing plant to the placement site. The other processes involved are lifting and placing the concrete at the exact location of placement.
- Compaction of concrete – After placing, concrete shall be compacted to the required degree by needle vibrators and surface shall be finished.
- Curing – To attain required strength concrete shall be cured with adequate moisture at a favourable temperature.

The details about river diversions works and foundation treatment of dams have been dealt in separate chapters. The processes of concrete production and placement in a dam, as given above, are graphically presented in a line diagram in Fig. 14.1. These are briefly described in following paras.

**Fig. 14.1** Concreting process in a dam.



### 14.2.3 Concrete Production

Concrete is a mixture of inert materials like aggregates with cement and water. The fine and coarse aggregates are joined together into a solid mass by the solidification of a paste made of cement and water.

#### 14.2.3.1 Ingredients of Concrete

##### 1. Cement

- (a) **Portland Cement** – Commonly Portland cement is used, though some special types of cements are also available. Portland cement is a mixture of siliceous, argillaceous and calcareous materials fused at a temperature of  $1400^{\circ}\text{C}$  and is ground to required fineness. The chemical combination of cement with water is known as hydration which is an exothermic reaction generating a heat of about 84 calories/gm after a few days of mixing the concrete. This causes temperature rise in mass concrete with steep temperature gradient between the centre and the surface of concrete block which may result in thermal stresses and cracking.

- (b) Low heat cement – This type of cement contains less quantity of calcium tri-silicate ( $C_3S$ ) and heat of hydration is limited to 65 calories/gram and is suitable for mass concrete in dams.
- (c) Portland-Slag cement – The Portland cement clinker and blast surface slag are ground to required fineness. It has less heat of hydration than Portland cement, so it is suitable for mass concrete works. With less  $Ca(OH)_2$ , it is more resistant to water containing sulphates and acidic water and so used in marine works with advantage.
- (d) Portland-Pozzolana cement – This is Portland cement blended with fine pozzolana. Pozzolana content varies from 15 to 20% by weight of cement. Its advantage lies in low heat of hydration. It gains strength slowly but ultimate strength is practically same as of Portland cement. It is also resistant to sulphate action. Hence it is of use in mass concrete works like concrete gravity dams.

## 2. Aggregates

Concrete aggregate generally consists of natural sand and gravel. Quarried and crushed rock is also used when natural sand and gravel is not economically available. If natural material is not available in sufficient quantity, a mixture of both natural and crushed aggregate is used. Crushed aggregate usually require more cement and water. However the workability of concrete with crushed aggregate can be improved by using air entraining agents.

Aggregates are classified as fine and coarse. The maximum size of fine aggregate is limited to 4.75 mm and beyond this size it is coarse aggregate.

The aggregate is commonly found to contain silt, clay and other organic matter which are not desirable for a sound and durable concrete of good texture. Washing of aggregate can to a large extent remove such undesirable material. The permitted quantity of silt and clay is limited to 5% by weight in fine aggregate and 3% in coarse aggregate.

The aggregate should be physically and chemical sound. Physically sound aggregate should be able to resist changes and disintegration on freezing and thawing, heating and cooling and wetting and drying. The aggregate should not chemically have reactive substances such as silica which may react with the alkali in cement. Generally shales, friable sandstones, micaceous rocks are found undesirable and physically unsound material for concrete aggregates. But quartz, quartzites and dense volcanic rocks are good for making strong and wear resistant concrete.

Flat and elongated particles of aggregate are not desirable as these affect the workability of concrete and necessitate highly sanded mixes and more use of cement and water. However, a small percentage of such particles may be permitted in large sizes of coarse aggregate. The particles with maximum to minimum dimension of 3 to 1 should not be more than 30% in coarse aggregate.

Specific gravity is a quick indicator of good quality aggregate. High specific gravity indicates good and sound aggregate and low specific gravity value is an indicator of porous and weak material. The minimum value of specific gravity should be 2.6.

The coarse aggregate to be used in concrete should be well graded from maximum size to the minimum size of 4.75 mm. The gradation has a definite effect on workability and so efforts should be made to achieve as close a gradation at the job as specified in mix design. To achieve it, the deficiencies in the gradation of natural deposits shall be supplemented with crushed particles. The maximum size of coarse aggregate, which can be handled conveniently in mixing, transporting and placing of concrete, is 150 mm. The bigger size results in reduction of both cement and water for a given slump which is an indicator for workability.

The fine aggregate is termed sand with a maximum size of 4.75 mm. The sand characteristics effect the workability to even greater degree than that of coarse aggregate. It shall be well graded. Coarse sand results in harshness, bleeding and segregation and requires more cement to make up for deficiencies for finer sizes. More fines in sand result in high water requirement. For good quality workable concrete the fineness modulus of sand should be between 2.3 and 3. The sand to be used in concrete should predominantly be natural sand. It may be supplemented with crushed sand to make up the deficiencies in natural sand gradings.

### 3. Water

Mixing water for concrete should be reasonably clean and free from silt, organic material, alkalies, salts etc. If water from the river carry excessive quantity of suspended sediment and other solids, it should be clarified by any means before use. Settling basins or ponds can be made to settle suspended solids before the stream water is used for mixing.

### 4. Admixtures

Admixtures are the materials which are added in small quantities during mixing to modify the properties of concrete. These are mainly to accelerate or decelerate rate of hydration, to improve workability etc.

- (a) Accelerators – The early strength of concrete can be achieved by adding small quantity of calcium chlorides. It should be limited to 1 to 2% by weight. It is primarily of advantage in concreting operations during winter season or in the structures where early removal of form work is required or where the early loading is to be permitted. However, it should not be used where metal works are to be embedded and also in pre-stressed concrete structures to avoid possibility of corrosion.
- (b) Retarders – Organic chemicals such as sulphuric acid or carboxylic acid salts are used as retarders to extend the setting time of concrete which may be required either to compensate acceleration in setting time due to high placement temperatures and to avoid cold joint on the job. These chemicals also act as water reducers which have the property of improving workability.
- (c) Air entraining agents – These are added to improve workability due to ball bearing action of air bubbles. These also improve the resistance of concrete to weathering and reduce permeability. These are frequently used with advantage

in lean and harsh mixes which are commonly used in construction of concrete dams. These are similar to soap and detergent and derived from resins, fats and oils. For a given workability the use of air entraining agents reduces water requirement of the mix. The disadvantage is in slight reduction in strength of concrete. Hence, its use should be first tested in the laboratory.

- (d) Pozzolanic materials – Pozzolanic materials are inert material and their use in concrete result in better workability, less heat of hydration, lower permeability, greater resistance to aggressive soils and water, and saving in cost. The pozzolana retards the development of strength in initial stage but a replacement upto 20% cement by pozzolana is likely to result in same strength in a year as that of concrete without pozzolana. Hence it is commonly used in mass concrete for dams. Usually fly ash is used as pozzolana which is a bye-product of coal burning in thermal power plants. Fly ash has been found effective in controlling alkali-aggregate reaction but the alkali content in flyash should not be more than 1.5 percent.

Silica fume which is bye product of ferro-silicon industry is also used as pozzolanic material. It is mainly fine grained  $\text{SiO}_2$ . Its use improves strength, durability, increases resistance to alkali-aggregate reaction, and decreases permeability. The use of a given amount of silica fume can replace three times the quantity of cement which may result in reduced rise in temperature. The use of both fly ash and silica fume can replace about 30% of cement which would result in large reduction in temperature rise.

The admixtures should be used after carefully examining the instructions of manufactures and adequate testing in a laboratory. The use of two admixtures should not be permitted unless the adverse effect of one on the other has not been investigated in detail.

#### 14.2.3.2 Handling of Concrete Ingredients (Materials)

- (a) Water is stored in a tank near the batching plant. It is necessary to provide a method of accurately measuring the quantity of water per batch. Concrete mixers are usually equipped with water measuring tank but these should be checked periodically to verify the amount supplied. Other devices include water meters and water weighing tanks.
- (b) Cement: It is supplied by the manufactures either in bags of 50 kg or in bulk through rail-road cars, box cars or trucks. In major projects it is generally received in bulk as it is cheaper than bag cement. Bag cement must be stored in a dry place in such a manner that it does not get set. Depending on the quantity required per batch, the cement bags are supplied to batching operation/site.

The bulk cement is unloaded from cars or trucks and stored in suitable silos or in a fully enclosed overhead bins. The pozzolana, if to be used, should also be

stored in another silo near the cement silo. A weighing hopper suspended beneath the storage bin is used to measure the correct amount of cement released.

- (c) **Aggregate:** Aggregates, both coarse and fine, are obtained either from natural deposits or by crushing quarried rock or from the combination of both so that a well graded aggregate containing all required sizes could be produced for production of concrete. Generally natural sources of aggregates contain deleterious materials which are ordinarily removed by washing and the unsatisfactory gradation is corrected either by wasting surplus sizes or adding deficient sizes. This needs aggregate processing plant which may comprise crushing and screening arrangements to produce the required sizes of aggregates from the natural gravel and boulders or quarried rock. The production of crushed aggregate involves drilling, blasting at the quarry, then loading, transporting, crushing, screening, handling and storing of the aggregate in stock piles of different sizes near the project site. The types of crushers and screens to be used depend on the maximum size of crushed rock and the various sizes of aggregate required for production of concrete.

Figure 14.2 illustrates the flow diagram of the aggregate processing plant for Philpott dam (USA) showing the arrangement of crushing and screening processes to produce sand and aggregates of sizes varying from 150 mm to 20 mm. This plant was established at the quarry site and the finished aggregate was hauled by trucks to concrete mixing plant. The aggregate processing plant can also be located near the project site and quarried rock may be transported from quarry to the processing plant by trucks. The finished aggregate can be fed to the batching and mixing plant through belt conveyers.

### 14.2.3.3 Batching and Mixing

The concrete materials should be weighed before mixing for all major jobs. The weighed materials should be properly and carefully handled so that the batches reaching the mixer shall be uniform and complete when released by the measuring equipment.

In small jobs or projects the batching equipment may be separate from the mixer. Concrete mixers are of various types defined by the capacity and drum type. Tilting mixers are generally more efficient than other types because they can discharge with minimum of segregation.

A central mixing plant is generally installed to produce concrete for a large structure such as a concrete dam. It includes equipment for handling and storing aggregate and cement, batchers and one to four tilting type concrete mixers of capacity 1 to 2 m<sup>3</sup>. The mixed concrete may be discharged generally into buckets or agitator type trucks.



#### **14.2.3.4 Precooling of Concrete Ingredients**

If pre-cooling of aggregates is required, it is generally done by forcing chilled air through concrete aggregate on the belt conveyor carrying aggregation from stock piles to the batching plant. Vacuum cooling process has recently been popular. It uses the fact that boiling temperature of water can be reduced by reducing vapour pressure. If the aggregate with water (moisture) is placed in such an atmosphere the water will quickly evaporate at less temperature. The heat required for evaporation will be taken from aggregate itself resulting in lowering the temperature of aggregate.

Cooling of cement and sand can be achieved by moving the material by means of hallow-screw conveyor through which chilled air is circulated. Cement is not lowered to a value of less than 33 °C to avoid condensation. Normally cement is not cooled. Mixing water is cooled to a temperature of 0 to 4.5 °C. Cooling of water is normally done by adding ice flakes. For purposes of precooling of concrete materials a refrigeration plant is generally built near the mixing plant.

A typical flow diagram for handling batching and mixing concrete is shown in Fig. 14.3.

#### **14.2.3.5 Quality of Concrete**

In order to produce quality concrete economically, it is necessary to regularly inspect batching and mixing plants. The concrete produced from the mixture should be tested for consistency, temperature, air content and density. Cubes/cylinders should be made from fresh concrete for compressive strength tests. Mix adjustments should regularly be made whenever required on the basis of the results of the tests on fresh concrete samples and the tests on aggregate gradation and the moisture. Such mix adjustments assures production of quality concrete.

### ***14.2.4 Transporting Concrete***

Concrete produced at the mixer should be carefully transported to the placement location. During transportation of concrete, separation, bleeding and loss of slump should be avoided. Buckets are commonly used. Concrete from the mixing plant is directly discharged into the buckets which are hauled by a truck or rail road to the placement location where they are lifted by a crane (trestle and crane system) and opened at the point where concrete is to be placed. The buckets used are of varying capacity ranging from about  $\frac{1}{2}$  to 3 m<sup>3</sup>. Buckets have bottom gates which may be opened in such a manner that concrete is discharged vertically downwards. Gates of small size buckets can be opened manually but gates of large size buckets are opened by compressed air and the opening can be regulated.

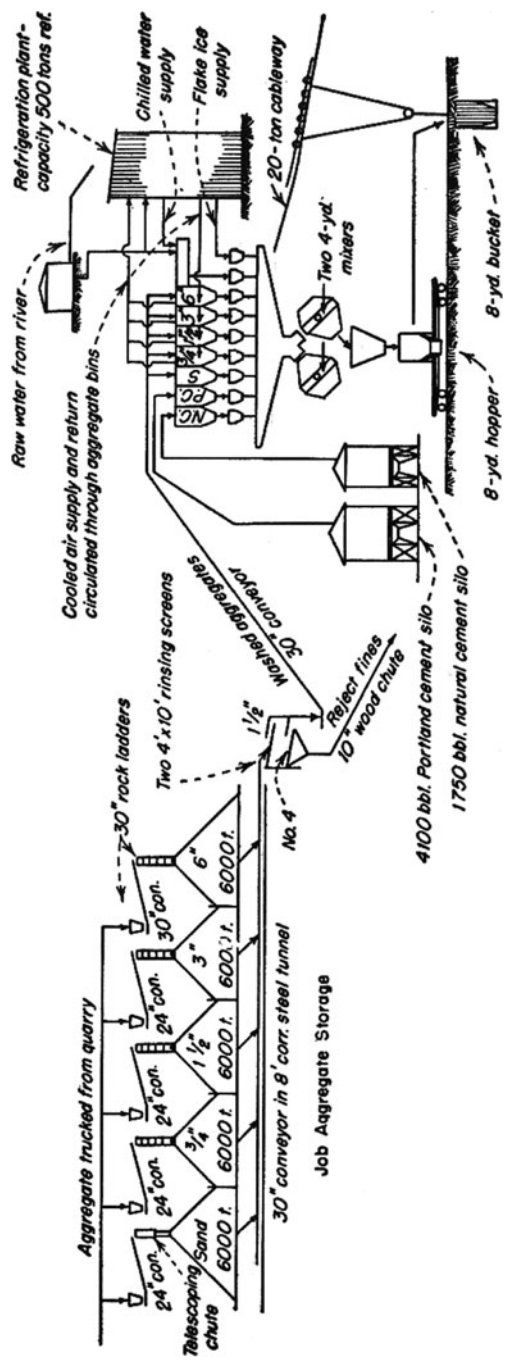


Fig. 14.3 Flow diagram for the concrete mixing at the Philpott dam. (Construction Methods and Equipment)

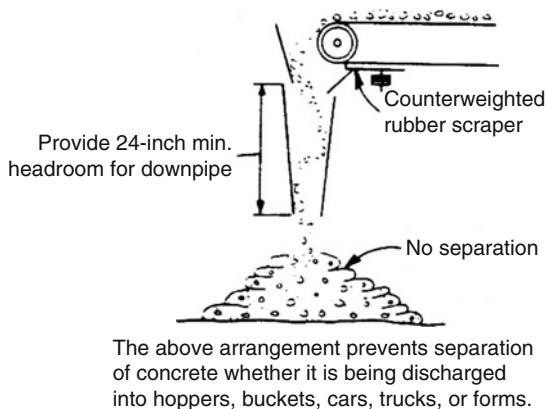
Buckets carrying concrete can be hauled to the placement site by using cableways or ropeways. In this case, at one end of dam a tower is installed at a point on the abutment above the top level of the dam. At the other end, the tower moves on a rail track laid as an arc of a circle of the radius more than the length of dam at top. A cable is tied to the towers, so one end of cableway is moveable to cover the base width of the dam. The moveable end of cable is placed near the mixing plant. The concrete bucket from the mixer is brought to the moveable cable point on a truck. The bucket is lifted by lowering the lifting hook and then the bucket moves on the rope and is brought precisely to the location where the bucket is to discharge the concrete. A sketch showing plan and section of cable arrangement is shown in Fig. 14.4.

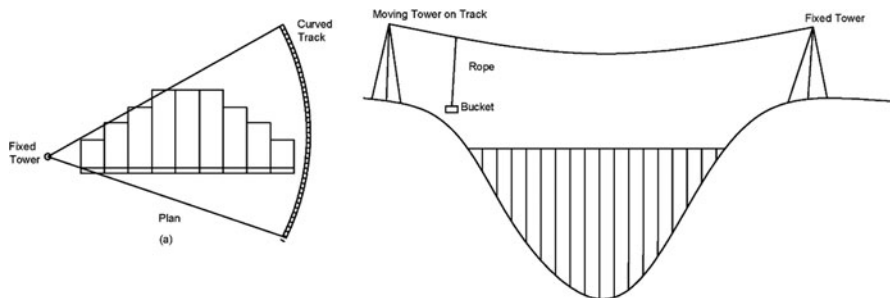
Bulk conveyor is another satisfactory method to transport concrete under some conditions. Uniform flow rate and high capacity are the advantages. But segregation at the discharging end and reduction in slump are the disadvantages. If these disadvantages are prevented, use of belt conveyors is desirable. To prevent segregation, a suitable type ladder or down pipe should be installed at the discharge end to ensure vertical drop of concrete. A rubber scraper should be placed on the return belt to prevent the loss of mortar and to feed the mortar into concrete receiving hopper. It is shown in Fig. 14.5. Slump loss is largely preventable by protecting the belt from sun and wind.

Pumping concrete through steel pipe lines is another satisfactory method of transporting concrete to long distances (not longer than 300 m) and placing it in narrow or tight locations. It needs high slump (7.5 to 10 cm) concrete. It should contain 2 to 3% more sand than otherwise required for gravity method of placement. Concrete with maximum size aggregate of 80 mm can possibly be pumped. The output achieved is normally 15 to 60 cum/hour.

In the locations where the concrete bucket cannot be opened directly for concreting, it may be opened at the nearest location and concrete may be placed at the required location either manually or by using a chute placed at an inclined position.

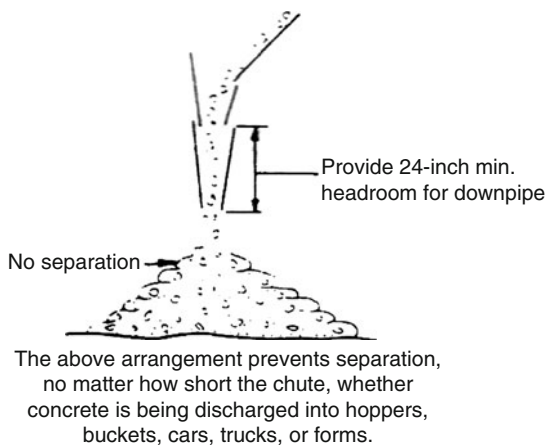
**Fig. 14.4** Cable arrangement for concrete placement.





**Fig. 14.5** Correct method of placing concrete using belt conveyor.

**Fig. 14.6** Correct method of placing concrete using chute.



The length of chute should be limited to about 3 m. The vertical drop should be avoided to prevent segregation. The correct method of placing concrete using chute is shown in Fig. 14.6.

Concrete can also be transported in transit mixers and agitator trucks. A truck mounted with a concrete mixer, in which the concrete materials are charged at a central batching plant and mixing job is done en-route to the job, is a transit mixer. These are available in various sizes ranging from 1 to 6 m<sup>3</sup> capacity. The disadvantage is the long period of mixing time which may affect slump and strength of concrete. The truck which hauls readymade concrete with only agitation en-route to the project to prevent segregation is called agitator. The concrete should be discharged from the transit mixer or agitator within 1½ hours after the water is added to concrete materials.

The methods of transporting concrete described above have their advantages and disadvantages and each is suitable for use under certain conditions at site. However, the method selected should be capable of delivering concrete of requisite properties such as consistency and without segregation of maximum size of aggregate.

### ***14.2.5 Placing Concrete***

#### **14.2.5.1 Preparations before placing**

Before placing concrete at a specific site it should be ensured that:

- Surface is properly prepared to receive fresh concrete. It should be moist but should not contain water.
- Construction joints are roughened and cleaned and are free from defective concrete.
- Forms are strong and joints are tight and are in proper alignment.
- All the reinforcement and other embedments are in place.
- The concreting equipment and other facilities are available at the site.

#### **14.2.5.2 Placing of Concrete**

The concrete shall be placed as far as practical in its final position. During placement segregation should be prevented. The vertical drop from the bucket to the placement point should not be more than 3 m. The concrete should be placed in horizontal layers and each layer should be thoroughly vibrated. The depth of each layer may vary from 30 to 50 cm. Soon after the concrete is placed and spread, it should be compacted through vibrators. Generally needle vibrators are used. During vibration the vibrator should be inserted vertically into fresh concrete but it should slightly penetrate into previous layer. Hence the time lapse between the placing of two layers should be limited to assure perfect bond between the fresh and previously placed concrete. Over vibration should be avoided, otherwise mortar will appear on the surface and will weaken the concrete.

In dams, concrete is placed in blocks in lifts of usually 1.5 m thickness. The concreting in a block in one lift should be done in a continuous operation. The next lift should be placed after 72 hours from temperature control considerations.

### ***14.2.6 Curing***

After removal of forms, which are generally removed after 24 hours, the exposed surfaces should be protected by curing. It is done so that initial moisture of concrete, which is for hydration of cement, does not evaporate before it is used in the process. Hence curing prevents loss of initial moisture and also replaces the moisture that has evaporated. This is accomplished by either leaving the forms in place for a longer time which may be seven days or more or keeping the surface wet or covering the surface with curing compound after removal of forms. Ordinarily curing is done by keeping surface wet for about 14 days.

### ***14.2.7 Concreting in Spillway***

Normally spillway is an integral part of concrete dam and is made of concrete in the same manner as the non-overflow section. High velocity flow along the downstream face of spillway generally cause cavitation damage. Hence the finish of the concrete surface needs extra precaution to avoid cavitation. Aerators are provided where cavitation is expected. These are provided on high head spillways. The design of aerators may be based on I.S. 12804. Their location and size is generally determined on a model. In case, the high velocity flow along spillway surface contains coarse sediment, erosion of concrete surface may take place. To prevent such damage high performance concrete ( $M_{50}$  or  $M_{60}$  with silica fume or epoxy concrete) is used on spillway surface in the region of high velocity and the basin floor or the buckets used for energy dissipation.

The crest and face of spillway should have formed surface. The alignment should be accurate and surface should be smooth to prevent damage from flowing water. Gradual surface irregularities should not exceed 6 mm. The chute, bucket and jump basin should have floated unformed surface. Floating should be done either by vibro-screed or metal edged screed. Concrete surface should be left undisturbed for 30 to 45 minutes until surface water disappears and there is no visible skin. Joints and edges should be finished with steel edging tool.

Abrupt offsets should not be permitted on surfaces subject to high velocity flow. If there is any such offset it should be completely eliminated by grinding to the required bevels as per guidelines given in IS 11155.

### ***14.2.8 Roller Compacted Concrete Dam (RCC Dam)***

This type of dam is basically a concrete gravity dam but it has different kind of concrete mix and construction technique. The concrete mix is a no slump concrete and has large proportion of pozzolana and less quantity of cement. The process of concrete production is similar to that of concrete dam described above. The method of transportation, placement and compaction are different than concrete gravity dam. In this case concrete from mixer is transported to placement site through dumpers. The concrete dumped at placement site is spread in uniform layers of thickness of 300 mm by dozers and compacted by vibratory rollers. The aspects of mix design and construction of these RCC dam have been dealt in Chapter 4. For further details reference may be made to USBR Publication “Roller Compacted Concrete – Design and Construction Consideration for Hydraulic Structures”.

## 14.3 EMBANKMENT DAMS

### 14.3.1 *General*

Embankment dams are made of locally available natural material. These are homogeneous type of small height made of clayey soil with drainage arrangement at downstream toe and large height embankment dam made of a zoned section. Large height dams have been possible with advances in soil and rock-mechanics as well as development in earth moving equipment. The construction of a safe and economical embankment dam depends on optimal use of local material, selection of proper type of earth moving equipment and adequate supervision, quality control and testing during construction. These aspects are briefly described in following paras.

### 14.3.2 *Materials*

Almost all types of soils and rocks can be used for construction of embankment dams. The river deposits and terrace deposits in the vicinity of dam site should be investigated for geotechnical characteristics. The borrow areas of the suitable materials should be identified. The available volume of material in borrow areas should be about 1.5 to 2 times the volume required for the dam fill. Before assessing the requirement of materials from borrow areas and quarries, the suitability of the excavations from dam foundation and other associated structures such as diversion works, spillway etc. should be investigated for their use in dam fill. The quantity of usable excavated material should be accounted for in computing the material required from borrow areas.

The materials required for zoned embankment are as below:

- Impervious soil (clay) for core.
- Filter material
- Pervious material for shell
- Rip-rap for slope protection.

Each of the above type of materials has to meet characteristic functional requirement. The borrow areas and quarries for each type of material should be tested for the required characteristics and properties and if the naturally available material does not satisfy the requirement, processing of material has to be worked out and carried out at construction site. The functional requirement of each type of material is briefly given below.

**Core material:** The core acts as barrier to seepage through the body of dam. It should have low permeability (less than  $10^{-5}$  cm/sec). It should have adequate moisture content (around optimum moisture content). The plasticity index should range between 15 and 20. Highly plastic clay should be avoided because this may crack under high pressure. The clay content in the selected material/soil for core should be in the range of 15 to 20%.

**Filter:** Filter layers are provided between clay core and the shell to prevent movement of the fines of core material with the pressure of seepage flow. It should also provide free passage of clean seepage water. It is made of well graded granular material comprising sand and gravel. The grain size distribution of filter material should satisfy the filter criterion. The permeability of filter material should be around  $10^{-1}$  cm/sec.

**Shell material:** The purpose of shell material is to provide stability to the dam. Any type of pervious material such as mix of sand, shingle, gravel, boulder, crushed rock can be used as shell material. The gradation should be such that the permeability is greater than  $10^{-3}$  cm/sec. The fines (below 4.75 mm) should be less than 5%. The quantity required for shell depends on slopes required from stability consideration which depend on the shear parameters of shell material.

**Rip-rap:** It is commonly provided on the upstream slope in sufficient thickness which should be adequate to protect the slope from wave action. The size and thickness of rip-rap depend on the wave height which is a function of reservoir fetch and the wind velocity. It may be hand placed or dumped type. It comprises large size rock pieces. The rock should be of good quality to withstand weathering due to wetting and drying.

### ***14.3.3 Activities Involved in Construction of an Embankment Dam***

The construction of an embankment dam generally requires following activities/processes:

- River diversion arrangement
- Identification of dam borrow areas and quarries
- Stripping of foundation and borrow areas
- Foundation treatment
- Excavation of construction material in borrow areas and quarries
- Transportation of construction material to dam site or stock piles if processing is required
- Crushing, screening, washing and processing of material, if required. In this case transportation from stock piles to dam site will be required
- Laying, spreading and compacting the material at the dam fill site
- Quality control

Diversion of river arrangement and foundation treatment for embankment dams have been dealt in separate chapters. All vegetal growth and loose material from dam foundation is removed till a firm and compact surface is reached which can support the weight of dam fill. The other activities of dam construction are briefly described below.

### 14.3.3.1 Borrow Areas

The borrow areas and quarries for the type of material in adequate quantities should be identified. The material available in borrow area should be tested for the properties required in design. The distance of borrow areas and quarries from dam site should be worked out. The depth of borrow pits should also be estimated. These data are required for the selection of type of earth moving equipment which are necessarily to be used in the construction of a major embankment dam.

### 14.3.3.2 Main Activities in Construction

The main activities in construction of high embankment dam requiring earth moving equipment are the following:

- Excavation in borrow areas
- Transportation or hauling the excavated material to dam fill area
- Spreading and compaction

Each of the above activities requires different kind of equipment; equipments generally used are described in following paras.

### 14.3.3.3 Earth Moving Equipment

The equipment required for excavation is called excavator. For transportation of borrowed material, the equipments used are called hauling equipment and sometimes belt conveyors are used. For compaction the equipment used is called compactor.

#### (a) Excavators

Excavators primarily excavate earth and load it in the hauling equipment or belt conveyor. They can excavate all kind of earth and soil except solid rock. Rock will need proper loosening before it can be excavated. They are crawler mounted, rubber-tyred wheel mounted and truck mounted and are used as per site requirement. Various types of excavators are as below:

1. Shovel: It is used for digging at or below operating level and dumping the load at or above the operation levels, to spoil banks or loading into the hauling equipment standing at or below the operating level. The boom of shovel is small, so operational area is small.
2. The hoe: It is generally used for digging trenches and foundations below operating level and discharging loads in the manner similar to shovel. The boom of the hoe is not interchangeable with other attachments.
3. The dragline: These are used to excavate earth and load it into a hauling unit just as a shovel. It has a crane boom which is longer than the boom of a shovel. The

shovel boom and bucket can be replaced with dragline boom. The advantage of the dragline is that usually it does not have to go into the pit. It may operate from the ground and the hauling unit also can be loaded while standing on the ground. The depth of cut depends on type of material. However, the output of dragline is less (about 70 to 80% of the shovel). Most draglines can handle more than one size bucket depending on length of boom.

4. The clam shell: These are primarily used to handle loose material such as sand, gravel etc. They are especially suited to vertically lifting the material from one location to another. They can discharge in hauling unit also. These can lift material from under-water condition. These can be interchanged into dragline with the same boom as these also have crane boom.

All the excavators are crawler mounted and have bucket capacities ranging from 1/4 to 3 cum. Small capacity equipments are also available as mounted on rubber tyre wheels. The large boom (in excess of 70 m) and large bucket (7 cum or more) units are called walking draglines.

The above excavators, their capacities and other specifications are given in Table 14.1.

#### (b) Hauling equipment


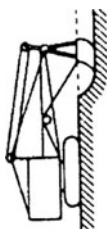
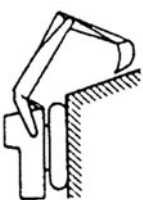
These are of two types: (i) excavation and hauling combined and (ii) hauling equipment only. The second type hauling equipments need loaders.

##### (i) *Excavation and hauling combined*

1. **Tractor Mounted:** The basic unit is tractor whose primary purpose is to pull or push the load. These are both crawler type and wheel type. These serve a number of purposes such as prime mover for pulling or pushing loads, and can be mounted with bulldozer blades and front-end bucket loaders for hauling each. The crawler tractor mounted with bull dozer blades can cut and doze the earth. The operation range is about 60 m one way hauling. The tractors, when fitted with front end loaders, can move the load within a range of 30 m. The bucket capacity ranges from ½ to 3 cum.
2. Scrapers: These are important hauling equipment. These can load, haul and discharge material. These are not dependent on loading and excavating equipment. These are able to discharge the material in uniformly thick layers and do not require spreading operation at the fill. These can work efficiently with earth and soft rock. The hard strata should be first ripped adequately before engaging scrapers. These scrap only a thin slice of earth, a few cms thick. The loading distance is from 30 to 100 m. These are available with capacities upto 30 cum or more.



Scrapers are of two types: (i) towed scrapers and (ii) motorized scrapers. The towed scrapers are pulled by tractors. The crawler tractor pulled scrapers are economical for relatively short haul distances. For longer haul distances wheel type tractors are economical. The wheel type tractors hauling the scraper cannot deliver great tractive effort in loading a scraper, so a crawler type tractor can be

Table 14.1 Earth excavating and lifting equipment

Types of equipments	Feature	Bucket Struck capacity								
	<i>Bucket Size</i>	YD^3	0.05	0.75	1.00	1.50	2.00	3.00		
		M^3	0.38	0.57	0.76	1.14	1.53	2.30		
	<i>Engine HP</i>		50	75	100	130	160	200		
1. Face shovel	Maximum cutting height	M	7.0	7.0	7.0	7.5	8.5	10.0		
	Ideal output/hour	LM^3	60	90	120	180	220	300		
	(Loose, easy dig)									
	Power shovel									
2. Backhoe	Maximum Digging Depth	M	5.0	6.5	7.5	8.0	9.0	9.0		
	Ideal output/hour	LM^3	48	72	96	144	176	240		
	(Loose, easy dig)									
 	Excavator									

(continued)

Table 14.1 (continued)

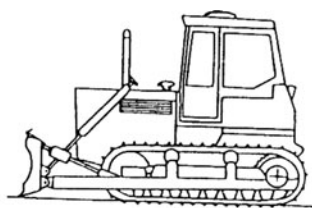
Types of equipments	Feature	Bucket Struck capacity									
		M	21	21	21.27	21.27	21.27	21.27	27.30	30.33	
3. Dragline 	Maximum Boom length	M	21	21	21.27	21.27	21.27	21.27	27.30	30.33	
	Ideal output/hour (Loose, easy dig)	LM^3	45	67	78	135	165	225			
4. Clamshell 	Maximum Boom length	M	21	21.27	21.27	21.27	27.30	30.33			
	Ideal output/hour (Loose, easy dig)	LM^3	24	36	48	72	89	120			

engaged to push the scraper during loading. On the job one crawler tractor can help a number of scrapers in loading operation. The wheel type tractors hauling the scraper can move at a speed exceeding 40 km/hr, depending on the condition of haul roads.

The motorized scrapers do not need tractors to pull. These are self-loading and hauling equipment. These are single engine and twin engine type. These are available in sizes ranging from 4 to 25 cum capacity and these have an operating range upto 3000 m one way lead.

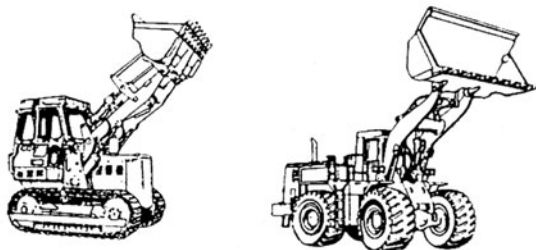
The above described hauling equipments in tractors and scrapers are shown in Fig. 14.7.

#### 1. Bull dozers

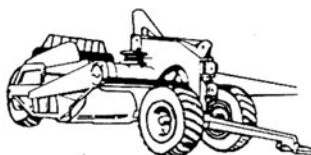


Tracked dozer

#### 2. Front-End Loaders



#### 3. Towed Scrapers



Towed

#### 4. Motorised Scrapers



Single engine conventional



Twin engine conventional



Elevating

Fig. 14.7 Different types of excavating and hauling equipments.

### (ii) *Hauling equipment*

The hauling equipments are wagons, trucks and dumpers. These can handle earth, aggregate, rock etc. These need loading units. The loading is accomplished, when the hauling unit is stationery, by means of shovels, draglines, clam shell, front end loaders and belt conveyors. These hauling units can operate in a range of a few hundred metres to several kilometres.

The wagons which are two wheel type need power unit to pull. Commonly four-wheel truck type tractors are used to pull the wagons. The wagons are generally bottom or side dump type. These can move at a top speed of 50 km/hr and the capacity range is from 4 to 40 cum.

Trucks and dumpers are also used as hauling units. These are self-propelled and do not need pulling unit. These can move on high speed on good haul roads where surface is sufficiently firm and smooth and grades are not excessive. These are bottom, rear and side dump type. Their capacities range from 4 to 35 cum.

Another type of hauling equipment is front end dumpers with capacities of 2 to 6 cum. These are used for shorter hauling distance of 150 to 200 m.

The tractor pulled bottom dump wagon and rear dump truck are shown in Fig. 14.8.

### (c) **Compactors**

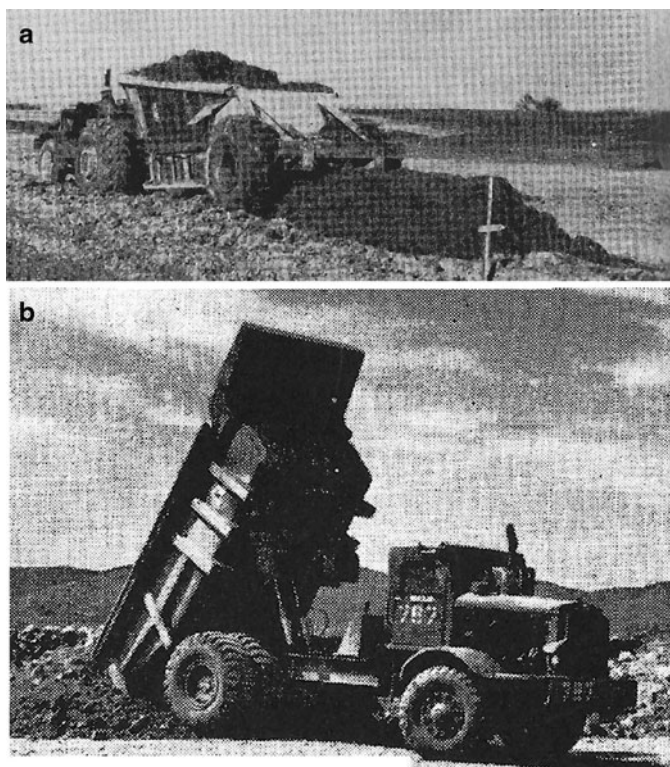
The scrapers discharge the load on the fill in thin layers of uniform thickness and no spreading operation is required. Other hauling units discharge load on a fill in heaps. This has to be spread in uniformly thick layers before compaction. Spreading is done with the use of dozers.

The equipments used for compaction are known as compactors. Various types of compactors are used. These are sheep foot roller, vibratory rollers, steel drum roller and pneumatic roller type. These are shown in Fig. 14.9. The sheep foot rollers are specifically used for compaction of clayey soil. Others are used to compact sand and rockfill.

#### **14.3.3.4 Guidelines for Selection of Earth Moving Equipment**

According to job requirement, the choice of equipment, type and size has to be made to complete the job within the scheduled time limit. Some of the factors affecting the selection of equipment are:

- Plant and machines already available, if any; the additional machines to be procured should be compatible to the existing fleet of equipment.
- Limitations on transport facilities between the suppliers plant and the work site.
- Ease of operation and maintenance with reference to availability of trained persons for the job.
- Cost of production: The operation cost for each unit of production of the machine should be minimum.



**Fig. 14.8** Hauling equipments.

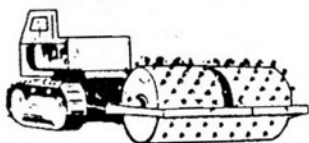
The total operating cost, in which depreciation is a major item, decreases as the number of hours the machine is put on productive work increases.

In India, due to weather limitation, there are eight working months and each month has about 21 working days. If work is done in one shift of 8 hours, we get about 1350 hours of work in a year. The useful operating life of earth moving equipment is generally accepted as below:

- |       |  |              |
|-------|--|--------------|
| (i)   | Tractors and tractor mounted or tractor drawn machines | 10,000 hours |
| (ii)  | Shovels  |              |
|       | 3/8 to 3/4 cum   | 10,000 hours |
|       | 1 to 1½ cum  | 12,000 hours |
|       | 2 cum and above  | 16,000 hours |
| (iii) | Draglines and clam shells                              |              |
|       | 3/8 to 3/4 cum   | 10,000 hours |
|       | 1 to 1½ cum  | 18,000 hours |
|       | 2 cum and above  | 24,000 hours |

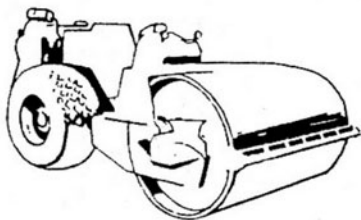
Generally a major dam project will have a completion schedule of 4 to 5 years. Hence to use a machine to its full useful life on a project, it should work atleast 2000

### 1. Sheep Foot Rollers



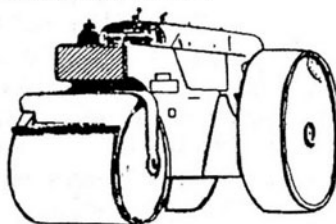
Towed Sheep's Foot roller

### 2. Vibratory Rollers



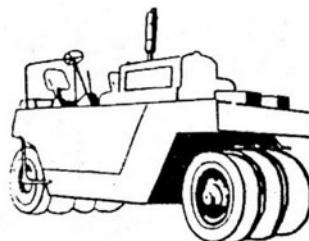
Self-propelled vibrating roller

### 3. Steel Drum Rollers



Smooth, steel wheel roller.

### 4. Multi-Tyred Pneumatic Rollers



Small, multi-tyred pneumatic roller.

**Fig. 14.9** Different types of compactors.

to 2500 hrs in a year. Thus, machine must work atleast in two shifts in a day if production cost is to be kept low.

The rough guidelines for selecting hauling equipment based on haulage distance are as given in Table 14.2.

The output of a hauling equipment depends on the condition of haul roads. The output on fairly good haul roads is taken as 85% of the output on good haul roads and output on poor haul roads is taken as 75%.

Considering the operating efficiency of the equipment, the output is commonly estimated on an hour of 45 minutes. The bank measure of the load is taken as a fraction of the hauling capacity of equipment. If the capacity of hauling equipment is 12 cum, the bank measure is taken as a low figure of 9 cum for estimation purpose.

The number of haulage vehicle required per unit of loading equipment can be worked out as

$$\text{Haulage vehicle required} = 1 + \frac{\text{Cycle time per trip of vehicle}}{\text{Loading time of vehicle}}$$

The cycle time per trip depends on distance travelled divided by speed. For average haul roads the speed can be taken as 10 to 15 km/hr.

The above is one approach for matching two types of earth moving equipments to work as a team. Another approach is to match the output of the two types of

**Table 14.2** Selecting hauling equipment for a hauling distance

Type of equipment	Range of haulage distance (m)
Front-end loaders (track)	Upto 80 m
Front-end loaders (wheel)	Upto 200
Bull dozers	Upto 80
Towed scrapers	100 – 300
Elevated self-loading scrapers	100 – 1000
Dozer pushed single engine scrapers	500 – 3000
Double engine scrapers	2000 – 3000
Trippers and dump trucks	800 and above

earthmoving equipments working as a team. The experience is that the average operating efficiency of the combination is lower than the individual operating efficiency of the two types of equipments which make the combination, because the coordination between the two type of machines is not always possible for long time and there is some waiting or delay involved during the working hours.

#### 14.3.3.5 Planning Earth Moving Equipment through Systems Approach

In the recent past use of waiting line theory has been suggested for matching two types of equipments. The analysis has revealed that the number of units of the two type of equipments work out the same by both waiting line theory and the method of equating the output of the two type of equipments working as a team. But the output of the combination is found lower by using waiting line theory and it appears logical because of delays in coordinated working of two types of equipment for long period. Thus for realistic planning waiting line theory should be used for estimation of output of the combination of two types of equipment working as a team.

Similarly the use of transportation model for allocating combinations of earth-moving equipments to different work sites of the project has been suggested for realistic planning.

#### 14.3.4 Construction of Zoned Embankment

The zoned dam section has a clay core, shell zones and filter between core and shell. The upstream slope is protected by rip-rap. The construction requirements of each component are briefly given below.

#### 14.3.4.1 Clay Core

Clay for the construction of core is excavated from the borrow area and hauled directly to the placement site. If blending of clay with the pervious material is required then it is brought from borrow area and placed in stalks comprising alternate layers of clay and pervious material at a site near to dam. The layers are mixed by cutting and overturning by the front end loader and then hauled to placement site. At placement site the material is spread by dozers in uniform layers of specified thickness which is generally 30 cm. Then the layers are compacted by sheep foot rollers. Compaction is achieved through specified number of passes which are predetermined through experiments at the test section with the type and size of roller to be used at the job. These are generally 6 to 12 passes. The density should be tested and if it is less than desired, more passes of the roller should be made. Thereafter another lift shall be placed over the previous one.

Moisture content of clay is an important factor in achieving compaction of desired density. The density is optimum at the optimum moisture content which is determined by laboratory tests. Compaction at site should normally achieve about 95% of optimum density. The moisture content of clay at the time of placement should be  $\pm 2\%$  of optimum moisture content. If it is less, moisture may be added in the borrow area either by wetting of the borrow area or spraying water on the excavated clay to increase moisture content. If the moisture content is more, the warming of the soil should be carried out by using blowers, drying drums or spreading the clay in thin layers. However, reducing moisture content is an expensive and difficult process and is rarely adopted. Therefore, it is advisable not to select a borrow area with high moisture content.

Rainy season makes work impossible in clay. Hence work schedule should be made in such a way that no work is done in the core area in rainy season.

The rate of construction of zoned embankment is dependent on the rate of raising core. It is recommended that both core and shell should be constructed simultaneously to make a homogeneous mass and the excessive height difference between the two should be avoided.

#### 14.3.4.2 Filter

The filter between clay core and shell should be of a specified gradation and permeability. The filter material is commonly obtained after processing of either river bed deposits or quarrying the rock slopes. For producing filter material from the quarried rock or river deposits a crushing and screening plant is set up. The output of this processing plant is stock piled separately for each size. Then these are mixed in required proportion to obtain desired gradation. The samples of the mix should be tested in the laboratory for gradation and permeability.

The mix should be hauled to the placement site in trippers or dump trucks. The filters are usually laid in layers 0.3 to 0.6 m in thickness and are compacted with

smooth vibratory roller. Usually four passes are made. The required number of passes should normally be determined by the tests on the test section.

#### **14.3.4.3 Shell Material**

The shell material can be any type of quarried rock or sand-gravel-boulder mix from the river deposits. It should be well graded with fines (less than 4.75 mm) less than five percent and maximum size 70 to 80% of the lift thickness. The lift thickness is commonly around 1.0 m. The material is brought directly to the placement site, then it is spread in layers and compacted. Water may be added using sprinklers if required before compaction. Water can also be added on route to the placement site in the dump trucks. Compaction is done through vibratory rollers in 4 to 6 passes. Density and permeability should be tested before the second lift is placed. Test fills are useful to establish the procedure of construction.

#### **14.3.4.4 Rip-rap**

Generally quarried rock of required size and gradation is placed/dumped on upstream face to provide required thickness for protecting the slope from wave action. The finished surface should be dense but rough enough to resist wave action. Generally a filter layer is placed between the shell and the riprap.

### ***14.3.5 Quality Control***

Quality control at site of an embankment dam is very essential for a safe and economical construction. Quality control is required both at the borrow area and at the fill placement site. It will ensure that the material of required specification is being hauled to the fill placement site and it is being placed at proper location in specified layers with adequate compaction following the procedure evolved at the test fill before starting the construction.

Material within each zone of fill have specified gradation, density, strength and permeability and it has to be ensured during fill placement.

It is desirable that dumping, spreading and compacting operations in each zone should be such that the material when compacted is sufficiently blended to achieve a homogeneous fill. Successive loads of material are deposited parallel to dam axis at proper spacing to ensure uniform spreading. The differential elevation at the contact between core and adjacent zone should desirably not exceed 1.0 m. It is desirable to keep core higher than adjacent zone.

For core it is important to control moisture content and density. The moisture content of core material should be  $\pm 2$  percent of the optimum moisture content. At least 80% of the samples tested should be within this limit. If there is variation more

than this limit, then necessary changes should be made. The density should be controlled by compaction efforts.

The quick field tests for density and moisture content as per IS 2720 should be carried out for which a laboratory should be established at work site.

For pervious material i.e. filter and shell, permeability and gradation are the properties of importance and should be controlled during placement. Compaction controls the relative density of the compact mass. The field density test should be carried out by water replacement method following IS 2720 (Part 33). Field permeability test should also be carried out at suitable locations according to IS 5529 (Part I). The average relative density for pervious zone should not be less than 70 percent.

The recommended frequency for tests (IS 14690) is as below:

- (i) Impervious materials: Gradation and moisture content before compaction and field density after compaction should be tested once for every 2000 cum of fill or in each shift whichever is earlier. Atleast one in-situ density test should be carried out for each compacted layer. Permeability and shear tests should be carried out for every 15000 cum of fill or atleast once per week.
- (ii) Pervious material: Initially one gradation and field density test should be carried out for every 1000 cum of shell material and if it proves satisfactory then one test for every 10,000 cum of shell material or one test for each shift should be carried out. For filter one test for gradation and field density should be carried out for every 1000 cum of filter material and preferably for every 500 cum.

### **Example of Quality Control**

As an example the quality control exercised at Tehri dam is given below:

#### **1. Clay core material**

The clay was blended with pervious material having a maximum size of 200 mm. Clay and pervious material in required proportion was laid in alternate layers in stockpiles (5 to 6 layers of each type of material) and water was added in each stockpile to achieve required moisture content (near to OMC). After mixing and before placement, gradation and moisture content was tested with a frequency of 1 in 1500 cum. Layer thickness at placement site was 40 cm compacted to 35 cm. Dry density was tested in each layer after compaction. In general the required density of 1.9 T/cum was normally achieved. The permeability was also tested on compacted clay layers and it was achieved of the order of  $10^{-6}$  cm/sec.

#### **2. Shell material**

The river bed material was directly placed in the shell zone. The gradation and moisture extent was checked in the borrow area. The frequency was 1 in 10000 cum. The moisture content test was used to calculate quantity of water to be added to make it 8–10% for proper compaction.

The material was placed in 80 cm thick layers to be compacted to 70 cm after eight passes of 15T vibrating rollers. Dry density tests by ring and water replacement

method were conducted after compaction for each layer. The density achieved was around  $2.36 \text{ T/m}^3$ .

### 3. Fine and coarse filler

The filler material of required gradation was produced by mixing material of different sizes obtained from crushing and screening plant. Before placement the fill material was tested for gradation and moisture content. The filler was laid in 40 cm thick layers to be compacted to 35 cm at a moisture content of 5% which was achieved by adding water when required. Dry density was tested after compaction in each layer. Relative density of 80% was achieved.

### 4. Rip-rap material

It is blasted rock fragments of required gradation with maximum size of 1200 mm. The gradation was tested before placement with a frequency of 1 in 1000 cum. It was placed in layers of 1.5 m thickness (loose) compacted with watering @ 500 litres/cum by 15T vibratory rollers to achieve a density of  $1.85 \text{ T/cum}$ . After compaction density was tested with a frequency of 1 in 1,00,000 cum.

## ***14.3.6 Examples of Equipment Used on Earth and Rockfill Dams***

Two examples of major earth and rockfill dams are given below to illustrate the site conditions and the criteria followed to adopt a specific mode for hauling the material and the type of equipment chosen for their construction.

### **14.3.6.1 Salal Dam**

It is a hydro-electric project ( $3 \times 115 \text{ MW}$ ) on river Chenab in J & K State (India). An earth and rockfill dam 115 m high and 615 m long is the main component of the project. It required placement of 7.2 M cum of fill material comprising:

Rockfill (shell)	5.36 Mcum
Clay (core)	1.16 Mcum
Filter	0.68 Mcum

Maximum quantity to be executed in a working season was 2.45 Mcum. Considering 200 working days in a season the placement quantity was worked out as 12000 cum/day. Besides placement of fill, other major items of work involved excavation of 1.2 Mcum and drilling and grouting of 96000 m.

It was decided to use shovel and dumpers for excavation and hauling of fill material as the average distance of borrow area was 3 km. The equipment was planned on following assumptions:

The hauling speed of equipment

Loaded hauling 10 km/hr

Empty hauling 12 km/hr.

Average density of material	Loose ( $t/m^3$ )	Compacted ( $t/m^3$ )
Pervious material	2.0	2.32
Impervious material	1.65	2.2
Partly blended rock	1.6	2.0

Standby capacity (as per CWC recommendations)

Single shift working 10%

Two-shift working 20%

Three-shift working 30%

Three-shift working was proposed; hence standby capacity was made 30%. Following type of equipment were therefore used at the project:

- 4 cu yard hydraulic shovels with matching dumpers for clay and rockfill.
- 2 cu yard hydraulic shovels with matching dumpers for clay and filter.
- Hydraulic shovels for foundation excavation.
- Tyred dozers for spreading filter and clay.
- Crawler tractors for spreading rockfill.
- Crawler tractors for ripping rock quarry.
- Vibratory rollers for compacting rockfill and filter.
- Sheep foot roller for compacting clay
- Tankers for adding moisture.
- Motor graders and water tankers for maintaining haul roads.

#### 14.3.6.2 Tehri Dam

It is one of the highest earth and rockfill dam (260.5 m high) completed a decade ago across river Bhagirathi in Uttarakhand (India). It required total fill placement of 27.8 Mcum. The hauling equipment adopted on several other similar projects in other countries were studied for selection of mode of hauling and type of equipment for the project. The details of other projects are briefly summarized in Table 14.3.

At Tehri project the shell material (20 Mcum) was proposed to be taken directly from river deposits located at about 2 km from placement site. The clay (3.5 Mcum) was available at a distance of 6 km. It was to be stock piled in between the borrow area and dam site for blending with pervious material ( $<200$  mm) and was to be hauled to placement site after mixing. Filter material (1.5 Mcum) was planned to be manufactured by crushing and screening plant to be installed at about 3 km from dam site. It was to be hauled to placement site after mixing materials of different sizes in desired proportion. Rip-rap (2.8 Mcum) was to be obtained from the rock quarry about 25 km from dam. It was proposed to be stock piled near dam site to be used when required.

**Table 14.3** Construction method and equipment used in other projects

Name of project	Country	Height of dam (m)	Fill volume M.cum	Method adopted	Remarks
Nurek	Tajikstan	300	45	Conventional	Average haul distance 3 km. 60MT dump trucks circulatory road net-work.
Chicosen	Mexico	264	15	Belt conveyor and conventional	Shovel and trucks used for rockfill and rip-rap. Alluvial and clayey deposits excavated and hauled by belt conveyors
Oroville	USA	236	60	Belt conveyor and rail/road	Borrow areas about 10 to 19 kms from dam.
Tarbela	Pakistan	143	122	Belt conveyor and conventional	Belt conveyor for about 2/3 <sup>rd</sup> quantity from borrow areas at a distance of about 7 km. For boulder and gravel material conventional shovel and bottom dumpers used.
Chivor	Colombia	237	11.0	Conventional	In borrow area and rock quarries were in the vicinity of dam. Valley slopes were steep and length of dam on top was 310 m. Tunnels in the abutment were used to haul the material. 30 T bottom dumpers were used.
Ramganga	India	125	11.0	Conventional	Borrow areas were within 3 kms. Shovels and clam shell used as excavators and scrapers and dumpers as hauling units.

Conventional means shovels and dumpers.

In view of the site requirement of handling the fill material, the hauling distance, use of the already available equipment and the cost and time factor involved in procurement and installing the belt conveyor system as it was not indigenously available, it was decided to adopt conventional system of shovel and dumpers.

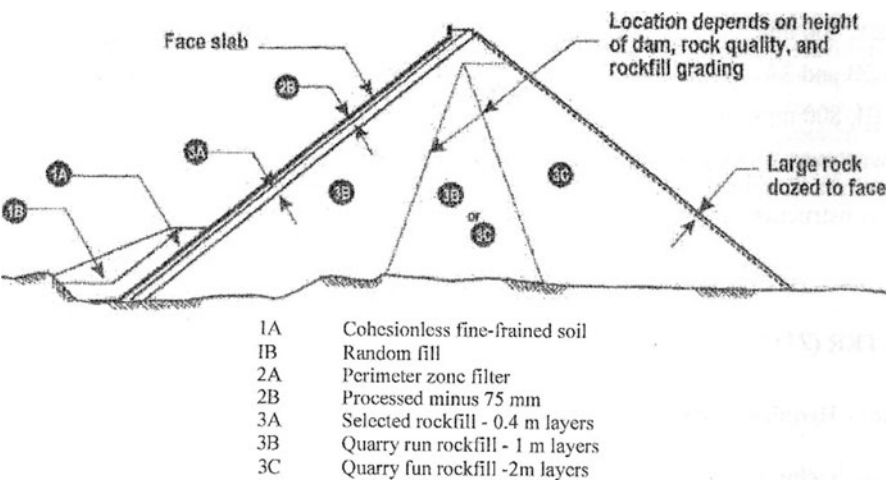
The dam filling was proposed to be completed in five working seasons with three-shift working. Hence standby equipment was considered as 30%. The equipment planned and deployed at site were as shown in Table 14.4.

### 14.3.7 Concrete Faced Rockfill Dam (CFRD)

The main components of CFRD are the rockfill and the face slab which is of reinforced cement concrete. The standard type zoning of a CFRD is shown in

**Table 14.4** Equipment used at Tehri Dam

Sl. No.	Name of equipment	Total nos.
1.	Russian shovel	6
2.	Hydraulic excavators Pe-650	3
3.	Hydraulic excavators UH-189	6
4.	Hydraulic excavators CK-90	1
5.	Loader 4 cum	4
6.	Loader 2 cum	7
7.	Dozer D-355	4
8.	Dozer D-155	20
9.	Dozer D-65/80	10
10.	Dumper 25T	150
11.	Tripper/dumper 12T	100
12.	Water tanker 25 KL	15
13.	Water tanker 10 KL	10
14.	Vibratory compactor 15 <sup>T</sup>	18
15.	Vibratory compactor 10 <sup>T</sup>	18
16.	Pneumatic tamper	50
17.	Aggregate processing plant 250 <sup>T</sup>	1
18.	Batching plant 120 cum/hr (for concreting spillway etc.)	1



**Fig. 14.10** Typical section of CFRD.

Fig. 14.10. The construction of the rockfill is similar to the construction of the shell zone of the earth and rockfill dam described in the previous section. The construction of reinforced concrete face requires all the specifications and methodology of manufacturing and placing of concrete to be followed as for a concrete dam. The extra precautions are to be taken in placing concrete slab so that cracks may not

**Table 14.5** Gradation of rockfill zone below face slab

Size in mm	Percent passing by weight
76.2	100
38.1	70-100
19.1	55-80
4.76	35-60
1.19	18-40
0.297	6-18
0.074	0-7 (non-cohesive)

develop. As the slab is the seepage barrier of the dam, the cracks if developed may cause seepage which may not be acceptable.

### Cracks and their Control

In many CFRD dams cracks have been observed. These are classified as type A, B and C. Type A cracks are shrinkage cracks which are identified as relatively horizontal and of small width (a fraction of a millimetre). These can be minimized and controlled by proper concrete placement and curing during construction. Type B cracks are because of settlement and movement of rockfill. These cracks are generally observed in the middle one-third of the dam height. These are evenly spaced at intervals of 0.5 to 1 m and are of the width less than a millimetre. These normally tend to close on filling the reservoir. It is experienced that placing reinforcement near the top face (with a cover of 150 mm or so) may be helpful in minimizing type B cracks. These can also be treated by placing fluid cement over the cracks.

Type C cracks are structural cracks which are caused due to differential movement in the embankment due to embankment construction or the effect of adjacent material of different deformation characteristics. These cracks could occur due to cracks in the transition fill beneath the slab. Proper selection of gradation, placement and compaction of face slab supporting zone material (zone 2B shown in Fig. 14.10) are important in limiting the occurrence of these cracks. If cracks appear in face slab supporting zone before laying the face slab, they should be repaired. Cement grout mixes can successfully fill the cracks. The transition zone material which supports the face slab is processed minus 75 mm material. It should be well graded. The gradation as per modified ICOLD Bulletin 70 (revised ICOLD number 141) is as shown in Table 14.5.

If cohesive soil is more, the cracks may develop. The permeability should be around  $10^{-2}$  cm/sec. This face slab supporting zone should be 2.4 m wide and should be well compacted, by atleast four passes of 10T vibratory rollers.

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# Chapter 15

## Dams and the Environmental Issues



### 15.1 GENERAL

There will not be any life on earth without fresh water. It is received on earth annually, through precipitation in the form of snow or rain. A part of it is transformed in the form of surface water in the rivers and as groundwater under the earth surface. The history of managing of this resource for irrigation and domestic needs is as old as the history of mankind. Reservoir construction activity all over the world is centuries old. In India this activity of dam construction is also centuries old specially in south. The topography, climate and rainfall has made southern part of India more suitable for tank irrigation and north Indian plains for well irrigation. Rao has described the ancient earth dams of India constructed from 800 to 1600 AD. Till the beginning of 20<sup>th</sup> century, dams were built in many countries but their height was not more than 30 m. The rapid growth in technology and mechanization during 20<sup>th</sup> century made it possible to construct high dams of different types with large storage capacity in many parts of the world. Some of such dams are listed below.

Name of dam	Height (m)	Type	Storage (Mham)
Kariba (Zimbabwe-Zambia)	130	Arch	16.1
High Aswan (Egypt)	97	Embankment	15.6
Akosombo (Ghana)	112.5	Rockfill	14.8
Hoover (Boulder USA)	222	Arch gravity	3.86
Mica (Canada)	244	Rockfill (clay core)	2.47
Oroville (USA)	235	Rockfill (clay core)	0.43
Nurek (CIS)	300	Rockfill (clay core)	1.05
Grand Dixence (Switzerland)	284	Concrete gravity	0.04

In India the construction of high dams started after independence in 1947. Through big and medium storage projects, India would create a live storage of about 22 Mham (on completion of under construction projects). The few major projects constructed in India are listed below:

Name of dam	Height (m)	Type	Storage (Mham)
Bhakra	226	Concrete gravity	0.987
Nagarjun Sagar	125	Stone masonry gravity	1.13
Rihand	93	Concrete gravity	1.05
Ramganga	125.5	Embankment	0.22
Tehri	260.5	Rockfill (clay core)	0.35

It can be seen that in India biggest reservoir is of 1.1 M ham capacity which is insignificant as compared to the reservoirs of other countries.

After constructing major reservoirs, various countries specially the developed ones started realizing the adverse effects of the reservoir. Their concern was reflected in the U.N. Conference on Environment at Stockholm in 1972, which focused attention to environment problems with increasing concern over the adverse impacts of reservoir and thereafter several U.N. organizations engaged themselves in propagating the awareness about environmental impacts. The international funding agencies also focused attention to environmental issues of the projects. There is no denying to the fact that dam construction has environmental repercussions of physical, chemical and biological nature alongwith economic benefits, but adverse impacts are found project specific. The over-emphasis on adverse effects of dam construction has seriously hampered the water resource development through dams in India. Since 1980 only two storage projects, Tehri dam (in Uttarakhand) and Sardar Sarovar (on river Narbada in Gujarat) which were already under construction since long have been completed. Other projects constructed in this period are diversion dams of small height for hydro power generation.

## 15.2 ENVIRONMENTAL IMPACTS OF RESERVOIRS

The issue of environmental impact of reservoir has been presented and discussed at International level in various Conferences, Symposia and Workshops (UN Water Conference 1977, UN Environmental Programme, 1982, UN World Commission on Env. and Development, 1987, UN Conference on Env. and Development, 1992 ICOLD 1973. International Seminar on Environment Impacts of Water Resources Projects, 1985 and Indo-Soviet Workshop 1985) and the concerns regarding consequences of water resource development through dams on environment have been strongly expressed. "The State of India's Environment", 1982, emphasizes on adverse effects of Indian dams.

The adverse effects of dam on environment can be grouped under physical, biological, chemical, social and economic effects as below:

- (i) Physical effects include changes in river morphology, climatic changes, induced seismicity, siltation of reservoir, soil fertility, etc.

- (ii) Biological effects include submergence of forest and threat to extinction of rare species of flora and fauna which may affect ecological balance, impact on migratory fish, and increase in water-borne diseases.
- (iii) Chemical effects include change in water quality due to storage and decay of organic matter in reservoir bed producing methane and hydrogen sulphide resulting in depletion of dissolved oxygen (DO).
- (iv) Social effects include uprooting of the population and also loss of historical and religious structures in the submergence.
- (v) Economic effects include loss of forest, mineral deposits and agricultural land. It also includes economic loss in case of a dam failure.

These adverse effects may be divided location-wise either in the submergence of dam or in the downstream of the dam. This division is as below.

**(i) Effects in Submerged Area of Reservoir**

These include displacement of people, submergence of forest area, agriculture land, and mineral deposits, historical and religious structures, loss of flora and fauna, climate change; reservoir siltation; induced seismicity, deterioration of water quality etc.

**(ii) Effects in the Downstream**

These include the effect on migratory fish, the effect of release of poor quality water (Low temperature and decreased DO) on the aquatic life as well as on the people living in the downstream, effect on river morphology due to reduced flow with less sediment, effect of practically no flow during non-monsoon on the aquatic life and people living in the downstream.

The above are the possible adverse effects of a dam construction but all of these necessarily do not occur in each case.

The benefits of a dam project are mainly economic in terms of agriculture production, power generation, flood control, industrial development and social and economic upliftment of the people.

## 15.3 ENVIRONMENTAL IMPACT OF DAMS IN INDIA

A large number of major and medium reservoirs have been constructed after Independence. The adverse effects of these dams in the submerged area are briefly discussed below.

- (i) The dams in India have submerged forest land and displaced people, but much greater area (20 to 30 times of the submerged land) and large population (many times of the displaced persons) have been benefitted directly and indirectly by these projects. Similar is the finding of 'World Commission on Dams', which is as below.

[‘Dams have made an important and significant contribution to human development and the benefits derived from them have been considerable’.]

So far as the loss of forest is concerned, the data reveals that in last 50 years we have lost about 50 Mha of forest cover i.e. we are losing at a rate of 1 to 1.5 Mha of forest every year for many other reasons. In its comparison the forest lost by all the dams in 50 years is only 2 M ha. Hence dam projects are responsible only for about 4% of the total loss of forest. The loss of forest due to dams is compensated by afforestation of the areas more than that which is submerged, at the cost of the project. In case of Tehri Project, in lieu of 1600 ha of forest land which has been submerged in reservoir, compensatory afforestation has been proposed in the project in an area of 4500 ha of non-forest land in Jhansi and Lalitpur districts of Uttar Pradesh at the cost of the project. Equally important to note is the fact that against 1600 ha of land which is getting submerged the project on completion will provide irrigation benefits to 2.3 lac ha area.

- (ii) Loss of flora and fauna is another important issue in each project. If properly identified, the loss of flora and fauna can be prevented by shifting them in the same watershed in the vicinity of project because our reservoirs are not large enough to submerge one type of forest environment completely. The zoological and botanical parks can be established for the endangered species at the cost of project. Such parks are provided for Tehri project area. Wild life is protected at Dudhganga reservoir (Maharashtra). Similarly wild life sanctuary is proposed at Sardar Sarovar dams project (Gujarat). Priyar wild life sanctuary has been developed on the rim of Idukki reservoir. The lakes of the dams have a positive feature of providing water pool for the wild life. Ramganga reservoir is a water pool for the wild life of Corbet National Park. The reservoirs in many cases have also been used in developing new species of flora and fauna.
- (iii) Silting of reservoir is a natural phenomenon and is unavoidable. Rate of silting in the reservoirs in Himalayan rivers is more than the reservoirs in the south. The observations have shown that the loss of live storage each year is around 0.5 percent of the capacity. It ranges from 0.2% in Bhakra to 0.8% in Panchet Hill. The reservoir in India are expected to serve for more than the economic life of reservoirs. Measures are also being taken on all major projects to reduce the rate of silting in the reservoirs.
- (iv) Mineral deposits, monuments of historical and religious importance are generally protected from submergence. The carborandum mines were saved by reducing the height of Rihand dam (UP) and lime stone deposits are saved by putting bund around the quarries in the submergence of Bansagar lake (MP). In most of the projects the important monuments are shifted. Shifting and re-erecting Budha’s statue in Nagarjunsagar lake is an example.
- (v) Temperature variation along the depth of reservoirs has not been found significant in our reservoirs. The loss of dissolved oxygen (DO) has also not been very alarming. Chemical analysis of water of reservoirs has shown marked changes in quality in a few cases only. The studies should be carried

out in all cases so that required remedial measures could be adopted because water quality affects the life in the downstream.

- (vi) Another controversial issue is reservoir induced seismicity. It is being reported that Koyna earthquake of magnitude 6.7 on Richter scale in 1967 was the effect of construction of reservoir, but studies did not conclusively support it. The same is true for Aswan dam (earthquake of 1981). Reservoirs like Bhakra, Pong, Ramganga, Tarbela and Mangla being located in highly seismic region of Himalayas are being regularly monitored but the observations have not shown any significant influence on the seismicity of the areas. In a few cases in the world it has been noticed that the presence of reservoir has increased the frequency of small magnitude tremors (3 to 4 on Richter scale) in a small area of 25 to 30 sq. km around the dam. This effect is also generally observed in first few years of filling of reservoir.
- (vii) Most of our reservoirs are not found responsible for spreading diseases. However, it is reported that fluorosis is endemic in Nagarjunsagar dam area. Malaria has however been controlled in DVC dams area.
- (viii) Landslides are very common in Himalayas. Any landslide in the reservoir may reduce the storage capacity or cause overtopping. So valley slopes along the rim of reservoir should be checked for stability and slope stabilization measures should be adopted.
- (ix) Since submergence spread of our reservoirs is small, these may cause only some micro level climatic changes.

Adverse effects in the downstream of the dam are generally caused due to changes in flow pattern, sediment content and the quality of flow.

- (i) Natural river flow pattern is disturbed due to storage in reservoir. The flood peak may be reduced and non-monsoon flows may increase. The non-monsoon flow may reduce to nil in a small reach of river if the project is run of river type hydropower project. This affects the life style of people in the downstream, the ecology and pisciculture. To mitigate these adverse effects a minimum environmental flow depending on the downstream requirements should be released from the dam. This has been made mandatory for all projects existing under construction or in planning stage. The environmental releases are presently being provided equal to 10 to 15% of average non-monsoon flows of river.
- (ii) The reduction in the silt content during floods through large storages causes retrogression in the river bed resulting in change in bed levels and water levels. This may affect the foundation of existing structures located across and along the river. This aspect needs investigation and proper planning and implementation of the required measures before a reservoir is constructed.
- (iii) The change in flow pattern and silt content may cause change in river meanders, bank erosion and change in land use. This aspect also needs study through models in each case.
- (iv) The dam construction affects the migration of fish. In low dams the provision of fish ladder takes care of this problem. The loss of migratory species of fish in some rivers due to dam construction is also compensated by rearing other

species of fish in the reservoir. Reservoirs on Krishna and Cauvery rivers, Gobind Sagar, Ukai, Mahi-kadana and Matatila are a few examples where sustainable fish development has taken place.

- (v) The disaster management in the event of dam failure is an important issue of concern. It is being taken care of in the new storage projects through disaster management plans.

The issues which need special attention from environmental considerations are the following:

- a. Rehabilitation policy of displaced people.
- b. Minimum flows in the downstream from ecological and water quality considerations.
- c. Disaster Management Plan – planning and implementation.

## **15.4 PRESENT STATUS REGARDING ENVIRONMENTAL ISSUES OF DAMS**

There is world-over agreement regarding the adverse environmental impacts of reservoirs. But, there is also a concern of third world and developing countries that harnessing surface water through storage dams, for which time tested technology is available, is essential to improve the economy and life of poor people because all sectors of economy depend on water use. It is also well recognized that poverty is the worst polluter. Hence storage dams projects are a necessity.

As far as India is concerned, which is a vast country depending on agrarian economy, the construction of storage dams is essential to meet the irrigation requirements. Since about 90% rainfall occurs during three monsoon months and its distribution in this period is temporarily and spatially very uneven, the irrigation facilities are required for both Kharif and Rabi crops. Besides irrigation facilities these projects would also help in power generation, flood control etc.

According to an estimate by CWC, the average annual surface water flows in India are 186.9 Mham out of which only 69 Mham can be utilized if appropriate storages are created. Estimated groundwater potential is 43.2 Mham. It is also estimated that irrigated agriculture to meet 2050 food demands needs 100 Mham of water including 60 Mham through storages against existing storages of 17.4 Mham. The projects under construction have a storage capacity of 7.5 Mham and those identified have a capacity of 13.2 Mham. These reservoirs in India, besides irrigation, contribute significantly to power generation, flood management, domestic and industrial supply etc. India is not gifted with many good storage sites. Thus, to abandon any identified site on environmental considerations does not appear justified. Efforts are, therefore, required to be directed to strike a balance between development and environment impact mitigation to facilitate dam construction.

In view of the above, the Ministry of Environment and Forest, Government of India have issued directives from time to time for the planners of water resources

projects to get a study done for environmental impact of project and the environmental management plan containing measures to mitigate the adverse impact through experts and this study should be the part of the Detailed Project Report along with a check list.

Similar situation prevailed in U.S.A. After the enactment of U.S. National Environment Policy Act 1970, water agencies moved away from large dam projects. USBR, the primary agency for water development in USA, changed its objective from water development to water management. But strong societal needs have resulted in a mechanism which permitted to go ahead with several large storage dam projects.

One of the world's biggest storage dam project, the Three Gorges Project is constructed in recent years in China. It is 175 m high dam and its power house has an installed capacity of 17,680 MW. Many more dams are under construction in China and also in other third world countries in South Asia and Africa.

## **15.5 BRIEF DESCRIPTION OF ENVIRONMENTAL IMPACT STUDIES OF DAMS IN INDIA**

Environmental impact studies for a dam project are now essential as per directives of Ministry of Environment and Forest. In order to give the idea of these studies to the readers, brief description of environmental impact studies and management plans worked out for two dam projects in India is given here. It illustrates the action which has been taken to meet the environmental concerns regarding dam projects. This may act as a guide for future dam projects. The CWC guidelines (draft stage) may also be followed for environmental studies of future projects.

### ***15.5.1 Tehri Dam Project***

Tehri Dam Project has a 260.5 m high earth core rockfill dam. The project was envisaged earlier than the time. The environmental concerns of dams were well recognized in India. Due to delays in the construction of this project, the project clearance required environmental clearance also. The environmental clearance was accorded by MoEF, Govt. of India in 1990 with conditions which were to be complied with by Tehri Hydro Development Corporation (THDC) before implementing the project. All the required studies were got completed by experts and reputed institutions and the remedial measures required addressing all the concerns expressed by Botanical and Zoological Survey of India and NEERI were incorporated in the report submitted to the MOEF. The report concluded that with the implementation of the suggested remedial measures no significant environmental damage would be caused due to construction of project. Additional environmental

measures suggested by Expert Committee appointed by Government of India on the demand of local people were also included in the project. No dam project earlier to Tehri was investigated for environmental impacts in so much detail. Thus the environmental studies of Tehri project may be considered as a role model for future dam projects.

The environmental impact studies and the mitigation measures required were carried out mainly for the following aspects on the suggestions of MoEF and the Expert Committee and are briefly described in subsequent paras.

- (i) Safety against seismic hazards
- (ii) Catchment area treatment
- (iii) Afforestation due to submergence
- (iv) Command area development
- (v) Effect of reservoir on flora and fauna
- (vi) Effect of lake formation on water quality
- (vii) Water quality and downstream impact
- (viii) Impact on human health
- (ix) Dust pollution during construction
- (x) Rim stability and formation of green belt along the rim
- (xi) Socio economic study including rehabilitation
- (xii) Disaster management plan.

Seismic design of Tehri dam section is discussed in Chapter 7. In addition some built-in measures have been provided such as wide crest width of 20 m increased to 25 m at the abutments to counter the effect of earthquake acceleration at the top and a liberal free board of 9.5 m above FRL and 4.5 m above MWL to take care of extra settlement due to earthquake.

Under the catchment area treatment (CAT) plan, the entire degraded area of 52,204 ha of high and very high erosion class was proposed to be treated at the project cost. The plantation was to be done on the recommendation of Botanical Survey of India (BSI) with the involvement of local people. In addition structural measures such as check dams, bank protection, stabilization of slopes, terracing etc. were proposed to prevent soil erosion. In CAT plan locals were also discouraged to use wood as fuel.

The MoEF accorded forest clearance for diversion of forest land of 4193 ha for construction of project and submergence in reservoir against compensatory afforestation through Forest Department of Government of Uttar Pradesh in an area of 4586 ha in Lalitpur and Jhansi districts of U.P. at the cost of the project.

Command Area Development Plan was prepared by the Govt. of Uttar Pradesh, as the water of Tehri reservoir will be utilized in extending irrigation facilities through existing and new canal networks to a command area of 2.7 lac ha in Uttar Pradesh. It was submitted to MoEF for approval and was implemented by Govt. of U.P.

**Flora & Fauna:** The studies were conducted through Botanical Survey of India (BSI) for flora and Zoological Society of India (ZSI) for fauna to identify the species expected to be affected due to formation of reservoir. BSI study revealed that no rare

species come under submergence and so dam in no way lead to extermination of any species but recommended planting of certain species outside the reservoir. Accordingly a Botanical garden in an area of 14.28 ha has been established by Uttarakhand Forest Department adjoining the reservoir for plantation of special species coming under submergence at the cost of project.

As per ZSI studies, it was revealed that there will be no adverse impact on mammals, birds, reptiles, frogs, pisces due to creation of reservoir except on Mahseer which is a migratory fish and ascends upstream for breeding and it will be affected due to dam. Action plan for possible mitigation of Mahseer fish was framed. The Mahseer fish hatchery and fish-farm site was selected and developed near Koteswar dam (20 km below Tehri Dam, a part of Tehri Project) at the project cost.

Water quality in reservoir and its impact in the downstream was investigated. The reservoir water quality modeling study was conducted by Civil Engineering Department of University of Roorkee which concluded that no specific measures are required as there would be no adverse impact on water quality due to impoundment. The dissolved oxygen (DO) and biochemical oxygen demand (BOD) in entire reservoir are expected to remain within permissible limits of 6 or more and 2 or less respectively. The water quality monitoring in the upstream and downstream of reservoir has been carried out by Pollution Control Research Institute, BHEL after completion of project and it has also indicated that water quality is within permissible limits.

The study of potential impact on health aspects due to Tehri dam reservoir was taken up by Director General of Health Services (DGHS), Government of India. On the direction of DGHS, the National Malaria Eradication Programme (MMEP) and Malaria Research Centre (MRC) had carried out detailed investigations and recommended measures for future. These were taken-up for implementation by Government of Uttarakhand.

The recommendations of Central Pollution Control Board (CPCB) regarding measures to control dust pollution at project site and habitat area during construction such as sprinkling water and monitoring air quality were implemented to keep dust pollution within permissible limit.

Rim stability studies are described in Chapter 9. The project of creating green belt along the reservoir rim is being implemented by Forest Department of Uttarakhand.

The rehabilitation of the displacement of persons of rural and urban population expected to be affected by the construction of dam and reservoir was carried out by the project authorities. For the resettlement of displaced people from affected villages, several colonies are located in the fertile areas of the districts of Dehradun and Hardwar. These colonies are provided with all civic facilities such as electricity, irrigation, drinking water, roads, schools, dispensaries, community centre etc. which improved general living standard of the people. To resettle urban population of old Tehri town a new Tehri town has been constructed by the project authority (THDC). It is situated at higher altitude (1350 to 1850 m) overlooking the lake created by the dam. It has all modern civic facilities including schools, colleges and a university,

hospital, financial institutions, district administration offices, markets, places of worship, bus stand, stadium etc.

The rehabilitation and compensation to the displaced persons is socio-economic problem. In the last two decades, the compensation packages for the projects have been considerably improved within country's resources with the objective that the displaced persons should get better environment, quality of life and a chance to come to the mainstream of nation's development.

The examples are the compensation and rehabilitation packages of Sardar Sarovar and Tehri project, which have been evolved by the concerned govts in consultation with the people's representatives and also upgraded from time to time. For Tehri project, initially U.P. Government at the highest level, had finalized compensation package in consultation with the people's representatives and the same was adopted by THDC when project was transferred in around 1989. Upgradation of the package has been carried out from time to time as follows:

- Upgradation of U.P. Govt. approved package from time to time for escalation.
- Additional measures introduced w.e.f. 1.9.1995.
- Enhancement of package in Dec. 1998 based on Hanumant Rao Committee Report.
- Additional provisions for Tehri town approved by Government of India were to be implemented before 31.3.2001.

Total cost of Rehabilitation Package is Rs. 862.57 crores on August 1999 price level.

Environmentalists and sociologists often argue against such compensation packages on the issues of social and cultural disruptions in the life of displaced persons. If we look to the large influx of involuntary migrants from rural and the country side to the cities every year for better quality of life, then such a criticism does not appear to be very valid.

Socio-economic study regarding living standard of Tehri project affected families was conducted by Administrative Staff College of India, Hyderabad and it concluded that quality of life of the families after rehabilitation is far above and better than what the situation was before rehabilitation of these families.

The disaster management plan (DMP) was prepared by THDC after dam break analysis and was approved by Ministry of Agriculture who are the nodal Ministry for this purpose. On the suggestion of the nodal ministry it was decided to associate the State Governments with DMP.

### ***15.5.2 Kirthai-I Hydroelectric Project***

It is a project proposed on river Chinab in J&K. It envisages construction of a concrete gravity dam of 160 m height above deepest foundation and an underground power house of installed capacity of 380 MW, in left bank about 150 m downstream of dam axis. The revised updated DPR is under examination of CWC, CEA and

other related agencies of Govt. of India. For the purpose of DPR, the project authorities (JKSPDC Ltd.) engaged a consulting private firm EQMS Infra Pvt. Ltd. for the preparation of EIA and EMP report for the project.

The study area for the Environmental Impact Assessment (EIA) of the project is considered as the area within 10 km radius from the dam. Baseline study was carried out for pre-monsoon, monsoon and winter season of year 2010-11.

The EIA study was based on field surveys and suitable sampling of population and on the data collected from various agencies such as IMD, Survey of India, National Remote Sensing Agency, Forest Department, and Geological Survey of India. The findings of the report are briefly described below as an example for a run of river hydro power project.

**(a) EIA Findings**

*(i) Impact on Land Environment*

The project for construction purposes required about 160 ha of forest land and 46 ha other land under government control. The govt. will have to change the land use of this land permanently. The underground components of the project will not have any effect on forest area. The quarrying for construction material will not have any significant impact on land use as most of the excavation of surface and underground works is proposed to be used as construction material.

No significant change in general is expected in the overall geology of the area as the hill slopes appear stable and can withstand drilling and blasting. In some geologically fragile locations, controlled blasting is proposed to be used to prevent any damage.

Soil erosion due to excavation of some project components is expected during construction. This along with muck disposal on river slopes may temporarily cause some increase in sediment intensity in the river. But finally the sediment intensity in the river will reduce due to proposed catchment area treatment of the degraded and erosion prone areas.

*(ii) Impact on Water Environment*

The project is a run-of-river type scheme for power generation with small storage capacity for daily peaking. Thus, the water quality in the reservoir or in the river downstream is not expected to be affected. During construction some pollutants from the washing of screening and mixing plants, labour camps, dumping yards, workshops etc. may slightly affect river water quality in the downstream but the effect will not be significant as the quantity of affluent will be very small as compared to even non-monsoon flows of the river.

The reservoir area created by the dam is only 210 ha. Hence it is not expected to cause any significant impact on micro climate of the area and the hydrological cycle. However, the reservoir may slightly increase the evapo-transpiration in the area.

The surface water use in the river basin will not be affected due to project because there is neither any industry nor any irrigation scheme to use river water.

The river water has a pH value ranging between 7.1 and 8.1 and the Nalas meeting it have similar values. There are no natural or man-made sources to create any possibility of acidification of reservoir lake. The seepage from reservoir is thus not expected to pollute the groundwater which is of good quality as per base line study.

Since it is a peaking station, a minimum release of water of about 9 cumecs is ensured round the clock to meet downstream ecological requirements.

The downstream projects will be benefitted regarding the impact of sediment because of catchment area treatment of this project.

#### (iii) *Impact on Air and Noise Environment*

The air quality at the project site and the surrounding habitat area will be affected due to dust and other emissions from vehicles and construction equipments. It will have to be monitored and rectification measures will be taken, if required.

#### (iv) *Impact on Flora*

The study area is extremely mountainous with rugged terrain. The hill slopes are moderate to very steep. The hill slope vegetation is dominated by conifers. Some slopes are barren with no vegetation. In the upstream region vegetation is less. Right bank is rocky with no vegetation and left bank is dominated by Deodar trees. The dominant horticulture crop in the area is apple. Some other fruits as plum and peach are also grown. The study has revealed that none of the species of trees, herbs or shrubs are endangered to extinction due to submergence. However, any loss of riverine vegetation during project activity will be restored on the reservoir periphery in course of time when implementation of green belt and work of restoration and landscaping will be completed.

#### (v) *Impact on Fauna*

The study revealed that the mammals like Black bear, Langur and Bos grunniens (cross breed of Yak and Cow) belong to the study area. The study also identified existence of some bird species and butterflies in the area. The project activity is not submerging all the habitats. Hence, there is little chance of endangering any of the species as there is sparse human habitation on both the banks in the study area. There is also no wild life sanctuary, national park and biosphere reserve near the project area. However, there will be some disturbance to the wildlife due to noise and human interference in project area because of construction activity.

#### (vi) *Impact on River Ecology*

The proposed project's submergence consists of rocky bottom and steep slopes having scanty vegetation. Thus the chances of decomposition of organic material will be quite low. Therefore, generation of greenhouse gas emission is not expected.

Dams generally block migration of fishes. However in this case no species of migratory fish was found in the study area. But to promote fresh water fishery a provision for setting up a trout hatching in the upstream of dam site has been made.

Dam will induce sediment deposition in reservoir and this may also affect the river ecology and it will require careful study of deposition pattern with respect to

power intake. This sedimentation study for the proposed reservoir was initially carried out to examine the sedimentation impact on the life of reservoir and design of intake by the empirical methods of working deposition pattern. Later, mathematical modeling studies have also been carried out by DHI consultants to estimate the pattern of reservoir sedimentations, the adequacy and effect of sluices on sedimentation and to finalize the intake location and its design. The DHI using Mike-11 one dimensional model has concluded that the reservoir would take around 20 years to reach equilibrium condition i.e. the sediment deposit will not reach the intake for a period of around 22 years but it was found that by this time most of the live storage between FRL and MDDL will be lost. Hence two dimensional studies with Mike 21C were also carried out to work out the sediment pattern near the dam with the flushing conduits to determine the adequacy of intake level. The results with sluicing showed that equilibrium in sedimentation will reach in 10 years and adequate live storage would be available even after 25 years and the sediment profile was found well below crest level of intake. During sluicing in monsoon period the reservoir was kept at MDDL.

Catchment area treatment plan for the free catchment between this project and the upstream project has been proposed to reduce rate of sedimentation.

(vii) *Impact on Habitants*

Generally, the effect of storage in reservoir in tropical areas is to give rise to mosquito breeding causing malaria in the region. However, no such negative effect has been anticipated in the proposed project due to temperate climate and quick release of stored water during peaking hours of the day.

Submergence, generally, causes rehabilitation problems but in this case not a single family is required to be shifted.

Ecological releases to meet requirements of the river downstream of dam such as supporting aquatic life, drinking water requirement, wild life, riparian rights etc. a continuous release of 9.5 m<sup>3</sup>/sec is being ensured through the base load unit of the power house.

(viii) *Socio-Economic Impact*

Due to submergence only three villages shall be partly affected. A few families are likely to lose more than 50% of their agriculture land. These people will be compensated. No school, temple, mosque and other structure of public interest will be affected by the construction of project. On the other hand the infrastructural facilities such as school, hospital, drinking water, bank etc. which will be created by the project will have an access to the affected people and local population. This will improve overall economy of the area and the life of the local population.

**(b) Environmental Management Plan (EMP)**

In order to mitigate negative impact of the project, an environmental management plan has been prepared. The total estimated cost of EMP is Rs. 89.2 crores. Its salient features are briefly given below:

(i) *Catchment Area Treatment Plan (CAT)*

The CAT Plan envisages afforestation and structural measures to check erosion of potentially severe and very severe areas of soil erosion in the free draining catchment of 691 km<sup>2</sup> between the proposed project and the upstream Dugar Hydro Project. In critically degraded areas, plantation of locally useful diverse and indigenous plant species, fodder species, fuel wood, grasses, shrubs and herbs are proposed to be planted to bring areas under vegetation. For developing saplings, a new nursery will be established. The structural measures such as check dams, bank protection, benching and terracing, landslide prevention measures etc. are also proposed. The cost of CAT is estimated as Rs. 3434 lac to be borne by the project.

(ii) *Compensatory Afforestation*

Total forest land required for the project including submergence is 160 ha. Compensatory afforestation is proposed to be carried out through the Forest Department of J & K State in degraded forest area of 360 ha (about double the area as per J & K Forest Act 1997) at the cost of Rs. 1635 lac to the project.

(iii) *Rehabilitation*

The study revealed that there will be 255 affected persons belonging to three villages who will be losing part of their land in the submergence due to dam. Out of the total affected persons majority of landholders are small and marginal and large number of persons fall in landless category. For compensation purposes families losing more than 50% of land are categorized as fully affected families and other as partly affected. The compensation package is worked out as per norms fixed in R&R policy Act of J&K. The package also includes infrastructure development of local area. It costs Rs. 313 lac to the project

(iv) *Green Belt Development*

For the green belt development, to improve environment, it is proposed that plantation in degraded forest land around reservoir rim and the road side shall be done. It will cost Rs. 48 lac to the project.

(v) *Protection of Flora and Fauna*

EIA study report does not indicate endangering the extinction of any specific species of flora and fauna due to the project. But some disturbance/damage to the habitats of some species due to construction activity is expected. To compensate the anticipated damage, provision is made for creation and maintenance of biodiversity park and sanctuary in the project area. It is estimated to cost Rs. 300 lac to the project.

(vi) *Reservoir Rim Treatment*

The toe of the steep hills over the river bed will be subjected to elevated water surface due to reservoir. The slopes between FRL and MDDL will be subjected to

water level change every day. Thus the slopes need to be examined for stability in detail. The entire rim of the reservoir will have to be examined and investigated for unstable reaches for which protection measures would be required. On a rough assessment, provision of Rs. 960 lac for the cost of reservoir rim treatment has been made in the project estimate.

(vii) *Muck Disposal*

A quantity of about 11 lac m<sup>3</sup> of muck from surface and underground works will have to be disposed in form of muck piles and their slopes would be maintained through engineering and biological measures. A provision of Rs. 476 lac is made for these works in the estimate.

(viii) *Restoration of Quarry Site and Landscape*

The quarry/borrow area sites used for obtaining construction material will be restored through biological measures to negate the environmental impact. Similarly landscaping of area around dam site and residential area will be carried out through biological measures such as development of parks, garden, plantation etc. A provision of Rs. 373 lac is made in the project.

(ix) *Health Management Plan*

Though there is no indication for the incidence of malaria in the study area and there is no chance of breeding mosquitoes due to run-of-river type operation of project, a provision to strengthen existing health facilities available in the area is made in the EMP. A provision of Rs. 125 lac is made in the project.

(x) *Subsidized Fuel*

In order to reduce pressure on adjoining forest for fuel wood by the labour force of the project, it is proposed to provide subsidized LPG connection and kerosene oil and electricity for heating purposes for which a provision is made in EMP. A provision of Rs. 340 lac is made in the project.

(xi) *Solid Waste Management*

The labour force, the other service providers and technical staff and their families will be stationed at work site in colonies. The sewerage and solid waste management arrangements would be warranted under the Municipal Solid Waste Management and Handling rules to maintain the health of the people. A provision of Rs. 490 lac is made for its cost in the project.

(xii) *Environment Monitoring*

In EIA study report, it has been mentioned that temporary changes during construction may occur in air, noise and water quality. EMP has developed a plan to monitor and manage these factors and a provision of Rs. 125 lac for its cost is made in the project.

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# Chapter 16

## Dam Safety



### 16.1 GENERAL

The dams have been constructed the world over and have been recognized for their role in the progress of human civilization. The water stored or diverted by a dam is put to several beneficial uses such as domestic, industrial, irrigation, power generation, fisheries, recreation etc. This storage sometimes possesses a potential hazard to public safety in the event of failure of the dam. A failure of dam results in loss of life and disastrous economic consequences in the downstream.

Dam engineering has evolved on scientific lines depending on advances in concrete technology, soil science and rock mechanics in 20<sup>th</sup> century but being site specific inter disciplinary and highly complex subject it is still not an exact science which can completely eliminate the risk of dam failure. Failure of some embankment dams has been described in Chapter 8.

The purpose of the dam safety is to predetermine potential hazards and to take measures to reduce them to the acceptable limits, so that the risk from dam failure is minimized. It shall also provide emergency preparedness plans.

The concept of dam safety has been adopted in India quite late. The dam safety service was started in Central Water Commission in around 1980 and later it was made mandatory to set up such organizations in the State water resource departments with the main function of reviewing the design flood of existing dams as per existing criteria for new dams and to take measures to ensure safety against it. Besides design flood it was also required to evaluate and ensure structural safety of the dams. CWC has also prepared a manual on dam safety procedures which serves as a guide for dam safety engineers.

In view of concerns for dam safety, it has become increasingly important to provide sufficient instrumentation in dams, whether embankment or concrete, for monitoring the performance of the structure during construction and throughout the operational life of the project. Instrumentation in dams has been dealt in Chapter 13. The CWC publication 'Guidelines for Instrumentation of Large Dams' may also be

referred. The evaluation and interpretation of instrumentation data for the dam, foundation, abutments and appurtenant structures provide primary information to the engineers of dam safety organization regarding safety of dam. To highlight the distress condition of dam, if indicated by the instrumentation data, is a step forward towards dam safety.

Greater emphasis on regular inspection, operation and maintenance of dam is required for the safety of a dam. The State Governments who are generally the owners of the dams have varying standards for the dam safety. Hence, there is a need for legislation which may standardize the procedure for dam maintenance to be followed for all dams in the country because dam failures have potential for heavy loss of life and property to the nation. The Central Government of India is now working on this issue and a legislation is expected soon. Some important aspects of dam safety such as review of inflow design flood, structural safety including seismic risk, need of systematic inspection for identifying causes of distress, and plan for action during emergency are described briefly in following paras. The Dam Safety Rehabilitation Directorate of CWC is working on preparing and publishing manuals for all these aspects which may be used for reviewing the safety of existing dams.

## 16.2 REVIEW OF DESIGN FLOOD

The existing guidelines for determining design flood and fixing the capacity of spillway to safely pass the design flood are contained in CWC guidelines and the relevant IS Codes. For major storage projects the design flood is the PMF and it is routed through spillway with 10% gates inoperative. These specifications are generally considered quite stringent. So new projects designed following the existing guidelines are considered reasonably safe from hydrological hazards. However, emergency action plan due to dam breach should be kept ready for such projects also to safeguard against unforeseen situations.

The existing dams need review of the adequacy of spillway capacity as these were designed a couple of decades ago when the present guidelines were not available. The review of all existing dams which are more than 15 m in height or having a capacity of 60 Mm<sup>3</sup> or more should be carried out on priority by applying the existing guidelines and IS Codes. The review of existing dams should give the value of flood which can safely be passed through the spillway and how much more can be passed with certain relaxations in the criteria such as:

1. Lowering the reservoir level before the flood impinges
2. Considering the properly designed solid parapet as part of free board.

In case the flood with the above relaxation does not ensure reasonable safety of dam and there are chances of failing the dam, immediately action should be initiated to frame emergency action plan for disaster management. Besides, the action should

also be initiated for permanent corrective measures such as diversion of part flood through provision of an auxiliary or side channel spillway or by raising the dam.

This aspect is of greater concern in case of an embankment dam as it cannot withstand any overtopping.

## 16.3 STRUCTURAL REVIEW

The new dams are being made structurally safe following the criteria and guidelines given in IS Codes and other international practices for design, construction and quality control. The new dams are also provided with instruments for monitoring the structural behaviour of dam during construction, reservoir filling and operation. The review of observations of instruments, if indicate any deviation from the design parameters, the corrective measures may be adopted. Hence it may not be incorrect to assume that new dams are reasonably structurally safe. Still for future review of the dams, it is of utmost importance that all technical records regarding investigations, design, drawing and quality control, measures during construction and any deviations from design during construction are properly compiled. All the observations of visual inspections carried out from time to time and of the instruments should also be compiled and maintained by the dam safety organization. The observations of the dam instruments should be periodically analysed by experts to ensure structural integrity of the structure.

The existing dams constructed a couple of decades ago need a structural review on priority because at that time the present knowhow of dam engineering was probably not available. Hence, during review of structural safety of existing dams it is desirable to collect the technical details of investigations, design and construction of existing dams and to see whether these conform to the criteria given in IS Codes. The CWC Manual on structural safety of dams may also be referred. It is a difficult job to carry it out because it is very unlikely that systematic technical old records of an existing dam would be available. Most likely such dams would be having no recording instruments installed inside the dam body. In case there would be some instruments, it is likely that their data would be either missing or incomplete. Moreover, such a study requires an inter-disciplinary effort. Hence to do this job, consultation with experts may generally be required. After carrying out the review in as best as possible manner, if it is revealed that the existing dam does not conform to present standards, it is of priority to prepare emergency action plan after carrying out the dam break analysis. In this process due weightage should also be given to the facts and circumstances due to which the dam has been safe in all these years before going for implementing rehabilitation measures.

In structural safety review, seismic aspects of design should be given due weightage, specially in dams located in seismic zones of high intensity. The design of existing dams would normally be based on pseudo-static method of analysis. The

new concrete dams of height above 30 m in highly seismic areas and in zones IV and V are generally designed and checked for safety by dynamic analysis using FEM method as per practice of dam design. If the existing old dam on checking for seismic forces as per present practice is not found safe, engineering solutions should be evolved but it is always not easy and economical to implement them. However, before adopting such a course, it is also necessary to consider other factors like age of dam and the disaster potential in case of failure. Hence each case should be treated on its own merit.

A study of seepage through body of dam and through foundation is an important component of structural review of dams. Whether the dam is of concrete or masonry or is an embankment type the seepage through body of dam will be the function of permeability of material used in construction of dam and the head of water. Concrete though considered impervious but water also seeps through it. This seepage is normally trapped through form drains and seepage through transverse joints in dams is prevented through water-stops. In embankment dams, the clay core acts as water barrier for seepage through dam body, but seepage is not completely checked. Thus, whatever seepage takes place through the core it is collected in the downstream toe drain via filter material placed along the downstream slope of core and the horizontal drainage blanket. Seepage through foundation is prevented through curtain grouting in concrete and masonry dams and by cut-offs in embankment dams. In concrete and masonry dams the seepage is collected through foundation gallery and is measured. Similarly in earth dams the seepage is measured in toe drain. In case the seepage observations show abnormal increase it should be investigated, locations identified and remedial measures should be taken. In case of embankment dams if action is not taken on priority, piping (seepage water carrying fines of soil) may lead to the failure of dam.

## **16.4 INSPECTION, OPERATION AND MAINTENANCE**

Regular inspection by engineer-in-charge of dam and its appurtenances, review of reservoir operation regulation order and periodical maintenance schedule and its implementation for the dam and its various appurtenances are the important components of dam safety. Guidelines of CWC on safety inspection of dams and IS Codes in this regard are available for reference.

For every dam there should be written instructions for inspection with a check list. The frequency of inspection should be specified which may decrease after several seasons of operation. There should also be standing regulation orders for reservoirs as well as maintenance schedule of works. The issuance of these orders and their implementation should be the job of the engineer-in-charge of the dam. The inspection of dams at closer intervals during flood season by senior officers and the proper regulation of flood discharge is very essential for the safety of dam. Some aspects of inspection and maintenance of dams are highlighted in following paras.

### ***16.4.1 Inspection and Maintenance of Embankment Dams***

Inspection of an embankment dam should be regularly carried out with the objective of looking for development of unfavourable conditions. The downstream slope should be inspected for cracks, slides, subsidence, damage to protection works, and boggy areas due to seepage. The upstream slope should be inspected for adequacy of rip-rap protection and disturbance in it due to wave action etc. Embankment dams must be observed during inspection for the occurrence of piping or boiling in the downstream of the dam. The measuring instruments and their monitoring system should be inspected and the observations of the instruments should be reviewed.

During maintenance the debris etc. must be cleared from upstream slope and the disturbed protection works should be set right. Undesirable vegetation should be cleared and monitoring devices on the dam and adjoining areas should be maintained for proper functioning. The unfavourable conditions observed in the downstream should be immediately remedied.

### ***16.4.2 Inspection and Maintenance of Concrete Dams***

During inspection of a concrete dam, the exposed abutment contacts, upstream and downstream slopes, crest, parapet, galleries and contraction joints should be observed for any abnormal condition such as cracks, settlements, deflections, lateral movements, and deterioration. The measuring instruments and their monitoring systems should also be observed. The seepage in foundation gallery and other galleries should be assessed. Documentation of regular inspections and observations and data from measuring instruments should form the basis of timely maintenance, repair and rehabilitation of the dam.

### ***16.4.3 Inspection and Maintenance of Mechanical Equipment***

The regulation gates at the outlets and the spillway are the main mechanical fixtures which need regular inspection and maintenance. The schedule inspection of these features includes observing the concrete and metal surfaces for any abnormality and deterioration of the protective coating. Besides the gates, the inspection of gate hoisting mechanism and its operational capability is also important. Hence, during inspection, if possible, the gates and valves etc. should be operated and their satisfactory functioning should be observed. The gates and valves during scheduled maintenance should be lubricated and serviced according to manufacturer's manual.

## 16.5 SAFETY CORRECTIONS

In case if during safety review and regular inspection of a dam some safety deficiencies are found, necessary corrections should be adopted on priority if these deficiencies are of a nature which can lead to dam failure resulting in heavy losses in the downstream. The corrective measures will depend on type of deficiencies and will be project specific (refer CWC publication 'Manual for Rehabilitating Existing Dams') but the basis of such corrections should be as below:

- The corrective measures should be able to ensure structural safety.
- These should involve minimum cost retaining the project benefits.
- These should not cause any adverse effect on environment.
- These should be a combination of structural and non-structural changes to minimize cost.
- These should be possible with available design and construction facilities in the project organisation.

## 16.6 EMERGENCY PREPAREDNESS

The emergency preparedness is a plan of action known as Emergency action plan (EPA) to be followed to save the dam in distress and, if the dam failure is eminent, then how to reduce the potential damage to property and loss of life from the large flood wave resulting from dam breach. It should be prepared for all dams. But the priority should be given to those dams which are found in distress after the review studies and those whose spillway capacity is found inadequate and implementation of possible corrective measures does not ensure reasonable safety against overtopping. Priority of preparing EPA should be given to those cases also whose hazard potential in the downstream in the event of dam failure is large. The systematic activities under an EPA (CWC-Doc, Feb. 2016) are:

- (i) To identify emergency conditions threatening a dam
- (ii) To expedite action to prevent dam failure
- (iii) To reduce loss of life and property damage in case of dam failure.

The first two are the engineering activities to be taken care of by the owner of the dam and the engineer-in-charge of dam operation. He should have required man-power and resources to handle the situation and prevent dam failure. The third activity involves administrative agencies who should have the knowledge of inundation of area due to dam failure. Their function should be to carry out timely evacuation and to come notification of the dam failure to the public.

It is seen that an important input for EPA is the inundation plan in the downstream of dam for floods of different return period such as 25 years, 50 years, design flood and PMF + dam break when reservoir is at FRL. The inundation plan for any flood or

PMF + dam break will require dam break analysis. This analysis will need the river cross sections in the downstream for a long reach and the roughness coefficient (Mannings 'n' values). It is also necessary to survey the flood expected area for property and population which may get affected. The time rate of dam break is generally assumed practically instantaneous in case of concrete dam and varying from 1/2 to 6 hours for an embankment dam. In concrete dam it is assumed that one block will be damaged completely whereas in embankment dam a trapezoidal breach section develops. Thus important breach parameters are geometry of breached section, time of failure and the peak discharge. Frochlich has developed empirical and semi-empirical expressions for the breach parameters (CWC-Doc.2018). After obtaining peak discharge from the breach, it shall be routed through the river channel to get the information about the area to be flooded due to the breach. It is generally assumed that flood corresponding to PMF would impinge the dam when the reservoir is at FRL.

Modelling softwares are available for analyzing the dam breach process and routing the peak breach outflow to determine inundation depth downstream of dam. DAMBAK was the modelling software developed in 1977. Thereafter several other models have been developed. HEC-1, HEC-HMS and HEC-RAS are the models developed by US Army Corps of Engineers, and HEC-RAS model is being widely used. It can give 1D and 2D flow solutions. One dimensional models are best suited for channels with moderate to steep slopes and where flow is confined within narrow flood plain and flow direction is single stream line without major divergence. For routing over wide and flat surfaces it is better to use 2D model.

The next step is to prepare inundation maps to assess the property and population lying in the area prone to inundation. Important structures and towns/villages should be identified which may be affected in the event of dam break. These maps should be made available to the local administrative officials which may be responsible for evacuation during emergency. It should be made clear with the maps that the area shown to be inundated and the travel time of flood wave are approximate and may differ with actual events.

Finally it is required by the dam operation-in-charge to establish an efficient warning system and the arrangement to notify all affected State and Central agencies about the emergency condition of the dam and possible consequences. These agencies may inform the public and may take action for evacuation so that loss to life and property is reduced in the event of dam failure.

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