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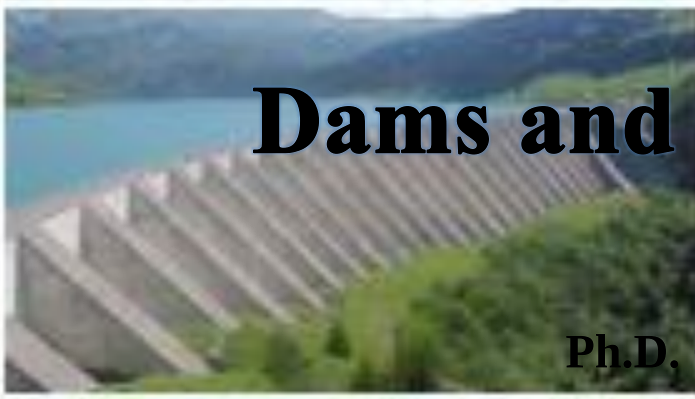

Dams and Reservoirs

Ph.D. Course

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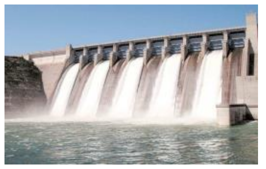
Contents



CHAPTER ONE

INTRODUCTION

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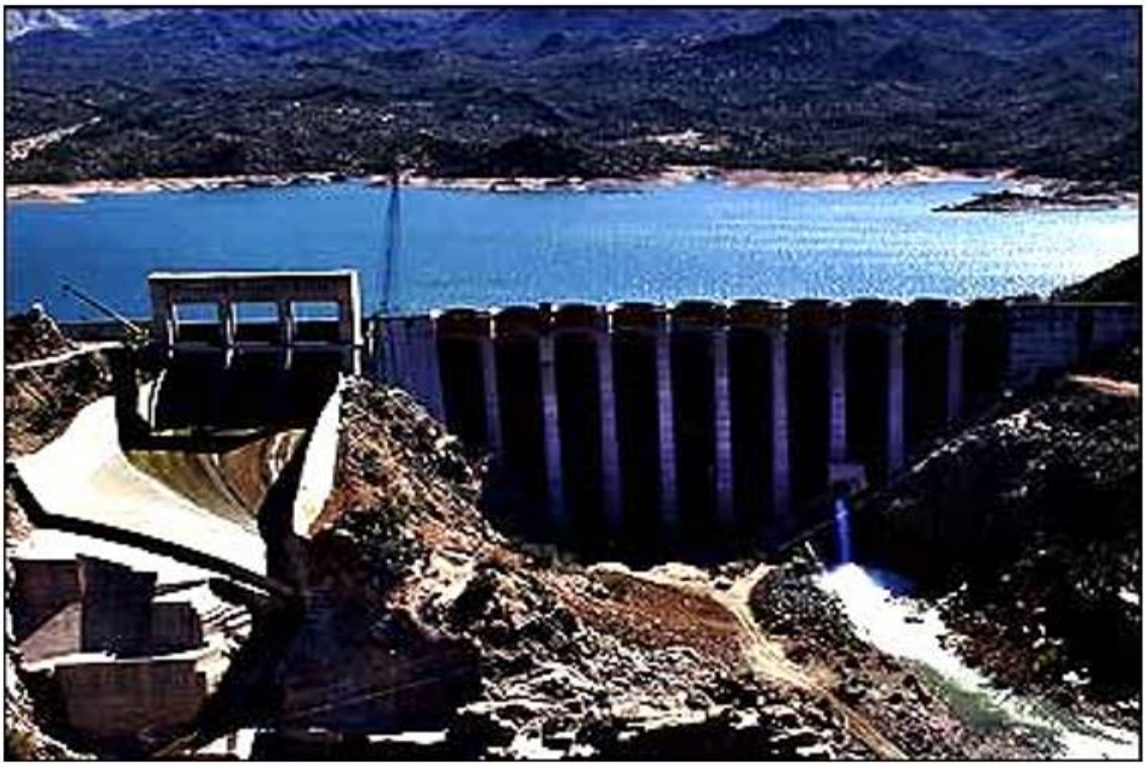


Figure 1.1 Dam and Reservoir

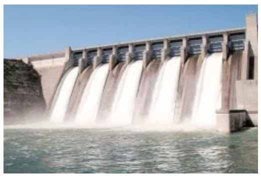
1.1. Definition:

A dam is a hydraulic structure built across a river to create a reservoir on its upstream side for impounding water for various purposes such as irrigation, municipal and industrial supply, hydropower, recreation as shown in figure1.1. A dam and a reservoir are complements of each other.

Dams are generally constructed in the mountainous reach of the river where the valley is narrow and the foundation is good.

Dams are probably the most important hydraulic structure built on the rivers. These are very huge structure. Thousands of workers and engineers work for a number of years in the construction of a dam.

Generally, a hydropower station is also constructed at or near the dam site to develop hydropower. Sometimes, a pickup weir is constructed on the downstream of a dam in boulder reach or alluvial reach.



1.2. Classification of Dams:

Dams can be classified according to different criteria, as given below:

A. Based on Function Served:

Depending upon the function served, the dams are of the following types:

1. Storage dams: Storage (or conservation) dams are constructed to store water during the rainy season when there is a large flow in the river. The stored water is utilized later during the period when the flow in the river is reduced and is less than the demand. The water stored in the reservoir is used for a number of purposes, such as irrigation, water supply and hydropower. Storage dams are the most common type of dams and in general the dam means a storage dam unless qualified otherwise.
2. Detention dams: Detention dams are constructed for flood control. A detention dam retards the flow in the river on its downstream during floods by storing some flood water. Thus the effect of sudden floods is reduced to some extent. The water retained in the reservoir is later released gradually at a controlled rate according to the carrying capacity of the channel downstream of the detention dam. Thus the area downstream of the dam is protected against flood.
3. Diversion dams: A diversion dam is constructed for the purpose of diverting water of the river into an off-taking canal (or a conduit). A diversion dam is usually of low height and has a small storage reservoir on its upstream. The diversion dam is a sort of storage weir which also diverts water and has

a small storage. Sometimes, the terms weirs and diversion dams are used synonymously.

4. Debris dams: A debris dam is constructed to retain debris such as sand, gravel, and drift wood flowing in the river with water. The water after passing over a debris dam is relatively clear.

5. Coffer dams: A coffer dams is not actually a dam. It is constructed around the construction site to exclude water so that the construction can be done in dry. A coffer dam is thus a temporary dam constructed for facilitating construction. A coffer dam is usually constructed on the upstream of the main dam to divert water into a diversion tunnel (or channel) during the construction of the dam as shown in figure1.2.

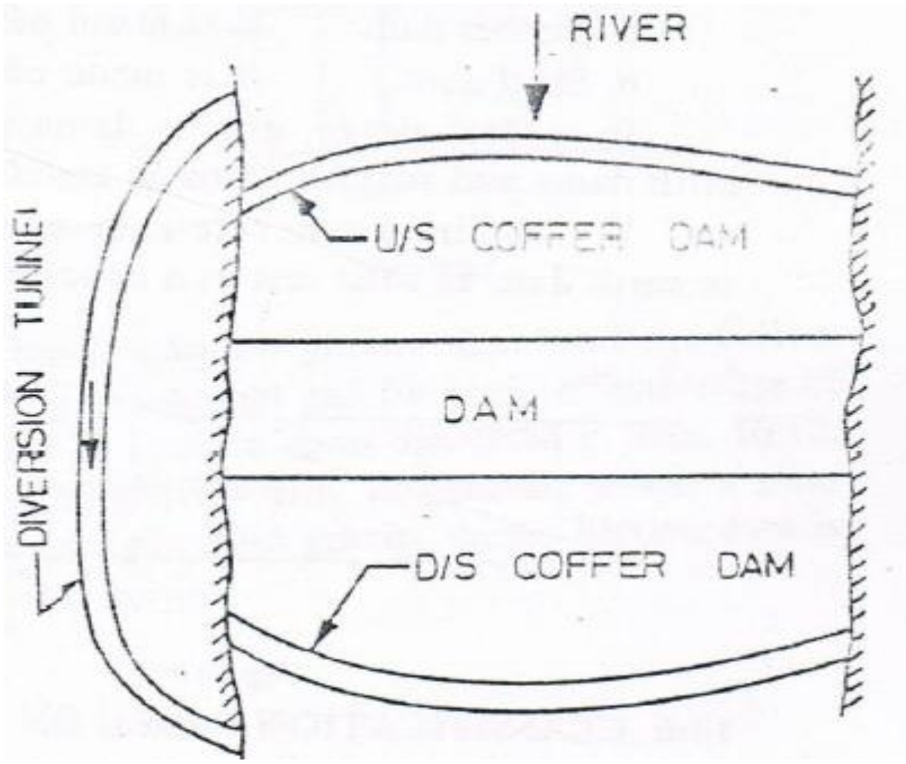


Figure 1.2 Main dam with coffer dam and diversion tunnel

B. Based on Hydraulic Design:

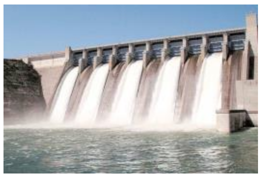
On the basis of hydraulic design, dams may be classified as:

1. Overflow dams: An overflow dam is designed to act as an overflow structure. The surplus water which cannot be retained in the reservoir is permitted to pass over the crest of the overflow dam which acts as a spillway. The overflow dam is made of a material which does not erode by the action of overflowing water. Generally, cement concrete is used in overflow dams and spillways. Most of the gravity dams have overflow sections for some length and the rest of the length as a non-overflow dam. However, sometimes the entire length of the dam of low height is designed as an overflow dam. The overflow dam is also called the spillway section.
2. Non-overflow dams: A non- overflow dam is designed such that there is no flow over it. Because there is no overflow, a non-overflow dam can be built of any material, such as concrete, masonry, earth, rock fill and timber. As already mentioned, the non-overflow dam is usually provided in a part of the total length of the dam. However, sometimes the non- overflow dam is provided for the entire length and a separate spillway is provided in the flanks or in a saddle away from the dam. Fig shows a non-overflow earth dam. Figure (1.3) shows the overflow and non-overflow dam.

Overflow Dam

Non-overflow Dam

Figure 1.3 Overflow and non-over flow dam



C. Based on Materials of Construction:

Based on the materials used in construction, the dams are classified as follows:

- Masonry dam
- Concrete dam
- Earth dam
- Rockfill dam
- Timber dam
- Steel dam

In modern usage, gravity dams, buttress dam and arch dams are classified as concrete dams and earth dams are rockfill dams as embankment dam.

Combined concrete-cum-earth dam:

In this type of dam, a part of the length is constructed as-an earth dam and the rest as a concrete.

Composite dam.

Composite dam has a section which consist of two materials. Generally, a composite dam has some portion as a rockfill and some portion as earthfill in the same section.

D. Based on Rigidity:

On the basis of the rigidity, the dams are classified into 2 types:

1. Rigid dams: A rigid dam is quite stiff. It is constructed of stiff materials such as concrete, masonry, steel and timber. These dams deflect and deform very little when subjected to water pressure and other forces.
2. Non-rigid dams: A non-rigid dam is relatively less stiff compared to a rigid dam. The dams constructed of earth and

rock fill are non-rigid dams. There are relatively large settlements and deformations in a non-rigid dam.

E. Based on the structural action:

This is the most commonly used classification of dams. Such classification are:

1. Gravity dams. 2. Rock fill dams.
3. Earth dams. 4. Arch dams.
5. Buttress dams. 6. Steel dams. and
7. Timber dams.

In figure 1.4 below different type of dam shown with respect to material construction.

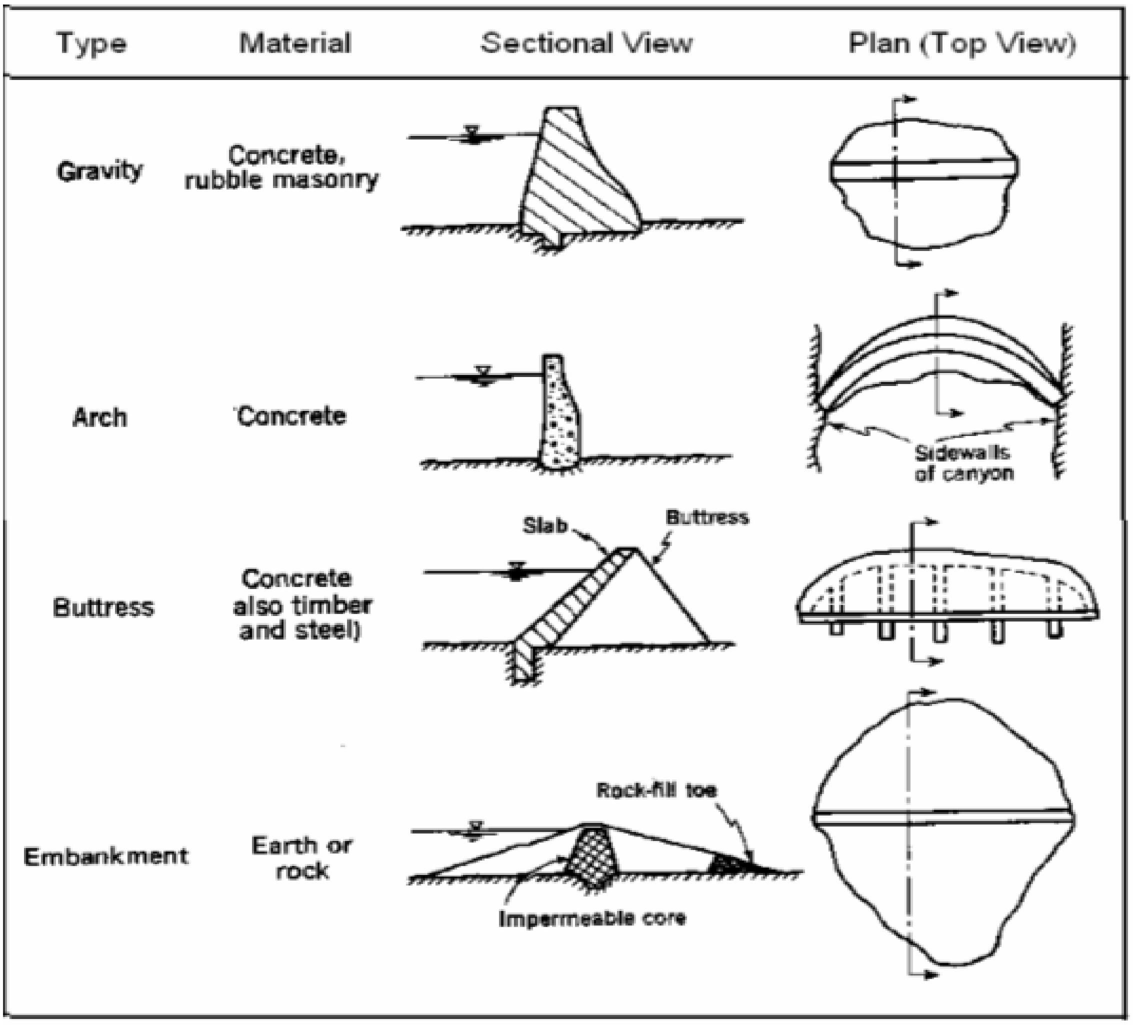
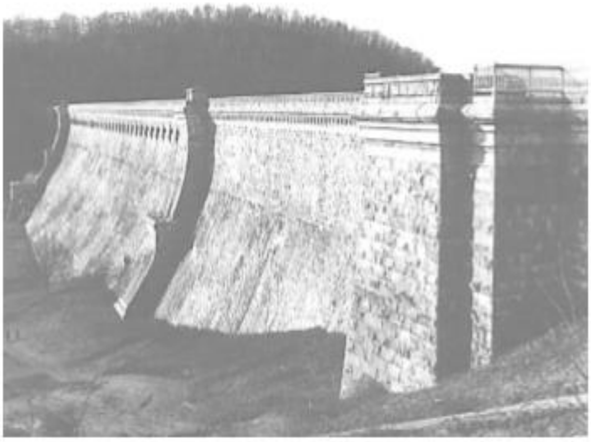


Figure 1.4 Dams type according to material construction

1.3. Gravity Dam:



Masonry Gravity Dam (non-over flow)

Concrete Gravity Dam (over flow)

Figure 1.5 Gravity dams

A gravity dam resists the water pressure and other forces due to its weight (or gravitational forces). Thus, the stability of a gravity dam depends upon its weight. The gravity dams are usually made of cement concrete. In the past, the gravity dams were made of stone masonry, but now the masonry dams are rarely constructed, except for very small heights, see figure (1.5).

Main Features of Gravity Dam:

1. The gravity dams are generally straight in plan (i.e. axis is straight from one a abutment to the other).
2. The gravity dams are approximately triangular in cross-section, with apex at the top.
3. The gravity dams are generally more expensive than earth dams but are more durable.
4. They are quite suitable for the gorges with very steep slopes.

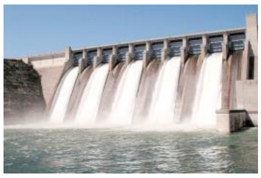
5. They require strong rock foundation. However, if the foundation consists of soil, the height of the gravity dams is usually limited to 20 m .

Advantages:

i. Gravity dams are quite strong, stable and durable.
ii. Gravity dams are quite suitable across moderately wide valleys and gorges having steep slopes where earth dams, if constructed, might slip.
iii. Gravity dams can be constructed to very great heights, provided good rock foundations are available.
iv. Gravity dams are well adapted for use as an overflow spillway section. Earth dams cannot be used as an overflow section. Even in earth dams, the overflow section is usually a gravity dam.
v. Gravity dams are especially suited to such areas where there is very heavy downpour. The slopes of the earth dams might be washed away in such an area.
vi. The maintenance cost of a gravity dam is very low.
vii. The gravity dam does not fail suddenly. There is enough warning of the imminent failure and the valuable property and human life can be saved to some extent.
viii. Gravity dam can be constructed during all types of climatic conditions.
ix. The sedimentation in the reservoir on the upstream of a gravity dam can be somewhat reduced by operation of deep-set sluices.

Disadvantages:

i. Gravity dams of great height can be constructed only on sound rock foundations. These cannot be constructed on weak or permeable



foundations on which earth dams can be constructed. However, gravity dams up to 20 m height can be constructed even when the foundation is weak.

ii. The initial cost of a gravity dam is usually more than that of an earth dam. At the sites where good earth is available for construction and funds are limited, earth dams are better.

iii. Gravity dams usually take a longer time in construction than earth dams, especially when mechanized plants for batching, mixing and transporting concrete are not available.

iv. Gravity dams require more skilled labor than that in earth dams.

v. Subsequent raising is not possible in a gravity dam.

1.4. Earth Dam:

Figure 1.6 Earth dam

An earth dam is made of earth (usually local soils), it resists the forces exerted upon it mainly due to shear strength of the soil. Although the weight of the dam also helps in resisting the forces, the structural behavior of an earth dam is entirely different from that of a gravity dam, see figure (1.6).

Main Features of Earth Dam:

1. The earth dams are usually built in wide valleys having flat slopes at (abutments).
2. The foundation requirements are less stringent than those of gravity dams, and hence they can be built at the sites where the foundations are less strong.
3. They can be built on all types of foundations. However, the height of the dam will depend upon the strength of the foundation material.
4. The section of an earth dam can be homogeneous when the height of the dam is not great.
5. Generally, the earth dams are consists different zoned sections (see Fig. below) , these sections categorized as follow :
6. The impervious zone (called core) mainly located in the middle of cross section of dam.
7. Relatively pervious zones (called shells or shoulders) enclosing the impervious zone (core) on both sides
8. If the earth dam is built on a pervious foundation, a concrete cutoff wall or a steel sheet pile line is also provided in the continuation of the core section.
9. Moreover, a drainage filter or a rock toe is provided on the downstream to carry away the water that seeps through the dam and its foundation.
10. Earth dams are usually cheaper than the gravity dams if suitable earth in abundant quantity is easily available near the site as shown in figure (1.7).

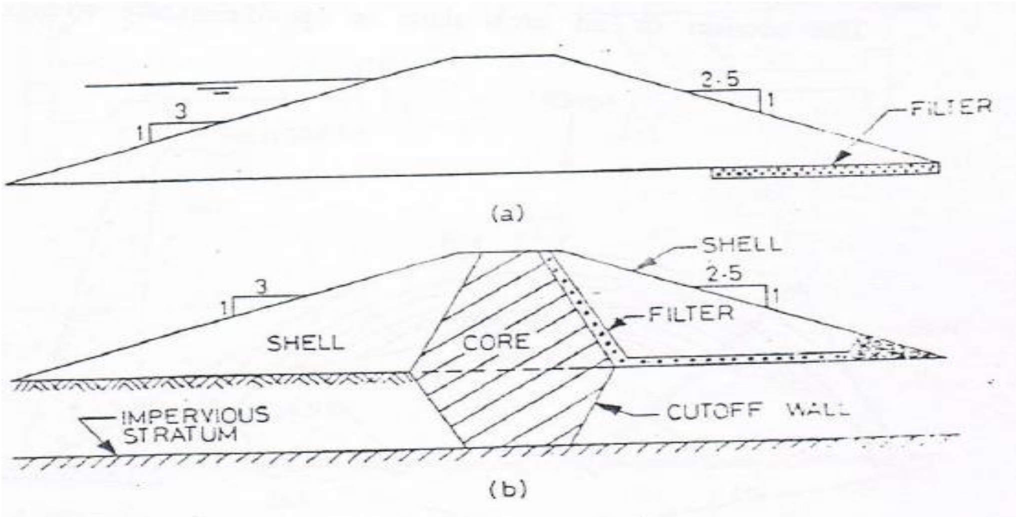


Figure 1.7 Earth dam component

Advantages:

i. Earth dams are usually cheaper than gravity dams if suitable earth for construction is available near the site.

ii. Earth dams can be constructed on almost all types of foundations, provided suitable measures of foundation treatment and seepage control are taken.

iii. Earth dams can be constructed in a relatively short period.

iv. The skilled labor is not required in construction of an earth dam. Earth dams can be raised subsequently.

v. Earth dams are aesthetically more pleasing than gravity dams.

vi. Earth dams are more earthquake-resistant than gravity dams.

Disadvantages:

i. Earth dams are not suitable for narrow gorges with steep slopes.

ii. An earth dam cannot be designed as an overflow section. A spillway has to be located away from the dam.

iii. Earth dams cannot be constructed in regions with heavy downpour, as the slopes might be washed away.

iv. The maintenance cost of an earth dam is quite high. It requires constant supervision.

v. Sluices cannot be provided in a high earth dam to remove slit.

vi. An earth dam fails suddenly without any sign of imminent failure. A sudden failure causes a major danger for lives and properties.

1.5. Rockfill Dam:

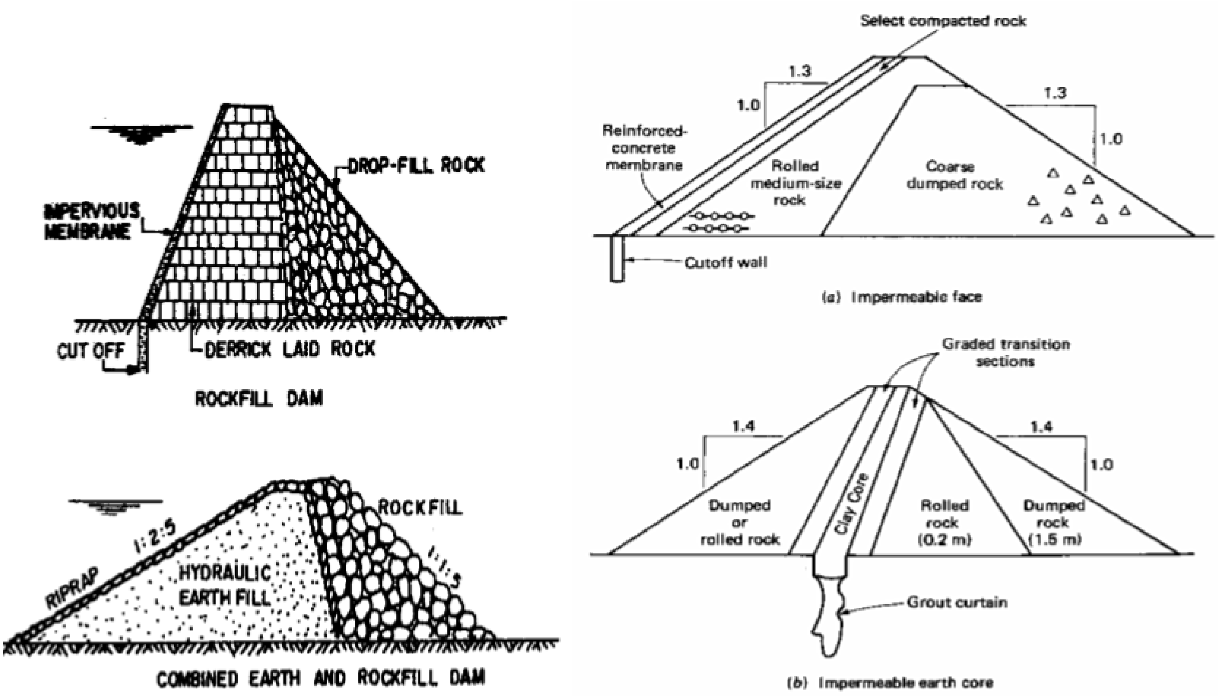


Figure 1.8 Rockfill dam

A rock fill dam is built of rock fragments and boulders of large size, see figure (1.8).

Main Features of Rockfill Dam:

1. An impervious membrane is placed on the rockfill on the upstream side to reduce the seepage through the dam.
2. The membrane isusually made of cement concrete or asphaltic concrete.

3. A dry rubble cushion is placed between the rockfill and the membrane for the distribution of water load and for providing a support to the membrane.
4. Sometimes, the rockfill dams have an impervious earth core in the middle to check the seepage instead of an impervious upstream membrane.
5. The earth core is placed against a dumped rockfill.
6. It is necessary to provide adequate filters between the earth core and the rockfill on the upstream and downstream sides of the core so that the soil particles are not carried by water and piping does not occur.
7. The side slopes of rockfill are usually kept equal to the angle of repose of rock, which is usually taken as 1.4:1 (or 1.3:1).
8. Rockfill dams require foundation stronger than those for earth dams. However, the foundation requirements are usually less stringent than those for gravity dams.
9. Rockfill dams are quite economical when a large quantity of rock is easily available near the site.

Advantages:

i. Rockfill dams are quite inexpensive if rock fragments are easily available.
ii. Rockfill dams can be constructed quite rapidly.
iii. Rockfill dams can better withstand the shocks due to earthquake than earth dams.
iv. Rockfill dams can be constructed even in adverse climates.

Disadvantages:

i. Rockfill dams require more strong foundations than earth dams.

ii. Rockfill dams require heavy machines for transporting, dumping and compacting rocks.

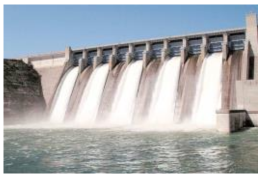
1.6. Arch Dam:

Figure1.9 Arch dam

An arch dam is curved in plan, with its convexity towards the upstream side, see figure (1.9).

Main Features of Arch Dam:

1. An arch dam transfers the water pressure and other forces mainly to the abutments by arch action.
2. An arch dam is quite suitable for narrow canyons with strong abutments which are capable of resisting the thrust produced by the arch action.
3. The section of an arch dam is approximately triangular like a gravity dam but the section is comparatively thinner.
4. The arch dam may have a single curvature or double curvature in the vertical plane.



5. Generally, the arch dams of double curvature are more economical and are used in practice.
6. The quantity of concrete required in an arch dam is less than that for a gravity dam, but it is not necessarily less expensive because of high cost of concrete and form work.
7. The arch dams are subjected to large stresses because of changes in temperature shrinkage of concrete and yielding of abutments.
8. The arch dam requires good quality concrete for resisting the stresses.

Advantages:

i. An arch dam requires less concrete as comparedto a gravitydam as the section is thinner.

ii. Arch dams are more suited to narrow, V-shaped valley, having very steep slopes.

iii. Uplift pressure is not an important factor in the design of an arch dam because the arch dam has less width and the reduction in weight due to uplift does not affect the stability.

iv. An arch dam can be constructed on a relatively less strong foundation because a small part of load is transferred to base, whereas in a gravity dam full load is transferred to base.

Disadvantages:

i. An arch dam requires good rock in the flanks (abutments) to resist the thrust. If the abutments yield, extra stresses develop which may cause failure.

ii. The arch dam requires sophisticated formwork, more skilled labor and richer concrete.

iii. The arch dam cannot be constructed in very cold climates because swelling of concrete occurs due to freezing.

iv. The speed of construction is relatively slow.

1.7. Buttress Dam:

Buttress dams are of three types:

1. Deck type: Consists of a sloping deck supported by buttresses. Buttresses are triangular concrete walls, which transmit the water pressure from the deck slab to the foundation. Buttresses are compression members as shown below in figure (1.10):

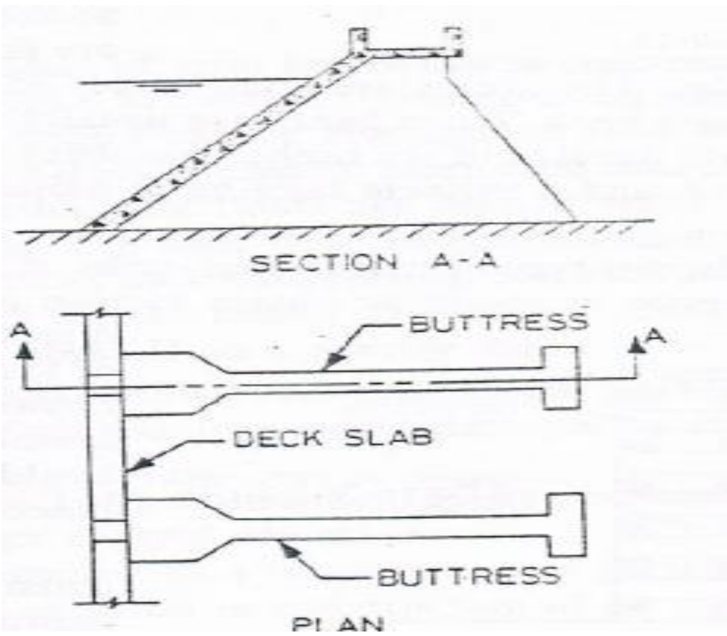


Figure1.10 Deck type buttress dam

2. Multiple –arch type: The deck slab is replaced by horizontal arches supported by buttresses. The arches are usually small span and made of concrete as shown below in figure (1.11):

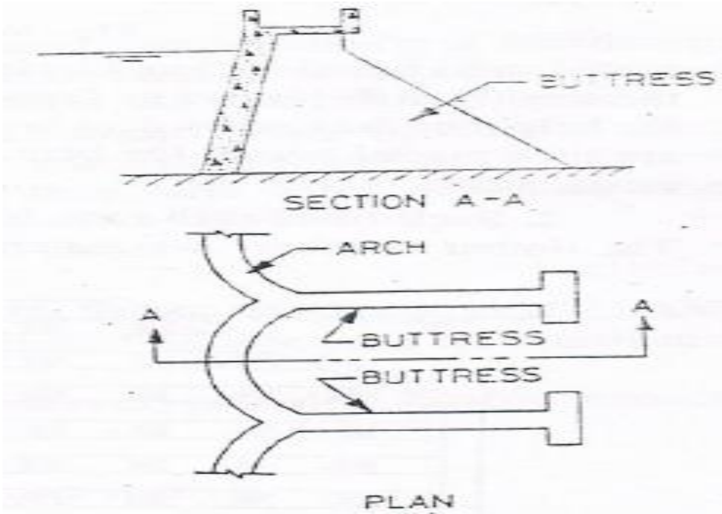


Figure1.11 Multiple-arch type buttress dam

3. Massive - head type: There is no deck slab. Instead of the deck, the upstream edges of the buttresses are flared to form massive head.

   Note: The buttress dam requires less concrete than gravity dam but they are not necessary cheaper because of extra cost of form work, reinforcement and more skilled labor.

1.8. Steel Dam:

Steel dam consists of a steel frame work, with a steel skin plate on it is upstream face. Steel dams are generally of two types:

1. Direct –strutted Steel dam: The water pressure is transmitted directly to the foundation through inclined strut .
2. Cantilever type Steel dam: There is a bent supporting the upper part of the deck, which is formed into a cantilever truss .This arrangement introduces a tensile force in the deck girder which can be taken care of by anchoring it into the foundation at the upstream toe. Direct strutted and cantilever steel dam shown in figure (1.12).

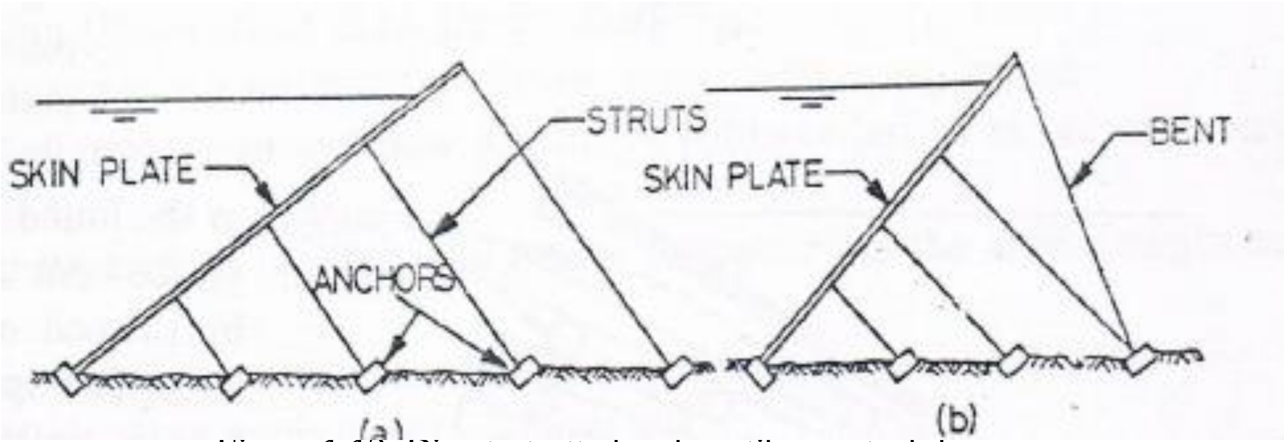


Figure 1.12 Direct strutted and cantilever steel dam

Note: The steel dam quite costly and subjected to corrosion. These dam are almost obsolete. Steel dams are sometimes used as temporary coffer dam.

1.9. Timber Dam:

A timber dam consists of a framework made of timber planks .This dams are mainly of three types :

1. A frame type : The stability depends upon the water in the deck and upon anchorage of sills . the structure of this dam as shown in figure(1.13).

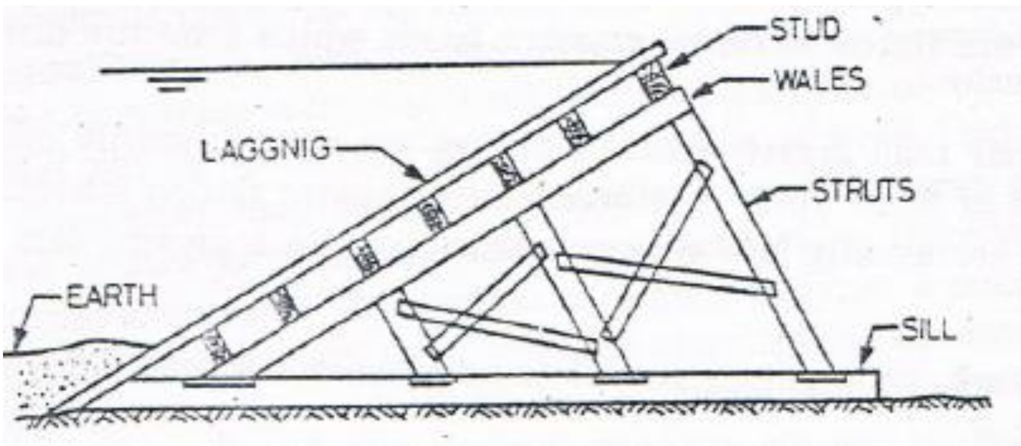


Figure 1.13 A frame timber dam

2. Rock – filled crib rock : Cribs of timber member are drift-blotted together . The bottom members of the cribs are generally pinned to

rock foundation . The space between various member is filled with rock fragment to give stability as shown in figure (1.14).

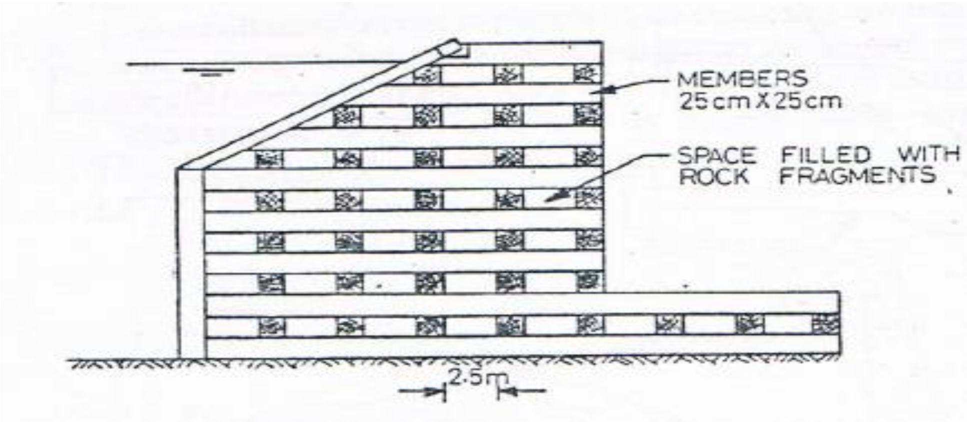


Figure 1.14 Rock-filled crib rock dam

3. Beaver type : Consists of timber members of round section forming a bent , Spacer logs are placed between the butts and drift-pinned to other logs . The bottom members are fixed to the foundation t anchor bolts as shown below in figure(1.15) .

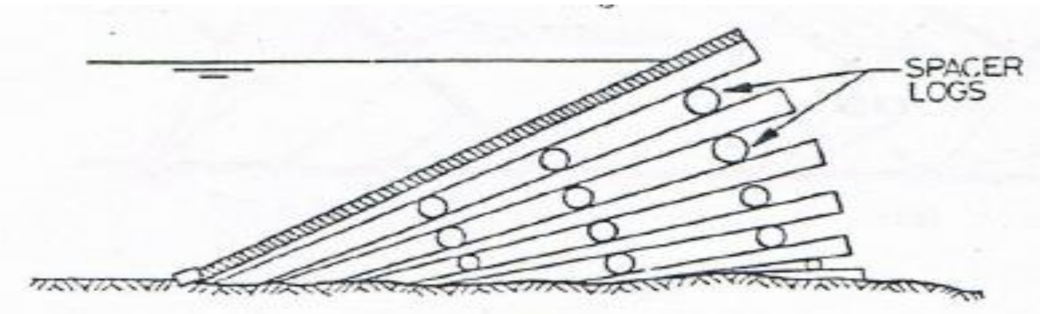


Figure 1.15Beaver dam

Advantage and Disadvantages:

Timber dams are generally used as temporary dams .These are short life but if well designed , constructions and maintained , they may last even 30-40 years . The timber dams are used at places where timber is available in plenty and the height of dam is low .

Because of shortage of timber , these dams are becoming obsolete.

1.10. Selection of Site for a Dam:

A dam is a huge structure requiring a lot of funds. Extreme care shall be taken while selecting the site of a dam. A wrong decision may lead to excessive cost and difficulties in construction and maintenance. The following factors shall be considered when selecting the site of a dam;

1. Topography: As far as possible, the dam should be located where the river has a narrow gorge which opens out upstream to create a large reservoir. In that case, the length of the dam would be small and the capacity of the reservoir on its upstream would be large.
   In case there is a confluence of two rivers in the selected reach, the dam should be located downstream of the confluence to take advantage of the flow of both rivers. The dam should be preferably located where the river bed is high, to reduce the height and cost of the dam.
2. Suitable Foundation: Suitable foundation should exist at the site for the particular type of dam. If suitable foundation is not available but it can be improved by adopting various measures, the site may be considered for selection. However, in that case, the cost of such measures should not be excessive. For gravity dams of great height, sound rock is essential. However, earth dams can be constructed on almost any type of foundation provided suitable measures are adopted.
3. Spillway site: A good site for a spillway should exist at or near the darn site. The valley should be sufficiently wide to locate the spillway if it is an integral part of the dam. If the spillway is to be located separately, the best site of spillway is that in which there is a saddle near the dam site which is separated from it by a hillock. In

that case, the main dam can be located in the gorge and the spillway can be constructed in the saddle.

4. Availability of materials: The dam requires a large quantity of material for its construction. Suitable type of material in sufficient quantity should be available at or near the dam site to reduce the cost.
5. Water tightness of reservoir : The bed and sides of the reservoir should be quite watertight to reduce leakage losses of the stored water .
6. Shape of reservoir basin : The reservoir basin on the upstream of the dam should preferable be cup-shaped , with a flat bottom but steep slopes.
7. Small submerged area : The area submerged by the reservoir on the upstream of the dam should be small.
8. Accessibility: The site should be easily accessible. It should be preferably well-connected by a road or a railway line. This would facilitate transportation of labor, materials and machinery.
9. Healthy surroundings: The surroundings of the site should be healthy and free from mosquitos so that the laborers can comfortably live in colonies constructed near the dam site.
10. Low sediments: The dam site should be such that the reservoir would not silt up quickly.
11. Development of backward areas : For the development of a particular backward area , the dam be constructed in that region
12. Other considerations: Sometimes political considerations and public opinion may affect the site of a dam.
13. Minimum overall cost: The site should be such that it entails the minimum overall cost of the project, including subsequent maintenance. Generally, two or three probable sites are selected and

rough estimates are made. The site which entails the minimum overall cost can be tentatively selected.

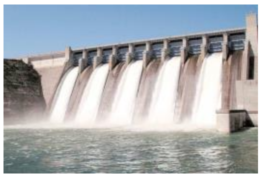
1.11. Selection Type of a Dam:

Selection of the most suitable type of dam for a particular site requires a lot of judgment and experience. It is only in exceptional cases that the most suitable type is obvious. Preliminary designs and estimates are usually required for several types of dams before making the final selection on economic basis. The salient features of different types of dams discussed in the preceding sections should be kept in mind while selecting the type of dam. Various factors which govern the selection of type of dam are discussed below:

1. Topography and valley shape. The choice or the type of dam for a particular site depends to a large extent on the topography and the valley shape. The following are the general guidelines. (a) If the valley is narrow, V-shaped and has sound rock in bed and abutments, an arch dam is generally the most suitable type. (b) If the valley is moderately wide, V-shaped and has sound rock in bed, a gravity dam or a buttress dam may be quite suitable. (c) For a low rolling plain country, with a fairly wide valley and alluvial soil or boulders in the bed. an earth dam or a rock fill dam may be quite suitable.
2. Geology and foundation conditions.A dam is a very huge structure. All the loads acting on the dam, including its own weight, are ultimately transferred to the foundations. While selecting the type of dam for a particular site, geologic character and thickness of rock, inclination of the bedding planes, existing faults and fissures, permeability of strata, etc. affect the selection.
3. Availability of construction materials. The construction of a dam requires a huge quantity of construction material. While selecting

the type of dam, the availability of the required construction materials should be considered. If a particular material is available in abundance at or near the dam site, the maximum use of that material should be made to reduce the cost. The materials which are not available near the site should be either avoided or the minimum use shall be made of such materials. For example, if suitable aggregates such as crushed stone, gravel and sand are available, a gravity dam may be suitable. On the other hand, if suitable soil is available in large quantity, an earth dam may be cheaper.

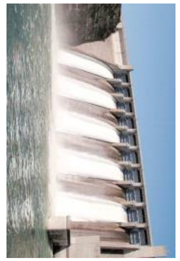
4. Overall cost. The overall cost is perhaps the most important factor which affects the selection of the suitable type of dam for a particular site. The initial cost of the dam depends upon the availability of material, the quantity of material required, labor and the construction methods. The cost of subsequent maintenance depends upon the durability of the materials used and the type of construction. The dam with a minimum overall cost is usually the best.
5. Spillway size and location. The selection of the most suitable type of dam for a particular site is sometimes governed by the size and location of spillway.
1. Earthquake hazards: If the dam site is located in a seismic zone, the most suitable type of the dam is one which can resist the earthquake shock without much damage. Earth dams and rock fill dams are generally more suitable for such sites, provided suitable modifications are made in the design. However, by adopting suitable measures and considering various forces and factors affecting the seismic design, other types of dams can also be provided.
2. Climatic conditions: Climatic conditions should also be considered while selecting the type of dam. In extremely cold climates, buttress and arch dams should be avoided.



3. Diversion problems: During the construction of the dam, the river water has to be diverted so that construction can be done in dry. If the river water cannot be diverted through a suitable tunnel (or channel) located in one of the flanks (abutments), it has to be passed over the partly constructed dam when the construction is done in the other part. In such a case, an earth dam cannot be provided, and the choice will be more in favor of a gravity dam or any other type of concrete dams.
4. Environmental considerations: The dam and its appurtenant works should be aesthetically acceptable and they should not have any adverse effect on ecology and environment. Generally, earth dams are more suitable than concrete dams for aesthetical consideration .
5. Roadway: If a wide, straight roadway is to be provided over the top of dam, an earth dam or a gravity dam is more suitable than an arch dam or a buttress dam.
6. Length and height of dam: If the length of the dam is great and the height is low, an earth dam is generally better than a gravity dam. On the other hand, if the length is small and the height is great, a gravity dam is better.
7. Life of dam: If the expected life of the project is long, a concrete dam is usually preferred. Earth and rock fill dams have moderate life, whereas timber dams have short life.
8. Miscellaneous consideration : If earth –moving machines are cheaply available at the site , an earth dam may be preferred. Similarly, if mixing plants, batching plants , etc. available near the site , a gravity dam or buttress dam may be selected.

1.12. Salient Feature of Types of Dam:

Table (1.1) describe the properties of different types of dam .



Salient Features of Various Types of Dams

S. No.	Feature	Gravity dam	Earth dam	Rockfill dam	Arch dam dam	Deck-slab buttress dam	Multiple-arch buttress dam	Steel dam	Timber dam
1.	Foundation	Sound bed rock	Any type	Any type	Sound rock in bed and flanks	Good rock	Good rock	Sound rock	Any type
2.	Valley	Gorge, moderately, wide	Wide-valley, with gentle slopes	Wide valley with gentle slopes	Very narrow, V-shaped valley	Wide valley	Wide valley	Any valley	Any valley
3.	Shape of cross-section	Triangle, with nearly vertical u/s face	Trapezium, with small top width	Trapezium, with small top width	Triangle, with different U/S & D/s slopes	Sloping, triangular slabs and buttresses	Sloping arches and buttresses	Sloping plate, with supporting frames	Sloping plank with supporting frames
4.	Initial cost	High	Low	Low	Less than gravity dam	Low	Low	Low	Low
5.	Maintenance cost	Low	High	Less than earth dam	Low	Moderate	Low	High	High
6.	Life	Long	Long	Moderate	Moderate	Moderate	Long	Long, if properly maintained	Short
7.	Kind of labour	Skilled	Unskilled	Unskilled	Highly skilled	Highly skilled	Highly skilled	Highly skilled	Skilled
8.	Facility of inspection	Not good	Not good	Not good	Not good	Good	Good	Good	Good
9.	Subsequent raising	Difficult	Easy	Easy	Not possible	Possible	Not possible	Easy	Easy
10.	Resistance against Temperature Changes and Settlements	Not good	Good	Good	Not good	Good	Good	Good	Good

CHAPTER TWO

EMBANKMENT DAMS

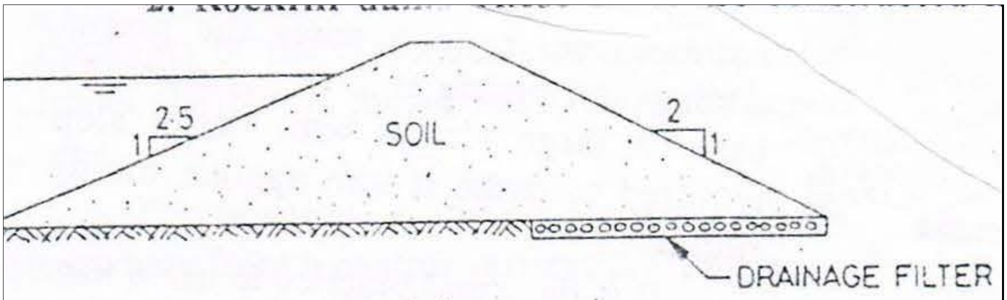
EMBANKMENT DAMS

2.1. Introduction:

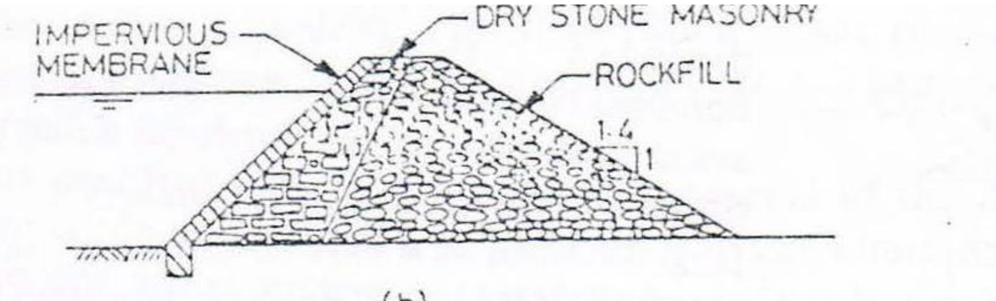
Embankment dam, build of soil or rockfill or both. As the soil and rockfill are non-rigid materials. The earth dams are more suitable than gravity dams if a strong foundation at a reasonable depth is not availab le at the site for the construction of a gravity dam. Earth dams are usually cheaper than gravity darns if the soil in abundant quantity is available near the site. Rockfill dams require somewhat stronger foundations as compared to earth dams, but the foundations need not be as strong as those for gravity dams: Rockfill dams are more economical than gravity dams if rockfill is easily available at the dam site.

Embankment Dams: are classified according to material of construction to:

1. Earth Dams: it is made of soil, Fig. 2.1 (a)
2. Rockfill Dams: it is made of rockfill , Fig. 2.1 (b)
3. Composite Earth And Rockfill Dam Fig 2.1 (c) .



(a)



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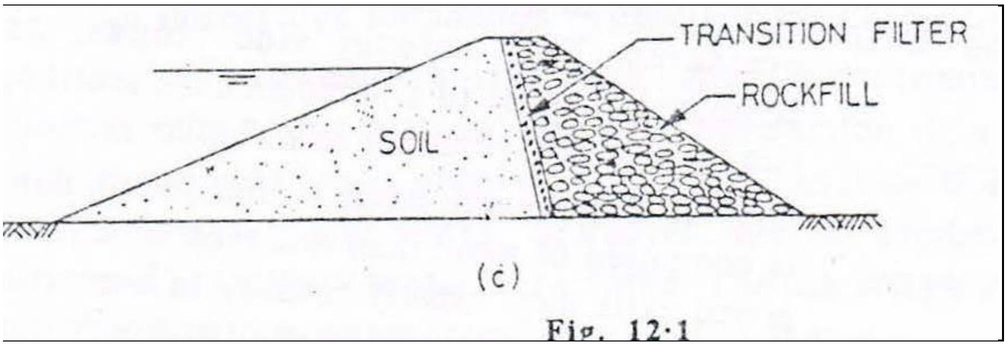
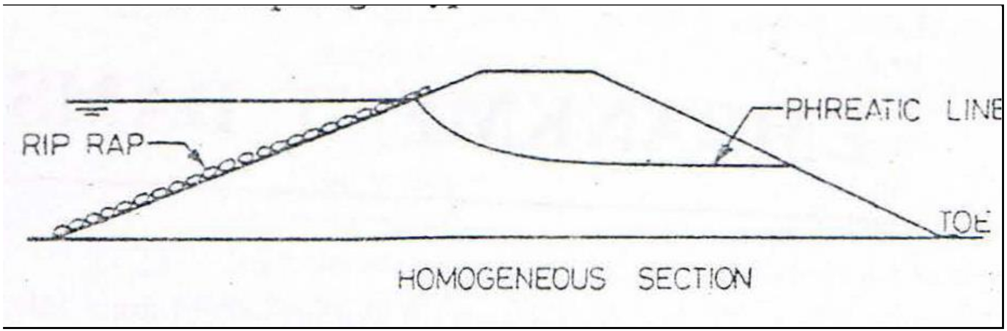


Figure (2.1) (a) Earth dam. (b) Rockfill dam. (c) Composite Earth And Rockfill Dam

2.2. Types of Earth Dams:

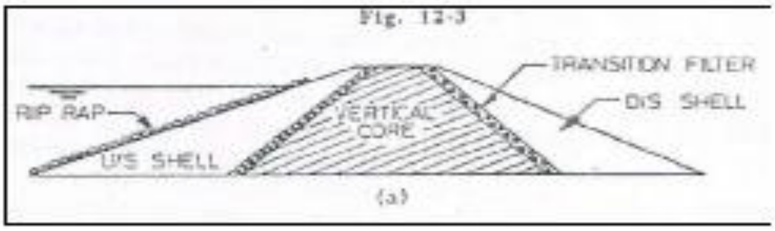
Figure (2.2) Earth Dam.

1. Earth Dams can be divided into the following, depending upon the section of the dam.
   a. Homogeneous earth dams.
   b. Zoned earth dams.
   c. Diaphragm-type earth dams
   a. Homogeneous earth dams, is composed of only one material Figure 2.3 a. such as sand and sand gravel mixture. A homogeneous earth dam is usually constructed where only one type of material is economically available near the dams site and the height of the dam is low.



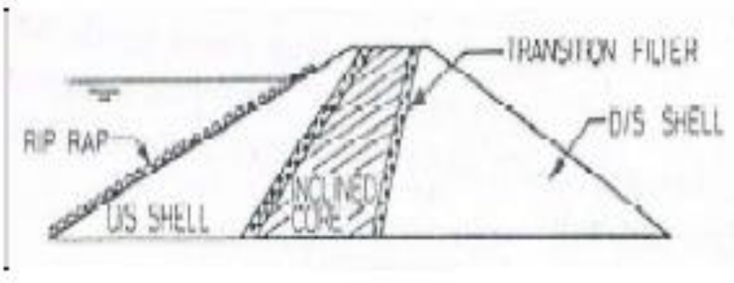
(a)

b. Zoned earth dams, A zoned earth dam is composed of more than one type of soil and usually consists of central impervious core flanked by shells of pervious materials on the upstream and downstream sides Figure 2·3 (b),(c). A transition filter is usually required between the core and the shell to prevent piping. The central core checks seepage through the dams. It is constructed of clay, silt, silty clay or clayey silt. The pervious shell gives stability to the dam and it consists of sand, gravel or a mixture of these materials. The upstream pervious zone provides free drainage during sudden drawdown. The downstream pervious zone acts as a drain to control the phreatic line. The pervious zones give stability to the core and also distribute the load over a large area of foundation. The transition filters prevent the migration of the core material into the pores of the shell material. The downstream transition filter is useful during the steady seepage conditions and the upstream filter is useful during the sudden drawdown conditions.



(b)

c. Diaphragm-type earth dams , A diaphragm-type earth dam consists of a thin impervious core. called diaphragm. If the thickness of the core at any elevation is less than the height of the embankment above that elevation or 10m, the dam is generally considered to be of the diaphragm type. On the other hand, if the thickness of the core equals or exceeds these limits, the dam considered to be of the zoned type . The diaphragm is usually constructed of the impervious soil or concrete, steel, timber and bituminous concrete. The position of the diaphragm may be a central vertical or inclined position directly on the upstream face .The construction of an internal diaphragm of the impervious soil and the transition filters is quite difficult, as it requires a high degree of precision and control.

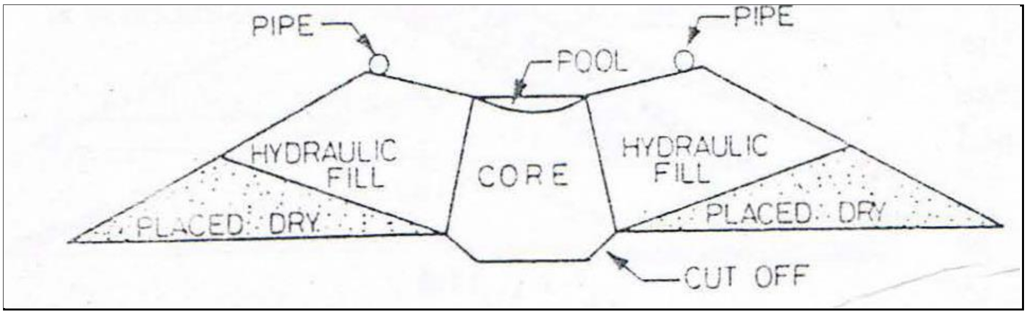


(c)

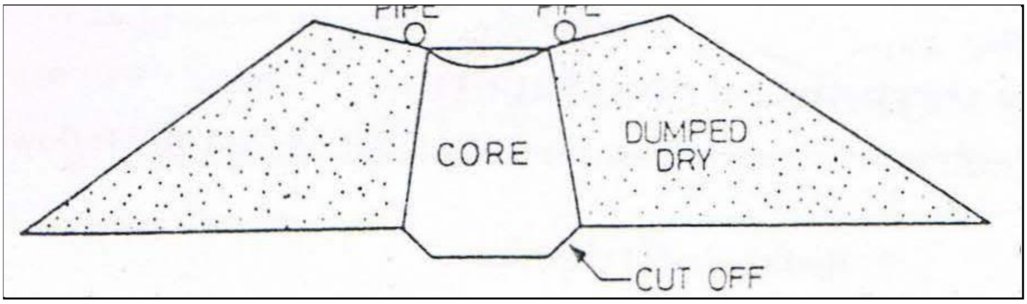
Figure (2.3) (a) Homogeneous earth dams.(b) Zoned earth dams.(c) Diaphragm-type earth dams

2. The earth dams may be classified based on methods of construction into the following:
    a. Rolled-fill earth dam.
    b. Hydraulic-fill earth dam.
    c. Semi-hydraulic earth dam
    a. Rolled-fill earth dam, these dams are constructed by thin layers, about 15 to 45 cm thick, and compacting each layer to the required dry density with heavy rollers . Most of the modern earth dams are rolled-filled dams Unless mentioned otherwise. the earth dam in this text means a rolled- fill dam.
    b. Hydraulic-fill earth dam, The material at the borrow pits is mixed with a large quantity of water to form slush or mud. This slush is transported through flumes or pipes and discharged along the outside edges of the fill of the earth dam {Figure 2 ·4 a}, As soon as the slush comes out the pipes, the coarser materials are deposited near the pipe exit. The main disadvantages of the hydraulic-fill dam is that the fill is saturated when placed and a very high pore-water pressure develops in the core material However, the finer materials are carried into the central pool at the location of the core.
    c. Semi-hydraulic earth dam, the coarse material is dumped from trucks into the required position to form shells and the jets of water are directed on the dumped fill. Because of these jets, the finer material moves towards the central portion of the dam to form an impervious core. The fines are sluiced into a core. This method of constructions, no compaction is required and if the jetting is not done properly, the dumped fill at the faces of the dam. May become more dense and impervious

than the material immediately below it. which may lead to the failure of the dam.{Figure 2.4 b}.



(a)



(b)

Figure (2.4) (a) Hydraulic-fill earth dam. (b) Semi-hydraulic earth dam

2.3. PRELIMINARYNARY SECTIONS OF EARTH DAMS:

The preliminary section of the dam is selected based on the experience, considering various factors as listed below:

i. Foundation conditions.
ii. Availability of materials.
iii. Physical properties of various materials.
iv. Method of construction and the construction control.
v. Diversion methods and construction schedule.
vi. Climatic conditions affecting the placement moisture control and subsequent moisture changes.
vii. Safety factors with respect to seepage.

viii. Safety factors with respect to stability.

The selected section should be analyses and checked to verify that it satisfies all the safety criteria with respect to seepage and stability .Figure (2.5). The following parameters are important for section of dam.

1. Crest width.
2. Free board.
3. U/s and D/s slopes of the dam.
4. Settlement allowance.
5. Cutoff wall in the foundation.
6. Impervious core.
7. Downstream drainage system.
8. Provision of rip rap.

1. Crest width is the top width of an earth dam, depends upon the following factors:

i. Nature of the embankment materials.
ii. Height of the dam.
iii. Importance of the dam.
iv. Practicability of construction.
v. Position of the phreatic line.
vi. Width of the roadway.
vii. Protection against earthquake forces .

The following empirical formulae are commonly used for the determination of the crest width.

a. For low dams (H < 10m): $\mathbf{a = 0.2H + 3}$ **…………………..(2.1)**

Where a is the crest width and H is the height of dam.

b. For medium dams (10 < H < 30m): $\boldsymbol{a = 0.55H^{0.5} + 0.2H \dots (2.2)}$

c. For high dams (H > 30 m): $\boldsymbol{a = 1.65(H + 1.5)^{0.5}} \dots\dots\dots (\mathbf{2.3})$

The crest width varies from 6 to 12 m, the larger values are for higher and more important

dams. In no case. the crest width should less than 4 m, which is the minimum required for maintenance work. The crest width should be adequate to withstand shock due to earthquake and wave action.

2. Free board, Free board is the vertical distance between the top of the dam and the maximum water level. The free board for the wave action is generally taken as 1·5 h_w where h_w is the height of wave. The actual free board is usually kept as follows:

Free board = 1.5 h_w + additional safety provision................(2.4)

The additional safety provision generally various from 0.6 to 3 m, depending upon the size of the reservoir, the height of the dam, The free board should not be less than 2m in any case. An additional free board allowance up to 1.5 m should be provided for dams in regions subjected to sub-zero temperatures.

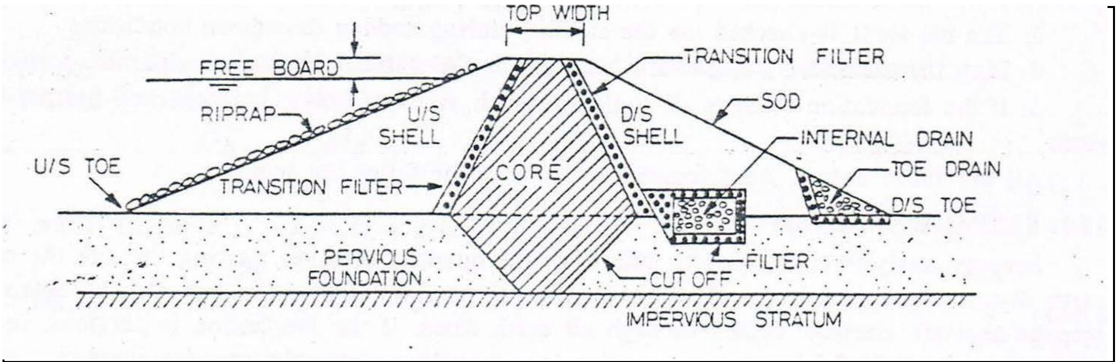


Figure (2.5) Section of Earth Dam.

3. U/s and D/s slopes of the dam, The upstream and downstream slopes of the dam depend upon the type of material, foundations conditions, the height of the dam, and many other factors. The main purpose of the shell is to provide structural supported to the core and to distribute the loads acting on the dam over a large area on the foundation. It provides stability to the dam.

Table 2.1 gives the tentative side slopes recommended by Terzaghi for the preliminary section.

Table (2.1): The tentative side slopes recommended by Terzaghi for the preliminary

S. No.	Type of section	Type of material	U/s slope	D/s slope
1.	Homogeneous section	Well-graded material	2·5:1	2:1
2.	-do-	Coarse silt	3:1	2·5:1
3.	-do-	Silty clay or clay		
		(a) Height < 15 m	2·5:1	2:1
		(b) Height ≥ 15 m	3:1	2·5:1
4.	Zoned section	Sand or gravel shells with clay core	3:1	2·5:1
	-do-	Sand or gravel shells, with R.C.C. core	2·5:1	2:1

The U/S slopes of most of the earth dams in actual practice usually vary from 2.5:1 to 4:1, and the D/S slopes are generally between 2:1 and 3.0 : 1.The stability of the slope depends mainly on the physical properties (c and Φ) of the materials . The coarse-grained materials can have steeper slopes as compared to the fine-grained materials. In the case of a zoned section, the slopes are relatively steeper as compared to a homogeneous section of the same material. If the foundation material is weak, The slopes will be kept relatively flat to reduce the stresses in the foundation. On the other hand, in the case of strong foundations, the slopes can be relatively steep. The side slopes in high dams are generally made steeper at the upper elevations and flatter at the lower elevations. Variable side slopes may prove advantageous for earth dams having a height greater than 30 m.

4. Settlement allowance, Settlement of an earth dam occurs due to consolidation of the soil mass in the dam and foundation. The magnitude and rate of the settlement depend upon a number of factors such as the character of the soil in the dam and foundation, the drainage conditions, the height of the dam, the depth of weak strata and the method of

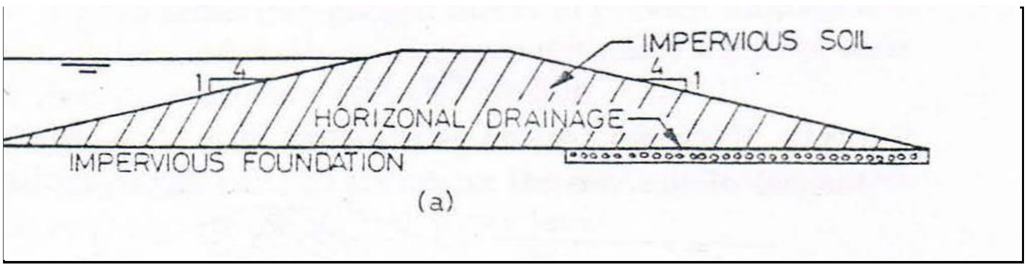
construction in the case of dams of height greater than 30 m, an extra allowance of 1% is made to account for the settlement due to earthquakes.

5. Foundations, To reduce seepage through the foundation and to avoid piping failure, can be used the following requirement based to type of foundation , Figure (2.6):

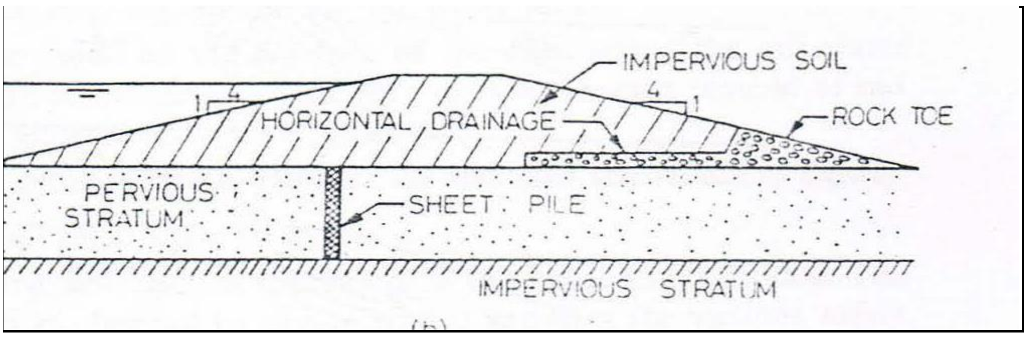
a. Impervious foundation, The horizontal drainage blanket is extended deep inside the body of the dam to reduce the pore water pressure and to increase the stability of slopes.

b. Pervious foundation with large depth, impervious blanket can be used. A positive cutoff wall up to the impervious stratum is not practicable. In that case, an u/s impervious blanket is provided on the river bed as an extension of the central core of the dam.

c. Foundation pervious with moderate depth, cutoff trench is provided in the foundation to the impervious stratum. Generally, the core is extended down to form a cutoff. For drainage system , chimney drain and horizontal drainage blanket can be used. steel sheet pile can be used instead of cutoff trench but if soil consists boulders , the sheet pile would bend and will not be effective.



(a)



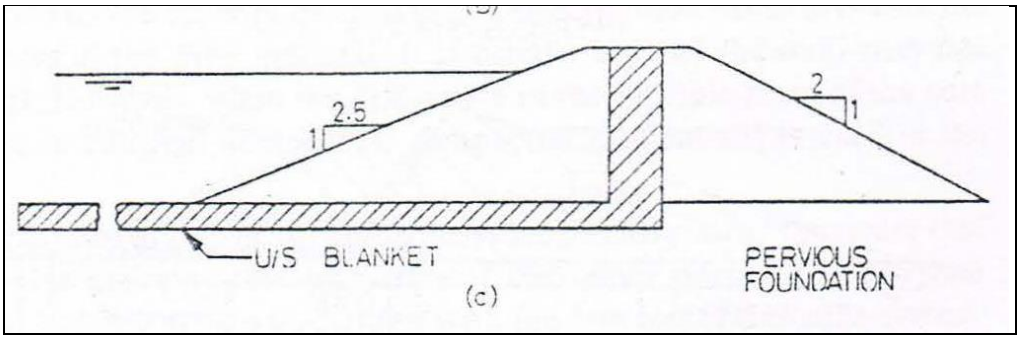


Figure (2.6) (a) Impervious foundation.(b) Pervious foundation with large depth. (c) Foundation pervious with moderate depth.

6. Impervious core, the core may be vertical or inclined. Impervious core is provided at the center of the zoned sections to control the loss of water by seepage through the dam .The pervious zones (or shells) in the zoned-section give the stability to the dam. The d/s pervious zone also acts as a drain and allows the seepage water to pass to downstream and reduce the pore water pressure. Maximum side slopes The side slopes of the core in any case should not be greater than (x – 0·5): 1 on the upstream and (y – 0·5):1 on the downstream, where x: 1 is the u/s slope of the shell and y:1 is the d/s slope of the shell. The maximum u/s and d/s slopes of the core are usually (1.5: 1) and (1: 1) respectively.

If the difference of the particles sizes of the shell material and the core material is large, transition filters are provided, if difference is not large transition filters are omitted.

7. Downstream drainage system, the downstream drainage system is required for all types of earth dams. The drainage system consists of materials appreciably more pervious than the embankment material so that the water seeping through the embankment is easily drained out. Types of drainage systems The following types of drainage systems are used for the dam and foundation.

(a) Drainage of the dam

i. Horizontal drainage blanket.
ii. Rock toe.
iii. Chimney drain.
iv. Strip drain

(b) Drainage of the foundation

i. Toe drain.
ii. Drainage trench.
iii. Relief walls.
iv. Vertical sand drains

An internal horizontal drainage system is provided to carry away the water that seeps through the core or the cutoff. It also prevents the saturation of the upper part of the d/s shell by rain or water spray. The rock toe is also sometimes provided along with the horizontal drainage system. The drainage system prevents sloughing of the d/s face due to seepage or the rain water. Due to the provision of the graded filters, the seepage water does not carry the soil particles into the drainage system and clog it or develop seepage erosion.

Instead of the horizontal drainage system, the chimney drains, is a vertical or nearly vertical drain which is located inside the dam so that it intercepts all layers of the dam in the seepage zone. Chimney drains are usually provided d/s of the impervious core and prevents the emergence of the seepage water on the d/s face of the dam , (Figure 2.7).

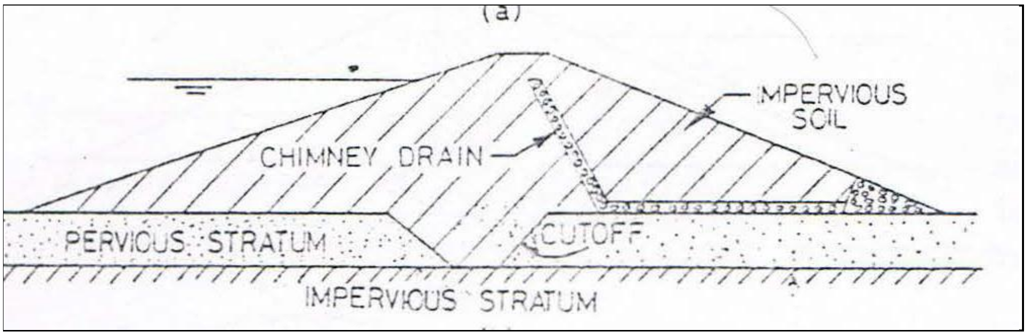


Figure (2.7) The chimney drains as drainage system.

Vertical sand drains, it provided in the foundation and consist of vertical holes drilled in the foundation all along the base of the dam.

8. provision of rip rap:

a. U/s slope The u/s slope of the dam is protected against erosion due to wave action.
b. D/s slope The d/s slope below the tail water is also usually protected against wave action by providing rip rap.
c. The top of the earth dams as well as the d/s face above the tail water level are exposed to atmosphere. They require protection against erosion due to rain and wind. A rip rap may be provided for protection if the stone is easily available. Rip rap consists of large stones either dumped or properly placed is layers, also called pitching (or fretting). Generally, dumped rip rap is more stable and is preferred. (Figure 2.8).

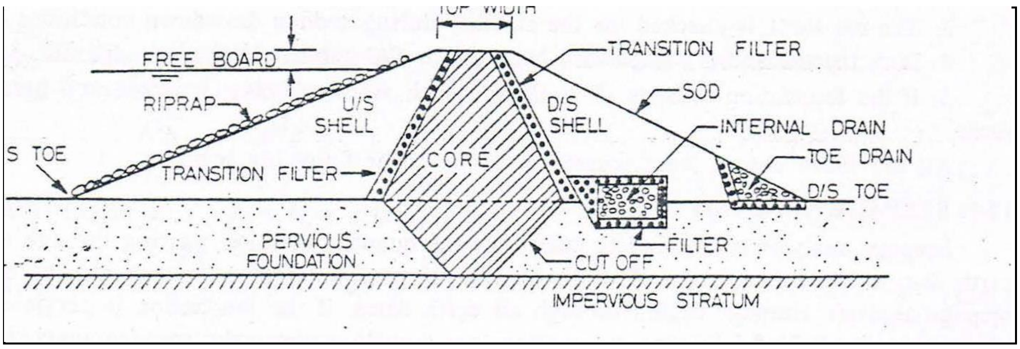


Figure (2.8) provision of rip rap

2.4. CAUSES OF FAILURE OF EARTH DAMS:

Earth dams are very huge earth structures which store a large quantity of water in the reservoir upstream. Their failure may cause vast damage, catastrophe and loss of human lives. Every precaution shall be taken that failures do not occur. Earth dam failures are mainly caused by improper design, lack of thorough investigations, inadequate care in construction and poor maintenance. Various causes of failures can be grouped into three categories:

1. Hydraulic failures.
2. Seepage failures.
3. Structural failures.

On the basis of investigations carried out on the actual failures of a large number of earth dams, it has been found that about 35% of the failures are hydraulic failures, about 38% are seepage failures and about 20% are structural failures. The remaining 7% of the failures are due to other miscellaneous causes.

1. **Hydraulic failures**, The hydraulic failures may occur due to one or more of the following causes:
   i. Overtopping.
   ii. Erosion of U/s face.
   iii. Erosion of D/s face.
   iv. Frost action.
   v. Erosion of D/s toe

i. Overtopping: An earth dam fails as soon as its overtopping occurs. Overtopping of the earth dam occurs if

a. The design flood is underestimated.
b. The spillway capacity is not adequate.

c. The spillway gates are not properly operated.

d. The free board is not sufficient.

e. Excessive settlements of the foundation and dam occur.

ii. **Erosion of U/s face:** the erosion of the upstream face by waves, rip rap is laid on a filter bed to prevent washing out of the soil below during sudden drawdown. If the rip rap thickness is not adequate or If the filter bed is not properly designed, the waves in the reservoir may cause erosion of the upstream face.

iii. **Erosion of D/s face:** Erosion of the downstream face may occur due to rains. Sometimes erosion of the downstream face also occurs because of high winds. To avoid erosion of the downstream face, Suitable berms are also provided and pitching (or rip rap) is provided up to a height slightly above the normal tail water depth.

iv. **Frost action:** If the earth dam is located at a place where the temperature fall is below the freezing point frost may form in the pores of the soil in the earth dam. When there is heaving, the cracks may form in the soil. To avoid the failure due to frost action, the soil susceptible to frost formation should not be used. A suitable extra free board should be provided for dams in regions of low temperature where frost is likely to form.

2. **Seepage failures**, Seepage failures may occur due to the following causes:

i. Piping through the dam.

ii. Piping through the foundation.

iii. Conduit leakage.

iv. Sloughing of downstream toe.

i. **Piping in the dam may occur due to one or more of the following causes:**

a. Poor construction: If the soil of the dam is not properly compacted, piping may occur.

b. Differential settlement Cracks may develop in the dam due to differential settlement in the foundation of the dam, which may lead to piping failure.

c. Burrowing animals.

d. Surface cracks Shrinkage and drying cracks in the soil surface may lead to piping failure.

e. Presence of roots, etc. The presence of roots of the trees, pockets of gravels or boulders in the embankment may lead to piping failure

f. Soluble salts: If there are soluble salts in the soil, they get leached out due to which hollows are created in the soil, which may lead to failure

ii. **Piping through the foundation**, occurs when the rate of pressure drop (i.e. hydraulic gradient) resulting from seepage through the foundation exceeds the resistance of the soil particles. Piping in the foundations may also occur when there are pockets of loose soil in the foundation.

iii. **Conduit leakages**, sometimes outlet(conduits) are provided through the earth dam. Cracks may develop in these conduits due to foundations settlements or due to the deterioration of the conduit itself. Leakage occurs through these cracks, which may lead to the failure of the dam or if there is a gap between the conduit and the surrounding soil. Such a gap usually occurs when the soil mass shrinks away from the underside of the conduit. The seepage occurring along the conduit walls may also lead to piping failure or

due to unequal settlement of the dam or due to overloading of the dam.

3. **Structural failures**, Structural failures in earth dams are generally shear failures leading to sliding of the embankments or the foundations. Structural failures in the earth dams are of the following types:
   i. Slides in embankments.
   ii. Foundation slides.
   iii. Liquefaction slides.
   iv. Failures by spreading.
   v. Failure due to earthquakes.
   vi. Failure due to holes caused by burrowing animals.
   vii. Failure due to holes caused by leaching of salts.

   i. **Slides in embankments**, Sliding of the slopes of the embankment occurs when the shear forces tending to cause sliding on any potential sliding surface exceed the resisting forces. The failure of slopes is the one of main causes of the failure of the earth dams . About 15% of the failures in the past occurred due to slope failure.

   The following three types of slope failure are quite common, Figure (2.9):

a. Failure of U/s slope during sudden drawdown.
b. Failure of D/s slope during steady seepage.
c. Failure of U/s and D/s slopes during construction

(a) FAILURE OF U/S SLOPE DURING SUDDEN DRAWDOWN

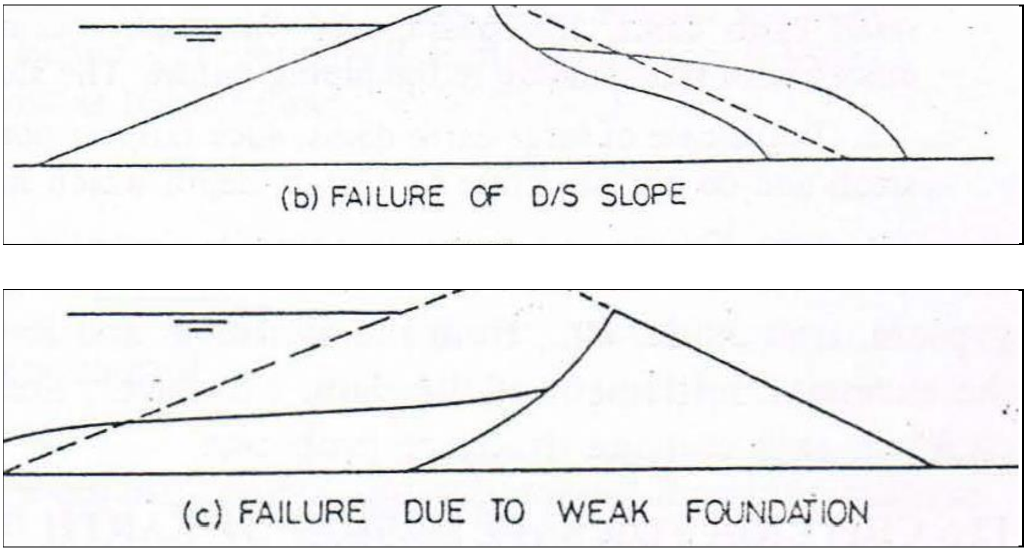


Figure (2.9) (a) Failure of U/s slope during sudden drawdown. (b) Failure of D/s slope during steady seepage.(d) Failure due to weak foundation.

ii. **Slides in foundation**, Sliding failure may occur in the foundation of an earth dam if it is weak {Fig. 2.9 c} If the foundation consists of fine sands and soft soil, the slide may occur in the foundation.

iii. **Liquefaction slides**, If the foundation consists of fine sand and silt in loose condition, liquefaction slides(or flow slides) may occur when the dam is subjected to vibrations.

iv. **Failure by spreading,** Failure by spreading usually occurs when the earth dam is located above a stratified deposited that contains layers of silt clay.

v. **Failure due to earthquakes,** Failure due to earthquakes may occur in the following cases :

a. Earthquakes may cause cracks in the core of the dam, leading to leakage and piping.
b. Earthquakes may cause excessive settlement of the dam due to which free board is reduced and overtopping may occur.

c. Earthquakes cause shaking of the reservoir bottom due to which slow waves (or seiches) are formed, which may cause overtopping.
d. Earthquakes may cause liquefaction of loose sand in foundation and consequent liquefaction slides.
e. Inertia forces due to the earthquake acceleration may cause shear slides.
f. Rockslides in flanks may occur due to earthquakes, causing a rise of the water level in the reservoir because of falling of the rock into the reservoir and overtopping of the dam may occur.

2.5. CRITERIA FOR SAFE DESIGN OF EARTH DAMS:

For the safe design of an earth dam, the following basic criteria should be satisfied.

1. No overtopping:

a. The dam should be safe against overtopping during occurrence of the worst flood.
b. An adequate free board should be provided so that the dam is not overtopped due to the wave action.
c. A suitable allowance in the height of the dam should be made to account for settlement.

2. No seepage failure:

a. The phreatic line (or the seepage line) should remain well within downstream face of the dam so that no sloughing of the downstream face occurs.
b. Seepage through the body of the dam, foundations and abutment should be controlled by adopting suitable measures.
c. The dam and foundation should be safe against piping failure.

d. There should be no opportunity for the free passage of water from the upstream to the downstream either through the dam or through the foundation.

3. No structural failure:

a. The upstream and downstream slopes should be safe during and immediately after construction.

b. The upstream slope should be safe during sudden-drawdown conditions.

c. The downstream slope should be safe during steady-seepage conditions.

d. The foundation shear stresses should be within the safe limits.

e. The dam as a whole should be earthquake-resistant.

4. Proper slope protection:

a. The upstream slope should be protected against erosion by waves.

b. The downstream slope and the crest(i.e. top) should be protected against erosion due to rain and wind.

5. Proper drainage portion of the dam downstream of the impervious core should be properly drained.

6. **Economic section**, The dam should have an economic section. As far as possible, the materials available near the dam she should be used to reduce the cost.

2.6. The Slope Stability of the Earth Dam:

1. Swedish Circle Method.

The Swedish circle method is the most common. The method for checking the stability of a slope. The method is general and can be used for homogeneous as well as non-homogeneous soil masses, stratified deposits, for partially or fully sub-merged conditions or dry conditions. However, the method is necessarily an approximate one because it

neglects the effect of forces acting on the sides of the vertical strips. Fortunately, the Swedish slip circle method errs on the safe side, because the factor of safety obtained by this method is less than that obtained from the more accurate methods. The method, therefore, can be safely used in practice.

In the Swedish slip circle method, the least factor of safety of about 1·3 to 1.5 is usually specified in the stability analysis of the earth dams.

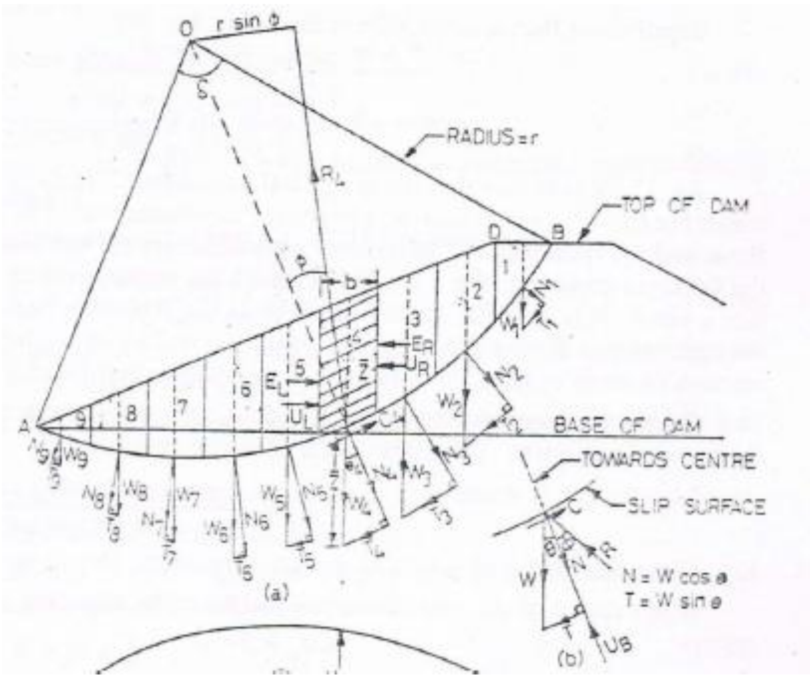


Figure 2.10 the stability of slopes of the earth dam.

In the conventional Swedish circle method, it is assumed that the reactions ER and EL are equal and opposite and cancel each other and do not affect the equilibrium. Likewise, it is assumed that the forces due to pore water pressure UL and UR on the two sides also balance each other and do not affect the equilibrium. Thus, there are 4 forces which are normally considered, W4, C4, R4 and UB. The force UB due to the pore water pressure is zero if the soil is dry, and the number of the forces in that case reduces to only three.

Figure 2.10 shows a portion of the slip surface for a slice. Let us resolve the weight W into its normal component N and the tangential component T. As the normal of a circle passes through the center, the direction of N is first marked in the normal direction. A perpendicular is drawn from the tip of the vector W to the normal direction to determine N and T. Thus

$$\boldsymbol{N = W \cos\theta} \qquad \text{and} \qquad \mathbf{T = W\ Sin\theta}$$

Where the normal components N are calculated considering the submerged unit weight w_s or γ_{sub} of the soil and the tangential components T are calculated considering the saturated unit weights w_{sat} or γ_{sat} of the soil. Then factor of safety of slope is determine from:-

$$\boldsymbol{F_s = \frac{\tan\phi\ \Sigma(W\ \cos\theta - ub\ \sec\theta) + \Sigma cb\sec\theta}{\Sigma W\ \sin\theta}} \qquad \mathbf{(2.5)}$$

If the soil is dry, the pore water pressure is zero and Eq. 2.5 becomes:

$$\boldsymbol{F_s = \frac{\tan\phi\ \Sigma\ W\ \cos\theta + \Sigma cb\sec\theta}{\Sigma W\ \sin\theta}} \qquad \mathbf{(2.6)}$$

Procedure:

The procedure for the determination of the factor of safety of the trial slip surface may be summarized as follows:

1. Take a trial slip surface and divide the wedge above the slip surface into 8 to 15 vertical slices.
2. Determine the weight W of each slice and its line of action. For convenience, the weight of the slice is generally taken proportional to the middle ordinate of the slice and its line of action is taken through the middle of the slice.

$$\boldsymbol{W = (b * Z') * \gamma} \qquad \textbf{(2.7)}$$

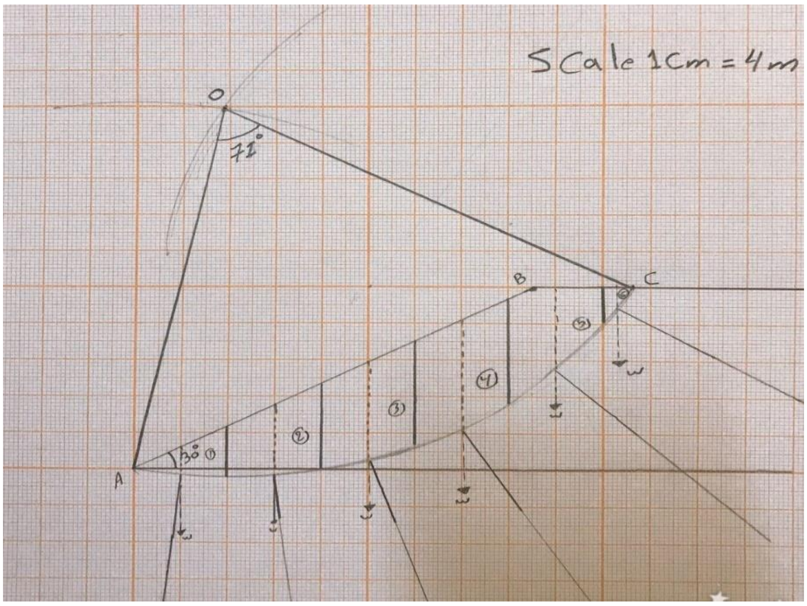
Where $\acute{Z}$ is the middle ordinate of the slice, b is the width of the slice, and γ is the unit weight of the soil.

3. Measure the angle θ which the normal makes with the vertical and compute the normal component N and the tangential component T.
4. Determine the cohesive force, **C = c b sec θ.**
5. Determine the factor of safety for the trial slip surface from Eq. 2.6.
6. Repeat the above procedure for a number of trial surface. The trial surface which gives the minimum factor of safety is the most critical circle. The minimum factor of safety should be greater than the specified safe value.

Example (1):

The earth dam has been made at an angle of 30° to the horizontal radius is 40m passing through the toe of the earth dam slope and through a point 8m away on the crest from the edge of the earth dam. c=15 kN/m², ø= 30°, γ=18kN/m³ depth of the earth dam 20. Check the stability of the upstream face of the dam by Swedish Circle Method.

Solution:



Slice no.	Ź (m)	b (m)	W (kN)	θ Deg.	N= W*cosθ	T= W*sinθ	N tanø	b secθ	c b secθ
1	0.7*4= 2.8	2*4=8	403.2	-5	401.6	-35.14	231.86	8.03	120.45
2	7.6	8	1094.4	6	1088.4	114.39	628.38	8.04	120.6
3	10.8	8	1555.2	17	1487.24	454.69	858.65	8.36	125.4
4	12	8	1728	29	1511.34	837.75	872.57	9.14	137.1
5	9.6	8	1382	44	994.41	960.29	574.12	11.12	166.8
6	2.4	0.7*4= 2.8	120.96	58	64.1	102.57	37	15.1	226.5
						2434.55	3202.58		896.85

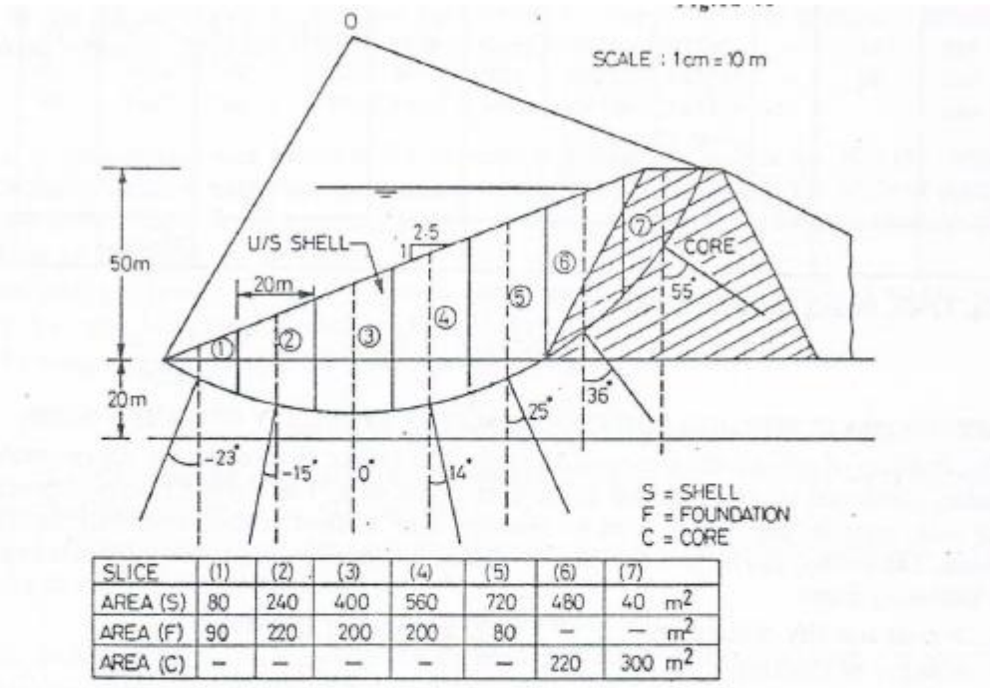
From Eq. (2.6)

$$F_s = \frac{3202.58+896.85}{2434.55} = 1.6 > 1.5 \text{ (safe).}$$

Example (2):

Check the stability of the upstream face of the dam for the assumed surface shown in Fig. below. The properties of the shell, core and foundation materials are as follows:

	Shell	Core	Foundation
Saturated unit weight (kN/m^3)	22	21	22
Submerged unit weight (kN/m^3)	12	11	12
Moist unit weight (kN/m^3)	21	20	21
Angle of internal friction	30°	20°	30°
Unit cohesion (kN/m^2)	10	60	10



Solution

The soil wedge above the slip surface is divided into 7 slices. The slices have been taken such that the base of each slice is in one type of material. The computations for the stability analysis are shown in the table below. The weight of each slice is computed from its area and the corresponding unit weight. The values of ø and *c* are taken corresponding to the material through which the base of that slice passes.

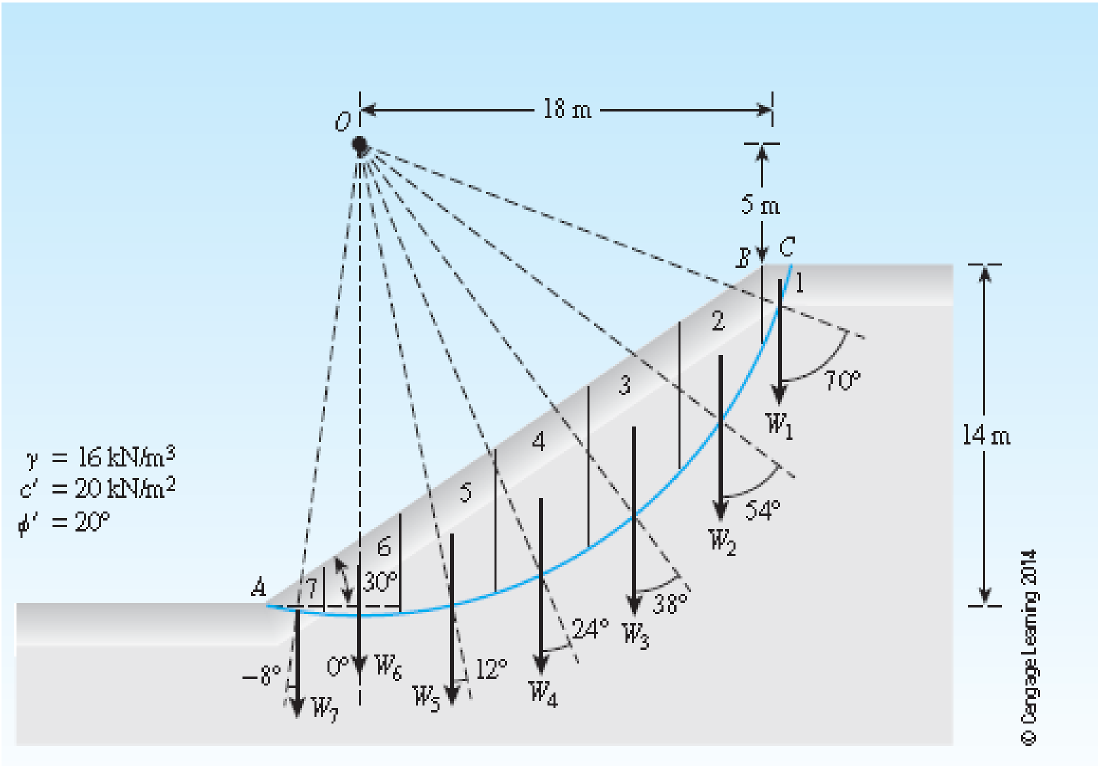
S.No.	Area of slice (m^2)			Weights (kN)		θ	$N' = W' \cos\theta$	$T = W \sin\theta$	ϕ	$N' \times \tan\phi$	c	$b \times \sec\theta$	$c\ b \times \sec\theta$
	Shell (S)	Foundation (F)	Core (C)	Saturated Weight (W)	Submerged Weight (W')								
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	80	90	–	3740 (=170×22)	2040 (170×12)	−23°	1878	-1461	30°	1084	10	21·7	217·0
2	240	220	–	10120	5520	−15°	5332	-2619	30°	3078	10	20·7	207·0
3	400	200	–	13200	7200	0	7200	0	30°	4157	10	20·0	200·0
4	560	200	–	16720	9120	+14°	8849	4045	30°	5109	10	20·6	206·0
5	720	80	–	17600	9600	+25°	8700	7438	30°	5023	10	22·1	221·0
6	480	–	220	15180 (=480×22 +220×21)	8180 (=480×12 +220×11)	+36	6618	8923	20°	2409	60	24·7	1482
7	40	--	300	7180	3780	+55°	2168	5882	20°	789	60	34·9	2094
						Σ =	40745	22208		21650			4627

From Eq. 2.6

$$F_s = \frac{21650+4627}{22208} = 1.18 < 1.5 \text{ (unsafe).}$$

Example (3):

For the slope shown in Figure below, find the factor of safety against sliding for the trial slip surface AC.



Slice no.	1	2	3	4	5	6	7
b m	1	4	4	4	4	4	3.2
Z m (mid)	1.4	4.6	6.8	6.8	6.1	4.2	1.3

Solution:

Slice no.	W kN	ϴ Deg.	sinɵ	cosɵ	b*secɵ	T=W*sinɵ	N=W*cos ɵ
1	22.4	70	0.94	0.342	2.924	21.1	7.66
2	294.4	54	0.81	0.588	6.803	238.5	173.1
3	435.2	38	0.616	0.788	5.076	268.1	342.94
4	435.2	24	0.407	0.914	4.376	177.1	397.8
5	390.4	12	0.208	0.978	4.09	81.2	381.8
6	268.8	0	0	1	4	0	268.8
7	66.56	-8	-0.139	0.99	3.232	-9.25	65.9
∑					30.501 m	776.75 kN	1638kN

$F_s = \frac{tan20*1638+30.501*20}{776.75} = 1.55 > 1.5$ (safe).

2. Approximate Method:

The stability of u/s and d/s slopes of an earth dam can be checked by the approximate method by considering horizontal shear developed at the base of the dam. The method is very approximate and rarely used in practice. The method can be used for the preliminary design or analysis. The method is explained below for the following cases:

i. Overall stability of the dam.
ii. Stability of D/s slope.
iii. Stability of U/s slope.

I. Overall stability of the dam: The overall sliding stability of the earth dam is determined considering it as a solid body. The factor of safety against sliding is defined as the ratio of the force resisting sliding of the dam to the force tending to cause sliding (Figure 2.11). Thus the factor of safety:

$$F_S = \frac{\text{Resisting force}}{\text{Sliding force}} \qquad (2.8)$$

The resisting force is taken as the shear resistance developed at the base of the dam given by:

$$\textbf{Resisting force} = \mathbf{W \tan ø} \qquad (a\ldots 2.8)$$

Where W is the effective weight of the entire dam per unit length and ø is the angle of internal friction of the soil of which the dam is composed of.

The force tending to cause sliding of the dam is the horizontal component of the water pressure acting on the upstream given by:

$$P_H = \frac{\gamma h^2}{2} \qquad (b\ldots..2.8)$$

Where h is the maximum depth of water and γ is the specific weight of water.

Thus, the factor of safety is given by

$$F_S = \frac{W \tan ø}{\frac{\gamma h^2}{2}} \qquad (2.9)$$

The factor of safety should not be less than 2.

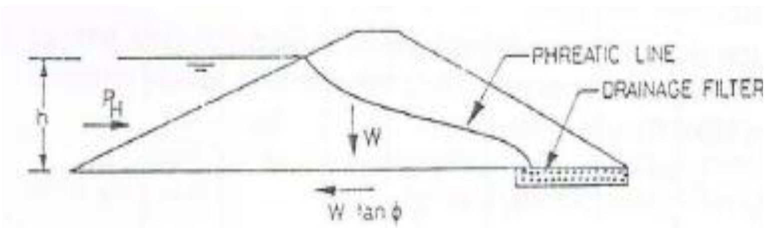


Figure 2.11 Overall stability of the dam.

II. Stability of the downstream slope Fig. 2.12 shows the downstream slope of the dam separated from the rest of the dam by a vertical

plane passing through the top shoulder of the d/s slope. The phreatic line during the steady seepage condition lying in the d/s slope of the dam is also shown. The average factor of safety Fs against shear is given by:

$$\boldsymbol{F_S = \frac{R_d}{H_d} = \frac{W_d \tan\emptyset + c b_d}{\frac{w_S H^2}{2} tan^2\left(45^\circ - \frac{\emptyset}{2}\right) + \frac{\gamma h_1{}^2}{2}}} \qquad \textbf{(2.10)}$$

Where

R_d: The resisting force against sliding is developed due to internal friction and cohesion of the soil mass in the d/s portion of the dam.

H_d: A horizontal force acts on the vertical plane A B in the downstream direction due to earth pressure caused by the left portion of the dam and due to water pressure.

W_d: the total effective weight of the d/s portion of the dam above the base.

c: the unit cohesion.

b_d: the width of the base of the d/s portion.

Ø: the angle of internal friction of the soil in the dam.

w_s: the weighted unit weight of the soil mass at the vertical section AB.

H: the vertical distance from the top of the dam to the base of the dam (= height of the dam).

γ: the unit weight of water.

h_1 : the vertical distance from the phreatic line to the base of the dam.

And $$\boldsymbol{w_S = \frac{W_1 h_1 + w_2 (H - h_1)}{H}} \qquad \textbf{(2.11)}$$

Where

w_1 : the submerged unit weight of the soil below the phreatic line.

w_2 : the moist (or dry) unit weight of the soil above the phreatic line.

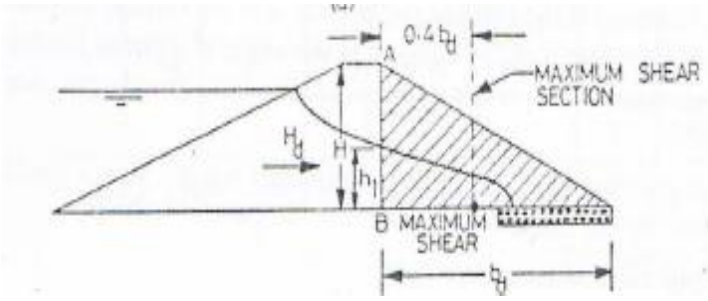


Figure 2.12 shows the downstream slope of the dam.

The factor of safety should be at least 2, because the actual failure surface is curved and the real factor of safety would be much less.

Factor of safety against maximum shear: The d/s slope should also be checked against the maximum shear which occurs at a point on the base at a horizontal distance of 0.4 b_d from the shoulder.

$$F_S = \frac{\textit{Shear strength at the point of maximum shear stress}}{\textit{Maximum shear stress}} \qquad \textbf{(2.12)}$$

The shear strength is determined by the Mohr-Coulomb shear equation as:

$$S = w_s\, h \tan \emptyset + c \qquad \textbf{(2.13)}$$

h: the vertical distance from the d/s slope of the dam down to the point of maximum shear stress.

The maximum shear (τ_{max})is twice the average shear (τ_a).

$$\tau_{max} = 2\,\tau_a = 2\,(H_d / b_d) \qquad \textbf{(2.14)}$$

$$F_S = \frac{(w_s\, h \tan \emptyset + c)\, b_d}{w_s H^2\, tan^2\left(45^\circ - \frac{\emptyset}{2}\right) + \gamma {h_1}^2} \qquad \textbf{(2.15)}$$

The factor of safety Fs against the maximum shear should be at least 1·50.

III. Stability of upstream slope Fig. 2.13 shows the upstream slope separated from the rest of the dam by a vertical plane CD passing

through the U/s shoulder of the dam. The average factor of safety F_s against shear is given:

$$F_S = \frac{R_u}{H_u} = \frac{W_u{}' \tan\emptyset + cb_u}{\frac{w_s H^2}{2} tan^2\left(45^\circ - \frac{\emptyset}{2}\right) + \frac{\gamma {h_1}^2}{2}} \qquad \textbf{(2.16)}$$

Where

Ru: A resisting force develops against the horizontal shear force.

Hu: The horizontal force acting on the upstream portion of the dam.

Wu: the total effective weight of the u/s portion of the dam above the base (the submerged unit weight of the soil).

bu : the width of the base of the u/s portion.

ws: saturated unit weight of the soil in the upstream portion.

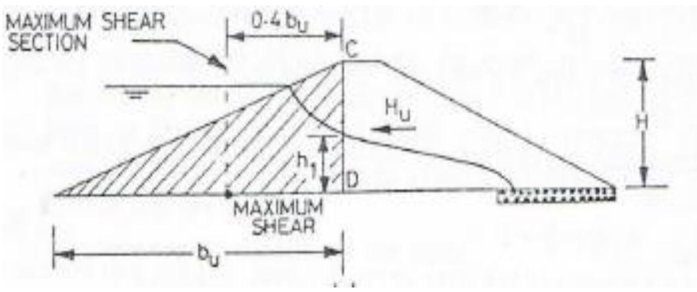


Figure 2.13 shows the upstream slope of the dam.

The factor of safety Fs should be at least 2.0.

Factor of safety against maximum shear:

The maximum shear stress occurs at a point at a distance of 0.4 b_u from the vertical line CD through the u/s shoulder. The factor of safety against maximum shear is obtained from the relation:

$$F_S = \frac{\textit{Shear strength at the point of maximum shear stress}}{\textit{Maximum shear stress}}$$

$$\text{or} \quad F_S = \frac{S_{max}}{\tau_{max}} \qquad (2.17)$$

$$S_{max} = w_s{}'\ h \tan \emptyset + c \qquad (2.18)$$

Ws’: the submerged unit of the soil,

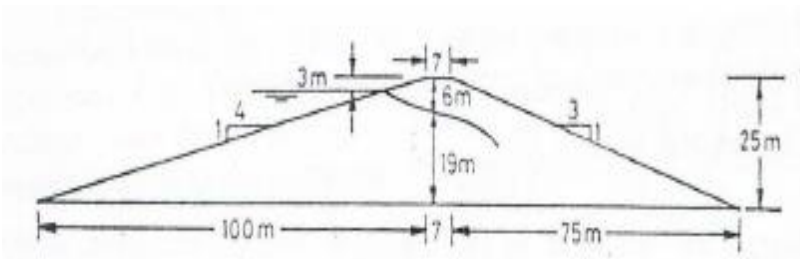
h : the height of soil column at that point.

$$F_S = \frac{(w_s{}' h \tan \emptyset + c) b_u}{w_s H^2\ tan^2\left(45^\circ - \frac{\emptyset}{2}\right) + \gamma {h_1}^2} \qquad (2.19)$$

The factor of safety Fs against the maximum shear stress should be at least 1.5.

<u>Example (4):</u>

A homogeneous earth dam is 25 m high and has a crest width of 7 m. The upstream and downstream slopes are respectively 4:1 and 3:1. Check the stability of U/s slope by the approximate method. Assume a free board of 3m and the height of the phreatic line above the base at the U/s shoulder as 19 m. Take saturated weight of soil = 22 kN/m³, submerged weight of soil = 12 kN/m³, specific weight of water = 10 kN/m³, ø = 24°, c = 50 kN/m².



Solution:

The factor of safety for the average shear

$$F_S = \frac{R_u}{H_u}$$

$$H_u = \frac{w_s H^2}{2} tan^2 \left(45^\circ - \frac{\emptyset}{2}\right) + \frac{\gamma {h_1}^2}{2}$$

$$H_u = \frac{22\ (25)^2}{2} tan^2 \left(45^\circ - \frac{24}{2}\right) + \frac{10\ (19)^2}{2} = 4704.4 \text{ kN}$$

$$R_u = W_u{}' tan\ \ \emptyset + cb_u$$

$$R_u = 12 * 0.5 * 100 * 25\, tan\ \ 24 + 50 * 100 = 11678.4 \text{ kN}$$

$$F_S = \frac{11678.4}{4704.4} = 2.48 > 2 \ \ \text{(safe)}$$

The factor of safety against maximum shear

$$F_S = \frac{S_{max}}{\tau_{max}}$$

$\tau_a = H_u / b_u = 4704.4/100 = 47.04$ kN/m^2

$\tau_{max} = 2\ \tau_a = 2 * 47.04 = 94.08$ kN/m^2

$$S_{max} = w_s{}'\ h \tan\emptyset + c$$

$$S_{max} = 12\ (0.6 * 25) \tan 24 + 50 = 130.14 \text{ kN/m}^2$$

$$F_S = \frac{130.14}{94.08} = 1.38 < 1.5 \text{ (unsafe).}$$

2.7. Stability of Foundation against Horizontal Shear:

If the foundation of an earth dam consists of a strong stratum, such as compact gravel, coarse sand, consolidated silt or clay, it has a high shear strength and generally safe against horizontal shear. On the other hand, if the foundation consists of fine, loose sand, unconsolidated silt or clay, it has low shear strength. Such foundations should be checked for stability against horizontal shear. An approximate method, is use to check the stability.

The method is based on the assumption that a c-ø soil can be replaced by an equivalent cohesionless soil which would produce the same earth

pressure as that by the c-ø soil. Fig. 2.14 shows the upstream portion of an earth dam on a weak foundation. The section AB passes through the upstream shoulder of the dam. The thickness of the foundation stratum is h2 and the overall height of the dam measured from the rigid boundary is h1.

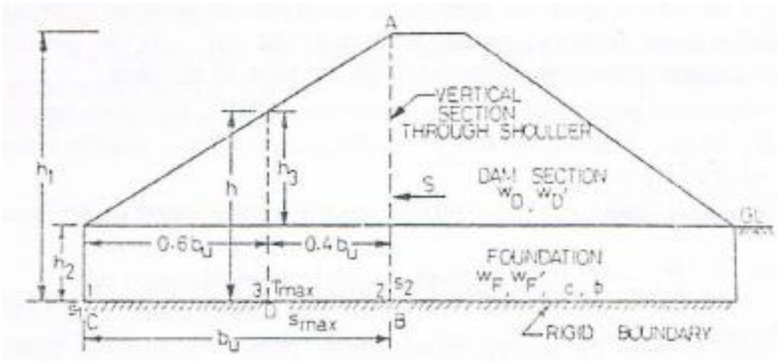


Figure 2.14 shows the upstream portion of an earth dam on a weak foundation.

Stability of foundation below u/s slope:

Factor of safety against average shear stress is:

$$F_S = \frac{Average\ shear\ strength}{Average\ shear\ stress} \qquad (2.20)$$

The average shear strength of the foundation is taken as the mean of the shear strengths of the foundation at the point C below the toe of the dam and the point B below the shoulder of the dam.

The shear strength below the toe is given by:

$$S_1 = w_F{'}\ h_2 \tan\emptyset + c \qquad (2.21)$$

Where w_F’ is the submerged unit weight of the soil mass of the foundation.

The shear strength below the shoulder is given by

$$S_2 = w_m{'}\ h_1 \tan\emptyset + c \qquad (2.22)$$

Where w_m' is the mean submerged unit weight of the soil mass of the dam and the foundation.

$$w_m{}' = \frac{w_D{}'(h_1 - h_2) + w_F{}'(h_2)}{h_1} \qquad (2.23)$$

Where w_D' is the submerged unit weight of the soil mass of the dam.

Then eq. (2.20) became:

$$F_S = \frac{0.5(S_1 + S_2)}{\tau_a} \qquad (2.24)$$

The factor of safety against average shear (Fs) should be at least 1.5.

$$\tau_a = \frac{S}{b_u} \qquad (2.25)$$

A horizontal shear force *S* acts on the vertical plane *AB*. Which is given by

$$S = w_m\left(\frac{h_1{}^2 - h_2{}^2}{2}\right)\tan^2\left(45° - \frac{\emptyset_1}{2}\right) \qquad (2.26)$$

Where w_m is the mean unit weight of the soil mass the dam and the foundation.

$$w_m = \frac{w_D(h_1 - h_2) + w_F(h_2)}{h_1} \qquad (2.27)$$

$\emptyset_1$ is the equivalent angle of internal friction of purely cohesionless soil.

$$w_m h_1 \tan\emptyset_1 = w_m h_1 \tan\emptyset + c \qquad (2.28)$$

Where ø and c are the angle of internal friction and cohesion of the foundation soil respectively.

<u>Factor of safety against maximum shear stress:</u>

$$F_S = \frac{S_{max}}{\tau_{max}} \qquad (2.29)$$

The maximum shear stress τ_{max} is taken as 1.40 times the average shear stress. The point of maximum shear stress occurs at the rigid boundary at a horizonial distance of 0.4 *bu* from the shoulder.

$$\boldsymbol{\tau}_{\mathbf{max}} = \mathbf{1.4}\,\boldsymbol{\tau}_{\mathbf{a}}$$

And

$$S_{max} = w_m{}'\; h \tan\emptyset + c \qquad (2.30)$$

$$w_m{}' = \frac{w_D{}'(h - h_2) + w_F{}'(h_2)}{h} \qquad (2.31)$$

Where h is the vertical distance from the upstream slope to the point D on the rigid boundary.

The factor of safety *Fs* should be greater than unity.

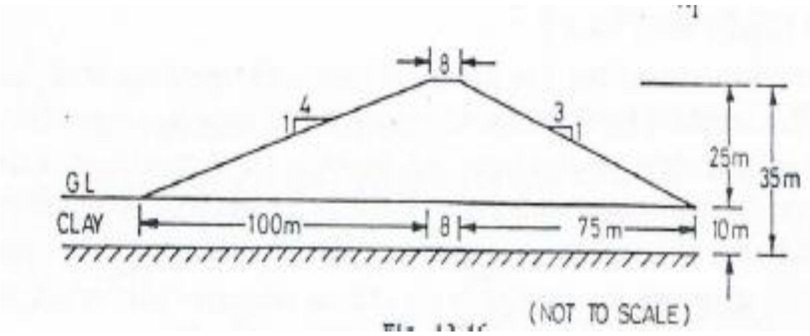
<u>Example 5:</u>

A homogeneous earth dam 25m high has a crest width of 8 m, u/s slope = 4:1 and d/s slope = 3:1. The foundation consists of soft clay up to a depth of 10 m and beneath that there is a rigid boundary. Check the stability of foundation against shear for the upstream portion of the dam. Assume the following properties:

a) Dam material. Saturated unit weight = 20 kN/m^3, ø = 15°, c = 20 kN/m^2

b) Foundation. Saturated unit weight = 21 kN/m^3, ø = 10°, c = 50 kN/m^2

Take specific weight of water = 10 kN/m^3



Sol.

$$F_S = \frac{0.5\,(S_1 + S_2)}{\tau_a}$$

From Eq. (2.27)

$$w_m = \frac{w_D\,(h_1 - h_2) + w_F\,(h_2)}{h_1}$$

$$w_m = \frac{20\,(35-10) + 21(10)}{35} = 20.29 \text{ kN/m}^3$$

From Eq. (2·28), the equivalent angle of internal friction

$$\tan \emptyset_1 = \frac{w_m h_1 \tan \emptyset + c}{w_m h_1}$$

$$\tan \emptyset_1 = \frac{20.29 * 35 \tan 10 + 50}{20.29 * 35} = 0.2467$$

$$\emptyset_1 = 13.86°$$

From Eq (2.25) & (2.26)

$$S = 20.29\left(\frac{35^2 - 10^2}{2}\right) \tan^2\left(45° - \frac{13.86}{2}\right) = 7001.82 \text{ kN}$$

$$\tau_a = \frac{7001.82}{100} = 70.02 \text{ kN/m}^2$$

$$S_2 = (20.29 - 10)\,35 \tan 10 + 50 = 113.50 \text{ kN/m}^2$$

$$S_1 = (21 - 10)\,10 \tan 10 + 50 = 69.40 \text{ kN/m}^2$$

$$F_S = \frac{0.5\,(69.40 + 113.50)}{70.02} = 1.31 < 1.5 \text{ (unsafe)}$$

$\tau_{max} = 1.4\ \tau_a$

$\tau_{max} = 1.4 * 70.02 = 98.03$ kN/m^2

$$S_{max} = w_m{}'\ h \tan \emptyset + c$$

h= h2+h3

h= 10+15=25m

$$w_m{}' = \frac{w_D{}'(\mathrm{h}-h_2)+w_F{}'(h_2)}{\mathrm{h}}$$

$$w_m{}' = \frac{(20-10)(25-10)+(21-10)(10)}{25} = 10.4 \text{ kN/m}^2$$

$$S_{max} = 10.4 * 25 * \tan 25 + 50 = 95.85 \text{ kN/m}^2$$

Fs= 95.85/98.03= 0.98 < 1(unsafe).

2.8. Stability Analysis Considering Earthquakes Forces:

In the seismic regions, the stability calculations of the slope of an earth dam should include earthquake forces because they reduce the margin of safety and even failure of the slope may occur in some cases. Whether the analysis is carried out by the circular arc for determination of the horizontal earthquake forces shall be based on the saturated unit weights of the material below the phreatic line and moist weights above it.

Figure 2.15 shows the inertial force due to earthquake is taken as α_h W in the horizontal direction, where α_h is the horizontal acceleration coefficient and W is the weight of slice. The stability of the slope of the dam is checked after taking the horizontal inertial force acting on each slice in the horizontal direction due to horizontal acceleration while analyzing the dam by the Swedish circle method.

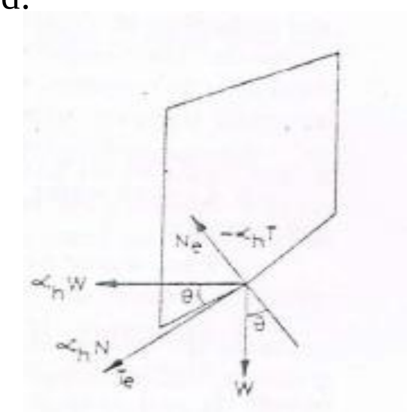


Figure 2.15 shows the inertial force due to earthquake.

The value of α_h usually varies from 0.05 to 0.20, depending upon the location of the dam with respect to various seismic zones. The inertial

force α_h W is resolved along the tangential and radial directions as follows.

The component along the tangential direction:

$$\boldsymbol{T_e = \alpha_h\, W\ \cos\theta = \alpha_h\, N} \qquad \textbf{(2.32)}$$

The component along the normal direction:

$$\boldsymbol{N_e = -(\alpha_h\, W\ \sin\theta) = -\alpha_h\, T} \qquad \textbf{(2.33)}$$

Hence for the most critical combination a force equal to (α_h N) is added to the tangential component T of the weight of the slice, and a force equal to (α_h T) is subtracted from the normal component N of the weight of the slice.

The factor of safety (Fs) is therefore, given by:

$$\boldsymbol{F_s = \frac{\tan\emptyset\ \sum(N - U - \alpha_h\, T) + cL_a}{\sum(T + \alpha_h\, N)}} \qquad \textbf{(2.34)}$$

The factor of safety should be at least unity.

2.9. Design Consideration for Earth Dams in Seismic Regions:

Damages caused by earthquakes, the earthquake may cause one or more of the following damages in an earth dam:

1. It may cause cracks in the core of the earth dam and leakage may occur through these cracks.
2. It may cause excessive settlements of the dam because of compression of the dam material or foundation or both. It results in a decrease in the free board and may cause overtopping.
3. Due to shaking of the reservoir bottom, the water in the reservoir may oscillate in relatively large but slow waves, called seiches, which may cause overtopping.

4. Natural hill slopes around the rim of the reservoir may fail and cause dumping of the rock mass into the reservoir, which may displace enough water to cause overtopping of the dam or may damage the appurtenant structures.
5. Deposits of loose sand or silt if existing in the foundation may liquefy due to vibrations and may move out from underneath the dam. It may cause cracking, sliding or even actual horizontal movement of a large part of the dam, resulting in its failure.
6. There may be crustal deformation associated with the movement of the fault zone. It may lift the bottom of the reservoir with respect to the foundation of the dam, resulting in the reduction of the capacity of reservoir and may cause overtopping.
7. The dam may actually be sheared off if there is a fault zone through the foundation of the dam and the two portions of the dam on either side of it may move with respect to each other, resulting in a large concentrated leakage and consequent failure.

2.10. Design considerations:

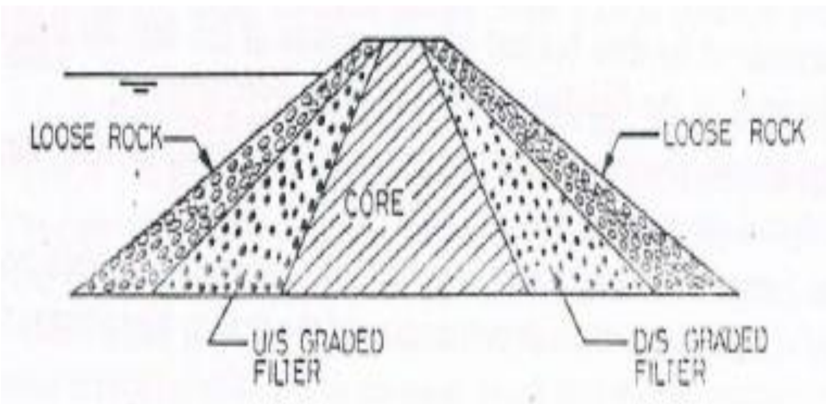


Figure 2.16 shows the cross-section of an earth dam in a seismic zone.

1. Consideration of Inertia Forces.
2. Graded filter at downstream of core.
3. Graded niter at upstream of core.
4. Highly pervious downstream shell, the downstream shell of the earth dam in a seismic region should be highly pervious so that the discharge through the cracks developed in the core is quickly removed and high pore water pressure does not develop.
5. Extra free board.
6. Thick core.
7. Larger section near top.
8. Strong foundation.

CHAPTER THREE

GRAVITY DAMS

Gravity Dams

3.1. Introduction:

A gravity dam is a solid structure, made of concrete or masonry, constructed across a river to create a reservoir on its upstream. The section of the gravity dam is approximately triangular in shape, with its apex at its top and maximum width at bottom as in Figure (3.1).

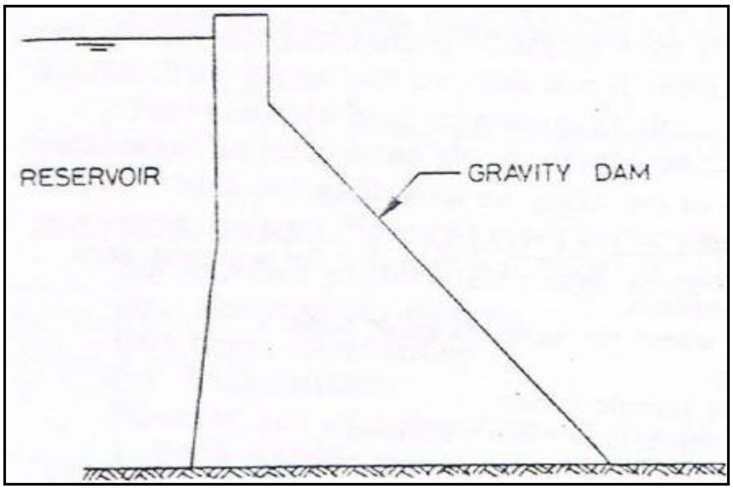


Figure (3.1) Gravity dam.

3.2. Basic Definitions:

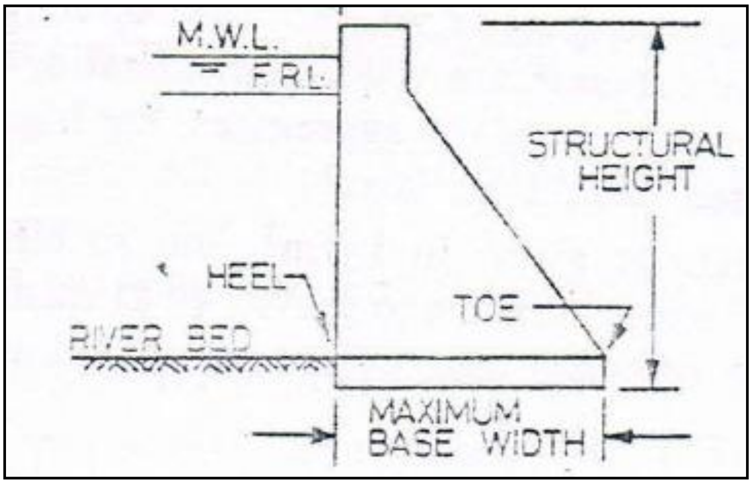


Figure (3.2) Basic Definitions of gravity dam.

1. **Length of the dam:** The length of the dam is the distance from one abutment to the other, measured along the axis of the dam at the level of

the top of the dam. It is the usual practice to mark the distance from the left abutment to the right abutment.

2. **Structural height of the dam:** The structural height of the dam is the difference in elevations of the top of the dam and the lowest point in the excavated foundation. In general, the height of the dam means its structural height.
3. **Maximum base width of the dam:** The maximum base width of the dam is the maximum horizontal distance between the heel and the toe of the maximum section of the dam.
4. **Hydraulic height of the dam:** The hydraulic height of the dam is equal to the difference in elevations of the highest controlled water surface on the upstream of the dam and the lowest point in the riverbed.

3.3. Forces Acting on A Gravity Dam: A gravity dam is subjected to the following main forces :

1. Weight of the dam.
2. Water pressure.
3. Uplift pressure.
4. Wave pressure.
5. Silt pressure.
6. Ice pressure.
7. Wind pressure.
8. Earthquake forces.

1. **Weight of the dam:** The weight of the dam is the main stabilising force in a gravity dam. The weight of the dam can be determined by divided the cross-section of the dam simple geometrical shapes, such as rectangles and triangles as in figure (3).

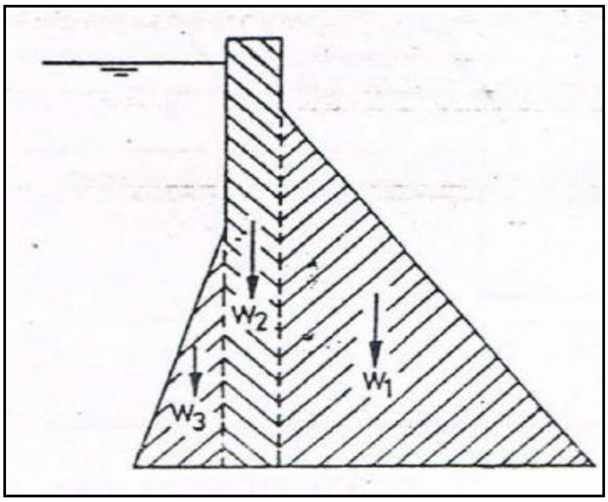


Figure (3.3) Computation The weight of the dam.

The weight of the dam per unit length is equal to the product of the area of cross-section of the dam and the specific. weight (or unit weight) of the material. The specific weight of the concrete is usually taken as 24 kN/m^3, and that of masonry as 23 kN/m^3 in designs.

2. **Water pressure:** The water pressure acts on the upstream and downstream faces of the dam. The water pressure on the upstream face is the main force acting on a gravity dam. The water pressure (**p**) varies linearly with the depth of the water and is expressed as:

$$\mathbf{p=\gamma\times H}$$

Where **p** is the water pressure (kN/m^2), **γ** is the specific weight of water equal to 9.81 kN/m^3 and **H** is the depth measured below the free surface (m).

The forces due to water pressure are discussed below separately for the non-overflow section and the overflow section.

(a) Non-overflow section:

1. U/s face vertical Figure (4). When the upstream face **AB** of the dam is vertical, The total water pressure is horizontal and is given by:

$$P_H = 1/2\, \gamma H^2 \quad \text{……….(3.1)}$$

It acts horizontally at a height of H/3 above the base of the dam.

2. U/s face ,inclined When the upstream face ABC is either inclined, or partly vertical and partly inclined the force due to water pressure can be calculated. in terms of the horizontal component $\mathbf{P_H}$ and the vertical component $\mathbf{P_V}$ Figure (5), The horizontal component is given by:

$$\mathbf{P_H = 1/2\, \gamma H^2 \;\ldots\ldots\ldots(3.2)}$$

Also, It acts horizontal at a height of (H/3) above the base.

The vertical component P_V of water pressure is equal to the weight of the water in the ABCD. the weight of water is found in two parts P_{V1} and P_{V2} by dividing the ABCD into a rectangle BCDE and a triangle ABE. Thus:

$$\mathbf{P_v = P_{V1} + P_{V2} \;\ldots\ldots\ldots\; (3.3)}$$

The lines of action of $\mathbf{P_{V1}}$ and $\mathbf{P_{V2}}$ will-pass through the respective centroids of the rectangle and triangle.

3. D/s as in Figure (5). The total water pressure is horizontal and vertical component is given by:

$$\mathbf{P_H' = l\,/2\, \gamma\, (H')^2 \;\ldots\ldots\ldots\; (3.4)}$$

$\mathbf{P_V'}$ = weight of water in triangle KFG, The force $\mathbf{P_V'}$ acts at the centroid of the triangle KFG. Obviously, when the water depth **H'** is zero, the water pressure force is zero.

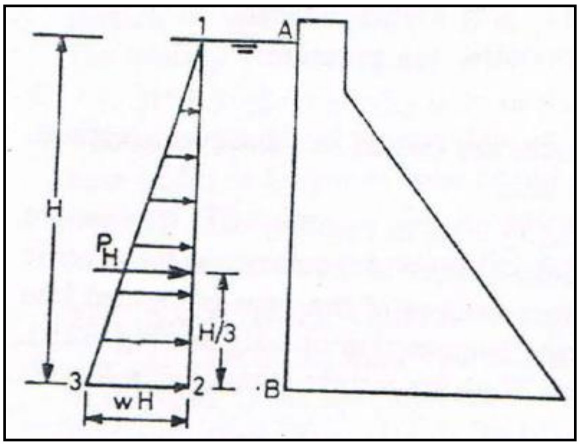


Figure (3.4) gravity dam with U/s Vertical face.

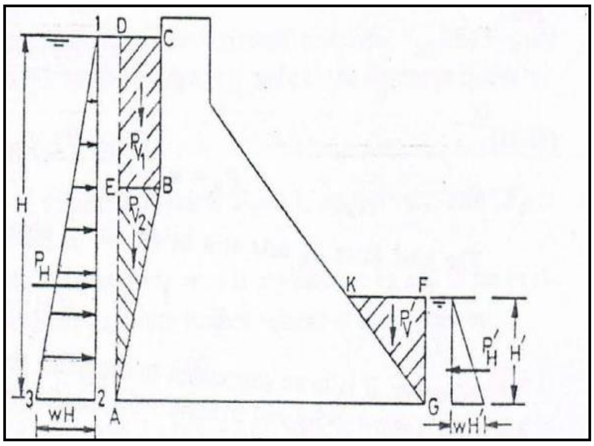


Figure (3.5) gravity dam with U/s inclined face.

(b) Overflow section:

1. U/s face vertical as in Figure (6) shows the overflow section (spillway section) of a gravity dam with the upstream face vertical. The water approaching the dam has a velocity of approach (Va). The head Ha due to the velocity of approach is equal to $Va^2/2g$, and the total energy line (TEL) is at a height of Ha above the water surface.

The horizontal component of force due to water pressure of the dam is given by:

$$\mathbf{P_H = \gamma(H_1 + H_a)\,(H_2 - H_1) + 1/2\,\gamma\,(H_2 - H_1)\,(H_2 - H_1)} \;\ldots\ldots\ldots\; \mathbf{(3.5)}$$

The total force $\mathbf{P_H}$ acts at a height of $\bar{Z}$ above the base, given by:

$$\bar{Z} = \frac{1}{3}(H_2 - H_1)\left[\frac{(H_2 + 2H_1) + 3H_a}{(H_2 + H_1) + 2H_a}\right] \ldots\ldots\ldots.\,(3.6)$$

2. Inclined U/s face as in Figure (7) shows the overflow section of a gravity darn with upstream face inclined. The force due to water pressure can be obtained from the vertical component and the horizontal component. as in the case of a non-overflow section.

3. Dynamic pressure on the D/s face as in figure (8) The downstream face of the overflow section is usually of the curved shape due to change in momentum, the water exerts a dynamic force on the dam. The horizontal and vertical components of the force can be determined by applying the impulse momentum equation to the water in the control volume a-b-e-d. The equation is applied in the horizontal and vertical directions.

In the horizontal direction $\sum \mathbf{FH} = \boldsymbol{\rho} \times \mathbf{q} \times (\mathbf{V_{2H}} - \mathbf{V_{1H}})$ **….(3.7)**

where $\sum$ FH is the sum of forces acting on the control volume in the horizontal direction, V_{2H} and V_{1H} are the horizontal components of velocities at points 2 and 1, respectively and q is the discharge intensity.

In the horizontal direction $\sum \mathbf{Fv} = \boldsymbol{\rho} \times \mathbf{q} \times (\mathbf{V_{2V}} - \mathbf{V_{1V}})$**………(8)**

Where $\sum FV$ is the sum of forces acting on the control volume in the vertical direction, V_{2V} and V_{1V} are the vertical components of velocities at points 2 and 1, respectively.

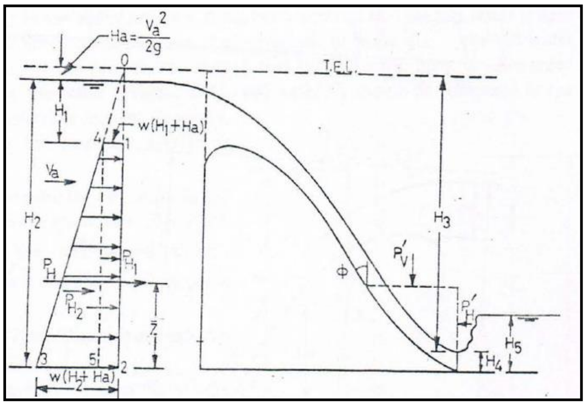


Figure (3.6) gravity dam with U/s vertical face.

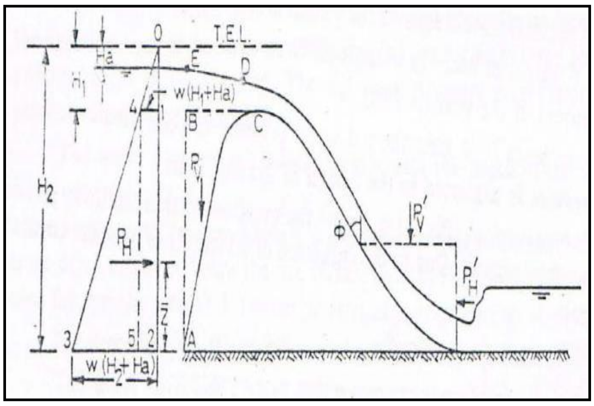


Figure (3.7) gravity dam with U/s inclined face.

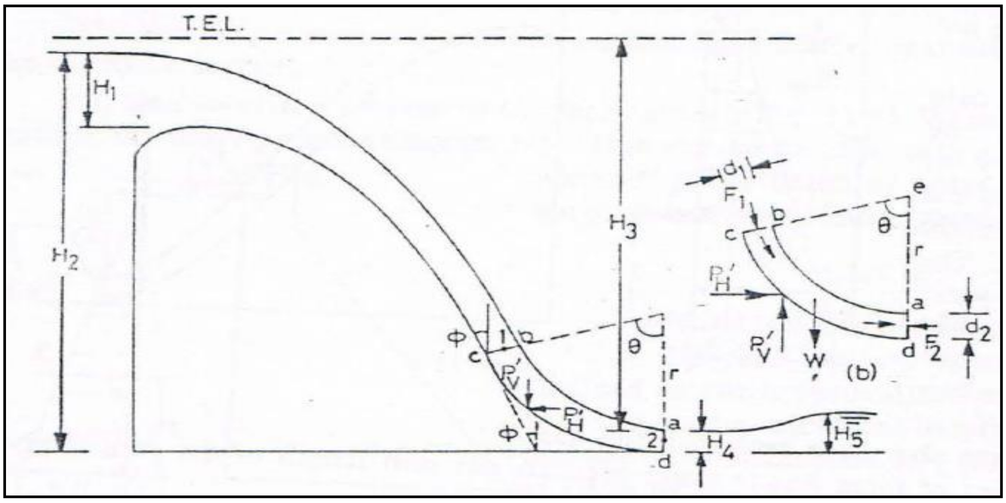
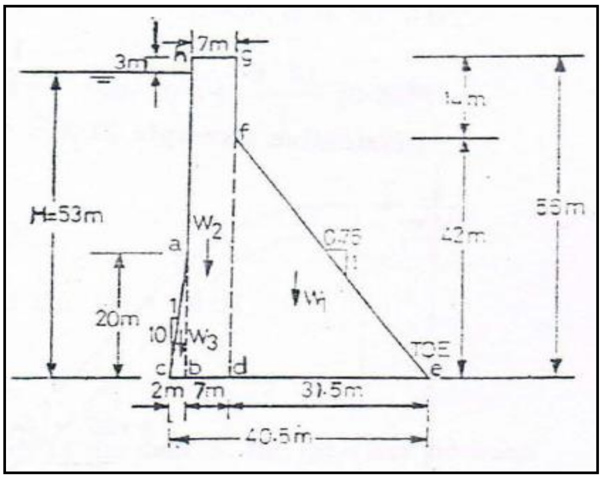
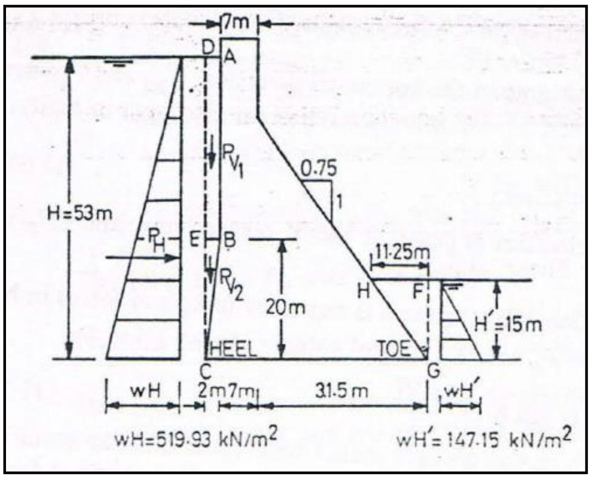


Figure (3.8) gravity dam Dynamic pressure on the D/s face.

Example 1:

Determine the forces due to self weight and water pressure on the non-overflow dam:





Solution: Weight The section is divided into three parts, as shown.

W1 = 0·5 x 42 x 31·5 x 24 = 15876·0 kN, acting at a distance, of 21 m from the toe.

W2 = 7 x 56 x 24 = 9408 kN, acting at a distance of 35·0 m from the toe.

W3 = 0·5 x 20 x 2 x 24 = 480 kN, acting at a distance of 3 9·17m from the toe.

Water pressure

(i) U/s side Horizontal force.

P_H= 0·5 x 9·81 x $(53)^2$ = 13778·15 kN, acting at a height of 53/3 m above the base.

Vertical force, Pv = P_{vl} + P_{v2}

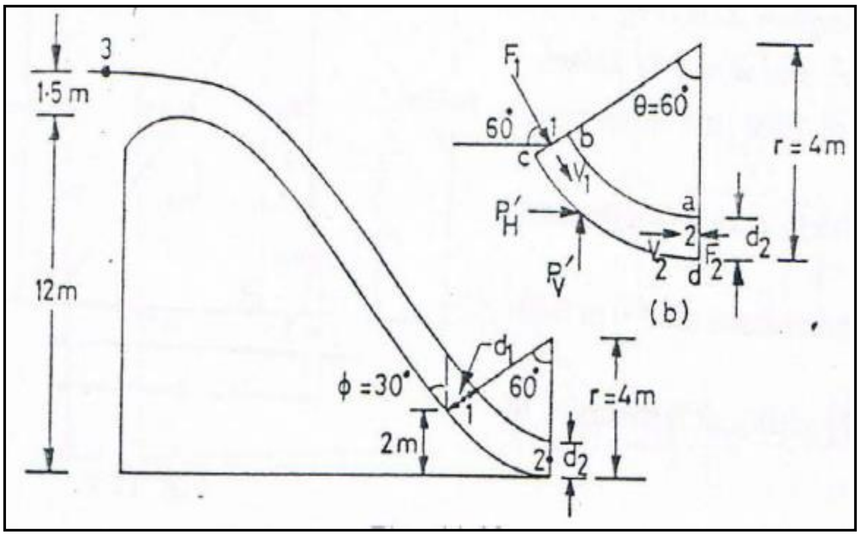
where P_{v1} = 2 x 33 x 9·81 = 647·46 kN, acting at a distance of 39·5 m from toe.

P_{v2} = 0·5 x 2 x 20 x 9·81 = 196·2 kN, acting at a distance of 39·83 m from toe.

(ii) D/s side P'_H = 0·5 x 9·81 x $(15)^2$ = 1103·63 kN, acting at a height of 5.0 m above the base. P_v' = 0·5 x 15 x 11·25 x 9·81 = 827·72 kN, acting at a distance 3·75 m from the toe.

Example 2:

For the overflow section shown in Figure determine the dynamic force on the curved surface. Take the coefficient of discharge of the spillway as 2·1 and the radius of the bucket (curved section) as 4 m.



Solution: Discharge intensity, $q = C\ H^{3/2}$

$q=2.1 \times (1.5)^{3/2} = 3.86$ cumecs/m

Let d_1 and d_2 the thickness of sheet of water at point 1 and 2, respectively, and V_1 and V_2 the corresponding velocities. Applying Bernoulli's equation to point 3 on the upstream water level and points 1 and 2, taking datum at the river bed,

$$\frac{p_1}{\gamma} + z_1 + \frac{v_1}{2g} = \frac{p_2}{\gamma} + z_2 + \frac{v_2}{2g}$$

$13.5 = 2.0 + d_1 \cos\theta + (V_1^2/2g) = d_2 + (V_2^2/2g)$

Taking $V_1 d_1 = V_2 d_2 = q = 3.86$, and $\theta = 60°$

$13.5 = 2.0 + d_1 \cos 60° + (3.86/d_1)^2 \times 1/2g = d_2 + (3.86/d_2)^2 \times 1/2g$

Solving by trial and error, $d_1 = 0.258$ m, $V_1 = 14.96$ m/s, $d_2 = 0.239$ m, and $V_2 = 16.15$ m/s.

Weight of water in the control volume $w = (\gamma \times V) = \frac{2\pi r\theta}{360} \times h$

$$w = \frac{9.81 \times 60 \times 2\pi \times 4}{360} \times \left(\frac{0.258 + 0.239}{2}\right) = 10.12\ \text{kN}$$

Hydrostatic force at face bc, $F1 = 0{\cdot}5\ \gamma d_1^2 \cos 60° = 0{\cdot}5 \times 9{\cdot}81 \times (0{\cdot}258)^2 \times 0{\cdot}5 = 0{\cdot}16$ kN

Hydrostatic force at face ad, $F2 = 0{\cdot}5\ \gamma d_2^2 = 0.5 \times 9.81 \times (0.239)^2 = 0.28$ kN

$\sum F_H = (\gamma/g)\ q\ (V_{2H} - V_{1H})$

$\sum F_H = (9{\cdot}81x10^3/9.81)$ x 3.86 (16.15- 14.96 x cos60°) x 1/1000 =33.47 kN

$0{\cdot}16$ co60° - $0{\cdot}28$ +P_H' =33.47

P_H'= 33·67 kN (towards d/s)

$\sum F_V = (\gamma/g)\ q\ (V_{2V} - V_{1V}) = (9{\cdot}81x\ 10^3 / 9.81)$ x 3.86 x (0·0 + 14·96 sin60°) x $l0^{-3}$

$\sum F_V$ = 3.86 (0 + 12·96) = 50.01 kN

P_v' - 0·16 sin60° - W=50.01

P_v'- 0·14 - 10·21 =50·01

P_v' = 60·36 kN (+upward)

3. **Upliff Pressure:** The water enters the pores, cracks and fissures within the body of the dam, at the interface between the dam and within the foundation. Because the water is under pressure, it creates uplift pressure on the dam. The pressure acts in all directions, but the pressure acting upwards · is important for the design of the dam.

The computation of forces due to uplift pressure requires the determination of the area on which it acts and the intensity of uplift pressure at various points:

- **Area factor**: The uplift pressure generally does not occur on the entire horizontal area, because in some portions, there are no pores in which water can enter. The fraction of the total area on which· the uplift pressure acts is called the area factor. recommended an area factor of 1/3 to 2/3 for both concrete and rock.
- **Intensity of uplift pressure:** The uplift pressure at any point depends upon the depth of water at that point. At the base of the dam the intensity of uplift pressure at the upstream is equal to the hydrostatic pressure corresponding to full reservoir level and that at the downstream is equal to the hydrostatic pressure corresponding to the tail water level.

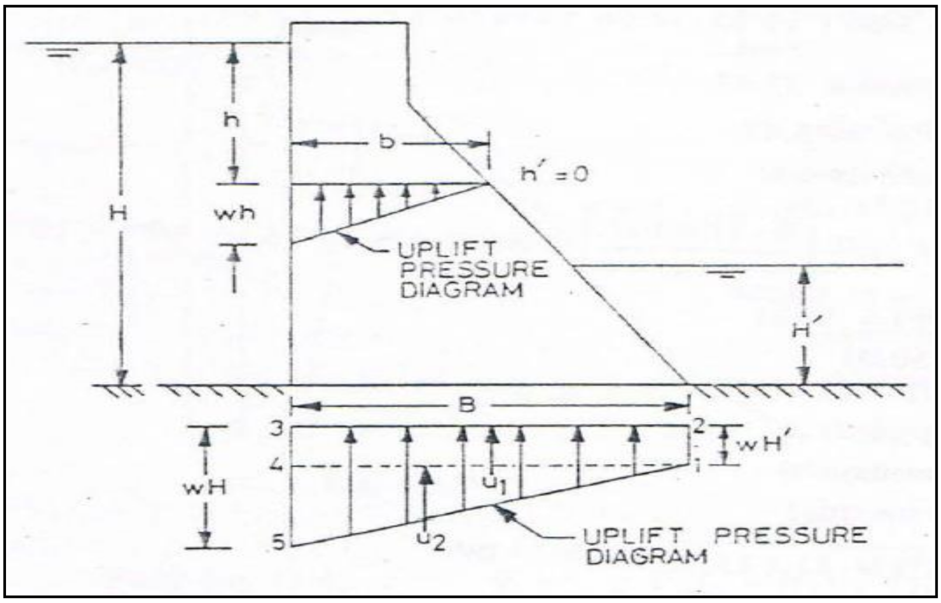


Figure (3.9) Total uplift force on the base of the dam.

Total uplift force on the base of the dam,

$$\mathbf{U = U_1 + U_2 = \gamma H' \times B + 1/2\, \gamma (H-H')B} \quad \text{..........(3.9)}$$

where U1 and U2 are the uplift pressure to the rectangle 1-2-3-4 and triangle 1-4-5, respectively.

The lines of action of U_1 and U_2 are at distance of B/2 and 2B/3 from the toe, respectively.

- **Uplift pressure in the body of the dam:** The uplift pressure distribution on any horizontal plane within the body of the darn is found in the same way, taking the corresponding pressures at the upstream and downstream faces. For example, at a depth **h** below the water surface, where the width is equal to **b** and the tail water depth is **h'**:

$$\mathbf{U = \frac{\gamma h}{2}(1 \times b)} \quad \text{..........(3.10)}$$

- **Effect of drains on uplift pressure:** To reduce the uplift pressure, drains are formed through the body of the dam.

  According to the U.S.B.R. (United States Bureau of Reclamation) recommendations, the uplift pressure at the line of the drains is equal to the hydrostatic pressure at the d/s face plus on-third of the differences· of pressures at the u/s and d/s faces.

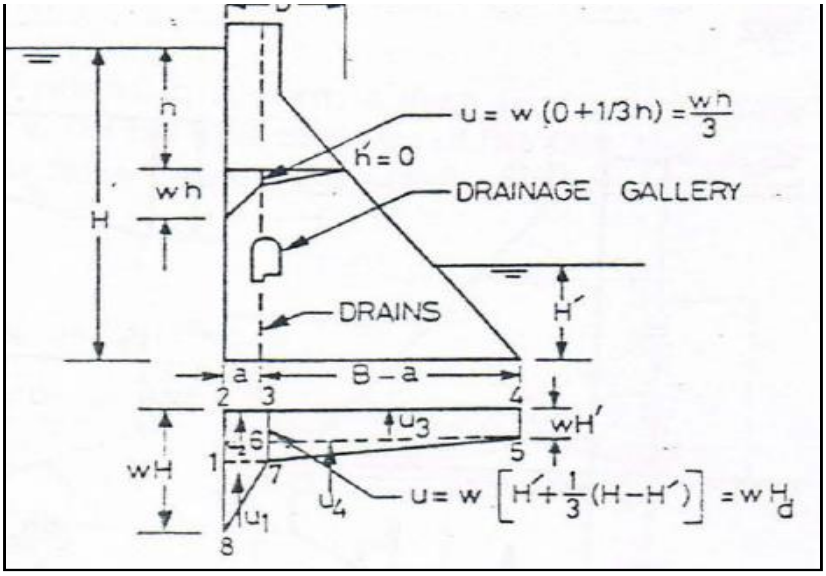


Figure (3.10) Effect of drains on uplift pressure.

The uplift pressure intensity at the line of drains is given by:

$$U = \gamma\, H_d = \gamma\, [H' + 1/3\,(H - H')] \quad \text{..........(3.11)}$$

H_d is the uplift pressure head at the line of drains.

Example 3: Determine the uplift force at the base of a gravity shown in Figure for the following two cases. (a) No drains, (b) With drains at a distance of 5 m from the U/s end.

Solution:

(a) Uplift force, $U = U_1 + U_2$

$U = 9{\cdot}81\,[5 \times 25 + 1/2\,(30 - 5) \times 25] = 4291{\cdot}88$ kN

(b) Uplift pressure head at drain,

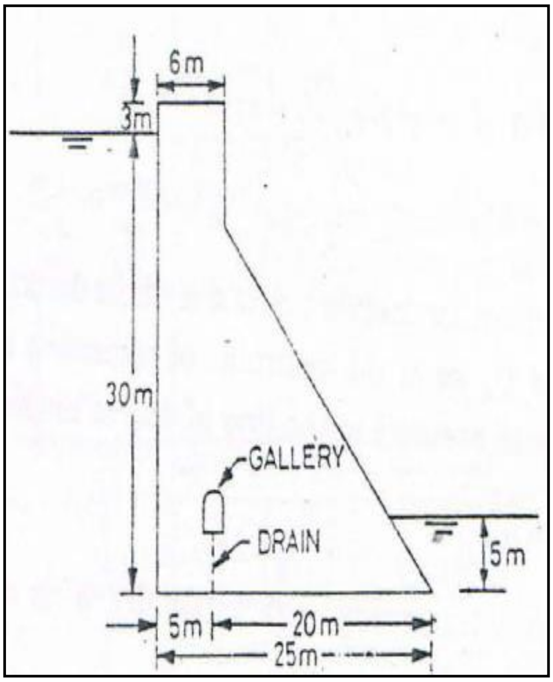
$Hd = [H' + 1/3\,(H - H')]$

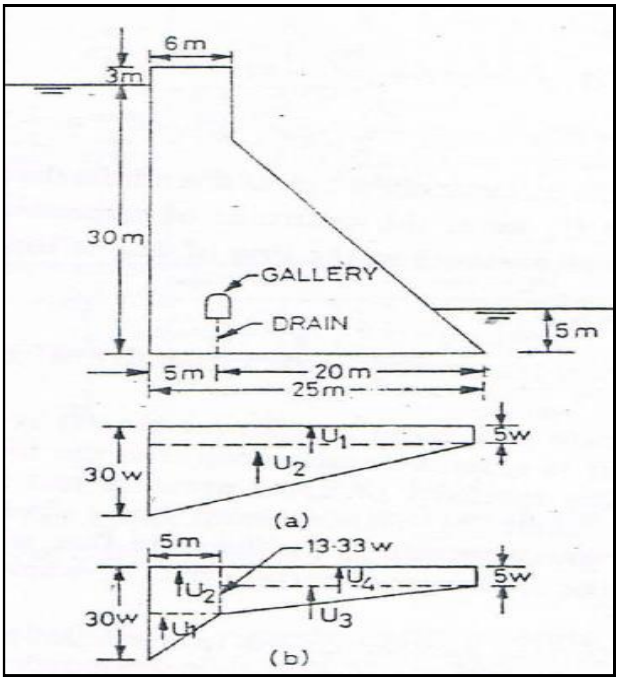
$Hd = [5 + 1/3\,(30 - 5)] = 13{\cdot}33$ m

Total uplift force, $U = U_1 + U_2 + U_3 + U_4$

$U = 9{\cdot}81\,[1/2 \times (30 - 13{\cdot}33) \times 5 + 13{\cdot}33 \times 5 + 1/2\,(13{\cdot}33 - 5) \times 20 + 5 \times 20]$

$= 2860.9$ kN





4. Wave Pressure: when the wind blows over the water surface of the reservoir, it exerts a drag on the surface, due to which ripples and waves are formed. When these waves strike the upstream face of the darn, they cause a force on the upper portion of the dam. The force due to wave pressure is horizontal and
is computed as follows:

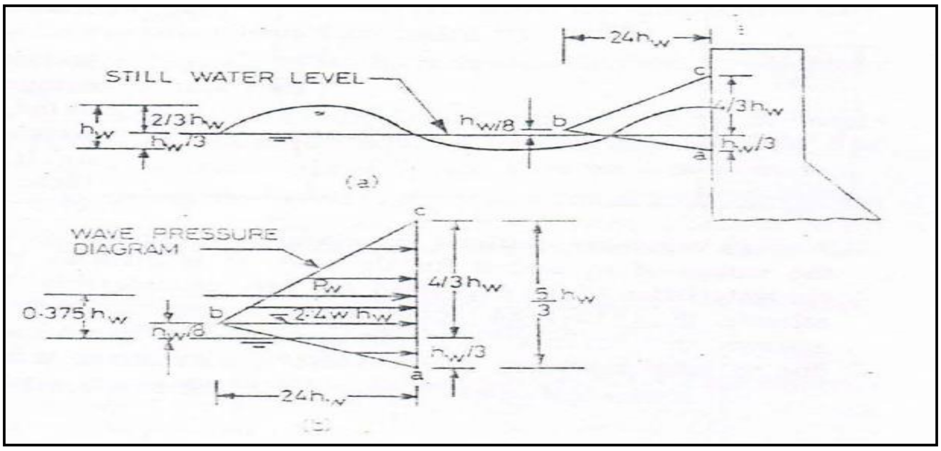


Figure (3.11) Effect Wave Pressure on gravity dam.

a) Height of wave: The force depends upon the characteristics of the waves.

$$\mathbf{h_w = 0.0322\sqrt{FV} + 0.763 - 0.271(F)^{1/4}} \quad \textbf{if } \mathbf{F \leq 32\ km} \textbf{ .(3.12)}$$

$$\mathbf{h_w = 0.0322\sqrt{FV}} \quad \textbf{if F>32 km ....(3.13)}$$

Where **hw** is the wave height (m), **F** straight length of water expanse measured normal to the axis of dam in (km) and **V** is the wind velocity (km/hr).

b) Wave pressure distribution: the actual distribution is difficult to determine. The total height of the triangle is (5/3) hw with a height of (4/3) hw above the still water level. The maximum pressure intensity occurs at a height of (hw/8) above the still water level, and is given by:

$$\mathbf{p_w = 2.4\,\gamma hw} \textbf{ .... (3.14)}$$

The line of action of P_w is at the centroid of the area a-b-c, which is at a height of 0.375 hw above the still water level.

The total water pressure force is given by:

$$\mathbf{Pw = 1/2\,(5/3\ hw)\,(2.4\,\gamma\ hw)} \textbf{ .... (3.15)}$$

or

$$\mathbf{Pw = 2{\cdot}0\,\gamma\ hw^2} \textbf{.......... (3.16)}$$

Example 4:

Determine the force due to wave pressure on a dam with the following data: F = 100 km, Wind velocity = 80 km/hr.

Solution:

$h_w = 0.0322\sqrt{FV}$ if F>32 km

hw= $0.0322\sqrt{100 \times 80} = 2.88$ m

Pw = $2{\cdot}0\,\gamma\,hw^2 = 2 \times 9{\cdot}81 \times (2{\cdot}88)^2 = 162{\cdot}74$ kN

The line of action of P_w will be 0·375 hw = 1·08 m above the still water level.

5. Silt Pressure: All rivers carry a large quantity of silt during floods. The silt is deposited in the reservoir on the upstream of the dam. This silt exerts the earth pressure on the dam. As the silt is submerged, the silt pressure is computed assuming the submerged conditions. It is the usual practice to assume that the silt is fully saturated, cohesionless soil.

The silt pressure can be evaluated in two cases:

1. If the upstream face of the darn is vertical, the force is horizontal and is given by:

$$\mathbf{P_{silt} = \frac{1}{2}\,\gamma_s\, hs^2\, ka \;\ldots\ldots\ldots\ldots (3.17)}$$

where: ka = (1-sinϕ) / (1+sinϕ) where γ_s, is the submerged unit weight of the soil, hs is the depth of silt above the bed and Φ is the angle of shearing resistance of the silt.

2. If upstream face of the darn is inclined The vertical component of the force (P_v) is equal to the weight of the submerged weight of silt in the area (abcd).

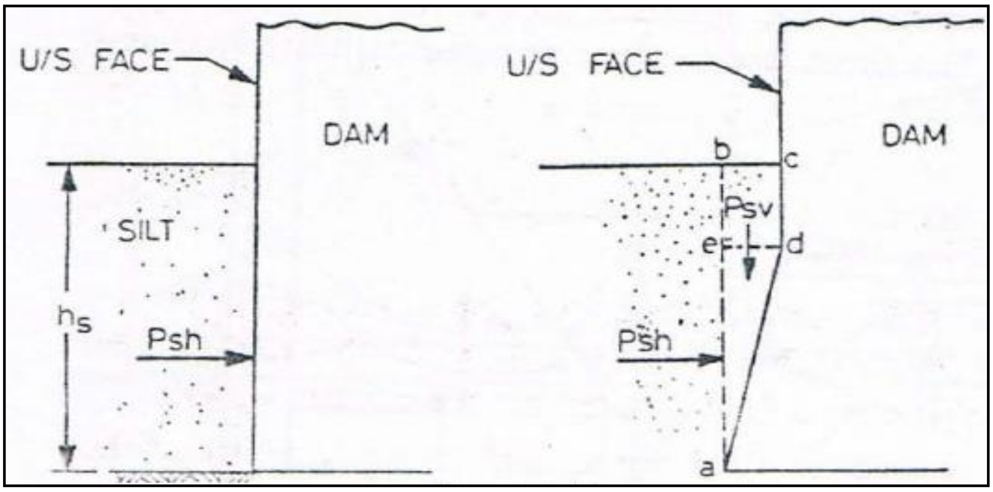


Figure (3.12) Effect silt Pressure on gravity dam.

6. Ice pressure: in the cold climates, ice is formed in the reservoir and the reservoir surface. gets covered with a sheet of ice The thickness of ice depends upon the climatic conditions.. Subsequently, when the temperature of atmosphere rises, ice sheets expands and causes a thrust

on the dam because the coefficient of thermal expansion of ice is about 5 times that of concrete. When the temperature falls, the ice sheet contracts and cracks, and these cracks subsequently get filled with water. The magnitude of the force due to ice pressure depends upon the following factors:

1. **Thickness of ice**: The thickness of ice in a particular reservoir depends upon a number of factors and cannot be easily estimated. Generally, it is obtained from the existing records in the region. The thickness is usually not more than 1 m, but in some sub-zero regions it may be even 2 m or more.
2. **Restraints of rim walls**: the ice pressure exerted on the dam depends upon the rigidity of the rim walls of ice. If these walls are strong the maximum force is caused on the dam. On the other hand, if these walls are yielding the force on the dam is reduced.
3. **Rate of rise of temperature:** the greater is the rate of rise of temperature, the greater is the force on the dam.

   The average value of (250 kN/m^2 =250 kpa) may be taken as an ice force.

7. Wind pressure: wind pressure acts on the exposed surface of the dam when winds blow. Generally, the wind pressure is not significant for the design of gravity dams and is therefore neglected. However, in the design of gravity dam the wind pressure can be taken of (1 to 1.5 kN/m^2) over the area exposed to high winds.

8. Earthquake forces:

1. Effect of Horizontal acceleration: the horizontal acceleration can occur in either upstream or downstream direction. Because the dam is designed for the worst case, the horizontal acceleration is assumed to occur in the direction which would produce. The directions are different for the reservoir full condition and the reservoir empty condition.

(a) Reservoir full condition For the reservoir full condition, the worst case occurs when the **earthquake acceleration acts towards the upstream direction and the corresponding inertia force acts in the downstream direction**.

(b) Reservoir empty condition For the reservoir empty condition, the worst case occurs when the **acceleration due to earthquake acts towards the downstream direction and the corresponding inertia force acts in the upstream direction.**

- **Horizontal force** The horizontal force due to the earthquake is equal to the product of mass M of the dam and horizontal earthquake acceleration.

  Thus F_h = M × horizontal acceleration

Because earthquake acceleration is expressed as ($\alpha_h \times g$), and mass is equal to weight W divided by g

$$\mathbf{F_h = \left(\frac{W}{g}\right) \times \alpha_h \times g = \alpha_h \times W \text{ .... } (3.18)}$$

The force is assumed to act at the centre of gravity of mass.

Where: α_h horizontal seismic coefficient.

Note: a constant value of the horizontal acceleration of 0·1g to 0·15g is assumed for high dams in seismic areas. Even for dams located in the regions not subjected to extreme earthquake, a value of 0·10g is sometimes taken. However, for dams located in non-seismic areas, earthquake forces are neglected.

- Methods of determine value of seismic coefficient (α):

1. **Seismic coefficient method:** for dams up to 100 m in height, the horizontal seismic coefficient α_h at the top of the dam is taken as 1.5 time the seismic coefficient α .The coefficient reduces linearly from 1.5α to zero at the base of the dam as in Figure. The inertia force for any part

of the dam above a horizontal section as well as for the entire dam above its base may be computed by integration:

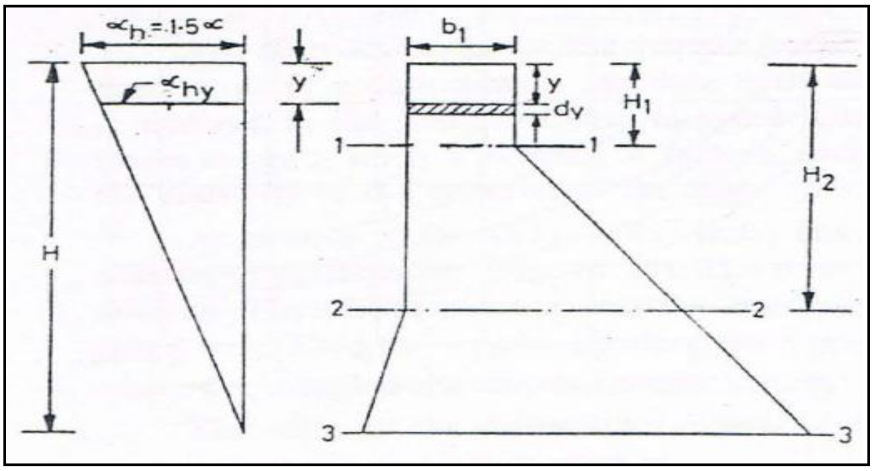


Figure (3.13) Seismic coefficient method.

Portion from the top to section 1-1 at depth H_1 Let us consider a small portion of the dam of height dy at a depth y below the top of dam. The inertia force acting on this small portion is given by:

$$\mathbf{dF_1 = (b_1 \times l \times dy)\, \gamma_c\, \alpha_{hy}} \quad \text{……….. (3.19)}$$

α_{hy} is the horizontal seismic coefficient at depth y.

$\alpha_{hy} = (1-\frac{y}{H})\times 1{\cdot}5\alpha$ **from** $\frac{1.5\alpha}{H} = \frac{\alpha_{hy}}{H-y}$

$$\mathbf{dF_1 = (b_l \times dy)\, \gamma_c\, (1-\frac{y}{H})\, (1{\cdot}5\alpha)} \quad \text{……….. (3.20)}$$

By integrating Eq. of dF_1 between the limits 0 to H_1 the inertia force F_1 for the part of the dam lying above section 1-1 can be computed. Thus

$$\mathbf{F_1 = \int_0^{H_1} (b_1 \times \gamma_c \times 1.5\alpha)\left(1-\frac{y}{H}\right) dy}$$

or

$$\mathbf{F_1 = 1.5\alpha \times b_1 \times \gamma_c \times H_1 (1-\frac{H_1}{2H})} \quad \text{……….. (3.21)}$$

The moment of the force dF_1, about section 1-1 is given by:

$$d_{M1} = (b_1 dy) \times \gamma_c \times (1.5\alpha)\left(1-\frac{y}{H}\right) \times (H_1 - y)$$

Total moment,

$$M_1 = \int_0^{H_1} dM1$$

$$\mathbf{M_1 = 1.5\alpha \times \gamma_c \times b_1 \left(\frac{H_1^2}{2}\right)\left(1 - \frac{H_1}{3H}\right)} \quad \text{.......... (3.22)}$$

Portion from section 1-1 at depth H_1 to section 2-2 at depth H_2 Let us consider a small portion of the dam of height dy lying between 1-1 and section 2-2, at a depth y below the top of dam. The inertia force is given by:

$$\mathbf{F_2 = F_1 + \int_{H_1}^{H_2}(1 \times b_y \times dy)\gamma_c\left(1 - \frac{y}{H}\right)(1.5\alpha)} \quad \text{.......... (3.23)}$$

where b_y is the width of the dam section at a depth *y* below the top of the dam, and is given by:

$$\mathbf{b_y = b_1 + n(y - H_1)} \quad \text{.......... (3.24)}$$

where n is the slope of d/s face (n horizontal to 1 vertical).

$F_2 = F_1 + 1.5\alpha \times \gamma_c \times$.........

$$(H_2 - H_1)\left[b_1\left(\frac{2H - H_2 - H_1}{2H}\right) + \left(\frac{H_2 - H_1}{2} - \frac{2H_2^2 - H_1H_2 - H_1^2}{6H}\right)\right]$$

The moment of the forces about section 2-2 is given by:

$$\mathbf{M_2 = M_1 + F_1(H_2 - H_1) + \int_{H_1}^{H_2}(1 \times b_y\, dy)\gamma_c\left(1 - \frac{y}{H}\right)(1.5\alpha)(H_2 - y)}$$

$$\text{...........(3.25)}$$

2. **Response spectrum method:** In the response spectrum method. the value of the horizontal seismic coefficient α_h is taken equal to the value of the seismic coefficient α.

$$\mathbf{\alpha = \beta \times I \times F_o \times (S_a/g)} \quad \text{.......... (3.26)}$$

where β is soil-foundation system factor, the value of which for gravity darns is equal w 1·0, I is the importance factor, the value of which for gravity darns is taken as 2·0, and F_o = Seismic zone factor for average acceleration spectra, the value of which is obtained from Table 1. and (S_a/g) =Average acceleration coefficient, the value of which depends upon the natural period (T) of vibration of the dam and damping of the

dam. Its value is obtained from Figure 14 which is known as Housner's chart.

$$\mathbf{T = 5.55\left(\frac{H^2}{B}\right)\sqrt{\frac{\gamma_c}{g \times E_c}}} \qquad \text{..........} \ \mathbf{(3.27)}$$

where H= height of dam (m), B= base width of dam (m), E_c= modulus of elasticity of material of dam (kN/m²).

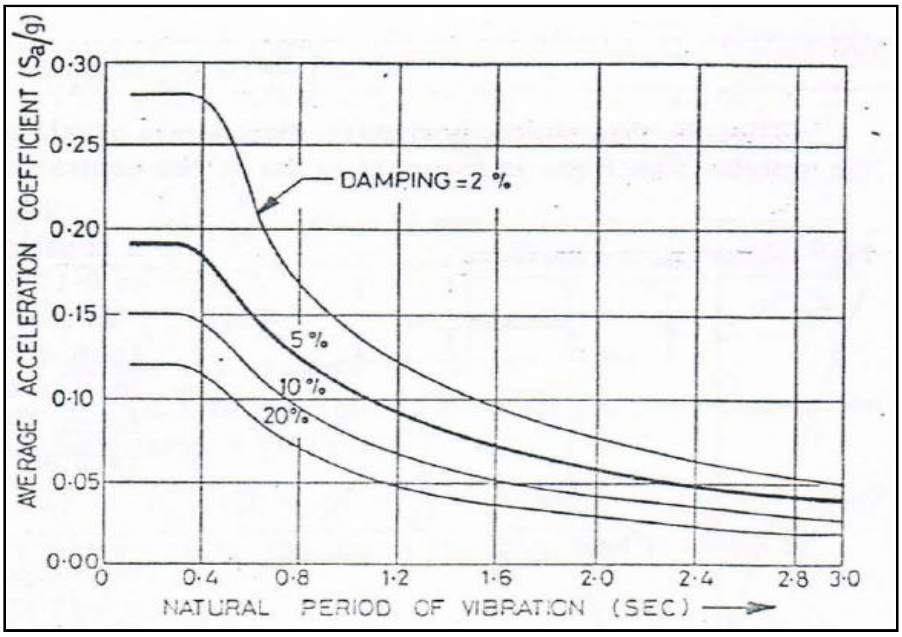


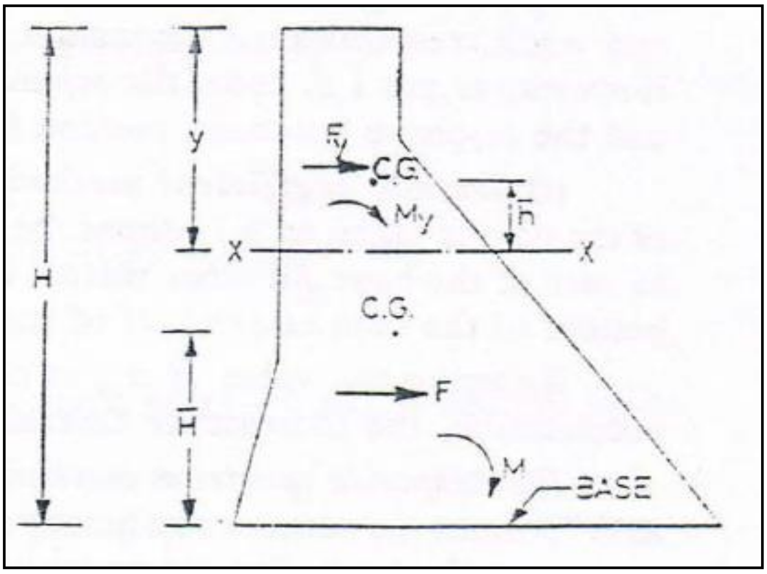
Figure (3.14) Average acceleration coefficient (S_a/g).

Table 1: the values of α_o and F_o

Zone	I	II	III	IV	V
Basic seismic Coefficient (α_o)	0·01	0·02	0·04	0·05	0·08
Seismic Zone factor (F_o)	0·05	0·10	0·20	0·25	0·40

Assuming a damping of 5 percent as suggested by, IS Code, the value of (S_a/g) may be obtained from Figure 14.

For a part of the dam up to section X-X at a depth y below the top of the dam, the forces F_y and moment M_y about X-X as in Figure are given by:



$$F_y = C_F \times F = C_F\,(0.6\ W\ \alpha_h)\ \ldots\ldots\ldots\ (3.28)$$

$$M_y = C_M \times M = C_M(0{\cdot}9\ W\ \bar{h}\ \alpha_h)\ \ldots\ldots\ldots\ (3.29)$$

where the values of coefficient C_F and C_M depend upon the ratio (y/H) and are taken from Figure 15, $\bar{h}$ is the height of the centre of gravity of the portion above X-X. The total force F and moment M above base of the entire dam are given by:

$$F = 0{\cdot}6 W \alpha_h\ \ldots\ldots\ldots\ (3.30)$$

$M = 0{\cdot}9\ (W\bar{H})\ \alpha_h$, $\bar{H}$ = height of the centre of gravity of the entire dam above the base.

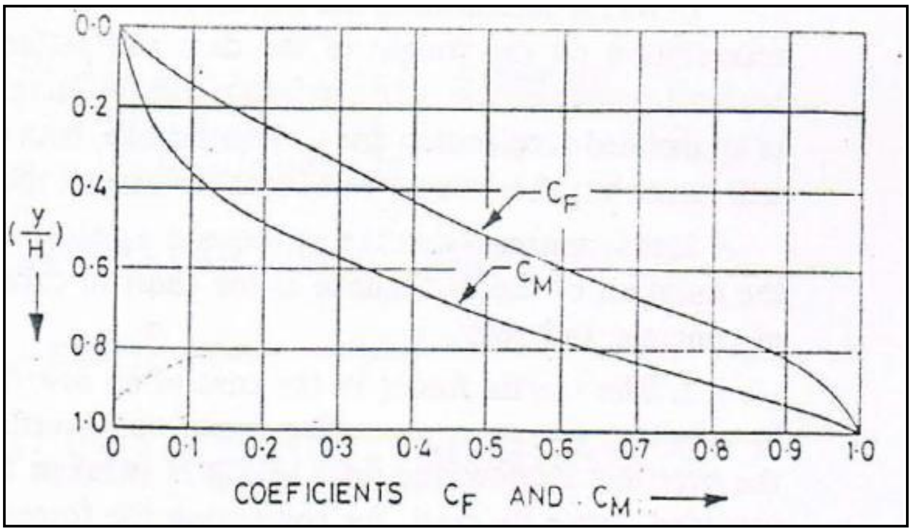


Figure (3.15) Coefficients C_F and C_M .

2. Effect of vertical acceleration: Due to vertical acceleration, the inertia forces act on the dam. The magnitude of the inertia force is equal to the product of the mass (M) and the vertical acceleration. Thus

$$\mathbf{F_v = M \times vertical\ acceleration\ldots\ldots\ldots\ldots (3.31)}$$

$$\mathbf{F_v = (W/g)\,(\alpha_v\, g) = \alpha_v\, W \ldots\ldots\ldots\ldots (3.32)}$$

where α_v is the vertical seismic coefficient.

Note: If the vertical acceleration acts downwards, the inertia force acts upwards, and the effective weight of the dam is decreased; hence the stability is reduced, because in a gravity dam the main stabilising force is the weight of the dam. on the other hand, if the vertical acceleration acts upwards, the inertia force acts downwards and increases the effective weight and further increases the stability of the darn.

Note: the modified weights of the dam may be used in the analysis, where Modified weight $W' = W \pm a_v W = W(1 \pm a_v)$. The modified weights can also be obtained by considering modified Specific weights. We can take $\gamma_c' = \gamma_c (1 \pm a_v)$ and $\gamma' = \gamma(1 \pm a_v)$ for computing the weights, where γ_c' and γ' are for the concrete and water, respectively. A constant value of $a_v = 0{\cdot}05$ is sometimes adopted in the design of a dam. However, the **seismic coefficient method** should be used for darns up to 100 m heights, and the **response spectrum method** for dams of heights greater than 100 m.

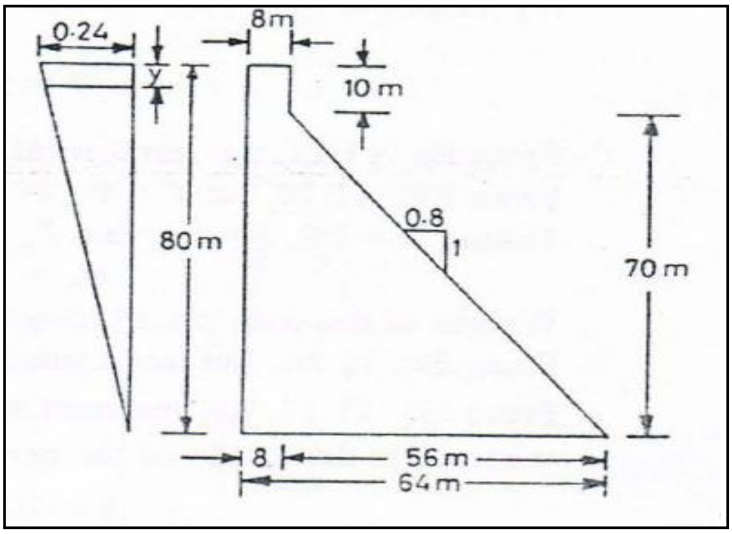
(i) Seismic coefficient method: In this method, the value of vertical seismic coefficient at the top of the dam is taken as 0·75 times the seismic coefficient determined and reducing linearly to zero at the base. In other words, the vertical seismic coefficient distribution diagram from the top to bottom of the dam is one-half of that for the horizontal seismic coefficient. Knowing the value of a_{vy} at depth y and adopting the same procedure as that for the horizontal acceleration.

(ii) Response spectrum method: In this method, the value of vertical seismic coefficient (a_v) is taken as 0·75 times the seismic coefficient α at

the top of the dam and reducing linearly to zero at the base. For determination of the increase br decrease in the weight of concrete as well as water, 1.\le same procedure is used as for the horizontal acceleration.

Example 5: Calculate the earthquake forces and moments acting on the base of the dam shown in Figure. The dam is located in Zone V Assume $\gamma_c = 24{\cdot}0$ kN/m^3 and $E_c = 2{\cdot}1\times10^7$ kN/m^2. Use

(a) Seismic coefficient method (b) Response spectrum method.



Solution: (a) Seismic coefficient method

α_h at the top = $1{\cdot}5\alpha = 1{\cdot}5\,(\beta\times I\times\alpha_o)$

In this case, $\beta=1{\cdot}0$, $I= 2{\cdot}0$,

$\alpha_o = 0.08$ is the basic seismic coefficient, the value of α_o for different seismic zones can be obtained from table 1:

Therefore, α_h at the top= $1{\cdot}5\text{x}1{\cdot}0\text{x}2{\cdot}0\text{x}0{\cdot}08=0{\cdot}24$ Thus α_h varies linearly from 0·24 at the top to zero at the base.

For the top 10 m height of the dam, the horizontal force is given by:

$$F_{10} = \int_0^{10} 24\times(1\times8\times dy)\left(1-\frac{y}{80}\right)\times0.24$$

$$F_{10} = 46.08\left[y-\frac{y^2}{160}\right]_0^{10} = 46.08\left(10-\frac{100}{160}\right) = 432\text{ kN}$$

The moment of force about the horizontal section at 10 m below the top is given by:

$$M_{10} = \int_{0}^{10} 24 \times (1 \times 8 \times dy)\left(1 - \frac{y}{80}\right) \times 0.24(10 - y)$$

$$M_{10} = 46.08\left[10y - \frac{9 \times y^2}{16} + \frac{y^3}{240}\right]_0^{10}$$

$$= 46.08\left(100 - \frac{9 \times 100}{16} + \frac{1000}{240}\right)$$

$$= 432\,[100 - 56.25 + 4.167] = 2208.01\ \text{kN}$$

The horizontal force at the base is given by

$$F_{10} = F_{10} + \int_{10}^{80} 24 \times [(8 + 0.8\,(y - 10)]dy\left(1 - \frac{y}{80}\right) \times 0.24$$

$$F_{10} = F_{10} + 5.76\int_{10}^{80} (0.8y - 0.01y^2)dy$$

$$F_{10} = F_{10} + 5.76\left[0.4y^2 - \frac{0.01y^3}{3} + \frac{y^3}{240}\right]_{10}^{80} = 432 + 4704 = 5136\ \text{kN}$$

The moment of the force about any point at the base is given by

$$M_{80} = M_{10} + F_{10}(80 - 10) + \int_{10}^{80} 24[8 + 0.8(y - 10)]\,dy\left(1 - \frac{y}{80}\right) \times 0.24 \times (80 - y)$$

$$M_{80} = 2208.01 + 432 \times 70 + 5.76\int_{10}^{80} [64y - 1.6y^2 + 0.01y^3]\,dy$$

$$= 32448.01 + 5.76\left[32y^2 - \frac{1.6y^3}{3} + \frac{0.01y^4}{4}\right]_{10}^{80} = 213552.03\ \text{kN} - \text{m}$$

(b) Response spectrum method The fundamental period (T) is given by:

$$T = 5.55\left(\frac{H^2}{B}\right)\sqrt{\frac{\gamma_c}{gE_c}} = 5.55\left(\frac{80^2}{64}\right)\sqrt{\frac{24}{9.81 \times 2.1 \times 10^7}} = 0.189\ \text{sec}$$

The horizontal seismic coefficient, $\alpha_h = \beta \times I \times F_o (S_a/g)$

T= 0·189 sec and a damping of 5%, S_a/g = 0·19

Taking β =1·0, I= 2·0 and F_O = 0·40 for zone V from Table 1,

α_h= 1x 2·0x0·4x0·19 = 0·152

Weight of the dam perm length, W = 24 (80 x 8 + 0·5 x 56 x70) = 62400 kN

the horizontal force F_{80}= 0·6 W α_h = 0·6 x 62400 x 0·152 = 5690·88 kN

the moment at the base $M_{80} = 0{\cdot}9\,(W\,\overline{H}\,\alpha_h)/m$

where $\overline{H}$ is the height of the centre of gravity of the entire dam above the base,

$$\overline{H} = \frac{(80 \times 8) \times 40 + (0.5 \times 56 \times 70) \times (70/3)}{80 \times 8 + 0.5 \times 56 \times 70} = 27.433 \text{ m}$$

M_{80}=0.9×(62400×27.433×0.152)=234180.149 kN-m

3. Another method for calculate the effect of earthquake on gravity dams:

a- Effect of horizontal acceleration:

$$\mathbf{F_h = W \times \alpha_h}$$

Where: (F_h) the horizontal force due to earthquake, (W) weight of the dam and (α_h) horizontal seismic coefficient.

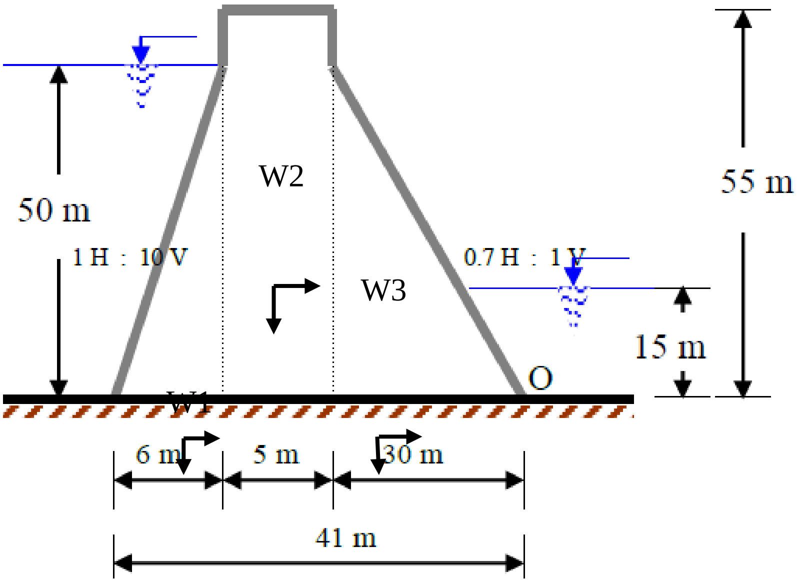
b- Effect of vertical acceleration:

$$\mathbf{F_v = W \times \alpha_v}$$

Where: (F_v) the vertical force due to earthquake, (W) weight of the dam and (α_v) vertical seismic coefficient.

Example 6:

Calculate the earthquake forces and moments acting on the base of the dam shown in Figure. If the values of (α_h=0.1 and α_v=0.05), γ_c = 25.0 kN/m^3.



Solution:

W1= 0.5×50×6×25=3750 kN

W2=5×55×25=6875 Kn

W3 = 50×30×0.5×25 = 18750 kN

F_{h1}=W1×α_h=3750×0.1=375 kN

F_{h2}=W2×α_h=6875×0.1=687.5 kN

F_{h3}=W3×α_h=18750×0.1=1875 kN

F_h= F_{h1}+ F_{h2}+ F_{h3}=375+687.5+1875=2937.5 kN

F_{v1}=W1×α_v=3750×0.05=187.5 kN

F_{v2}=W2×α_v=6875×0.05=343.75 kN

F_{v3}=W3×α_v=18750×0.05=937.5 kN

F_v= F_{v1}+ F_{v2}+ F_{v3}=187.5+343.75+937.5=1468.75 kN

The moment due to the horizontal forces about point (**O**) :

M1=375×16.67=6251.25 kN-m

M2=687.5×27.5=18906.25 kN-m

M3=1875×16.67=31256.25 kN-m

The total moment =56413.75 kN-m

The moment due to the vertical forces about point (**O**) :

M1=187.5×20=3750 kN-m

M2=343.75×32.5=11171.875 kN-m

M3=937.5×37=34687.5 kN-m

The total moment =49609.375 kN-m

3.4. STABILITY REQUIREMENTS OFAGRAVITY DAM:

The dam as a whole should be structurally safe and stable. Moreover every part of the dam should also be safe and stable. It should be able to with stand the stresses developed due to imposed loads. The foundations should be strong enough to carry the loads.

3.5. MODES OF FAILURE OF A GRAVITY DAM:

The gravity dam must be designed such that it is safe against all possible modes of failure with adequate factor of safety. The dam may fail in one or more of the following modes:

1. Overturning failure.
2. Sliding failure.
3. Crushing failure (or compression failure).
4. Tension failure.

3.5.1. FAILURE DUE TO OVERTURNING:

The dam may fail by overturning about its toe or about the downstream edge of any horizontal plane with in the dam. The overturning failure occurs when the resultant of all the forces acting on the base (or any other horizontal section) passes out side the base (or the horizontal section).

The factor of safety against overturning (F_o) is defined as the ratio of the sum of the stabilizing moments (or resisting moments) to that of the overturning moments about toe.

$$F_O = \frac{\sum \text{Stabilisingmoments}}{\sum \text{Overturningmoment}} \ldots\ldots\ldots \textbf{(3.33)}$$

Or

$$F_O = \frac{\sum MR}{\sum M0} \ldots\ldots\ldots \textbf{(3.34)}$$

A minimum factor of safety of l.50 is some times specified. However, if the dam is safe against tension and crushing failures, a factor of safety between l.50 to 2.50 is usually available and the dam does not fail in overturning.

Thus, the failure of the dam by overturning is usually pressed by the failure due to development of tension cracks (called tension failure) or due to crushing at the downstream edge (called crushing failure). Therefore, a gravity dam may be considered safe against overturning if the criterion of no tension at any point in the dam is satisfied and also the maximum stress does not exceed the allowable limit.

3.5.2. FAILURE DUE TO SLIDING:

The sliding failure occurs when the dam slides over its base or when a part of the dam lying above any horizontal plane slides over that plane. The sliding failure is resisted by the friction and the shearing strength of concrete. To a void the failure of the dam due to sliding at any horizontal section or at base, the dam should be designed such that the sliding forces do not exceed the resisting forces.

The factor of safety against sliding (Fs) is defined as the ratio of forces resisting sliding to the forces tending to cause sliding.

$$F_s = \frac{Sum\ of\ horizontal\ force\ resisting\ sliding}{Sum\ of\ horizontal\ force\ tending\ sliding} \ldots\ldots\ldots \textbf{(3.35)}$$

a) **Factor of safety (Fs) considering friction alone:** If the shear strength of the joint is neglected. The factor of safety against sliding is, therefore, given by:

$$F_s = \frac{\mu \sum V}{\sum H} \dots\dots\dots (3.36)$$

Where:

μ: is the coefficient of friction between the material above and below the horizontal section.

$\sum$V=Sum of all the vertical forces including uplift acting on the part of the dam above the horizontal section.

$\sum$H = Sum of all the horizontal forces acting on the part of the dam above the horizontal section.

The factor of safety against sliding (F_s) should be greater than1.0.

Sliding factor some time, instead of the factor of safety against sliding, the sliding factor (SF) is computed to check the stability against sliding,

$$SF = \frac{\sum V}{\sum H} \dots\dots\dots (3.37)$$

The value of **SF** should be less than the coefficient of friction where this indicate a factor of safety against sliding (**fs**) great than 1.0.

b) **Shear friction factor (SFF):** If the factor of safety (F_s) against sliding is less than unity it does not mean that the dam will fail due to sliding. In that case the shear friction factor (SFF) should be determined, If the S.F.F. is with in the safe limits the dam is considered to be safe against sliding. The shear friction factor is defined as the ratio of the total resistance, including shear strength to the total sliding force.

$$SFF = \frac{\mu \sum V + (b * q)}{\sum H} \dots\dots\dots (3.38)$$

Where:

b: is the width of the dam at the horizontal section under consideration. It is also equal to the area of cross section because the length of dam is taken as unity.

q: average shear strength of material at the horizontal section.

Other terms are the same as defined in Eq.(5.4). The value of *q* generally varies from 1400kN/m^2 to 5000 kN/m^2. A value of *q* =1400kN/m^2 (=1.4MPa) is generally taken for a preliminary design. The value of SFF should not less than 5 under normal (usual) condition and not be less than 4 under unusual conditions. The corresponding values for dams higher than 150 m are taken as 4 and 3, to effect economy.

3.5.3. FAILURE DUE TO CRUSHING (OR COMPRESSION FAILURE):

A dam may fail by the failure of its materials, i.e. the compressive stresses produced may exceed the allowable stresses, and the dam material may get crushed. The vertical direct stress distribution at the base is given by the equation:

P = Direct stress + bending stress.

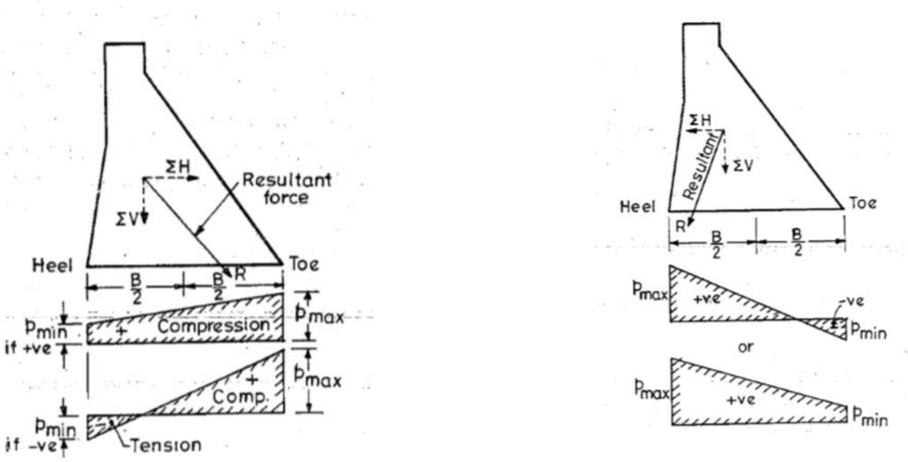
$$\mathbf{P_{max/min}} = \frac{\Sigma V}{B} \pm \frac{M}{I}\,Y = \frac{\Sigma V}{B} \pm \frac{\Sigma V * e}{B^2/6} = \frac{\Sigma V}{B}\left(1 \pm \frac{6e}{B}\right) \quad \text{……….(3.39)}$$

Where e = Eccentricity of the resultant force from the center of the base.

Σ V = Total vertical force.

B = Base width.

The plus sign is used to determine the maximum stress (P_{max}) and the minus sign for the minimum stress (P_{min}). as shown in figure (16) (a) and (b).



(a) (b)

Figure (3.16) vertical stress distribution for (a) reservoir full case, (b) reservoir empty case.

Thus we have the following equations for the determination of the vertical stresses (indicated by f) at the base (or any horizontal section) of the dam.

(a) Reservoir full:

- at toe(d/sedge), $$f_{yd}=\frac{\Sigma V}{B}\left(1+\frac{6e}{B}\right)$$
- at heel (u/sedge), $$f_{yu}=\frac{\Sigma V}{B}\left(1-\frac{6e}{B}\right)$$

Where the eccentricity(e) given by e=M/∑V

(b) Reservoir empty:

- at heel(u/sedge), $$f_{yd}=\frac{\Sigma V}{B}\left(1+\frac{6e}{B}\right)$$
- at toe(d/sedge), $$f_{yu}=\frac{\Sigma V}{B}\left(1-\frac{6e}{B}\right)$$

The suffixes u and d are used for the u/s and d/s edges respectively. The suffix y is used for the vertical stresses.

If ($\boldsymbol{Pmin}$) comes out to be negative, it means that tension shall be produced at the appropriate end.

If ($\boldsymbol{Pmax}$) exceeds the allowable compressive stress of dam material. (Generally taken as 3000 kN/m² for concrete; the dam may crush and fail by crushing.

Principal stresses:

The maximum direct stress in a complex stress system is the principal stress. The maximum direct stress is the major principal stress acting on the major principal plane passing through the point.

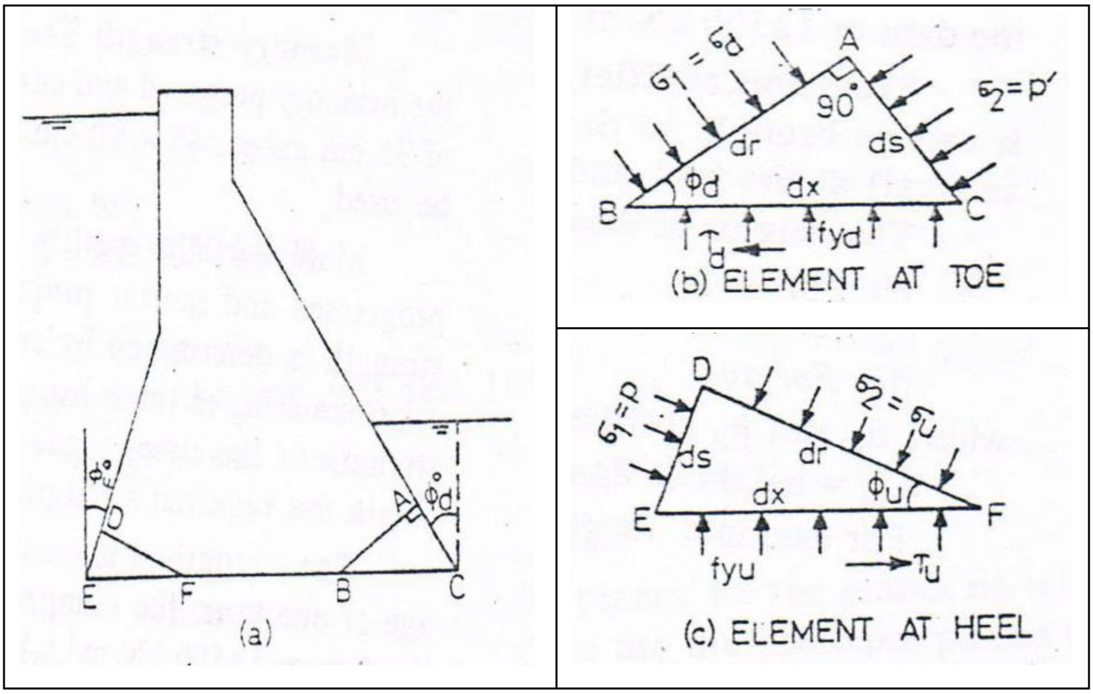


Figure (3.17) principal stress acting in a gravity dam.

a. Reservoir full condition:

The maximum vertical stress occurs at the toe (or the d/s edge of the horizontal section). The major ' principal stress is given by:

$$\boldsymbol{\sigma_d = f_{yd} sec^2 \emptyset_d - (\acute{p} - \acute{p}_e) tan^2 \emptyset_d \text{..........(3.40)}}$$

Where:

$\sigma_d =$ major principal stress , f_{yd} = maximum vertical stress , $\emptyset_d$ = angle which the d/s face makes with the vertical, $\acute{p}$= water pressure at the toe ($= \gamma \acute{H}$), $\acute{p}_e$= hydrodynamic pressure due to earthquake at toe.

If there is no tail water , $\acute{p} = \acute{p}_e = 0$ and the equation reduces to:

$$\boldsymbol{\sigma_d = f_{yd} sec^2 \emptyset_d}$$

The maximum shear stress at the d/s edge toe is given by:

$$\boldsymbol{\tau_d = [f_{yd} - (\acute{p} - \acute{p}_e)] \, tan \, \emptyset_d} \;\; \textbf{.......... (3.41)}$$

If $\acute{p} = \acute{p}_e = 0$ and the equation reduces to:

$$\boldsymbol{\tau_d = [f_{yd}] \, tan \, \emptyset_d}$$

The direction of the shear stress is towards the upstream.

b. Reservoir empty condition:

The plane DF at right angles to the upstream face ED of the dam is the minor principle plane and the maximum vertical stress occurs at the heel (or the u/s edge of the horizontal section) in the reservoir empty condition. The principle stress is given by:

$$\boldsymbol{\sigma_u = f_{yu} sec^2 \emptyset_u - (p + p_e) tan^2 \emptyset_u} \;\; \textbf{.......... (3.42)}$$

Where:

$\sigma_u =$ principal stress , f_{yu} = vertical stress at the hell, $\emptyset_u$= angle which the u/s face makes with the vertical,

p= water pressure at heel $(= \gamma H)$, p_e= hydrodynamic pressure due to earthquake at heel.

For the reservoir empty conditions p = p_e =0 and the equation reduces to: $\sigma_u = f_{yu} sec^2 \emptyset_u$

The maximum shear stress at the heel is given by:

$$\tau = -[f_{yu} - (p + p_e)] tan^2 \emptyset_u$$

If p = p_e =0 and the equation reduces to: $\tau = -[f_{yu}] \, tan\emptyset_u$

The direction of the shear stress is towards the downstream. The avoid overstressing of the material; the principal stresses should not exceed the allowable compressive stress in the dam and foundation. The compressive strength of the concrete (or masonry) must exceed the computed principal

stress by a safe margin.

Provision of fillet: if the compressive stress at the heel or toe is more than the permissible value t can be brought to permissible limits by providing fillet with slope 1:2 at the heel and 2:1 at the toe.

3.5.4. TENSION FAILURE:

Masonry and concrete gravity dams are usually designed in such a way that no tension is developed anywhere, because these materials cannot withstand sustained tensile stresses. As mention above if P_{min} comes out to be negative, it means that tension shall be produced at the appropriate end, so the eccentricity (e) should be less than B/6, i.e. the resultant should always lie within the middle third.

3.6. ELEMENTARY PROFILE OF A GRAVITY DAM:

An elementary profile of a gravity dam is the theoretical shape of its cross-section when it is subjected only to three main forces; which is self-weight, water pressure and uplift pressure. Moreover, the elementary profile has zero top width and no freeboard. Thus the elementary profile is a right-angle triangle with its apex at the water surface and a base width B.

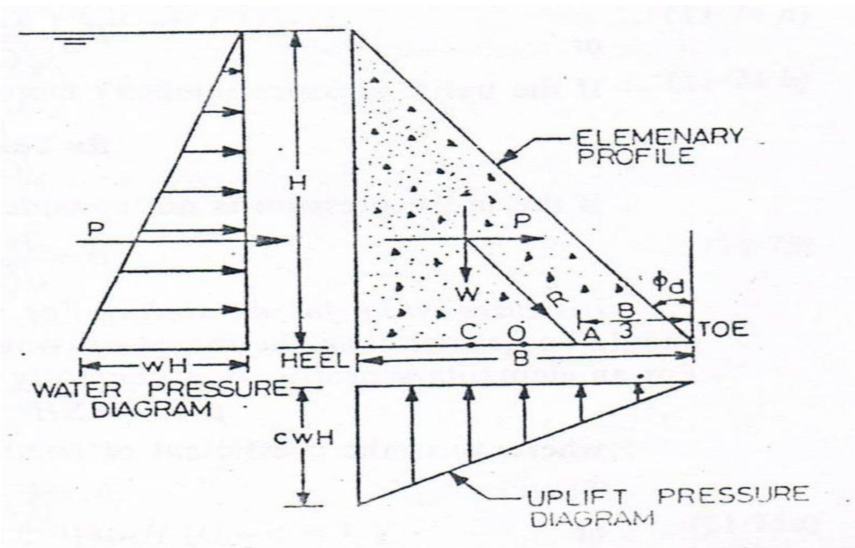


Figure (3.18) elementary profile of gravity dam.

Forces acting on an elementary profile:

The magnitudes and the lines of action of the three forces are found as follows:

i. **Self weight of the dam weight:** $W = \frac{1}{2}(BH)G\,\gamma_w$

Where G is the specific gravity of the dam material, which is usually taken as 2.40 for concrete, γ_w is the specific weight of water, *B* is the base width and *H* is the height of the dam. As already mentioned, the unit length of the dam is considered in the analysis.The weight W acts vertically downwards through the centroid of the triangle, at a distance of B/3 from the heel.

ii. **Water pressure:** $P = \frac{1}{2}(H^2)\gamma_w$

Total water pressure P acts horizontally at a height of H/3 above the base.

iii. **Uplift pressure:** $U = \frac{1}{2}(BH)\gamma_w$

The uplift force U acts at a distance of B/3 from the heel.

Range of Base width of elementary profile:

i. **Base width for non tension basis:**

$$B = \frac{H}{\sqrt{G-c}} \quad \text{..........(3.44)}$$

- If c = 0,G = 2.4, we have **B = 0.65H**
- If c = 1,G = 2.4, we have **B = 0.85H**

ii. **Base width for no sliding:**

$$B = \frac{H}{\mu\,(G-c)} \quad \text{..........(3.45)}$$

- If c = 0,G = 2.4, μ = 0.65, we have **B = 0.64H**
- If c = 1,G = 2.4, μ = 0.75 ,we have **B = 0.95H**

It is observed that the value of B varies from 0.64 to 0·95 times H . In actual practice, base width usually varies between 0·7 H to 0·8 H. The

slope of the d/s face is therefore, from 0·7:1 to 0·8:1 in most cases.

3.7. LIMITING HEIGHT OF A LOW:

The maximum height of the dam can be given by:

$$H_{cr} = \frac{f_a}{\gamma_w (G - c + 1)} \quad \text{..........(3.46)}$$

Where H_{cr} is the maximum height of the dam, called critical height, of the elementary profile which can be provided without exceeding the allowable stress (f_a) of the material.

A smaller value of H_{cr} will be obtained when the uplift pressure is not considered (i.e. c= 0.0), hence to be on the safer side, the limiting height of the elementary profile is usually given by:

$$H_{cr} = \frac{f_a}{\gamma_w (G + 1)}$$

For example, for fa = 3000 kN/m2 and G = 2.40,

H_{cr} = 3000/(9.81(2.4+ 1)) = 90 m.

It may be noted that the limiting height of 90 m for the elementary profile is for the assumed values of fa and G. If these values are changed, the limiting height will be different.

Low and high gravity dams distinction is usually made between a low gravity dam and a high gravity dam on the basis of the limiting height. A low gravity dam is the one whose height is equal to or less than the limiting height H_{cr}. (Figure 19a). On the other hand, if the height of the dams greater than the limiting height, it is called high dam (Figure 19 b).

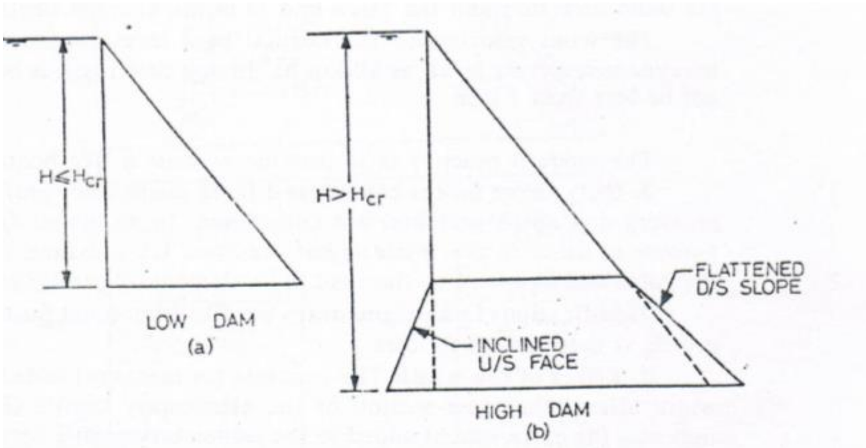


Figure (3.19) distinction of the of low and high gravity dam. (a) low dam, (b) high dam.

Thus:

a) For low dams, H ≤ H_{cr}

b) For high dams, H > H_{cr}

Where H is the actual height of the da m, and H_{cr} is the limiting height.

In a high dam. When the resultants trikes just at the outer middle third point, the maximum compressive stress is greater than the allowable stress if the elementary profile is provided. Therefore, in a high dam. The base width required is greater than that given by an elementary profile. Extra slopes are given on the upstream and downstream faces to increase the width in order to bring the compressive stress with in the allowable limits.

3.8. PRACTICAL PROFILE OF A GRAVITY DAM:

When the top width and free board are provided, the elementary section gets modified. Figure (2) shows a practical profile which can be used in practice. The required free board is provided. The top width (**a**) is equal to **0.14 H or the minimum required for the roadway**,

whichever is greater. The upstream face is vertical up to a depth of **2.0 a $\sqrt{G}$** measured below the water level and then it is provided with a slope from a depth of **2.0 a $\sqrt{G}$ to 3.1a $\sqrt{G}$** such that the horizontal projection is equal to a /16. The upstream face is again vertical below the depth of **3.1a $\sqrt{G}$.** Sometimes, a constant slope of 1 in 16 is provided on the upstream face below a depth of 2.0 a $\sqrt{G}$. In practice, the sections is usually adopted from experience.

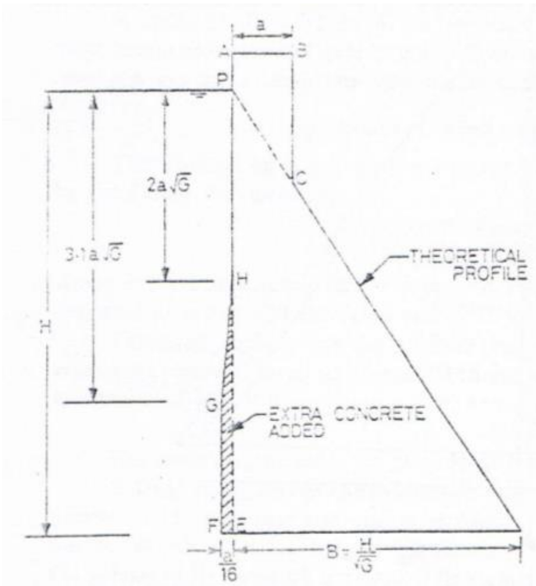


Figure (3.20) practically profile of a gravity dam.

3.9. ANALYSIS OF A GRAVITY DAM:

Once a section of a gravity dam has been assumed or designed, it has to be analysed and checked whether it satisfies the safety requirements. The various methods of analysis of a gravity dam can be classified as follows:

1. Gravity method (or two-dimensional method).
2. Three-dimensional methods:

(i) Trial load twist method.

(ii) Slab analogy method.

(iii) Lattice analogy method.

3. Experimental methods.
4. Finite element method.

In this section, the gravity method of analysis is discussed.

Gravity method of analysis:

In the gravity method of analysis, the dam is considered as a two- dimensional structure (a plane strain case). A unit length of the dam is considered for analysis. The dam is assumed to consist of a series of vertical cantilevers of unit length and fixed at the base. These cantilevers are assumed to be independent of one another. The load acting on the cantilever is transferred to the foundation through the cantilever action. The height of the cantilevers is considered up to the deepest foundation level. The distance between the two vertical planes of the cantilever normal to the axis of the dam is unity (i. e. 1 meter). The stability of these cantilevers is checked at different levels against all the possible modes of failure under worst loading conditions. If the assumed section does not satisfy the required stability requirements, it is modified and again checked . The process is repeated till an economic and safe section i s obtained.

The following procedure is commonly used . The calculations are usually done in a tabular form. Generally, the maximum section of the dam up to the deepest foundation level is checked first.

1. Determine all the forces acting per unit length of the dam. Determine the horizontal and vertical components of all forces.

2. Find the algebraic sum $\sum H$ of all the horizontal force components , and the algebraic sum $\sum V$ of all the vertical components .

3. Determine the moments of all the force components about any convenient point . It is the usual practice to take moments about the d/s edge or toe. It is convenient to write the resisting moment ($\boldsymbol{M_R}$) (counter clockwise) and the overturning moment $\boldsymbol{M_O}$ (clockwise) in separate columns of the table.

$$\sum \boldsymbol{M} = \sum \boldsymbol{M_R} - \sum \boldsymbol{M_O}$$

4. Determine the distance ($\bar{x}$) of the point where the resultant R strikes the horizontal section or the base:

$$\bar{\boldsymbol{x}} = \frac{\sum \boldsymbol{M}}{\sum \boldsymbol{V}}$$

5. Determine the eccentricity **e** of the resultant:

$$\boldsymbol{e} = \mathbf{0.5}\boldsymbol{B} - \bar{\boldsymbol{x}}$$

Where: B is the base width of the section. The **eccentricity** is considered positive when it is on the right side (or d/s side) of the center.

6. Determine the vertical stresses at the toe and heel of the dam from the relation.

$$\boldsymbol{F_y} = \frac{\sum \boldsymbol{V}}{\boldsymbol{B}}\left(\mathbf{1} + \frac{\mathbf{6}\boldsymbol{e}}{\boldsymbol{B}}\right)$$

Use plus sign for the toe and negative sign for heel for the reservoir full conditions. In the reservoir empty conditions, the signs are reversed because eccentricity (e) is negative.

7. Calculate the principal stresses from and check whether they are within the safe limits. Also check the shear stresses.
8. Determine the factor of safety against sliding Fs : if Fs is less than 1. Determine the shear friction factor (SFF) and check whether that is safe.
9. Determine the factor of safety against overturning F_o.
10. Determine the tensile stresses if the eccentricity is greater than B/6 , and check whether they are within limits within limits.

- ❑ Stability of other horizontal sections. The above procedure is for the base of the dam. The same procedure should be repeated for other horizontal sections at different elevations. Usually the dam is checked at every 3 to 5 m height.

3.10. EFFECT OF EARTHQUAKE FORCES ON WATER:

1. Effect of horizontal acceleration on water (Hydrodynamic pressure):

The horizontal acceleration acting upstream towards the reservoir causes a momentary increase in the water pressure. The additional water pressure exerted due to the earthquake is known as the hydrodynamic pressure. Generally, the following simplified methods are used to estimate the hydrodynamic pressure.

i. Von Karman's method:

Von Karman's suggested that the hydrodynamic pressure variation is parabolic and the total water pressure force due to earthquake is given by:

$$\boldsymbol{P_e = 0.555\alpha_h\gamma_w H^2} \text{..........} \textbf{(3.51)}$$

The line of action of $\boldsymbol{P_e}$ is at a height of ($4H/3\pi$) above the base.

ii. Zangar's method:

The intensity of hydrodynamic pressure at depth y below the water surface in the reservoir with total depth of water His given by:

$$p_{eH} = C\alpha_h \gamma_w h \text{ ....} (3.52)$$

Where: C is a dimensionless coefficient. The value of the coefficient C depends upon the slope of the u/s face of the dam and the depth of reservoir, as explained below.

a. U/S face of dam either vertical or having constant slope for the entire length. The value of C is given by

$$C = \frac{C_m}{2}\left(2 - \frac{y}{H}\right) + \sqrt{\frac{y}{H}\left(2 - \frac{y}{H}\right)} \text{ ....} (3.53)$$

Where C_m is maximum value of the coefficient C for a given slope of the u/s face of the dam. The value of C_m can be obtained from figure for different values of Φ, where Φ is the angle in degrees which the u/s face makes with the vertical. An approximate value of C_m can be obtained from the relation:

$$C_m = 0.735\left(1 - \frac{\emptyset}{90}\right) \text{ ....} (3.54)$$

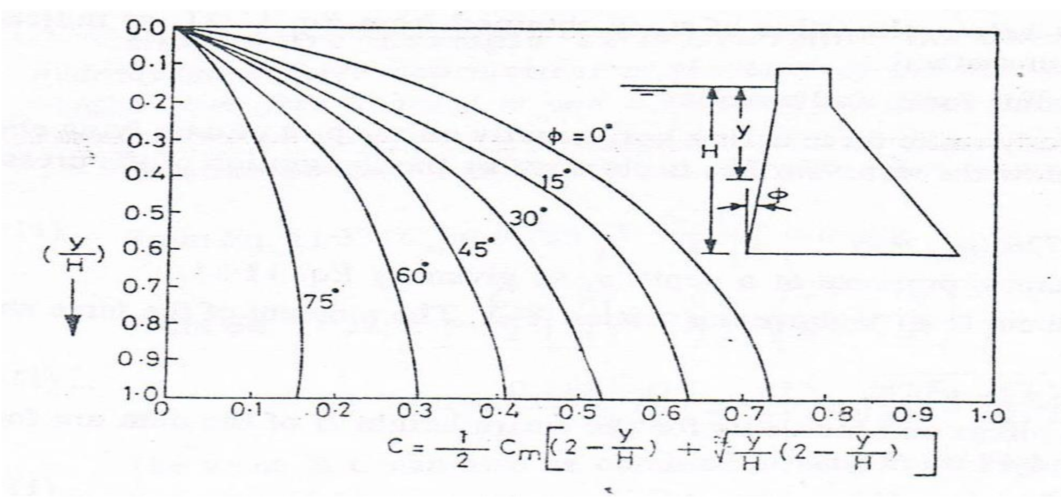
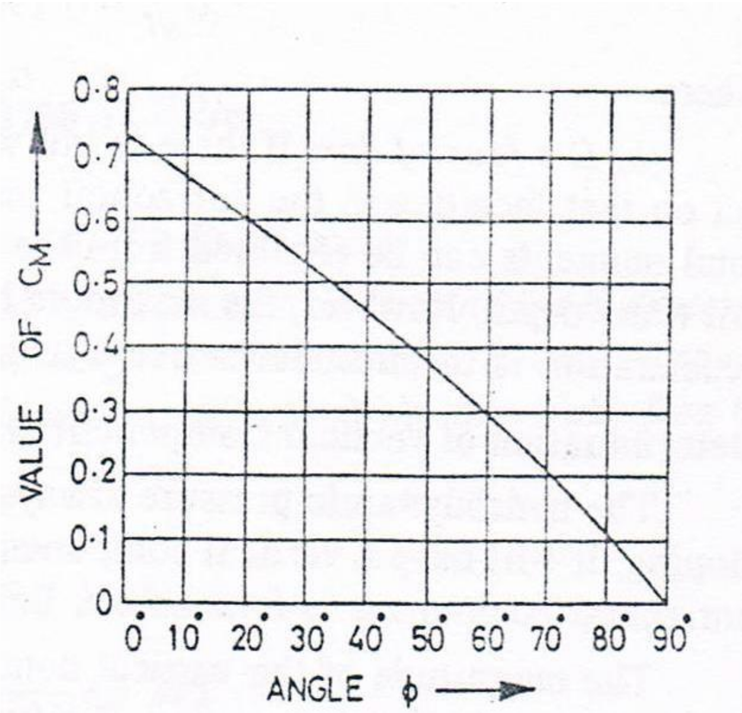


Figure (3.22) the value of C for different value of (y/H) and Ø.



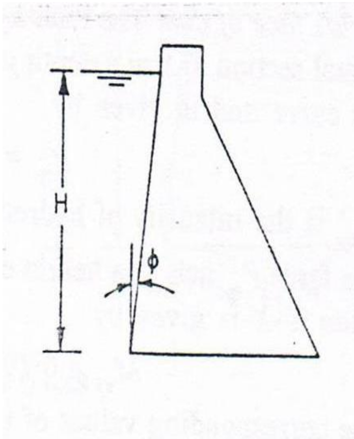


Figure (3.21) The value of Cm for different of Ø.

b. Partly vertical and partly sloping U/S face: for a dam in which the u/s face is partly vertical and partly sloping, the value of C is obtained as follows:

1. If the height h of the vertical portion of the u/s face is equal to or greater than one - half of the total height H of the dam, consider the entire face as vertical.
2. If the height h of vertical portion is less than one - half of the total height of dam, then consider the entire u/s face as sloping with a uniform slope. The slope is taken equal to the slope of the line joining the point of intersection of the u/s face of the dam with the water surface in the reservoir and the heel of the dam.

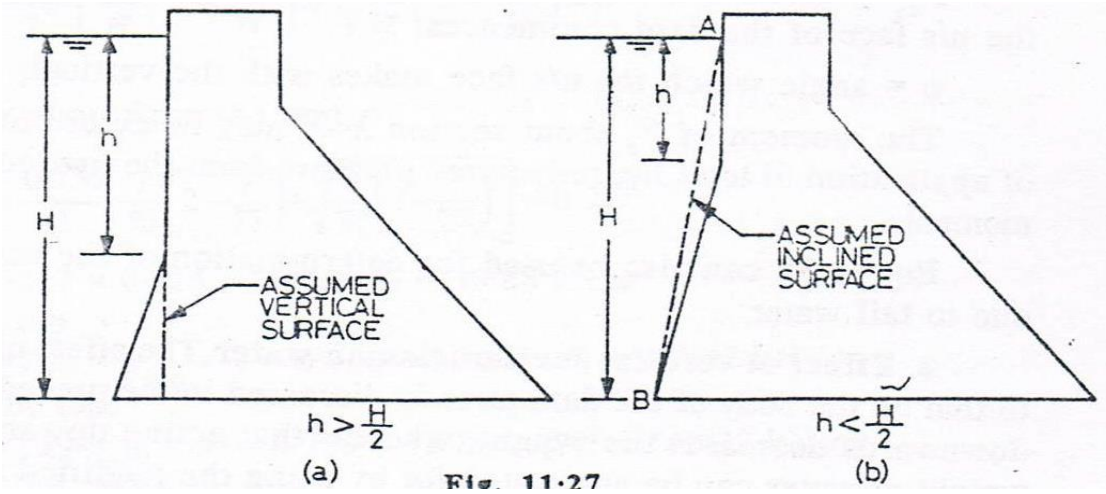


Figure (3.23 a and b) partly vertical and partly sloping U/S face dam.

Determination of the total hydrodynamic force and moment:

a) **u/s face of dam:** The total hydrodynamic force acting horizontally on the dam with a depth (H) below the water surface is given by :

$$\boldsymbol{P_{eH} = 0.726(p_{eH} * H)} \text{.........} \textbf{(3.55)}$$

Where: $\boldsymbol{P_{eH}}$ is the intensity of hydrodynamic pressure at a depth H, as given by: $\boldsymbol{p_{ey} = C\alpha_h \gamma_w h}$

The force $\boldsymbol{P_{eH}}$ acts at height of 0.41*H.

The moment of force is given by:

$$\boldsymbol{M_{eH} = 0.299(p_{eH} * H^2)} \text{ ....} \textbf{(3.56)}$$

b) **d/s face of dam:** If there is tail water on the d/s face of the dam, the hydrodynamic pressure will act on that face due to the horizontal acceleration. The values of the pressure intensity, total force and total moments can be obtained from the equations given by substituting $\boldsymbol{\overline{H}}$ for H where $\boldsymbol{\overline{H}}$ is the tail water depth. However, the maximum hydrodynamic pressure on the d/s face occurs when the earthquake acceleration is towards downstream and the corresponding inertia force acts in upstream direction.

2. Effect of vertical acceleration on water: The effect of vertical acceleration on the water is similar to that on the body of the dam already discussed in the preceding section. The vertical acceleration acting downwards decreases the weight, whereas that acting upwards, increases the weight.

3.11. TEMPERATURE CONTROL IN DAMS:

When cement concrete sets, a large amount of heat is liberated. This heat raises the temperature with in the body of the dam, where as the outside temperature remains equal to the atmospheric temperature. Thus, a temperature gradient develops due to the difference of temperatures. This results in the development of the temperature stresses in the dam. On the other hand. When the concrete cools down. It shrinks and the shrinkage stresses are developed. These temperature and shrinkage stresses cause cracking of concrete unless suitable measures are taken. In order to reduce cracking of concrete. Contraction joints are provided. But even the contraction Joints are of no avail to control cracking within the individual blocks which are also of quite large size. Therefore, within the individual blocks. cracks may develop if no measures are adopted to control the rise in temperature of concrete.

The following measures are usually adopted.

1. Precooling of concrete.
2. Post cooling of concrete.
3. Modifying design of mix.
4. Miscellaneous methods.

1. **Precooling of concrete:**

   Precooling is commonly used in practice. In this method. The temperature of the concrete is reduced before it is placed. The

ingredients of the concrete. (i.e. . fine aggregate, coarse aggregate and water) are precooled before mixing with cement which is not precooled. This lowered temperature of the concrete, to some extent, counteracts the rise in temperature.

2. **Post cooling of concrete:**

   In the post cooling method. The concrete is cooled after it has been placed in position. Post cooling of the concrete is immediately begun after the concreting of a block has been completed. And it is continued till the temperature of concrete falls to the mean annual temperature.

3. **Modifications in the design of concrete:**

   Mix in this method of temperature control, the design of concrete mix is suitably modified so that low heat of hydration is generated. To reduce heat of hydration, sometimes special low-heat cement is used in place of the normal Portland cement, or replacement a part of cement by pozzolanas.

4. **Miscellaneous methods:**

   The following miscellaneous methods are generally used to control the temperature.

i. By reducing the lifts. Temperature can be controlled to some extent, as the volume of concrete in the lift is reduced.

ii. By increasing the time period between the successive lifts, sufficient cooling time is available for dissipation of heat from the surface.

iii. As far as possible, curing by water should be done. As it reduces the surface temperature.

3.12. GALLERIES IN GRAVITY DAMS:

A galleries a small passage in a dam for providing an access to the interior of the dam. The galleries usually rectangular in shape with its top and bottom either flat or semi–circular and there are many types of it.

For a gallery with its top and bottom flat, it is necessary that all the corners should be rounded to reduce the stress concentration. If the gallery is with a semi- circular bottom, it is filled with unbounded concrete at the base to provide a flat surface for the walkway as shown in figure below.

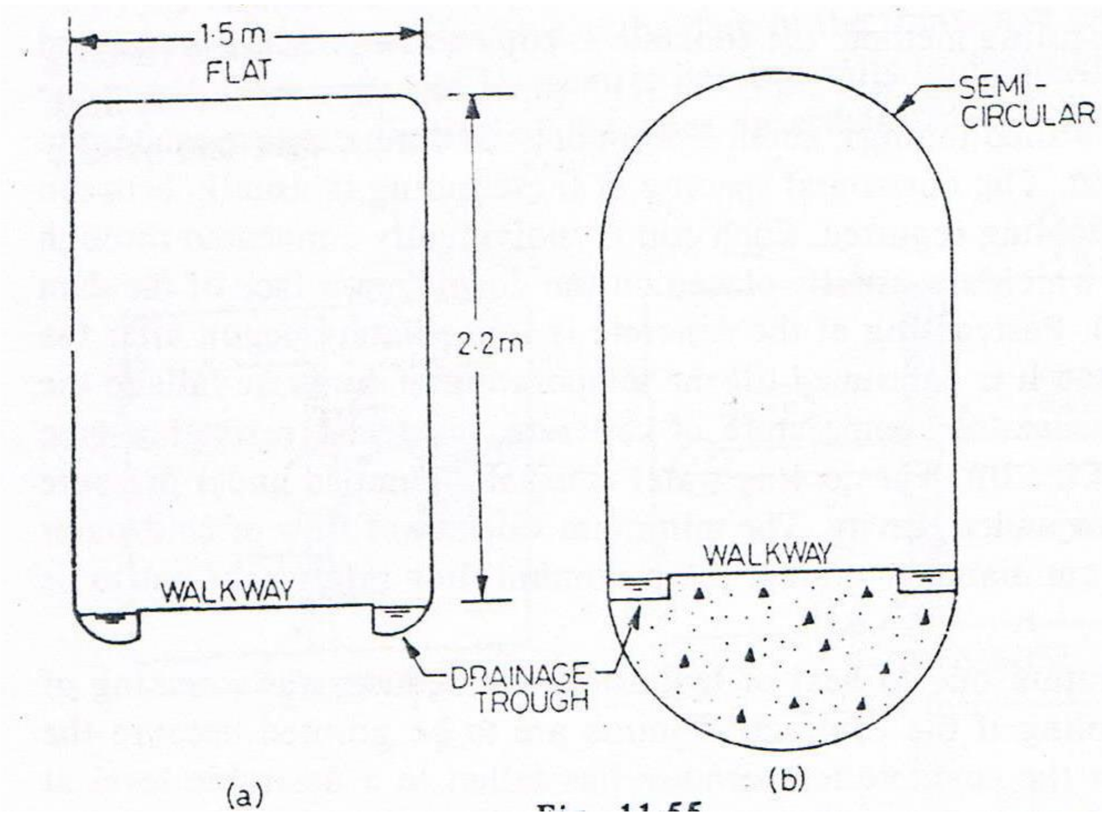


Figure (3.24) gallery in gravity dam.

The gallery should be sufficiently large to serve the required purpose. It should provide adequate working space and access for the equipment for normal maintenance. The width of the gallery generally varies from 1.5 to 1.8 m. the height of the gallery is generally between 2.2 to 2.4 m so that a person can easily walk inside it. However, it should be noted that galleries

are weak spot in the dam and they should be made as small as practically feasible to reduce the weakening effect. There are many types of galleries depending upon the purpose served such as :

1. Foundation gallery.
2. Grouting gallery.
3. Gate gallery.
4. Drainage gallery.
5. Inspection gallery.

Functions of a gallery:

The gallery may serve one or more of the following purposes.

- The gallery can be used as a drainage gallery to permit drainage of water percolating from the upstream of the dam into the body of the dam or its foundation.
- A gallery can be used for providing space for equipment required for drilling holes and grouting the holes to form a grout curtain in the foundation.
- A gallery provides an access to the interior of the darn for inspection and maintenance.
- A gallery also provides space for installing various instruments in the dam to study its structural behavior.
- A gallery can provide space for the mechanical and electrical equipment for the operation of gates for outlet conduits, penstocks or spillways.
- The piping system for post cooling of concrete can be accommodate ted in the gallery.

- The gallery can be used for placing equipment used for grouting of contraction joints.
- The gallery provides an access through the dam for control cable and power cabbies.
- The gallery provides access to the interior of the dam to the visitors.

Example 1:

Determine the maximum and minimum vertical stresses to which the foundation of the dam will be subjected from the following data :

Total overturning moment about toe ($\sum M_d$) = 1.2×10^6 kN-m

Total resisting moment about toe ($\sum M_R$) = 2.5×10^6 kN-m

Total vertical force above the base ($\sum V$) = 6×10^4 kN

base width of dam = 55 m. slop of d/s face = 0.8:1

Also calculate the maximum principal stress at the toe. Neglect tail water depth.

Solution Let $\bar{x}$ be the distance from the toe of the point where the resultant strikes the base. From

Eq 11.80, $\bar{x} = \frac{\sum M}{\sum V} = \frac{\sum MR - \sum Mo}{\sum V} = \frac{(2.5 \times 10^6 - 1.2 \times 10^6}{6 \times 10^4} = 21.67$ m

from Eq. 11.81, e = 0.5 B - x = 0.5 55 - 21.67 = + 5.83 m

From Eq. 11.82, the vertical stress is given by $f_y = \frac{\sum V}{B}(1 \pm \frac{6e}{b})$

The maximum vertical stress occurs at toe,

$f_{yd} = \frac{6 \times 10^4}{55}(1 + \frac{6 \times 5.83}{55}) = +\ 1784.73$ kNm^2(compression)

The minimum vertical stress occurs at the heel.

$f_{yu} = \frac{6 \times 10^4}{55}(1 - \frac{6 \times 5.83}{55})$ = + 397.09 kN/m² (compression)

From Eq. 11.62, the principal stress at toe

$\sigma_d = f_{yd}\sec^2\phi_d = 1784.73 \times [1 + (0.8)^2]$ = + 2926.96 kN/m²

Example 2:

Determine the maximum compressive stress and the shear stress at the toe of a dam if base width is 143 m and the eccentricity is 18m towards the d/s. The total vertical so - rce above the base is 326 MN and the slope of d/s face it 0.75:1. Neglect tail water.

Solution From Eq. 11.82,

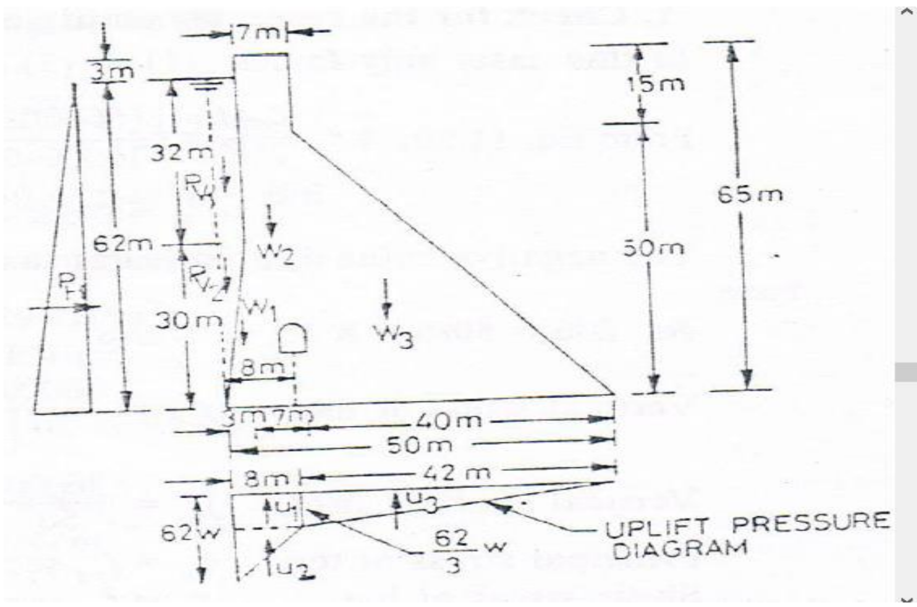
At toe, $f_{yd} = \frac{326}{143}(1 + \frac{6 \times 18}{143})$ = 4.00 Mpa

From Eq. 11.62, the principal stress at toe, $\sigma_d = f_{yd}\sec^2\phi_d = 4.00 \times (1 + 0.75^2)$ = + 6.25 Mp

From Eq. 11.65, the shear stress at toe, $\tau_d = f_{yd}\tan\phi_d = 4.00 \times 0.75$ = + 3.00 MPa.

Example 3:

Check the stability of the gravity dam shown in Fig. 11.43 for the reservoir empty reservoir full conditions. Also, find the principals and shear stresses at the toe and heel of the dam. Assume $\mu = 0.75$. Consider only the weight of the dam. Water pressure and uplift pressure. Take the average shear strength (q) = 1.4 MPa and w_c = 24 kN/m³. Also check the stability (i) when the uplift pressure s not act. (ii) when the drains are chocked.



Solution:

The calculations are shown in the table slow. The unit length of the dam is considered.

From Eq. 11.12, the uplift pressure intensity at the n line,

$$u = w\,H_d = w\left[H^{/} + \tfrac{1}{3}(H - H^{/})\right]$$

or $\quad u = w \times \left[0 + \tfrac{1}{3} \times 62\right] = 62\,(w/3) = 202.74\ \text{kN/m}^2$

The uplift pressure distributions diagrwn is drawn w the dam section in Fig. 11.43.

The pressure intensity at the heel, $p = w\,H = 62w = 608.22\ \text{kN/m}^2$

The water pressure distribution is triangular. The dam section is divided into three components 1,2

3. The corresponding weight are W_1, W_2, and W_3

The vertical components of the water pressure are marked as Pv_1 and Pv_2.

S.No.	Item	Description and dimensions	Forces (kN) Vertical (positive upwards)	Forces (kN) Horizontal (positive towards d/s)	Lever arm from the toe (m)	Moment about toe (kN-m) +ve (counter clockwise)	Moment about toe (kN-m) -ve (clockwise)
		(a) *Weight of dam*					
1.	W_1	$1/2 \times 3 \times 30 \times 24$	+ 1080·00		48·00	+ 51840·00	
2.	W_2	$7 \times 65 \times 24$	+ 10920·00		43·50	+ 475020·00	
3.	W_3	$1/2 \times 40 \times 50 \times 24$	+ 24000·00		80/3	+ 640000·00	
		(b) *Water pressure*					
4.	P_H	$9{\cdot}81 \times 1 \times (62)^2/2$		+ 18854·82	62/3		−389666·28
5.	Pv_1	$3 \times 32 \times 9{\cdot}81$	+ 941·76		48·50	+ 45675·36	
6.	Pv_2	$1/2 \times 3 \times 30 \times 9{\cdot}81$	+ 441·45		49·0	+ 21631·05	
		(c) *Uplift pressure*					
7.	U_1	$\left(\frac{62 \times 9{\cdot}81}{3}\right) \times 8$	− 1621·92		46·00		-74608·32
8.	U_2	$\frac{1}{2} \times \frac{2}{3} \times 62 \times 9{\cdot}81 \times 8$	− 1621·92		142/3		− 76770.88
9.	U_3	$\frac{1}{2} \times \frac{1}{3} \times 62 \times 9{\cdot}81 \times 42$	4257·54		28·00		+ 119211·12
Sum of forces and moments		(1) to (3)	+ 36000·00			+ 1166860·00	0·00
		(1) to (6)	+37383·21	+ 18854·82		+1234166·40	-389666·28
		(1) to (9)	+ 29881·83	+ 18854·82		+1234166·40	-660256·60

1. Check for the reservoir empty conditions

In this case, only forces (1) to (3) act.

From Eq. 11,80, $\bar{x} = \frac{\sum M}{\sum V} = \frac{1166860.00}{36000.00} = 32.41$ m

e = 0.5 B - $\bar{x}$ = 0.5× 5.0 - 32.40 = - 7.41 m.

The negative value of e indicates that the eccentricity is towards the upstream of the centre of the use.

As B/6 = 50/6 = 8.33, e < B/6.

Vertical stress at toe, $f_{yd} = \frac{36000}{50}\left(1 - \frac{6 \times 7.41}{50}\right) = +\ 79.78$ kN/m^2 (compression)

Vertical stress at heel, $f_{yu} = \frac{36000}{50}\left[1-(\frac{-7.41\times 6}{50})\right]$= + 1360.22 kN/m² (compression)

Principal stress at toe. σ_d=$f_{yd}\sec^2\phi_d$ = 79.78 × (1 + 0.8²) = 130.84 kN/m² (compression)

shear stress at toe. τ_d=$f_{yd}\tan\phi_d$ = 79.78 × 0.8 = + 63.82 kN/m² (towards upstream)

Princin at heelσ_u=$f_{yu}\sec^2\phi_u$

or τ_u= 1360.22 $\times[1+(0.1)^2]$ = + 1373.82 kN/m²(compression)

shear stress at heel, τ_u = - $f_{yu}\tan\phi_u$= - 1360.22 × 0.1 = - 136.02 kN/m² (towards downstream)

Check for tension Since e < B/6, there is not tension.

Check for sliding Since there is not sliding force, sliding will not occur.

Check for overturning As there is no overturning moment, check for overturning is not required.

2. Check for the reservoir full conditions, considering uplift

In this case, all the forces (1) to (9) will act.

$\bar{x}=\frac{\sum M}{\sum V}=\frac{1234166.40-660256.60}{29881.83}$ = 19.21 m

e = 0.5 B - $\bar{x}$ = 25 - 19.21 = 5.79 m

As B/6 = 50/6 = 8.33, e < B/6

Vertical stress at the toe, $f_{yd}=\frac{\sum V}{B}(1+\frac{6e}{B})=\frac{29881.83}{50}(1+\frac{6\times 5.79}{50})$ = 1012.87 kN/m²

Principal stress at the toe = 1012.87 $\times$ [1 + 0.8^2]= + 1661.11 kN/m²

shear stress at the toe = 1012.87 $\times$ 0.8 = + 810.29 kN/m²

Vertical stress at the heel, $f_{yu} = \frac{\sum V}{B}(1 - \frac{6e}{B}) = \frac{29881.83}{50}(1 - \frac{6 \times 5.97}{50})$ = 169.49 kN/m²

Principal stress at the heel = 169.49 $\times$ [1 + 0.1^2] = + 171.18 kN/m²

Shear stress at the heel = - 169.49 $\times$ 0.10 = - 16.95 kN/m²

Check for tension since e < B/6, there is no tension

Check for sliding Eq. 11.42, $F_s = \frac{\mu \sum V}{\sum H} = \frac{0.75 \times 29881.83 + 50 \times 1400}{18854.82}$ = 1.19> 1.00

As the factor of safety against sliding is greater than unity, there is no need to compule SFF

However, for the illustration, it is computed.

Form Eq. 11.46, $SFF = \frac{\mu \sum V + Bq}{\sum H} = \frac{0.75 \times 29881.83 + 50 \times 1400}{18854.82}$ = 4.90

Check for overturning From Eq. 11.40, $F_0 = \frac{\sum M_R}{\sum M_o} = \frac{1234166.44}{660256.60}$ = 1.87 > 1.50

3. check for the reservoir full condition, considering no uplift

In this case , only the forces (1) to (6) act.

$\bar{x} = \frac{1234166.40 - 389666.28}{37383.21}$ = 22.59 m

e = 0.5 B - $\bar{x}$ = 25 - 22.59 = 2.41 m < B/6

Vertical stress at toe, $f_{yd} = \frac{37383.2}{50}(1 + \frac{6 \times 2.41}{50})$ = 963.89 kN/m²

principal stress at toe, σ_d = 963.89 ×1.64 = 1580.79 kN/m²

Shear stress at toe, τ_d= 963.89 $\times$ 0.8 = 771.11 kN/m²

Vertical stress at heel, $f_{yu} = \frac{37383.21}{50}\left(1 - \frac{6 \times 2.41}{50}\right) = 531.44$ kN/m^2

Principal stress at heel, $\sigma_u = 531.44 \times 1.01 = 536.75$kN/m^2

Shear stress at heel, $\tau_u = 531.44 \times 0.1 = 53.14$ kN/m^2

Check for tension As e < B/6, there is no tension

Check for sliding Fs = $\frac{0.75 \times 37383.21}{18854.82} = 1.49 > 1.00$

SFF = $\frac{0.75 \times 37383.21 + 50 \times 1400}{18854.82} = 5.20$

Check for overturning F = $\frac{1234166.40}{389666.28} = 3.17$

4. Check for the reservoir full condition, drains choked

When the drains are choked, full uplift pressure acts on the base. The intensity of uplift pressure at the heel is $9.81 \times 62 = 680.22$ kN/m^2 and that at the toe is zero, because there is not tail water.

Total uplift, U = $\frac{1}{2} \times 608.22 \times 50 = 15205.5$ kN

This force acts at a distance of 100/3 m from the toe.

Moment about tve = $15205.5 \times 100/3 = -506850.00$ kN-m

Now $\sum V = 37383.21 - 15205.5 = 22177.71$ kN

$\sum M_R = +1234166.40$, $\sum M_o = -389666.28 - 506850.00 = -896516.28$ kN-m

$\sum M = 1234166.40 - 896516.28 = +337650.12$ kN-m

$\bar{x} = \frac{337650.12}{22177.17} = 15.22$ m

e = 0.50 B - 15.22 = 25 - 15.22 = 9.78 m > B/6

Vertical stress at toe, $f_{yd} = \frac{22177.71}{50}\left(1 + \frac{6 \times 9.78}{50}\right) = 964.11$ kN/m^2

Principal stress at toe, σ_d = 964.11 × 1.64 = 1581.14 kN/m²

Shear stress at toe, τ_d = 964.11 × 0.8 = 771.29 kN/m²

Vertical stress at heel, $f_{yu} = \frac{22177.71}{50}\left(1 - \frac{6 \times 9.78}{50}\right)$ = -77.00 kN/m² (tension)

Principal stress at heel, σ_u = -77 × 1.01 = -77.77 kN/m² (tension)

Shear stress at heel, τ_u= + 77 × 0.1 = 7.77 kN/m² acting towards upstream

Check for tension As e > B/6, tension is developed at heel.

Maximum tension = 77.77 kN/m²

Check for sliding F = $\frac{0.75 \times 22177.71}{18854.82}$ = 0.88 < 1.00 (unsafe)

SFF = $\frac{0.75 \times 22177.71 + 50 \times 1400}{18854.82}$ = 4.59 (safe)

Check for overturning $F_o = \frac{1234166.40}{896516.28}$ = 1.38 < 1.50 (unsafe)

Example 4:

Check the stability of the gravity dam section shown in Fig. 11.43 (Example 11.12), considering the seismic forces, in addition to self weight. water pressure and uplift. Assume: α_v = 0.10 and α_h = 0.20

(a) Consider only vertical acceleration (b) Consider only horizontal acceleration

Consider only the reservoir full condition.

Solution

For the worst case, the horizontal acceleration should act towards upstream and the vertical acceleration should act downwards when the

reservoir is full. The inertia forces in the vertical and horizontal directions are determined by multiplying the weights computed in Example 11.12 by 0.10 and 0.20 , respectively, as shown in the table below. These forces are shown in rows 10 to 15 and 16 to 21, respectively. The suffix e_v is used for vertical forces and e_h for the horizontal forces.

The hydrodynamic pressure is determined from Eq. 11.31. Since the height of the vertical portion of the upstream face is greater than one-half of the total height, the upstream face is assumed to be vertical for the determination of the hydrodynamic pressure on the upstream.

Thus $$C_m = 0.735\ (1 - \frac{\phi}{90}) = 0.735\ (1\text{-}0) = 0.735$$

From Eq. 11.31, the intensity of hydrodynamic pressure at the heel,

$P_e = C\ \alpha_h w\ H = 0.735 \times 0.2 \times 9.81 \times 62 = 89.41$ kN/m^2

The total hydrodynamic force acting at the u/s face, from Eq.11.37,

$P_e = 0.726\ \ P_e\, H = 0.726 \times 89.41 \times 62 = 4024.52$ kN

Moment about toe. From Eq .11.3

$M_e = 0.299\ \ P_e\, H^2 = 0.299 \times 89.41 \times (62)^2 = 102763.92$ kN-m

S.No.	Item	Description and Dimensions	Forces (kN)		Lever arm from the toe (m)	Moments about toe (kN-m)	
			Vertical (Positive downwards)	Horizontal (Positive towards d/s)		+ve M (counter clockwise)	-ve M (Clockwise)
1 to 9		From the table of Example 11·12	+ 29881.83	+ 18854·22		+ 1234166·40	– 660256·60
		(a) Vertical acceleration					
10.	W_{1ev}	0·1 × 1080	- 108·00		48·00		– 5184·00
11.	W_{2ev}	0·1 × 10920	- 1092·00		43·50		– 47502·00
12.	W_{3ev}	0·1 × 24000	- 2400·00		80/3		– 64000·00
13.	P_{Hev}	0·1 × 18854·82		- 1885·48	62/3	+ 1256·99	
14.	P_{1ev}	0·1 × 941·76	- 94·18		48·5		– 4567·73
15.	P_{2ev}	0·1 × 441·45	- 44·15		49·00		– 2163·35
		(b) Horizontal acceleration					
16.	W_{1eh}	0·2 × 1080		+ 216·00	10		– 2160·00
17.	W_{2eh}	0·2 × 10920		+ 2184·00	32·50		– 70980·00
18.	W_{3eh}	0·2 × 24000		+ 4800·00	50/3		– 80000·00
19·	P_{eh}	(Calculated earlier)		+ 4024·52			– 102763·93
20.	P_{v1eH}	0·2 × 941·76		+ 188·35	46·0		– 8664·10
21.	P_{v2eH}	0·2 × 441·45		+ 88·29	20		– 1765·80
	Sum (1) to (15)		+ 26143·50	+ 16969·34		1235423·4	– 783673·68
	Sum (1) to (9) and (16) to (21)		+ 29881·83	+ 30355·98		1234166·40	– 926590·42

(a) Considering Intertie forces due to vertical acceleration only Forces (1) to (15) act in this case.

$\bar{x} = \frac{+1235423.4 - 783673.68}{26143.50} = 17.28$ m

e = 0.5 B - $\bar{x}$ = 25.00 - 17.28 = 7.72 m < B/6

Vertical stress at toe, $f_{yd} = \frac{26143.50}{50}(1 + \frac{6 \times 7.72}{50}) = 1007.26$ kN/m^2

Principle stress at toe, σ_d = 1001.26 × 1.64 = 1651.90 kN/m^2

Shear stress at toe, τ_d = 1007.26 × 0.80 = 805.81 kN/m^2

Vertical stress at heel, $f_{yu} = \frac{26143.50}{50}(1 - \frac{6 \times 7,72}{50}) = 38.48$ kN/m^2

principal stress at heel $\sigma_u = f_{yu} \sec^2\phi_u$ - (p + p_e) $\tan^2\phi_u$

= 38.48 1.01 - (608.22 × 0.9 + 0.0) 0.01

= 38.86 - 5.47 = 33.39 kN/m^2

shear stress at heel $\tau_u = -\left[f_{yu} - (\mathrm{p} + \mathrm{pe})\right]\tan\phi_u$

$= -\left[38.48 - (608.22 \times 0.9 + 0.0\right] \times 0.1 = 50.89$ kN/m²

Check for tension As $e < B/6$, there is no tension.

Check for sliding. $F_s = \frac{0.75 \times 26143.50}{16969.34} = 1.16 > 1.00$ (safe)

$\mathrm{SFF} = \frac{0.75 \times 26143.50 + 50 \times 1400}{16969.34} = 5.28$ (safe)

Check for overturning, $F_o = \frac{1234166.40}{783673.68} = 1.58$ (safe)

(b) Considering inertial forces due to horizontal acceleration only

In this case. forces (1) to (9) and (16) to (21) act.

$\bar{x} = \frac{1234166.40 - 926590.42}{29881.83} = 10.29$ m

$e = 0.5\,B - \bar{x} = 25 - 10.29 = 14.71 > B/6$

Vertical stress at toe, $f_{yd} = \frac{29881.83}{50}\left(1 + \frac{6 \times 14.71}{50}\right) = 1652.58$ kN/m²

Principle stress at toe, $\sigma_d = 1652.58 \times 1.64 = 2710.23$ kN/m²

Shear stress at toe, $\tau_d = 1652.58 \times 0.80 = 1322.06$ kN/m²

Vertical stress at heel, $f_{yu} = \frac{29881.83}{50}\left(1 - \frac{6 \times 14.71}{50}\right) = -457.31$ kN/m²

(tension)

principal stress at heel $\sigma_u = f_{yu}\sec^2\phi_d - (\mathrm{p} + \mathrm{pe})\tan^2\phi_u$

$= -457.31 \times 1.01 - (608.22 \times 0.9 + 89.41) \times 0.01$

$= -461.88 - 6.98 = -468.86$ kN/m²

shear stress at heel $\tau_u = -\left[f_{yu} - (\mathrm{p} + \mathrm{pe})\right]\tan\phi_u$

$= - [-57.31 - (608.22 \times 89.41)] \times 0.1 = +115.49$ kN/m²

Check for tension Maximum tension = 457.31 kN/m²

Check for sliding. $F_s = \frac{0.75 \times 29881.83}{30355.98} = 0.74 < 1.00$

$SFF = \frac{0.75 \times 29881.83 + 50 \times 1400}{30355.98} = 3.04$

Check for overturning $F_o = \frac{1234166.40}{926590} = 1.33$

Example 5:

Check the stability of the overflow section of the gravity dam shown in Figure:

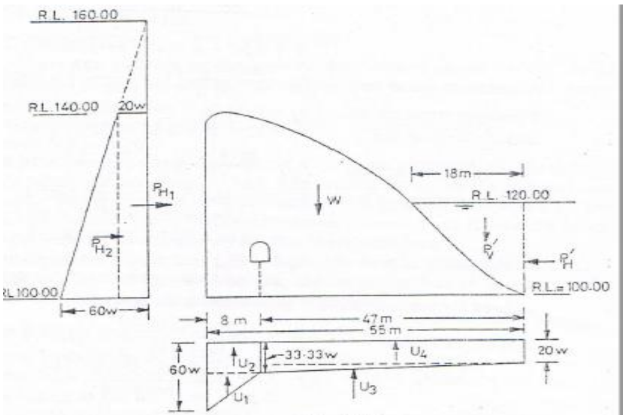


Fig. 11.44

Assume the weight of concrete, gates piers and water over crest, etc. = 3.0×10^4 kN..

Moment of weight of concrete ,gates, piers and water over crest,etc. about toe $=10^6$ kN-m. Neglect all forces other than weight ,uplift pressure and water pressure. Take μ=0.75 and q=1400 kN/m²

Solution

The uplift pressure at the drain line from Eq. 11.12

$\mu = w\,Hd = w\left[20 + \frac{1}{3}(60 - 20)\right]$

$= w \times 33.33 = 326.97$ kN/m²

The calculation are shown in the table below.

S.No.	Item	Description and Dimension	Force (kN) Vertical (Positive upwards)	Force (kN) Horizontal (Positive towards d/s)	Lever arm (m)	Moment about toe (kN-m) +ve (counter - clockwise)	Moment about toe (kN-m) -ve (clockwise)
1.	W	Weight (given)	+ 30000			1000000.0	
2.	P_v'	$\frac{18 \times 20}{2} \times 9.81$	+ 1765.8		6	10594.8	
3.	P_{H_1}	$20 \times 9.81 \times 40$		+ 7848.0	20		156960.00
4.	P_{H_2}	$\frac{9.81 \times (40)^2}{2}$		+ 7848.0	40/3		104640.0
5.	P_H'	$\frac{9.81 \times (20)^2}{2}$		− 1962	20/3	13080.00	
6.	U_1	$1/2 \times 26.67 \times 9.81 \times 8$	− 1046.53		52.33		54764.92
7.	U_2	$33.33 \times 8 \times 9.81$	− 2615.74		51.00		133402.74
8.	U_3	$1/2 \times 13.33 \times 9.81 \times 47$	− 3073.03		31.33		96278.03
9.	U_4	$20 \times 9.81 \times 47$	− 9221.40		23.50		216702.9
		Sum	+ 15809.1	+ 13734.00		1023674.8	762747.67

$$\bar{x} = \frac{1023674.8 - 762747.67}{15809.1} = 16.50 \text{ m}$$

e = 0.5 B - $\bar{x}$ = 27.5 - 16.50 = 11.00 m B/6

Vertical stress at toe, $f_{yd} = \frac{15809.1}{53}\left(1 - \frac{6 \times 11.00}{55}\right) = 656.64$ kN/m^2

Principle stress at toe, $\sigma_d = 1188.52 \times 158.92 = 1029.60$ kN/m^2

= 1188.52 - 158.92 = 1029 = 1029.60 kN/m^2

Shear stress at toe, $\tau_d = (656.64 - 20 \times 9.81) \times 0.9 = 414.40$ kN/m^2

Vertical stress at heel, $f_{yu} = \frac{15809.1}{53}\left(1 - \frac{6 \times 11}{55}\right) = -\ 59.66$ kN/m^2

principal stress at heel $\sigma_u = -\ 59.66 \times 1.0 - (60 \times 9.81) \times 0.0 = -\ 59.66$ kN/m^2

shear stress at heel $\tau_u = 0.0$

Check for tension Maximum tension = 59.66 kN/m^2

Check for sliding. $F_s = \frac{0.75 \times 15809.1}{13734.00} = 0.86 < 1.00$

$SFF = \frac{0.75 \times 15809 - 55 \times 1400}{13734.00} = 5.4$

Check for overturning, $F_o = \frac{1023674.8}{762747.67} = 1.34$ (unsafe)

CHAPTER FOUR

ARCH DAMS

Arch Dams

4.1. Introduction:

An arch dam is a concrete dam that is curved upstream in plan. The arch dam is designed so that the force of the water against it, known as hydrostatic pressure, presses against the arch, compressing and strengthening the structure as it pushes into its foundation or abutments. (An arch dam is curve shaped solid wall which is generally built with cement concrete. It is preferred where river valleys are formed).

An arch dam is most suitable for narrow canyons or gorges with steep walls of stable rock to support the structure and stresses. Since they are thinner than any other dam type, they require much less construction material, making them economical and practical in remote areas.(The working nature of arch dam is like partly cantilever retaining wall and partly arch action. The whole curved wall stands with its large base and transfers the loads to the two ends of the dam using horizontal thrust).

4.2. Types of Arch Dams:

1. Single Curvature Arch Dam.
2. Double Curvature Arch Dam.

Single Curvature Arch Dam:

A Single curvature arch dam is curved only in plan because it has a curvature only in the horizontal plan. There are three types of Single curvature arch dam based on the shape face of the arch dam:

- Constant Radius Arch Dam
- Variable Radius Arch Dam
- Constant Angle Arch Dam

In constant radius arch dam, the upstream face of the dam has a constant radius making it a linear shape face throughout the height of the dam. But the inner curves their radius reduces as we move down from top elevation to bottom and thus in cross-section it makes a shape of a triangle.

For variable arch dam, the radius of both inner and outer faces of the dam arch varies from bottom to top. The radius of the arch is greatest at the top and lowest at lower elevations. The central angle of the arch is also widened as we move upside.

The third type of the arch dam which is constant angle arch dam is the most economical. However, for the third type of arch dam stronger foundation is required as it involves overhangs at the abutment sections. The constant angle arch dam is that in which the central angles of the horizontal arch rings are of the same magnitude at all elevations. All different types of arch dam can be seen in Figre1 and 2 as it mention above.

Double Curvature Arch Dam:

Double curvature arch dam is provided not only in the horizontal direction but also in the vertical direction. It means the cross section of double curvature also looks like curve.

The whole dam is looks like shell type so, it is also called as shell arch dam. Because of double curvature, the thickness of arch wall is reduced. But this non-vertical type dams are difficult to construct compared to other three types. They also require stronger foundations. Shell arch dams are more suitable for very narrow valleys.

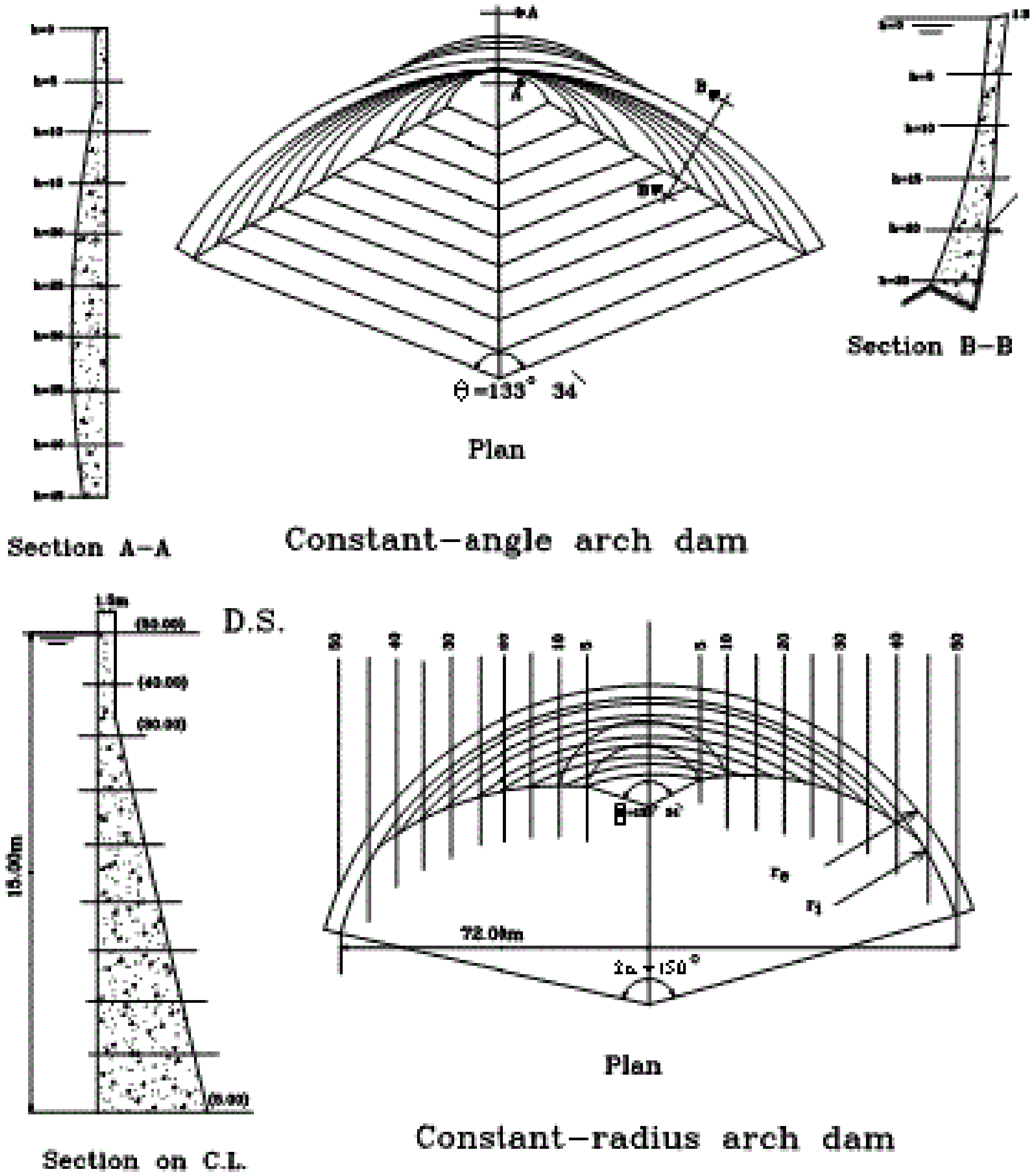


Figure (4.1) constant angle and radius dam

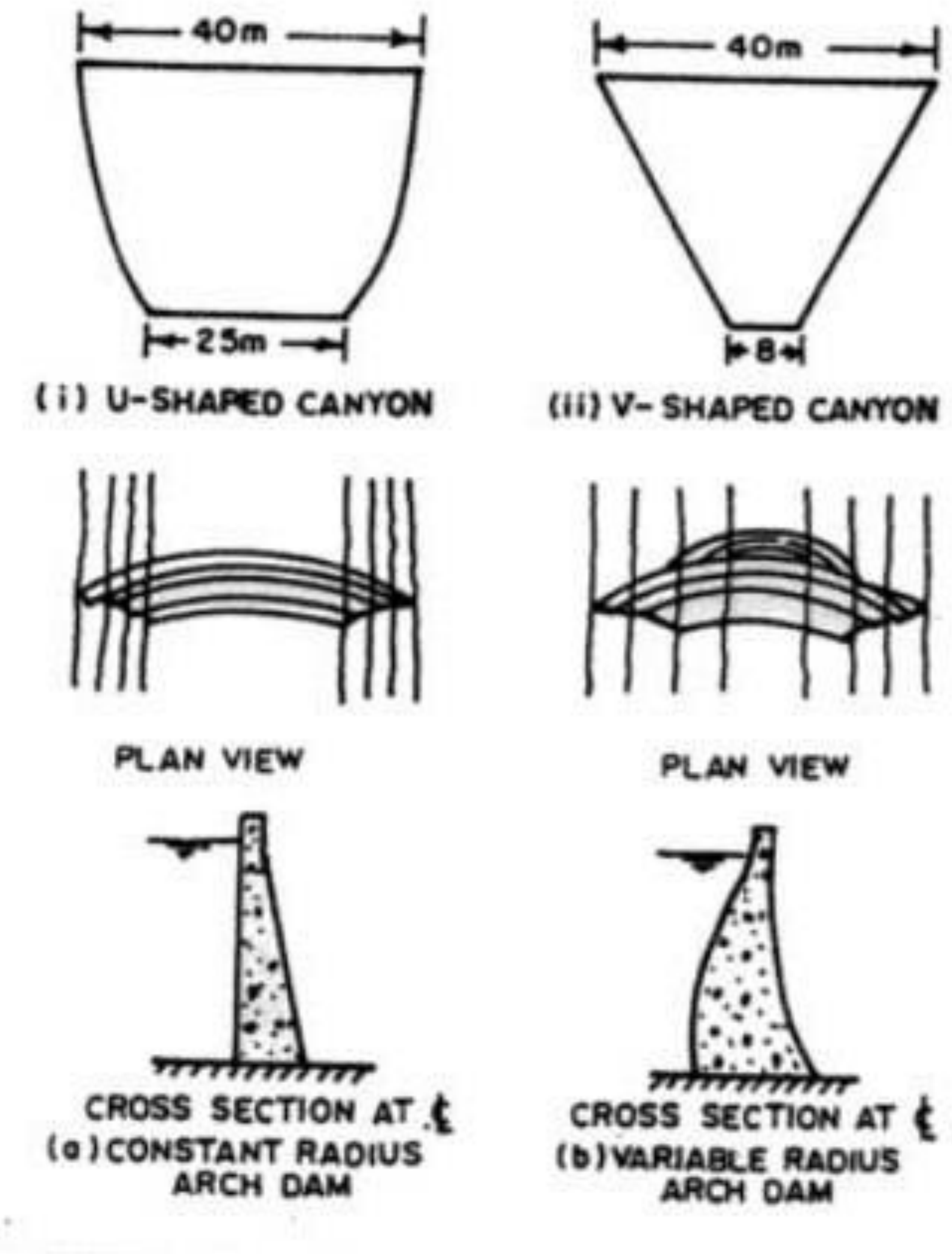


Figure (4.2) types of arch acceding to the valley shape

4.3. Classification of Arch Dams:

In general, arch dams are classified based on the ratio of the base thickness to the structural height (b/h) as:

- Thin, for b/h less than 0.2,
- Medium-thick, for b/h between 0.2 and 0.3, and
- Thick, for b/h ratio over 0.3.

Arch dams classified with respect to their structural height are:[1]

- Low dams up to 100 feet (30 m),
- Medium high dams between 100–300 ft (30–91 m),
- High dams over 300 ft (91 m).

4.4. Design of Arch Dams:

Arch dams can be designed on the basis of the following methods:

i. Thin cylinder theory;
ii. Theory of Elastic arches;
iii. The Trial load method;
iv. Shell theory;
v. Finite element method;
vi. Engineering Monograph (EM) No. 36 method

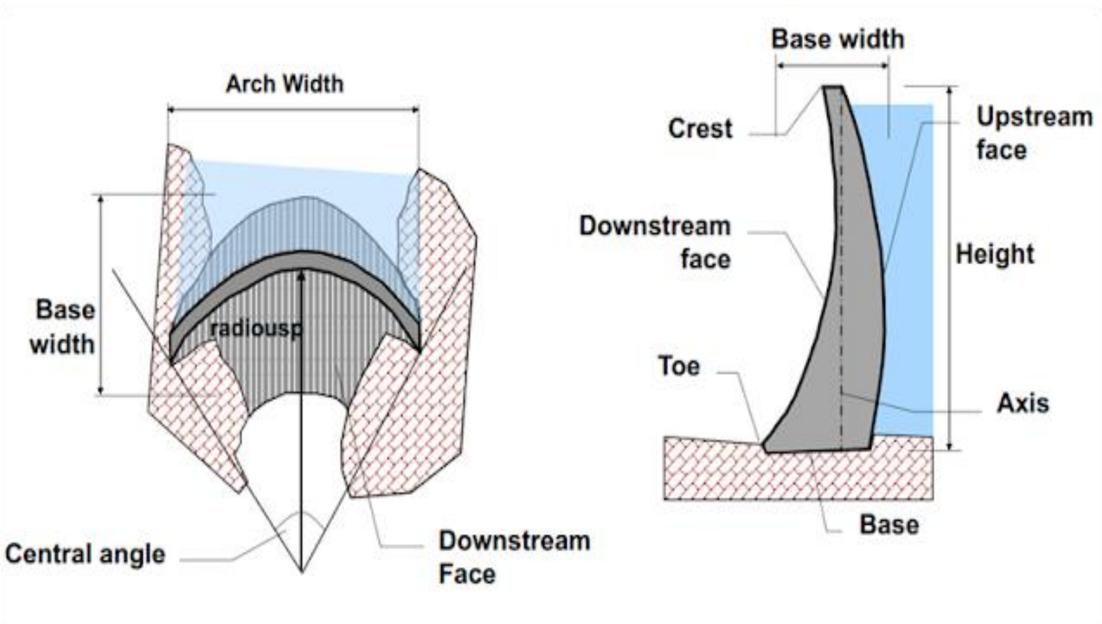


Figure (4.3) cross section of arch dam

The selection method of design an arch dam depends on the structural behavior that will be considered such as cantilever or ring member. For ring behavior the Thin Cylinder Theory method will be used and for cantilever behavior the Engineering Monograph (EM) No. 36 method will be used. An arch dam will be appropriate when "Width / Height of valley" [(B/H) < 6]. Rocks at the base and hillsides should be strong enough with high bearing capacity. Reduce the volume of concrete. Stresses are allowed to be as high as allowable stress of concrete. Generally, the same forces act on an arch dam, which do act on a gravity dam.

These forces are:

i. Water pressure.
ii. Uplift pressure.
iii. Earthquake forces.
iv. Silt pressure.
v. Wave pressure.
vi. Ice pressure.

Thin Cylinder Theory Method:

The weight of concrete and water in the dam is carried directly to the foundation; the horizontal water load is carried entirely by arch action. In thin cylinder theory, the stresses in the arch are assumed to be nearly the same as in a thin cylinder of equal outside radius

- If R is the abutment reaction its component in the upstream direction which resists the pressure force P is equal to R sin $\theta/2$
- The hydrostatic pressure acting in the radial direction

$$\boldsymbol{P = \gamma_w * h \ldots\ldots\ldots.4.1}$$

- The projected area is equal to $2 * r_e * \sin\theta/2$
- Total hydrostatic force = hydrostatic pressure x projected area

$$P = \gamma_w * h * 2 * r_e * sin\frac{\theta}{2} \ldots\ldots\ldots 4.2$$

Summing forces parallel to the stream axis = 0

$P = 2 * R * sin\,\theta/2$

$2*R * sin\,\theta/2 = \gamma_w * h * 2 * r_e * sin\,\theta/2$

$$R = \gamma_w * h * r_e \ldots\ldots\ldots 4.3$$

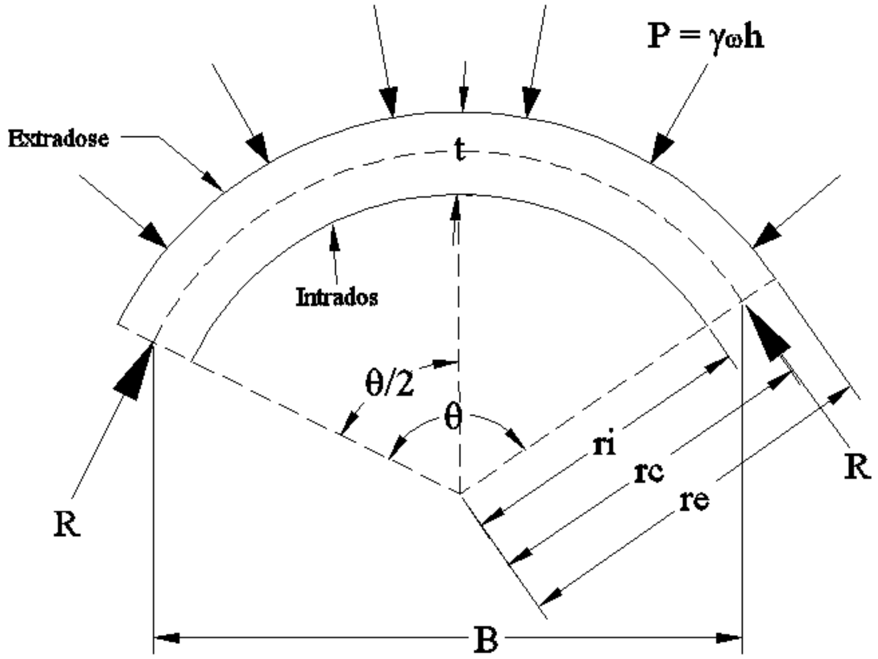


Figure (4.4) force arch dam in thin cylinder theory

If the thickness (t) of the arch ring is small compared with (r_e) it may be assumed that uniform compressive stress is developed in the arch ring.

The transverse unit stress $\sigma = \frac{R}{t*1}$ $\sigma = \frac{\gamma_w * h * r_e}{t} \ldots\ldots\ldots 4.4$

For a given stress, thickness (t) $t = \frac{\gamma_w * h * r_e}{\sigma_{all}} \ldots\ldots\ldots 4.5$

Note: the hydrostatic pressure $\gamma * h_w$ may be increased by earth quake and other pressure forces where applicable.

This equation indicates that the thickness t of the arch ring increases linearly with depth below the water surface and for a given pressure the

required thickness is proportional to its radius. Thickness relation in terms of intrados, ri and mean radius rc , can be derived as follows :

re = rc + 0.5*t $$\boldsymbol{t = \frac{\gamma_w * h * r_c}{\sigma_{all} - 0.5\ \gamma_w * h}} \ \ldots\ldots\ldots\ \mathbf{4.6}$$

re = ri + t $$\boldsymbol{t = \frac{\gamma_w * h * r_i}{\sigma_{all} - \gamma_w * h}} \ \ldots\ldots\ldots\ \mathbf{4.7}$$

For best central angle the concrete volume (V) of any given arch is proportional to the product of the arch thickness and the length of the centerline arc. The volume of unit height of arch:

$$V = (1 * t) * r * \theta$$

$$t = \frac{\gamma_w * h * r}{\sigma_{all}}$$

$$V = \left(1 * \frac{\gamma_w * h * r}{\sigma_{all}}\right) * r * \theta$$

$$V = \frac{\gamma_w * h}{\sigma_{all}} * \theta * r^2$$

$$r = (B/2)/sin(\theta/2)$$

$$V = \frac{\gamma_w * h}{\sigma_{all}} * \theta * \left[\frac{B}{2 * sin(\theta/2)}\right]^2$$

Let $K = 0.25 * \gamma_w * h * B^2/\sigma_{all}$

$$V = \frac{K * \theta}{sin^2(\theta/2)}$$

Differentiating V with respect to θ and setting to zero, $\frac{dV}{d\theta} = 0$

$$\frac{d}{d\theta}\left[\frac{K * \theta}{sin^2(\theta/2)}\right] = 0$$

$$\sin^2(\theta/2) - 2 * \theta * si\,n(\theta/2) * (0.5 \cos(\theta/2)) = 0$$

$$\sin^2(\theta/2) = \theta * si\,n(\theta/2) * \cos(\theta/2)$$

$$\tan(\theta/2) = \theta$$

$$\boldsymbol{\theta = 2.331\ radians} \qquad \boldsymbol{\theta = 133.5}$$

This is the most economical angle for arch with minimum volume

For $\theta = 133.50$

$r = (B/2)/sin(\theta/2)$

$r = (B/2)/sin(133.5/2)$

$$\mathbf{r = 0.544 * B \ldots\ldots\ldots 4.8}$$

The various limitations of this theory are:

i. The arch sections are not thin cylinders. They are also not free at abutments, as assumed in this theory.

ii. The theory does not consider shear and bending stresses in the arch.

iii. The analysis is based on only the hydrostatic water pressure. Temperature stresses and ice pressures, which are quite important in arch dams, get ignored in this theory.

iv. Stresses due to yielding of abutments and those due to rib shortening have not been accounted in this theory.

v. Plastic flows of concrete and shrinkage in concrete have not been accounted for.

Arch Dams, EM No. 36:

Reclamation has made significant engineering contributions to the advancement and evolution of arch dam analysis, design, and construction since the formation of Reclamation in 1902. There are several documents published by the Reclamation that are extensively used by the industry in layout and design of the arch dams. The most significant publications include:

- Guide for Preliminary Design of Arch Dams, EM No. 36, Third Ed. 1977.
- Design Manual for Concrete Arch Dams, 1977.

In 1966, Engineering Monograph (EM) No. 36 was written explaining the initial steps to laying out a double-curvature arch dam. The monograph presents formulas and charts to rapidly determine the initial dimensions and concrete volume for a 8 double curvature arch dam. The volume estimates are based on statistical data of geometrical properties of twelve arch dam layouts as requested by the District Offices. The monograph was primarily intended as an aid for planners in the field to estimate the volume of concrete and subsequently to estimate appraisal level costs for an arch dam.

Then empirical formulas for crown cantilever thickness. The shape of the crown cantilever is defined by three thicknesses, upstream projections (USP) and downstream projections (DSP) at the crest, at the base, and at 0.45H elevation by the following empirical formulas:

$$USP = 0.0$$
$$DSP = T_c$$

At $0.45H$ projection

$$max.\ USP = 0.95T_B$$
$$min.\ DSP = 0.0$$

Base projection

$$USP = 0.67T_B$$
$$DSP = 0.33T_B$$

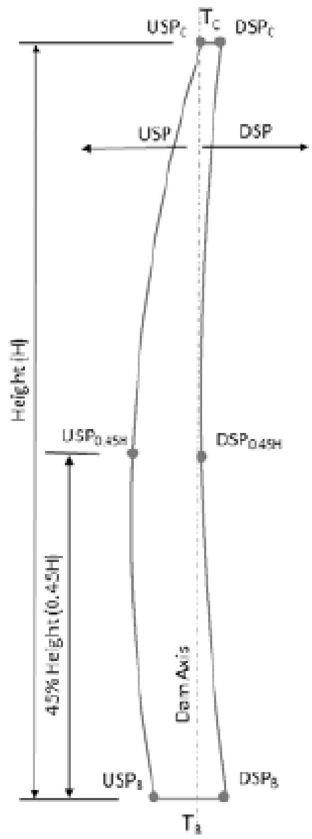


Figure (4.5) double curvature arch dam

As a result of the statistical analyses, empirical formulas were developed for computing the volume of concrete in a dam and for sufficient dimensions for a crown cantilever to produce an adequate shape. Dimensions, in feet, required for solving the equations are: H, the structural height (which is the vertical distance from the crest of the dam to the lowest assumed point of foundation); L_1, the straight line distance at crest elevation between abutments, assumed excavated to sound rock; and L_2, the straight line distance at (0.15 H) between abutments, assumed excavated to sound rock.

Crest thickness $$T_C = 0.01 * [H + 1.2\,L_1] \ldots\ldots\ldots 4.9$$

Base thickness $$T_B = \sqrt[3]{0.0012 * H * L_1 * L_2 * \left(\frac{H}{400}\right)^{\left(\frac{H}{400}\right)}} \ldots 4.10$$

Volume of dam

$$V = 0.000002\,H^2\,L_2 * \left[\frac{(H + 0.8 * L_1)^2}{L_1 - L_2}\right] + 0.0004\,H\,L_1 * [H + 1.1\,L_1] \ldots\ldots\ldots\ldots 4.11$$

Example 1:

Given a canyon with the following dimensions, compute and draw the layout of arch dams of **1-constant radius** and **2- constant angle** profiles.

Data :

- Maximum height = 100m

-Top width of the valley = 500m

-Bottom width of valley =200m

-Allowable stress in concrete σ *all*=5 *Mpa*.

Solution:

Solution-Using thin cylinder method

1. **Constant radius:**

Let the central angle be 150°

Assume top width is 1.5m or assume 0

$$r = \frac{\frac{B}{2}}{\sin\left(\frac{\theta}{2}\right)} = \frac{\frac{500}{2}}{\sin\left(\frac{150}{2}\right)} = 258.82$$

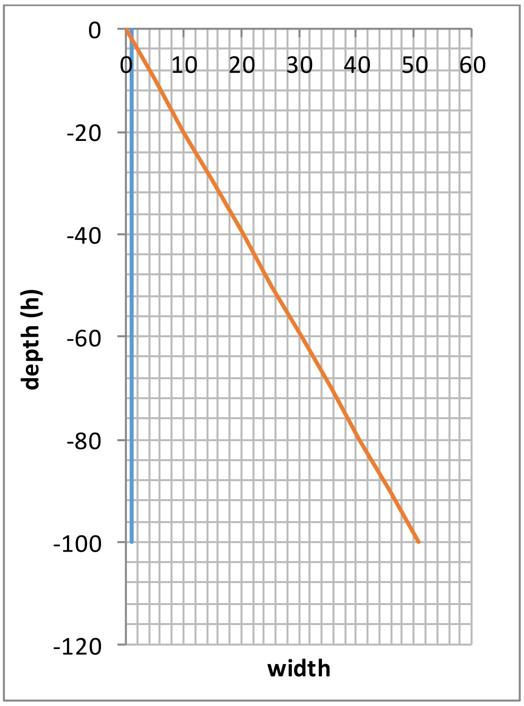
$$t = \frac{\gamma_w * h * r}{\sigma_{all}} = \frac{9.81 * h * 258.82}{5000} = 0.508\ h$$

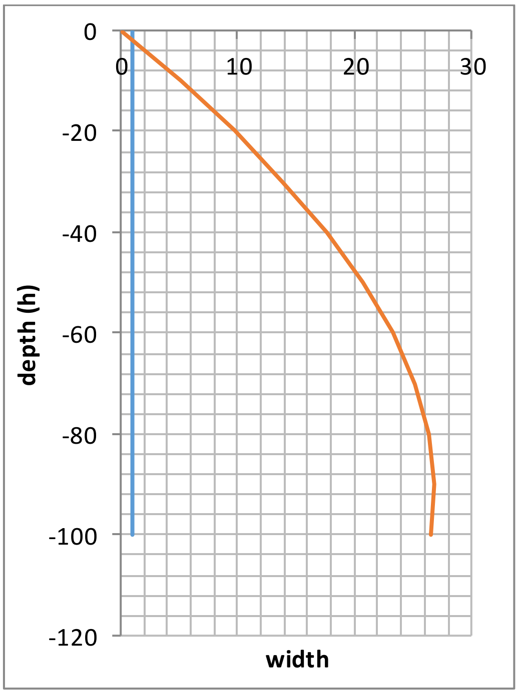
Depth (h)	width (B)	t=0.508 h	$r_i = r_e - t$	$\frac{B}{2r_e}$	$\theta = 2\sin^{-1}\left(\frac{B}{2r_e}\right)$
0	500	0.00	258.82	0.9659	149.9885
10	470	5.08	253.74	0.9080	130.4432
20	440	10.16	248.66	0.8500	116.4182
30	410	15.24	243.58	0.7921	104.7493
40	380	20.32	238.50	0.7341	94.45637
50	350	25.40	233.42	0.6761	85.08074
60	320	30.48	228.34	0.6182	76.36316
70	290	35.56	223.26	0.5602	68.13959
80	260	40.64	218.18	0.5023	60.29788
90	230	45.72	213.10	0.4443	52.75674
100	200	50.80	208.02	0.3864	45.4545

2. Constant Angle

The best central angle $\theta = 133.5^0$

depth (h)	Width (B)	$r_i = 0.544$ B	$\frac{\gamma h}{1000}$	$\frac{\gamma h r_i}{1000}$	$\sigma_{all} - \gamma * h$	t	$r_e = r_i + t$
0	500	272.00	0	0	5	0.00	272.00
10	470	255.68	0.0981	25.082	4.902	5.12	260.80
20	440	239.36	0.1962	46.962	4.804	9.78	249.14
30	410	223.04	0.2943	65.641	4.706	13.95	236.99
40	380	206.72	0.3924	81.117	4.608	17.61	224.33
50	350	190.40	0.4905	93.391	4.510	20.71	211.11
60	320	174.08	0.5886	102.463	4.411	23.23	197.31
70	290	157.76	0.6867	108.334	4.313	25.12	182.88
80	260	141.44	0.7848	111.002	4.215	26.33	167.77
90	230	125.12	0.8829	110.468	4.117	26.83	151.95
100	200	108.80	0.981	106.733	4.019	26.56	135.36





Example 2:

A constant radius (cylindrical) will be 80 m in height thickness is equal to ¼ of maximum dam height. allowable tension of rings 220 t / m², modulus of elasticity of concrete 2×10^6 t / m², find the defect of radius

Solution:

Note: σ_{all} in ton then γ_w should be in ton and its equal to 1 t/m³

$$t = 0.25 * h$$

$$t = 0.25 * 80 = 20\ m$$

$$t = \frac{\gamma_w * h * r_c}{\sigma_{all} - 0.5\,\gamma_w * h}$$

$$20 = \frac{1 * 80 * r_c}{220 - 0.5 * 1 * 80}$$

$$r_c = 45 \text{ m}$$

$$\sigma_{all} = E * e$$

$$e = \frac{\delta}{rc}$$

$$\sigma_{all} = E * \frac{\delta}{rc}$$

$$220 = 2 * 10^6 * \frac{\delta}{45}$$

$$\delta = 0.00495 \cong 0.005$$

$$\delta = 5\ cm$$

Example 3:

From the estimated structural height, H-290 feet, and the measured chord lengths, L_1 =550 feet and L_2=160 feet, find thicknesses volume of the double Curvature Arch Dam

Solution:

1- At crest

$$T_C = 0.01 * [\mathrm{H} + 1.2\,\mathrm{L}_1]$$

$$T_C = 0.01 * [290 + 1.2 * 550]$$

$$T_C = 9.5\,ft \text{ use 10 ft}$$

2- At base

$$T_B = \sqrt[3]{0.0012 * \mathrm{H} * \mathrm{L}_1 * \mathrm{L}_2 * \left(\frac{\mathrm{H}}{400}\right)^{\left(\frac{\mathrm{H}}{400}\right)}}$$

$$T_B = \sqrt[3]{0.0012 * 290 * 550 * 160 * \left(\frac{290}{400}\right)^{\left(\frac{290}{400}\right)}}$$

$$T_B = 28.9 \text{ ft use 29 ft}$$

3- At 0.45 H

$$T_{0.45} = 0.95 * T_B$$

$$T_{0.45} = 0.95 * 28.9$$

$$T_{0.45} = 27.45 \text{ use 27.5 ft}$$

4- Volume of dam

$$V = 0.000002\,\mathrm{H}^2\,\mathrm{L}_2 * \left[\frac{(\mathrm{H}+0.8*\mathrm{L}_1)^2}{\mathrm{L}_1-\mathrm{L}_2}\right] + 0.0004\,\mathrm{H}\,\mathrm{L}_1 * [\mathrm{H} + 1.1\,\mathrm{L}_1]$$

$$V = 0.000002 * 290^2 * 160 * \left[\frac{(290+0.8*550)^2}{550-160}\right] + 0.0004 * 290 * 550 * [290 + 1.1 * 550] = 36800 \text{ cubic yards}$$

CHAPTER FIVE

BUTTRESS DAMS

Buttress Dam

5.1. Introduction:

An ordinary concrete gravity dam, as we know, is a solid body of mass concrete, somewhat triangular in section, running across the entire width of the river valley. Such a solid wall requires huge amount of concrete, which partly remains unstressed to full extent, thereby leading to wastage of concrete. The uplift pressure also acts on the entire width of the dam body, from its bottom, which further increases its size, without giving any additional benefit as a dam.

Efforts have been made from time to time, to innovate methods for affecting economy in the use of concrete, by cutting down the concrete from those dam portions where it remains unstressed. Attempts have, therefore been made to provide hollow gravity dams. Buttress dams, are an improvement innovation over the hollow concrete gravity dams.

Buttress dams consists of principal structural elements: A sloping upstream deck that supports the water

5.2. Types of Buttress Dams:

There are five types of buttresses dams, out of which deck slab type and multiple arch types are most commonly used. Those five are as follows.

I. Deck slab buttress dam.
II. Multiple arch buttress dam.
III. Massive head buttress dam.
IV. Multiple dome buttress dam.
V. Columnar buttress dam

Deck Slab Buttress Dam:

In this type of dam, a deck slab is provided which is supported by the corbels of buttresses. This type of dam is constructed to smaller heights generally from 20 to 50 meters.

The slab provided is inclined to the horizontal by about 40 to 55 degrees. This inclination is needed to stabilize the dam and support the dead load of stored reservoir water and also it prevents sliding due to self-weight of dam.

In this case, each deck slab unit on two adjacent buttresses acts as one independent single unit. Hence, if one unit gets affected or damaged then no need to worry about other units.

Design considerations of deck slab buttress dam are same as gravity dam considerations. Again, in this case we have three types and they are:

- **Fixed Deck Slab Buttress Dams:** In case of fixed deck slab type dam, deck slab and buttresses are casted monolithically.
- **Free Deck Slab Buttress Dams:** This is also called as simple deck slab buttress dam. This is constructed when the foundation soil is very weak. In this case reinforcement is provided at the downstream face of the deck slab.
- **Cantilever Type Buttress Dams:** In this case, deck slab is cantilevered at both ends and upstream face is provided with reinforcement.

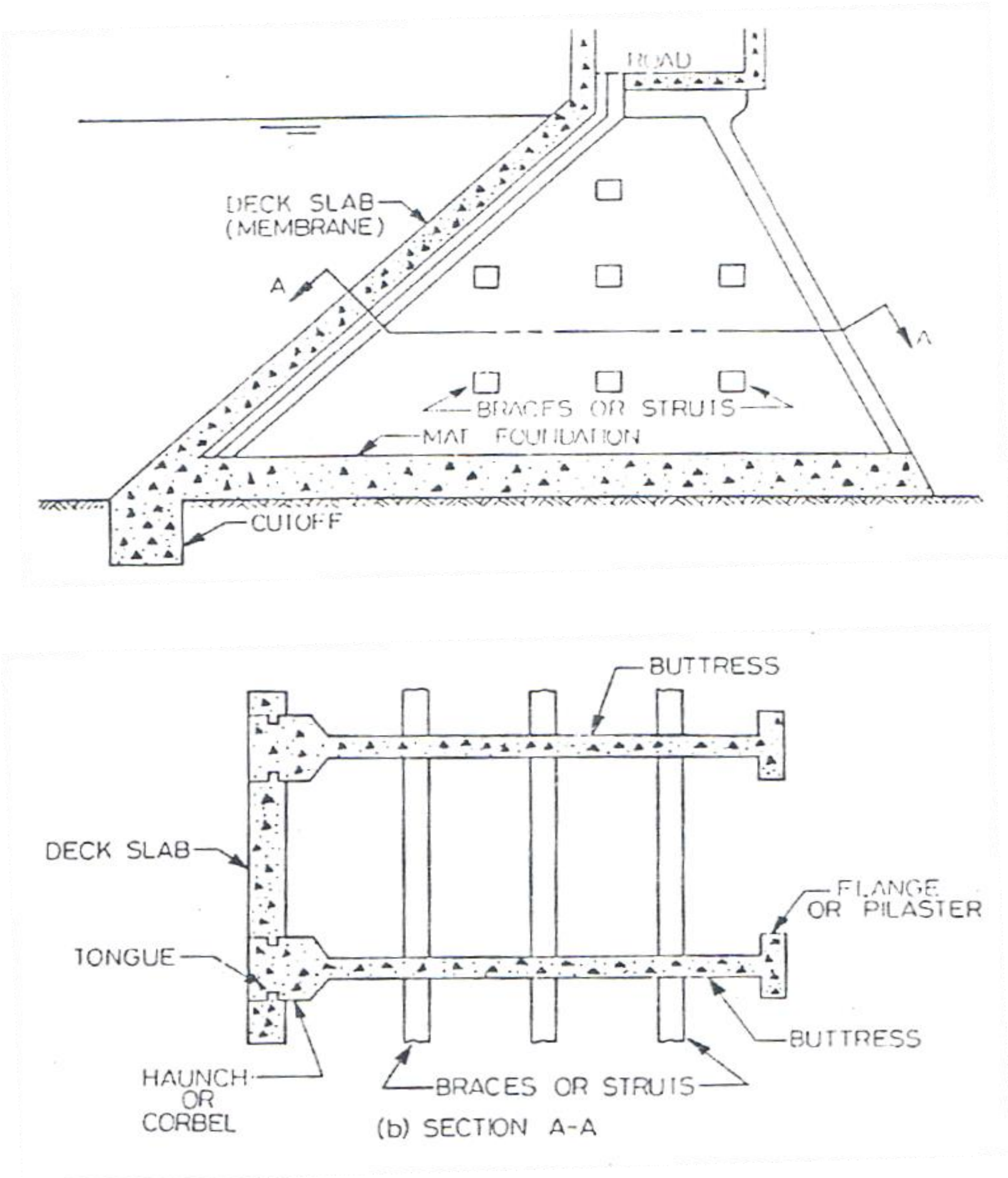


Figure (5.1) deck slab buttress dam.

Multiple Arch Buttress Dam:

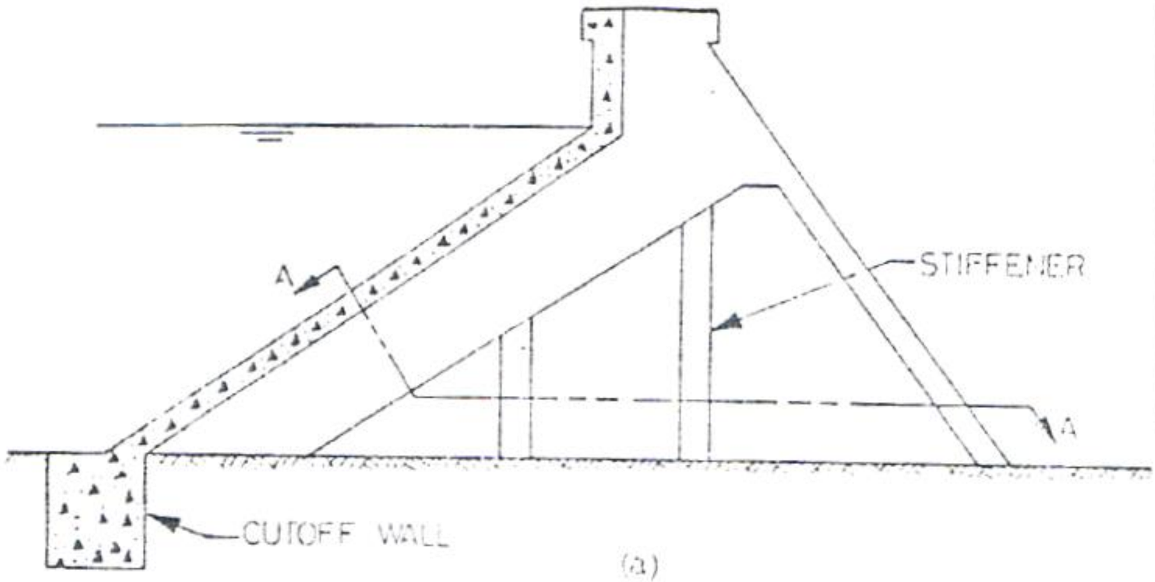
Multiple arch dam contains series of arches and in this case arch slabs are provided at the upstream face of the dam. These slabs are supported by buttresses. The buttress wall is constructed as single stiffened wall or double hollow wall.

This type of dam can be preferred for larger heights about more than 50 meters. As compared to deck slab type buttress dams, multiple arch buttress dams are more flexible and stable.

The main disadvantage in this case is the adjacent buttress units depends on each other. So, if one unit gets affected then it causes severe problems for the whole series of arches. To overcome this good foundation for each of buttresses wall should be provided.

The thickness of arch provided varies in case of larger spans and constant in the case of shorter spans. The central angle of arch should be in between 180 degrees to 150 degrees.

The space between buttresses should be in between 15 to 21 meters. However, larger spacing can also be provided by taking some consideration.



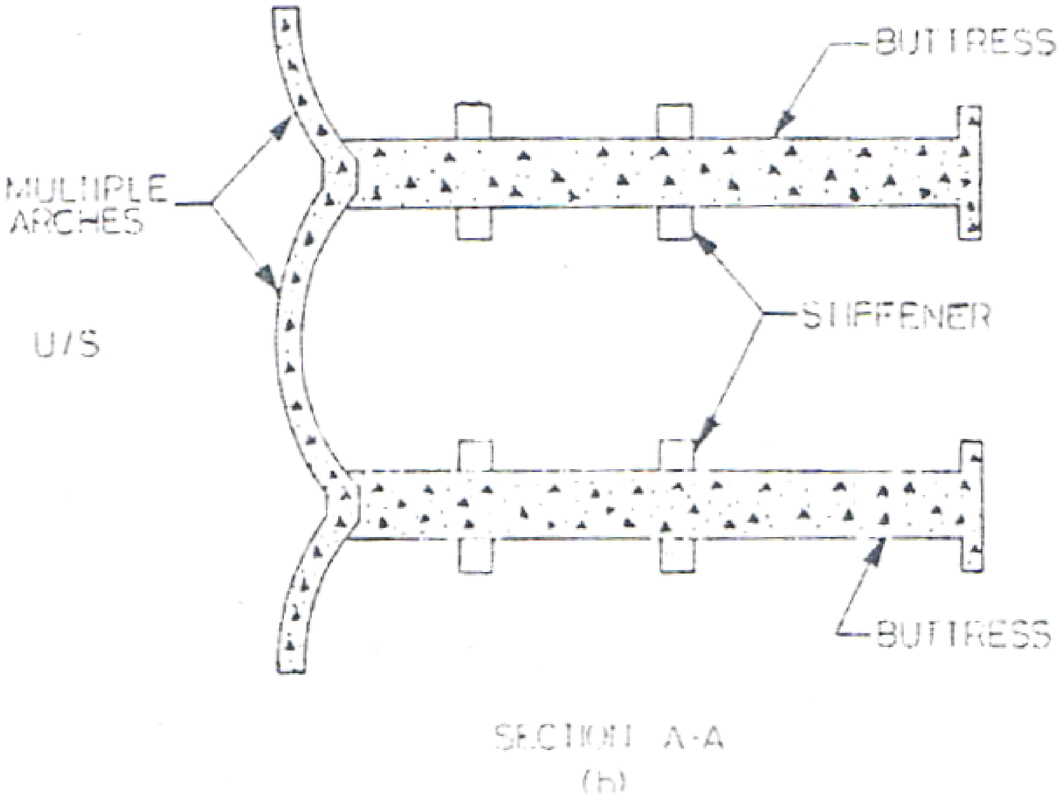


Figure (5.2) multiple arch buttress dam.

Massive Head Buttress Dam:

In this type of buttress dam, no slab or arch is provided at the upstream face, instead of that buttress head is enlarged and joined with adjacent buttress head. Like this all buttresses heads makes strong water supporting surface.

The enlargement of buttress heads can be done in different shapes like rounded, diamond shape etc. So, these shapes can resist the water pressure very well. The joining of heads is strengthened by providing copper strips. Construction of massive head buttress dam is easier compared to other types and no reinforcement is required only mass concrete is laid whole over the dam body. Because of such a huge concrete body, it is heavier and resists against sliding.

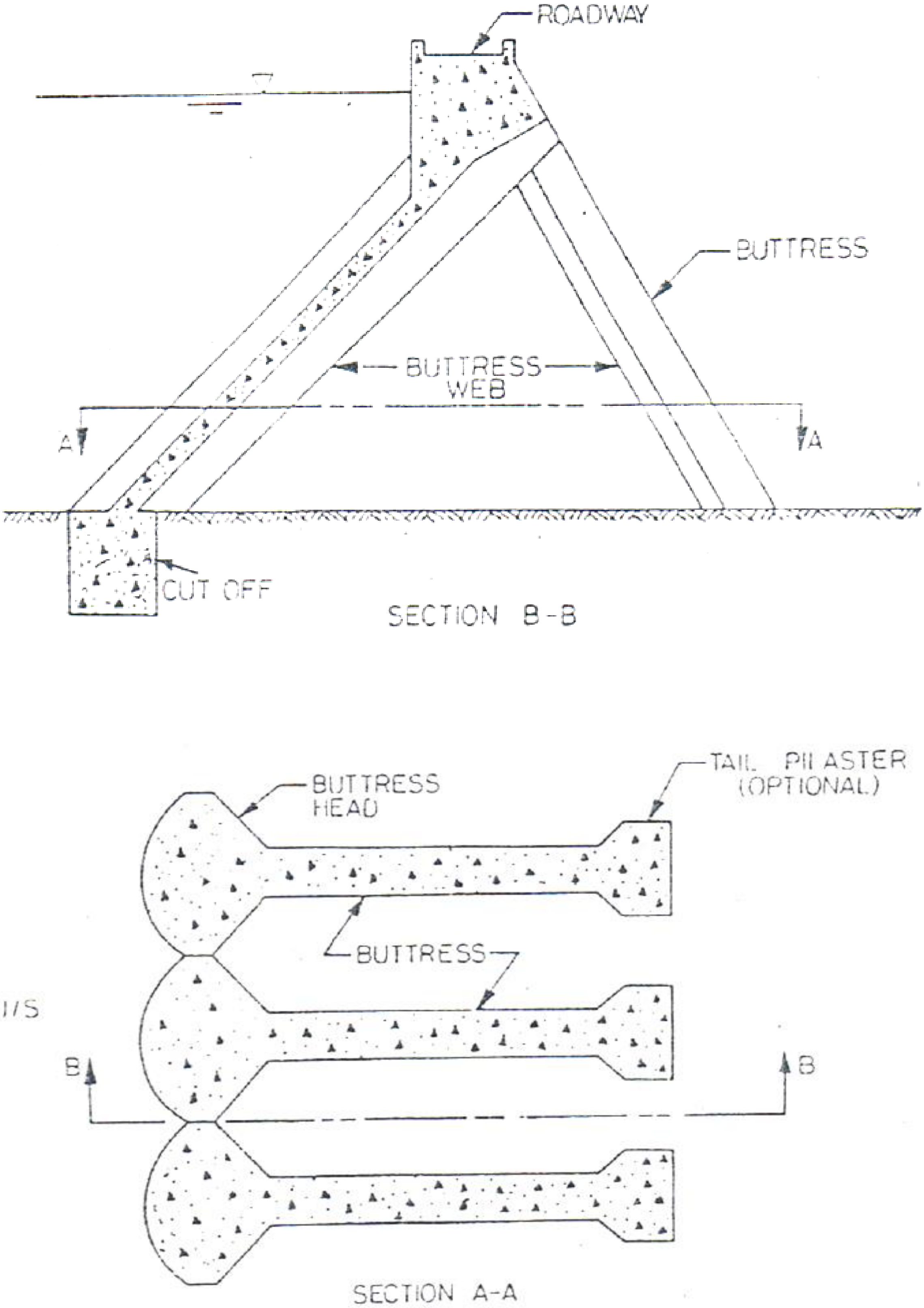


Figure (5.3) massive head buttress dam.

Multiple Dome Buttress Dam:

It is almost similar to multiple arch type buttress dam but in this case domes are constructed in the place of arches. Apart from that almost all the features are same.

Provision of such domes influence the setting of larger spacing between buttresses. Hence, longer spans can be provided and also number of buttress also reduced.

Columnar Buttress Dam:

In case of columnar buttress dam, inclined columns support the deck slab of dam. It is the modification of deck slab type buttress dam. It requires very strong and stable foundations. It also requires skilled persons to build buttresses. Hence, it is not widely used dam.

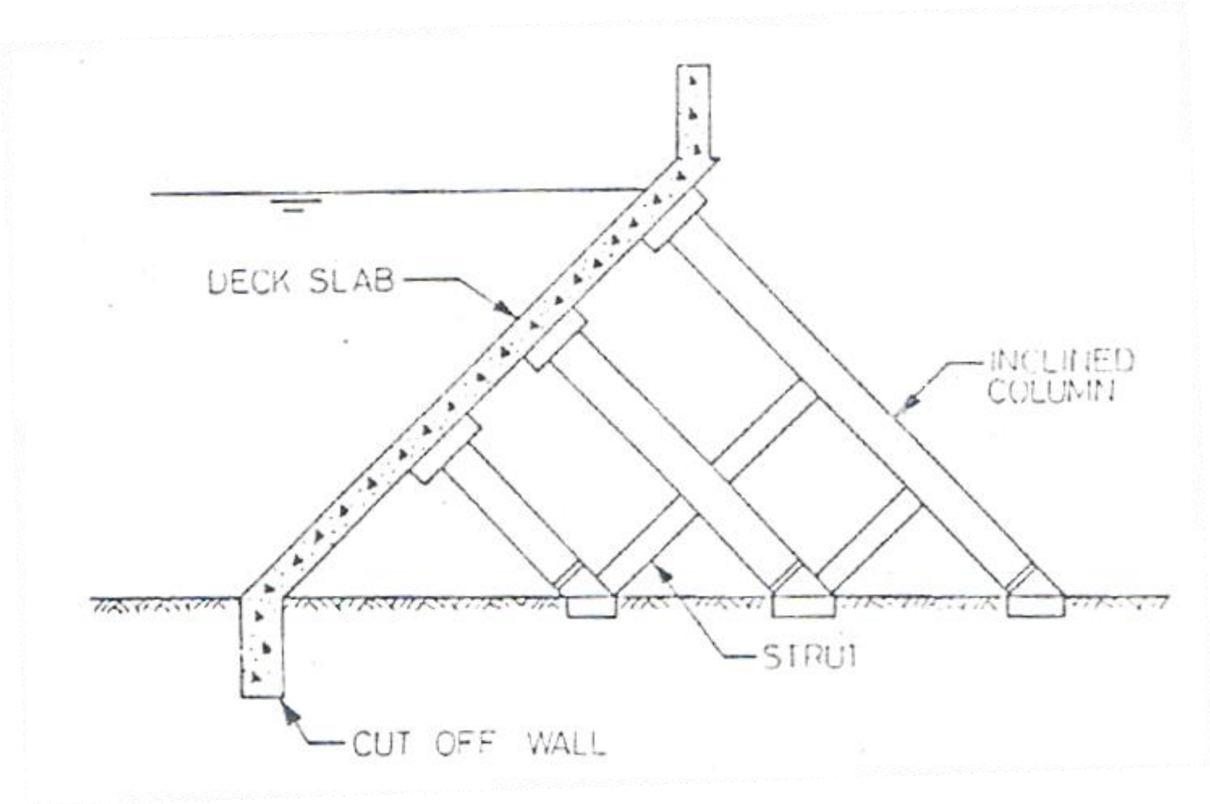


Figure (5.4) columnar buttress dam.

5.3. Buttress Dams Spacing:

The most economical spacing of buttresses is the one, in which the minimum thickness of concrete is fully utilized. This spacing is governed to a very large extent by the values of the upstream slope of the dam (Ø). For economy, it is also necessary that the buttress spacing be changed for dams of different heights. The suggested spacing for different dam heights, are shown in table:

Dam Height in m	Economic buttress spacing suggested-in m for a normal Ø value of 40" to 50°
Less Than 15 m	4.5 m
15 - 30 m	4.5 to 7.5 m
30 – 45 m	7.5 to 12 m
above 45 m	12 to 15 m

5.4. Height and Thickness of Buttress:

$$\textbf{Massiveness factor} = \frac{\textbf{Spassing of Butters}}{\textbf{Thickness of Butters}} \qquad \textbf{2.5 to 3} \ldots\ldots\ldots\ldots \textbf{5.1}$$

$$\textbf{Slenderness ratio} = \frac{\textbf{Height of Butters}}{\textbf{Thickness of Butters}} \qquad \textbf{12 to 15} \ldots\ldots\ldots\ldots \textbf{5.2}$$

5.5. Master Carve for Economical Spacing:

The forces that have major effect to the butters dams are concentrated mainly in the hydrostatic pressure of water and the dam weight of concert. These forces can be divided in two parts, the first one trying to stabilize the dam against the sliding and the second one trying to slide the dam to the downstream.

The sliding force is the horizontal component of hydrostatic force and the resisting forces are the vertical component of hydrostatic force and the concert weight of dam.

Horizontal force $$\mathbf{P_H = \frac{1}{2}\,\gamma\, H^2} \;\ldots\ldots\ldots\, 5.3$$

Vertical force $$\mathbf{P_V = \frac{1}{2}\,\gamma\, H^2\, \cot\emptyset} \;\ldots\ldots\, 5.4$$

Weight of concert $$\mathbf{Wc = \gamma_c\;\; V_c} \ldots\ldots\ldots\, 5.5$$

For constant buttress spacing, variation of few degrees in the u/s slope may result in an appreciable change. in the quantity of concrete. This concrete quantity required per meter length of dam (Vc) is, in fact, related to u/s deck slope (Ø), dam height (H), and the sliding factor

$$\mathbf{F_s = \frac{\sum H}{\sum V} = \frac{P_H}{P_V + Wc} = \frac{\frac{1}{2}\gamma H^2}{\frac{1}{2}\gamma H^2 \cot\emptyset + \gamma_c\;\; V_c}} \;\ldots\ldots\ldots\, 5.6$$

Considering the γ of water equal to 10 kN/m^3 and γ_c of concert equal to 24 kN/m^3 .

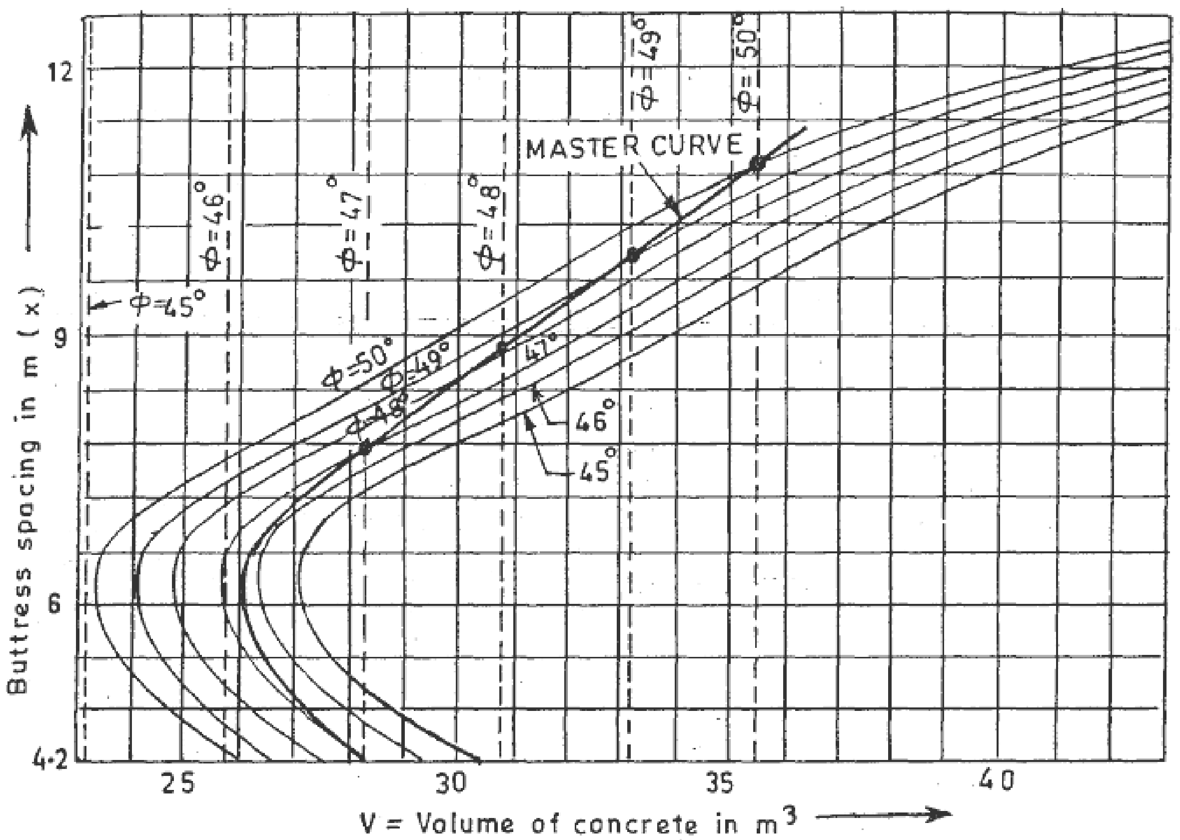
$$F_s = \frac{5\ H^2}{5\ \cot\emptyset + 24\ V_c}$$

$$\mathbf{V_c = 0.2083\;\; H^2 \left(\frac{1}{F_s} - \cot\emptyset\right)} \;\ldots\ldots\ldots\, 5.7$$

This equation clearly shows that for a fixed dam height (H) and a given value of Fs, the concrete quantity depends solely on Ø. Hence, for different values of Ø, at fixed values of Hand Fs, concrete quantities can be worked out. Vertical lines, like those shown in Figure, are, thus, drawn for each value of the concrete quantity corresponding to each value of Ø.

Concrete quantities are also worked out for -various angles of u/s slope (Ø) and different buttress spacing, for fixed decided dam height (H) and plotted to get a curve for each angle. A master curve is then drawn through the

junctions of the curves and the vertical lines corresponding to each angle. The master curve gives the absolute economic value of buttress spacing for different us slope (Ø) for the decided dam eight.



Design Example:

Design a butters dam for 30m height of dam using master curve with factor of safety exceed 2. Considering the γ of water equal to 10 kN/m^3 and γ_c of concert equal to 24 kN/m^3 .

Solution:

H =30 m

Let – S is the spacing of butters

t is the thickness of butters

Hb is the height of butters

Ø = 45 ^{o}

Base on dam height H = 30 m select trail spacing as S = 6 m

$\text{Massiveness factor} = 3 = \frac{S}{t}$ $t = \frac{6}{3} = 2\ \text{m}$

$\text{Slenderness ratio} = 12 = \frac{Hb}{t}$ $Hb = 12 * 2 = 24\ \text{m}$

Now finding Vc from master curve for optimum spacing. Using S= 6 m and Ø = 45 ^{o} the volume of concert Vc = 27.2 m^3

For γ of water equal to 10 kN/m^3 and γ_c of concert equal to 24 kN/m^3

$$V_c = 0.2083\ H^2 \left(\frac{1}{F_s} - \cot Ø\right)$$

$$27.2 = 0.2083\ 30^2 \left(\frac{1}{F_s} - \cot 45\right)$$

Fs = 0.87 < 2 that means the spacing and the angle of butters is not economic so that it not good – it should be re-design.

$$27.2 = 0.2083\ 30^2 \left(\frac{1}{2} - \cot Ø\right)$$

Ø = 70.46^o, use Ø =75^o and re analysis

For Ø = 75^o

$$27.2 = 0.2083\ 30^2 \left(\frac{1}{F_s} - \cot 75\right)$$

Fs= 3.73 > 2 O.K

CHAPTER SIX

RUBBER DAMS

Rubber Dam

6.1. Introduction:

Rubber dams are long tubular-shaped fabrics placed across channels, streams and weir crest to raise the upstream water level when inflated (Figure 1). In open channels, they are commonly used to raise water levels, to increase water storage and to prevent chemical dispersion. The interest in inflatable dams is increasing because of the ease of placement. Such structures can be installed during later development stages. The membrane is usually deflated for large overflows. It is however common practice to allow small spillages over the inflated dam.

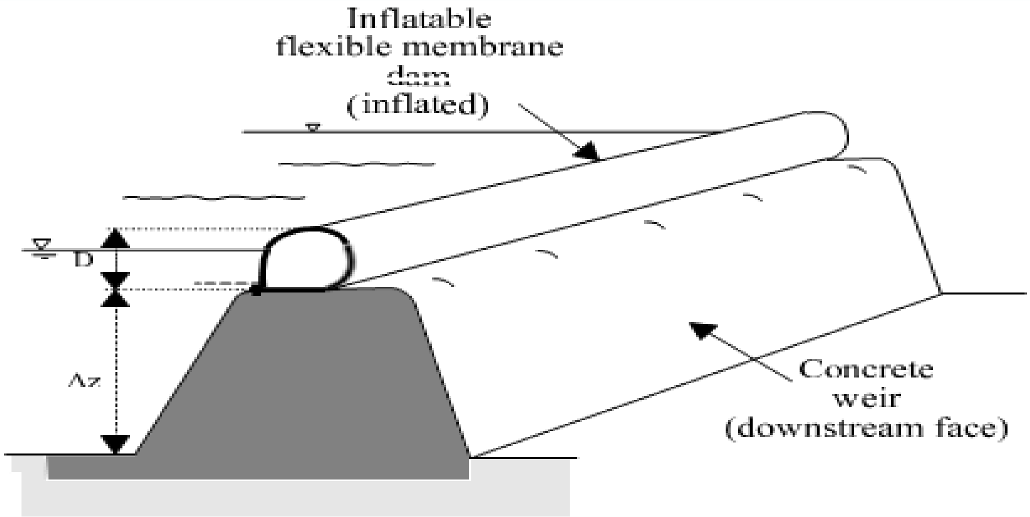


Fig. 1: Sketch of a rubber dam

Figure (6.1) Sketch of a rubber dam

6.2. Methodology:

The main part of the Rubber Dam is rubber bag, pump house and concrete floor (with which rubber bag is attached with steel pad and platen). The water coming from the upstream side is directly entered to the pump house. Then with the help of different valve and pump motor, the water is used to inflate the rubber bag. After the use of water for irrigation purposes, the bag is then emptied by valves and the bag is sinking at the

river bed. Generally, gravity drainage system is used for emptied of the rubber bag. So, it does not create obstruction for the passing of water in rainy season and also passage of boat freely through the River.

Inflation System: An air blower or water pump and ancillary devices such as valves are used to inflate the air or water filled dam respectively.

Deflation System: There are three types of deflation systems: bucket, float, and electrical.

Safety Systems: An air blow-off tank (air-filled dam) and siphon pipe (water-filled dam) can be used as a safety device in case principal deflation mechanism fails.

Construction procedure of Rubber dam: The rubber bag of the dam is attached with the concrete floor. At the beginning of the construction sheet pile wall or cut off wall is constructed at both the upstream and downstream side for controlling soil corrosion due to seepage. After that, the upstream, downstream, and the rubber bag is cast with concrete. At last, abutment wall, block, pump house, valve chamber etc. is constructed. But the main attached concrete structure of rubber bag is constructed very carefully. By using M.S. pipe, pad and platen; rubber bag is anchored with the floor bed.

INFLATED

DEFLATED

Figure (6.2) Inflated and Deflated of the rubber dam

6.3. Materials of Rubber Dam:

Main material use in Rubber dam is **rubber bag** which control the flow of water. Some characteristics of rubber bag are:

1. It is made up of multi-layer fabric of synthetic fiber (usually nylon). The fabric is quite flexible and exhibits good wear-resistance characteristics.
2. The fabric-bag should be water resistant, water tolerant, corrosion resistant and durable in atmosphere.
3. The layers of rubber coated fabric are joined together in the longitudinal direction. The actual number of layers of rubber coated fabric for each rubber body depends on the height and the tension.
4. The rubber sheet may be of single layer, double layer and multi-layer as per its height and strength.

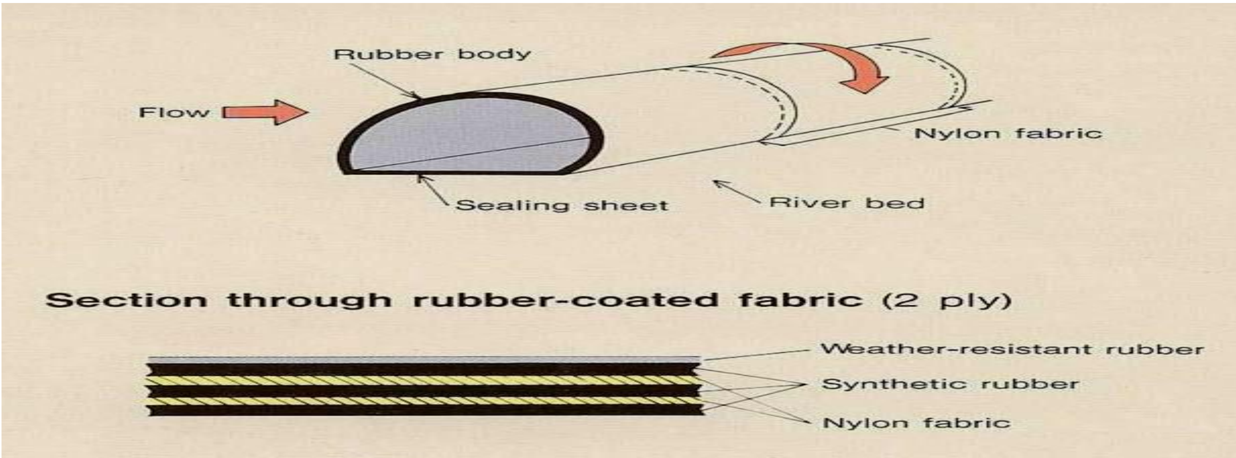


Figure (6.3) Materials of Rubber dam

6.4. Design principle:

A rubber dam basic data shall be collected, processed, analyzed and studied on:

- Hydrology.
- Socio-economic status and environmental impact.
- Engineering geology.
- Topography.
- Meteorology.

- Hydrological geology.
- Socio-economic status and environmental impact.

6.5. Hydraulic Design:

The design data/parameters required for hydraulic design of rubber dam data/parameters are as follows:

1. Design discharge (20 years discharge record of river).
2. Design water level.
3. Water retention level of the project.
4. Average lowest bed level of the river.
5. Soil information.
6. Embankment crests Level.
7. Cross section of the river up to 7-10 Km U/S.
8. Long section of the river.

6.6. Internal pressure:

The standard state of the rubber dam should be such that its upstream water level corresponds to the dam height and its downstream water level is zero and the dam height under this condition is used as standard dam height. The standard internal pressure is usually about 1.5 times as high at the head pressure of dam height for rubber dam.

6.7. Dam Height:

The dam height is selected from the necessity of the purpose.

For ex: If the rubber dam is used as irrigation purpose, the height of dam depends upon the requirement of crops & area of land would be cultivating by the dam project.

Rubber Dam height are small dams therefore we always constructed 3 to 4 meters height dam only.

6.8. Length of Rubber Dam:

$$\boldsymbol{L = 4.83\, Q^{1/2}} \quad \text{-------- } \textit{(6.1)} \qquad \text{(Empirical formula)}$$

Where Q = design discharge;

If Length of rubber dam more than “L”, then we constructed a dam in a section or make a concrete joint after every “L” length in rubber bag of dam because to resist overturning moment of deflated rubber bag.

6.9. Strength of Rubber Dam:

The main design loads acting on the dam bag are the static hydraulic pressure from outside the dam bag and the pressure inside it caused by the filled water or air. Various design parameters for this kind of dam bags is used from mathematical analysis by using the formulae given below:

Radial strength of dam bag:

$$\boldsymbol{T = \tfrac{1}{2}\gamma\,[\alpha - 1/2]\,H_1^2} \text{-------- } \mathbf{(6.2)}$$

Where

$\boldsymbol{\gamma}$ = unit wt. of water kN/m^3;

α = Internal pressure ratio= $\boldsymbol{H_0/H_1}$;

$\boldsymbol{H_0}$= internal pressure water head in m;

$\boldsymbol{H_1}$ = design height of dam in m.

Design tensile strength of rubber bag per meter

Tensile strength = K *T **-------- (6.3)**

Where, K= factor of safety (**5 to 8**).

6.10. Shape of Rubber Bag:

The shape of dam bag after inflation is divided into four parts: -

S1= length of curved segment of dam surface in the upstream direction;

S2= length of curved segment of dam surface in the downstream direction;

N= length of ground touching segment in the upstream direction and;

X= length of ground touching segment in the downstream direction.

Effective Circumference of the dam body is

$$\boldsymbol{Lo = S1 + S2} \text{-------- } \mathbf{(6.4)}$$

Effective length of bottom pad is

$$\boldsymbol{D = N + X} \text{ --------- } \mathbf{(6.5)}$$

$$D = (2/3 - 3/4)\ Lo \text{ ---------------(6.6)}$$

$$N = 1/3\ D \text{ -----------------(6.7)}$$

$$X = 2/3\ D \text{ -----------------(6.8)}$$

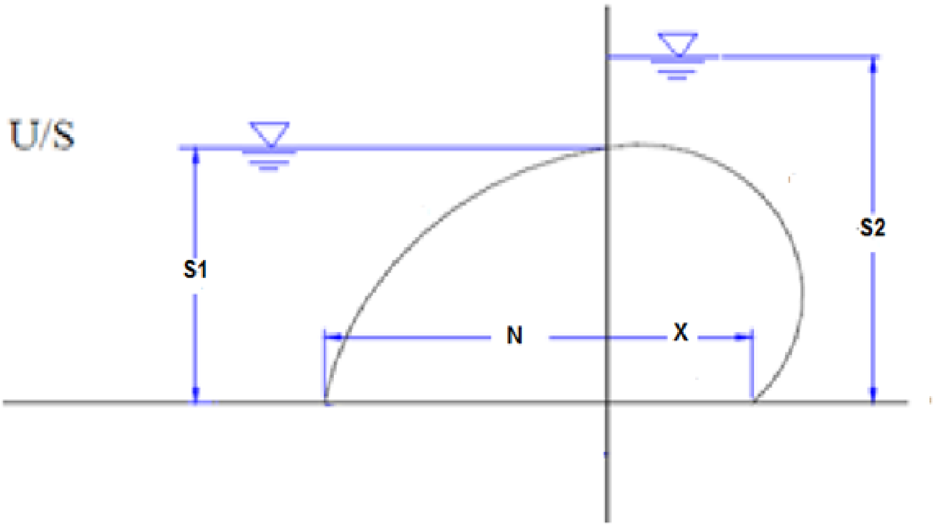


Figure (6.4) Shape of Rubber dam

6.11. Components of a Rubber Dam:

Rubber dam is mainly divided into four parts:

i. Dam body or dam bag.

ii. Anchorage system.

iii. Control system (including water or air filling and emptying system, monitoring system and safety (control system).

iv. Foundation (including base floor, abutment and side walls etc.).

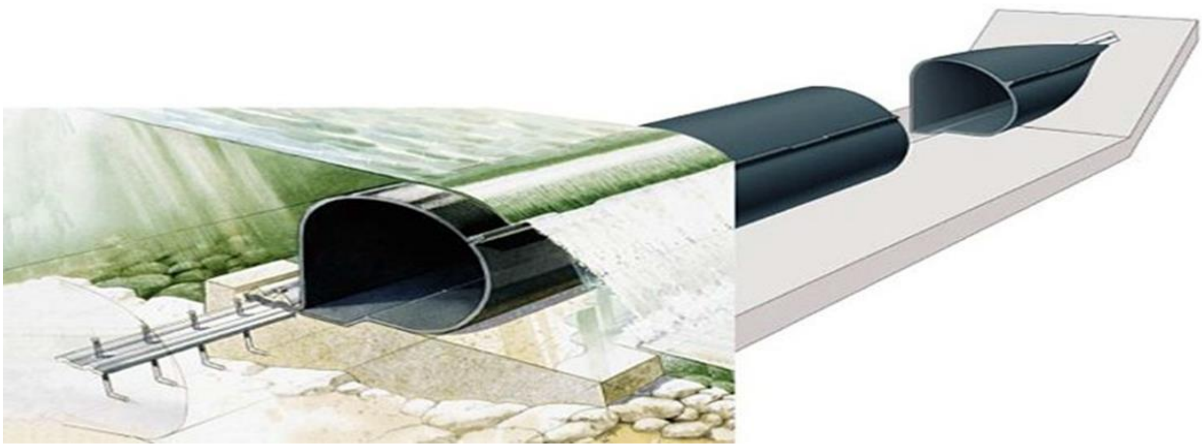


Figure (6.5) components of Rubber dam

Dam body: Dam body is made up of rubber which is reinforced by woven synthetic fabric that provides the tensile strength with rubber acting as the adhesive and water proofing elements. The fabric reinforcement is used in layers. The layers depend on the design strength requirements.

Anchorage of Dam Bag with Concrete Base: Many types of anchorage system are used in the world. Major types are given below:

Concrete Wedge Blocks: -This method of fixing or anchoring of rubber sheet in slab is every old. This system first used by in China. This system does not work properly and does not give good result. Slots are provided in the base slab and the ends of the rubber sheet are anchored by two sets of pre-cast concrete wedge blocks; the front wedge blocks pressing the rubber sheet against the face of floods and the back wedge blocks hammered into position ensuring the grip. The slots and the blocks should be in accurate dimension and sharpness for quality work. This method is low height dams of simple hydraulic forces; wooden anchorage blocks may also be used.

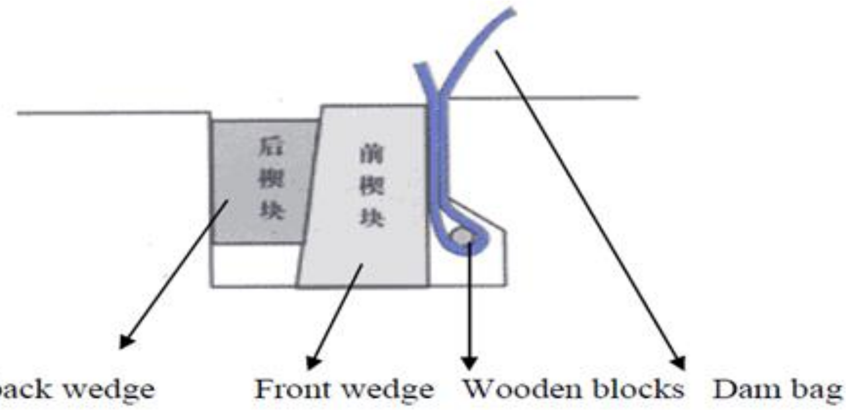


Figure (6.6) Concrete Wedge Blocks

Steel Clamps with Anchor Bolts: Fixing of rubber sheet on the concrete slab, steel clamp elements and stainless steel anchor bolts and nuts are used. Fixing of rubber sheet is not as simple as like as other construction work of the rubber dam project. At first, the anchoring slot is kept open for

second stage concrete for positioning and fitting and fixing the anchor bolt with dowel bar of the slot. Then lower anchoring plate is fixed with the help of anchor bolt and the anchor bolt is fixed with dowel bar with welding work. The design level of lower plate is done with the help of machine and this level is adjusted with the help of bottom nut of anchor bolt. Then second stage concrete is cast up to bottom level of lower plate. The thickness of lower and upper plate is designed with the height of rubber dam. The works are done very carefully with maintaining proper alignment of the anchor bolt, because any deviation of anchor bolts will create problems in fitting of anchor plate

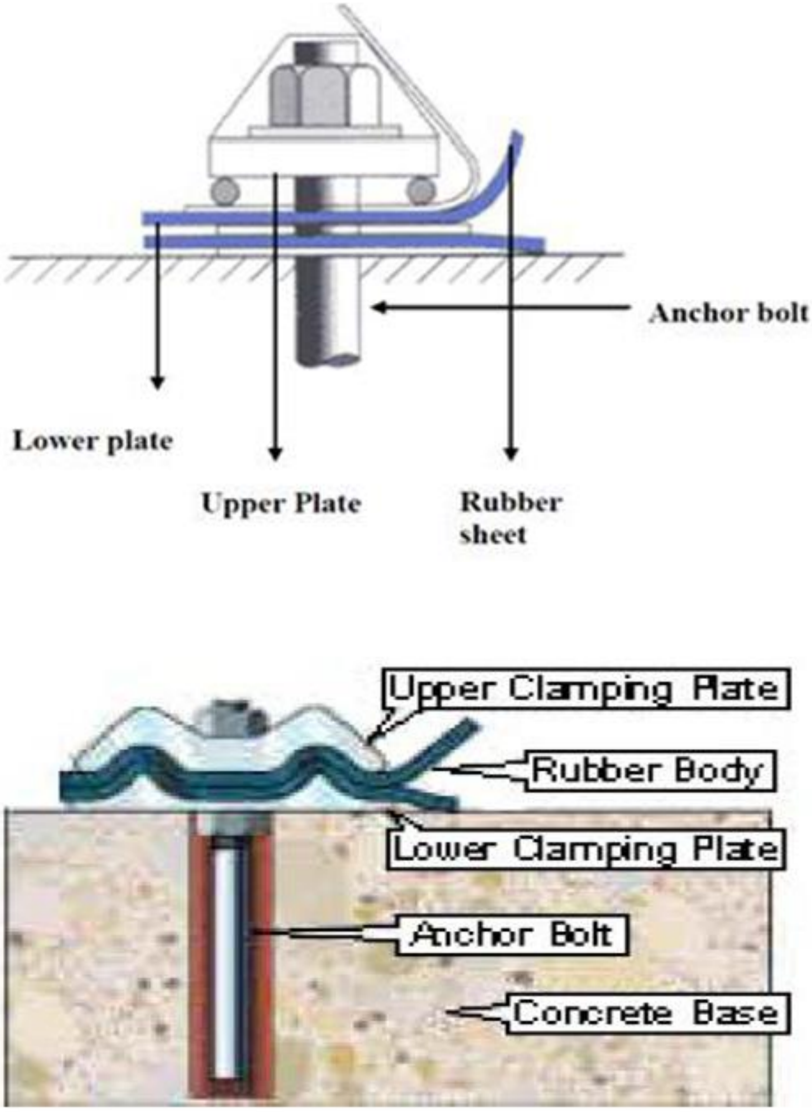


Figure (6.7) Steel Clamps with Anchor Bolts

6.12. SAFETY CONSIDERATIONS:

When the Rubber dam is inflated or deflated, water levels change suddenly and may create a hazard to adjacent people or their properties.

Preventive maintenance:

Innovative materials and designs have been applied to rubber dams to increase their life span and enhance their performance. It is included that rubber dams are vulnerable two types of damages,

i. Vandalism.

ii. Flood borne debris, especially sharp objects.

Periodic inspection and maintenance:

These includes mainly:

i. Inspecting the dam body and surrounding environment to identify if there any presence of unidentified object that may cause to rubber cut or puncture.

ii. Removing silt and debris from both upstream and downstream of dam.

iii. Keeping the chamber clean that contains the level sensor and associated piping system

Sediment removal:

Rivers have scouring effect on the dam body. Since rubber is less susceptible to scour than concrete so this does not pose a major problem to dam body.

Example:

Calculate the number of concrete joints and bags of rubber dam for a 0.5 m/sec velocity in river has 70 m width and 1.7 m height of water level.

Solution:

$Q = V * A$

$Q = 0.5 * (70*1.7) = 59.5\ m^3/sec$

$L = 4.83\ Q^{1/2}$

$L = 4.83 * (47.6)^{1/2} = 37.25\ m < 70\ m$

Number of bags = 70/37.25 = 1.88 use 2*34.5 m

Length of joints = 70 – 2*34.5 = 1 use 1 * 1 m

<u>Design Example:</u>

Design a rubber dam for a canal has a width equal to 4m, if the height and velocity of water is 2.2m and 0.7m/sec, respectively.

Cross section area = 2.2 * 4 = 8.8 m^2

$Q = 8.8 * 0.7 = 6.16\ m^3/sec$

$L = 4.83\ Q^{1/2}$

$L = 4.83 * (6.16)^{1/2} = 11.98\ m > 4\ m$ No need to concrete joint

Assume internal pressure is 1.5 H and K=5

$H_o = 1.5 * 2.2 = 3.3\ m$

$\alpha = H_o/H$

$\alpha = \frac{3.3}{2.2} = 1.5$

$T = ½\, \gamma \left[\alpha - \frac{1}{2}\right] H_1^2$

$T = 0.5 * 9.81\ (1.5-0.5) * (2.2)^2$

T= 23.74 kN/m

Tensile strength = K *T

Tensile strength = 5* 23.74 = 118.7 kN/m

Effective Circumference of the dam body is

$$Lo = S1 + S2$$

S1= 2.2

S2 = 3 …………….. Assumption

Lo = 2.2 + 3 = 5.5 m

D = 2/3 L0

D = 0.75 *5.5 = 4.125 m

N = 1/3 D

N = 1/3 *4.125 = 1.375 say 1.5 m

X = 2/3 * D

X = 2/3 * 4.125 = 2.75 say 3 m

CHAPTER SEVEN

RESERVOIR PLANNING

Reservoir Planning

7.1. Introduction:

A reservoir is a large, artificial lake created by constructing a dam across a river. Water store on the upstream of a dam constructed for this purpose. To regulate the water supplies a reservoir is created on the river to store water during the rainy season. The store water is later released during the period of low flows to meet the demand.

Besides releasing the water during the period of low flow, the reservoirs also help in flood control.

If a reservoir serves only one purpose, it is called a single – purpose reservoir. On the other hand if a reservoir serves more than one purpose, it is termed multipurpose reservoir.

The various purpose served by a multipurpose reservoir include:

i. Irrigation.
ii. Municipal and industrial water supply.
iii. Flood control.
iv. Hydropower.
v. Navigation.
vi. Recreation.
vii. Development of fish and wild life.
viii. Soil conservation.
ix. Pollution control.

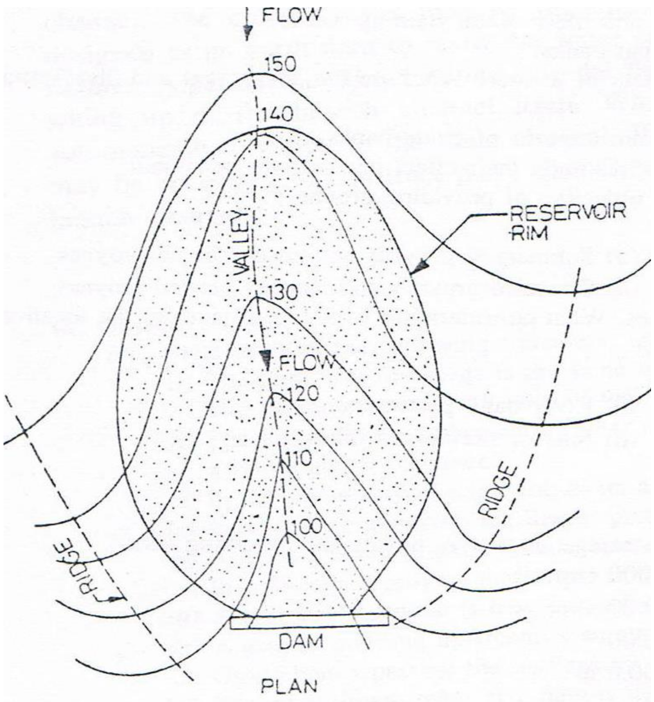


Figure (7.1) Plan of Reservoir

7.2. Types of reservoirs:

Depending upon the purpose served, the reservoirs may be broadly classified into four types:

1. **Storage Reservoir:**

   Storage reservoir are constructed to store water in the rainy season and to release it later when the river flow is low.

2. **Flood Control Reservoir:**

   It constructed for the purpose of flood control. This reservoir reduces the flood damage and it is also known as the flood – mitigation reservoir, flood protection reservoir. When the discharge exceeds the safe capacity, the excess water is stored in the reservoir and then released and does not exceed the safe capacity of the river.

3. **Multipurpose Reservoir:**

   This reservoir is designed and constructed to serve two or more purpose, such as irrigation and hydropower.

4. **Distribution Reservoir:**

   Small reservoir it stores water during the period of lean demand and supplies the same during the period of high demand. The storage is limited in this reservoir, it merely helps in distribution of water as per demand for a day.

5. **Balancing Reservoirs:**

   A balancing reservoir is a small reservoir constructed d/s of the main reservoir for holding water released from the main reservoir.

7.3. Types of Flood Control Reservoir:

1. **Detention Reservoir:**

   A detention reservoir stored water during flood and released it after the flood. It is similar to a storage reservoir but is provided with large gated spillways and sluiceways. The discharge is regulated by gates. There is basically no difference between the detention reservoir and a storage reservoir except that the former has a large spillway capacity and sluiceway capacity to permit rapid drawdown just before or after a flood.

2. **Retarding Reservoir:**

   A retarding reservoir with spillways and sluiceways which are not gated. There is an automatic released of water depending upon the level of water in the reservoir.

Advantage and Disadvantage of Detention and Retarding Reservoir:

a) **Detention reservoir:**

   Advantage:

1. The detention reservoir provide more flexibility of operation and better control of outflow than retarding reservoir.

2. The discharge can be adjusted according to carrying capacity of the d/s channel.

Disadvantage:

1. More expensive from retarding reservoir because of high initial cost and maintenance cost.
2. Due to possibility of human error or negligence, a disaster can occur.

b) Retarding reservoirs:

Advantage:

1. Less expansive.

2. As the outflow is automatic, there is no possibility of a disaster due to human error.

Disadvantage:

1. Do not provide any flexibility of operation.

2. Do not adjusted the d/s discharge and can cause flood in river downstream.

7.4. Available Storage Capacity of Reservoir:

The storage capacity of any type of reservoir is most important characteristics. The available storage capacity depends upon the topography of the site and the height of the dam. Engineering survey, topography survey of the reservoir area is usually conducted and a contour map of the area is prepared. The storage capacity and the water spread area at different elevations can be determined from the contour map, as explain below:

A. **Area-Elevation Curve:**

From the contour plan, the water is determined. Planimeter is used for measuring area and curve drawn between the surface area as abscissa and the elevation as ordinate.

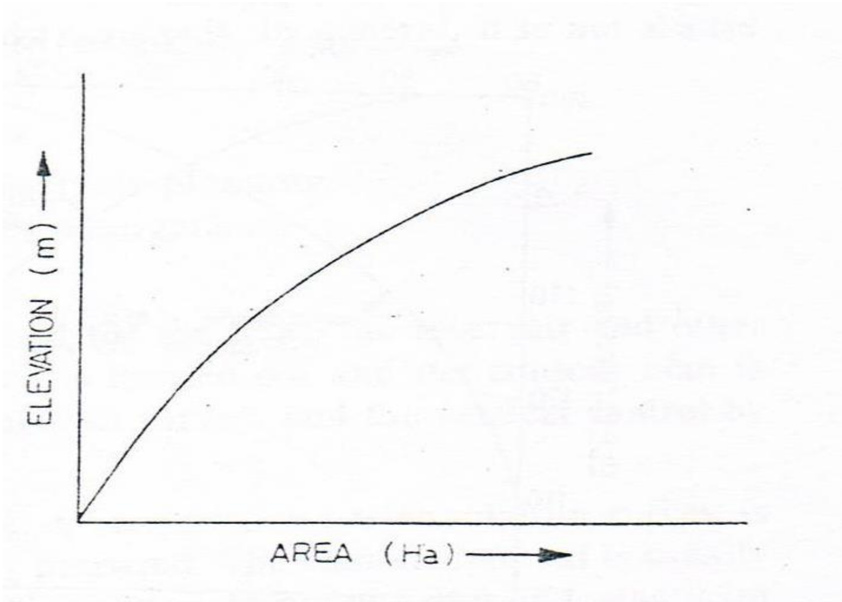


Figure (7.2) Area Elevation Curve.

B. **Elevation-Capacity Curve:**

The storage capacity of the reservoir at any elevation is determined from the water spread area. The following formulae are commonly used to determine the storage capacity.

1. **Trapezoidal Formula:**

The storage volume between two successive contours of areas A1 and A2 is given by:

$$\Delta \boldsymbol{V_1} = \frac{\boldsymbol{h}}{\boldsymbol{2}}(\boldsymbol{A_1} + \boldsymbol{A_2}) \ldots\ldots.\ \mathbf{7.1}$$

Where h is contour interval. Therefore the total volume V of the storage is given by:

$$\boldsymbol{V} = \Delta \boldsymbol{V_1} + \Delta \boldsymbol{V_2} + \Delta \boldsymbol{V_3} + \cdots = \sum \Delta \boldsymbol{V} \ldots\ldots\ldots.\ \mathbf{7.2}$$

$$\boldsymbol{V} = \frac{\boldsymbol{h}}{\boldsymbol{2}}(\boldsymbol{A_1} + \boldsymbol{2A_2} + \boldsymbol{2A_3} + \cdots + \boldsymbol{2A_{n-1}} + \boldsymbol{A_n}) \ldots\ldots\ldots.\ \mathbf{7.3}$$

Where n is the total number of areas.

2. **Cone Formula:**

The storage volume between two successive contour of areas A_1 and A_2 is given by:

$$\Delta V_1 = \frac{h}{3}\left(A_1 + A_2 + \sqrt{A_1 A_2}\right) \ldots\ldots\ldots 7.4$$

$$V = \Delta V_1 + \Delta V_2 + \Delta V_3 + \cdots = \sum \Delta V \ldots\ldots\ldots 7.5$$

3. **Prismoidal Formula:**

The storage volume between 3 successive contours is given by:

$$\Delta V_1 = \frac{h}{3}(A_1 + 4A_2 + A_3) \ldots\ldots\ldots 7.6$$

The total volume V of the storage is given by:

$$V = \frac{h}{3}[(A_1 + A_n) + 4(A_2 + A_4 + A_6 + \cdots) + 2(A_3 + A_5 + A_7 + \cdots)] \ldots\ldots\ldots 7.7$$

Where A1, A5 etc., are the areas of the odd number, A2, A4, A6 etc, are the areas of even numbers A_1, A_n are respectively the first and the last area. The prismoidal formula is applicable only when there are odd numbers of area.

C. **Combined Diagram:**

It is usual practice to plot both the elevation –area curve and the elevation storage curve on the same paper. As shown in figure below.

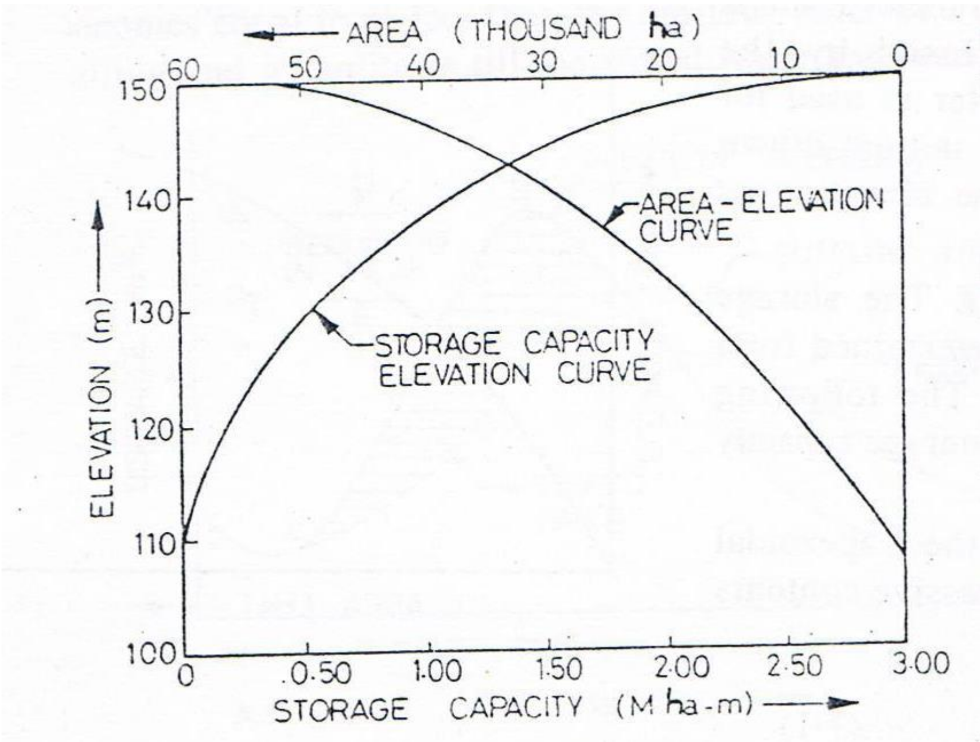


Figure (7.3) Combined Diagram.

Example:

A reservoir has the following areas enclosed by contours at various elevations. Determine the capacity of the reservoir between elevations of 200 to 300.

Elevation	200	220	240	260	280	300
Area of contour (m^2)	150	175	210	270	320	400

Use a) trapezoidal formula. b) prismoidal formula.

Solution:

a) trapezoidal formula.

$$\boldsymbol{V = \frac{h}{2}(A_1 + 2A_2 + 2A_3 + \cdots + 2A_{n-1} + A_n)}$$

$$V = \frac{20}{2}(150 + 2x175 + 2x210 + 2x270 + 2x320 + 400) =$$

$$25000\ m.km^2$$

b) prismoidal formula.

In this case, there are even number of areas. The prismoidal formula is applied to first 5 areas from equation.

$$\boldsymbol{V = \frac{h}{3}[(A_1 + A_5) + 4(A_2 + A_4) + 2(A_3)]}$$

$$V = \frac{20}{3}[(150 + 320) + 4(175 + 270) + 2(210)] = 17800\, m.km^2$$

$$\Delta V = \frac{h}{2}(A_5 + A_6) = \frac{20}{2}(320 + 400) = 7200\, m.km^2$$

Total volume $= 17800 + 7200 = 25000\, m.km^2$

7.5. Investigation for reservoir planning:

The following investigation are usually conducted for reservoir planning:

1. **Engineering surveys:** The topographic survey of the area is carried out and the contour plan is prepared. The horizontal control is usually provided by triangulation survey, and the vertical control by precise leveling.
2. **Geological Investigation:** Geological investigation of the dam and the reservoir site are done for the following purpose:

i. Suitability of foundation for the dam.
ii. Water tightness of the reservoir basin.
iii. Location of the quarry sites for the construction material.

3. Hydrological Investigation: The hydrological investigations are done for the following purpose

i. To study the runoff pattern and to estimate yield.
ii. To determine the maximum discharge at the site.

7.6. Selection of Site for a Reservoir:

A good site for a reservoir should have the following characteristics:

1. **Large storage capacity:** The topography of the site should be such that the reservoir has a large capacity.
2. **Suitable site for the dam**: A suitable site for the dam should exist on the downstream of the proposed reservoir. The reservoir basin should have a narrow opening in the valley so that the length of the dam is small.
3. **Water tightness of the reservoir:** The reservoir sites having pervious rocks are not suitable. The reservoir basins having shales, granite, etc. are generally suitable.
4. **Good hydrological condition:** The hydrological conditions of the river at the reservoir site should be such that adequate runoff is available for storage.
5. **Deep reservoir:** A deep reservoir is preferred to a shallow reservoir because in the former the evaporation losses are small; the cost of the weed growth is low.
6. **Small submerged area:** The site should be such that the submerged area is a minimum. It should not submerged costly land and property.
7. **Low silt inflow:** The reservoir site should be selected such that it avoids the water carry a high percentage of silt.
8. **No objectionable minerals:** The soil and rock mass at the reservoir site should not contain any objectionable soluble minerals which may contaminate the water.
9. **Low cost of real estate:** The cost of the real estate for the reservoir site, dam, roads, railways, etc, should be low.

7.7. Basic terms and definition:

A large number of terms are commonly used for reservoir planning. These terms are defined below:

1. **Full reservoir level (FRL):** is the highest water level to which the water surface will rise during normal operation conditions. In case of dams without spillway gates the FRL is equal to the crest level of the spillway. If the spillway is gated, the FRL is equal to the level of the top of the gates. See(figure 7.4).
2. **Maximum water level (MWL):** is the maximum level to which the water surface will rise when the design flood passes over spillway.
3. **Minimum pool level (MPL):** is the lowest level up to which the water is withdrawn from the reservoir under ordinary condition. The (MPL) generally corresponds to the elevation of the lowest outlet (or sluiceway) of the dam.
4. **Useful storage:** volume of the water stored between (FRL) and minimum pool level is called the useful storage.
5. **Surcharge storage:** is the volume of water stored above the (FRL) up to (MWL) .The surcharge storage is an uncontrolled storage which exists only when the river is in flood and the flood water passing over spillway.
6. **Dead storage:** the volume of water held below the minimum pool level. The dead storage is not useful.
7. **Bank storage:** if the bank of the reservoir are porous, some water is temporarily stored by them when the reservoir is full. The stored water in banks later drains into the reservoir when the water level in the reservoir falls.

8. **Valley storage**: volume of water held by the natural river channel in its valley up to the top of its banks before the construction of reservoir.
9. **Yield from a reservoir:** volume of water which can be withdrawn from a reservoir in a specific period of time.
10. **Safe yield (Firm yield):** maximum quantity of water which can be supplied from a reservoir in a specific period of time during a critical dry year.
11. **Secondary yield:** is the quantity of water which is available during the period of high flow in the rivers when the yield is more than the safe yield.
12. **Average yield:** is the arithmetic average of the firm yield and the secondary yield.
13. **Design yield:** is the yield adopted in the design of the reservoir.

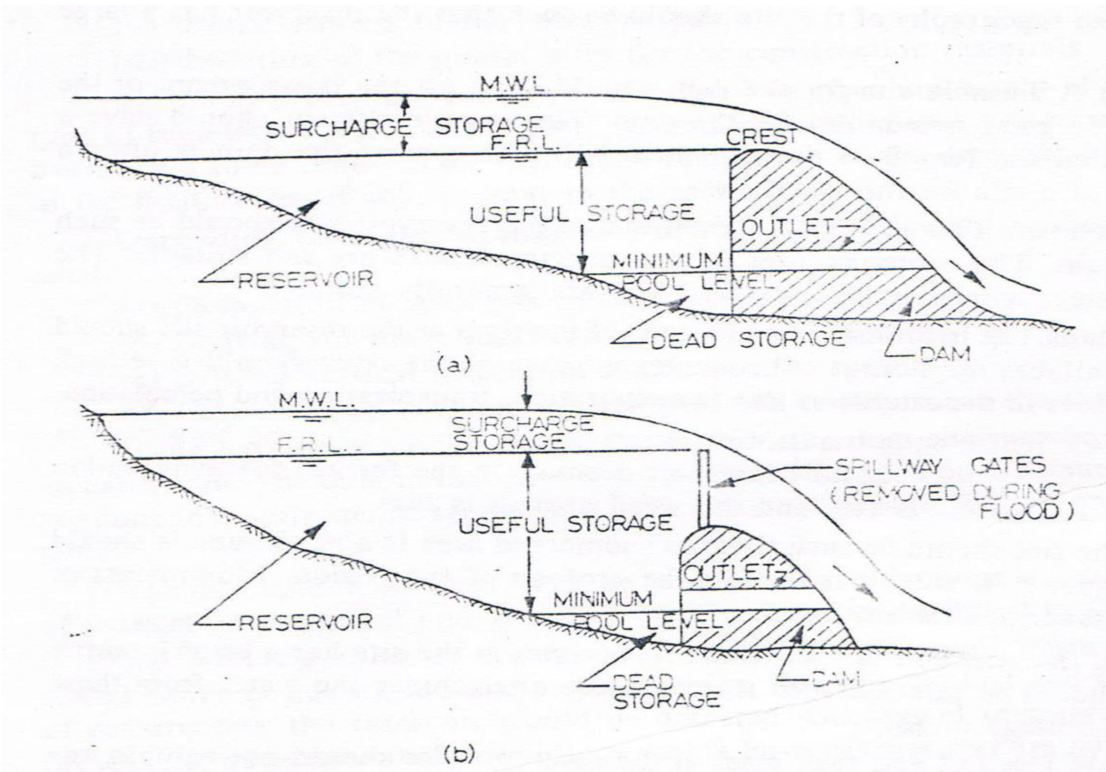


Figure (7.4) Gated and Ungated Spillway

7.8. Mass Inflow Curve and Demand Curve:

For the determination of the reservoir capacity, it is useful to study the mass inflow curve and demand curve:

a) **Mass inflow curve** also called mass curve, is pl ot between accumulated inflow volume as ordinate and time as abscissa. A mass inflow curve is prepared from the inflow hydrograph of a river for a large number of consecutive years. From the Fig.7.5 the area A 1 under the hydrograph from the starting year 1960 to 1961 represents th e volume of water in cumec-year t hat has flowed through the river . Fig. 7.5(a) shows mass inflow curve .The ordinate of the curve at the year 1960 is zero and that at the year 1961 i s equal to the volume of water flowed from the year 1960 to 1961, etc. If there is no flow during a certain period . The mass curve can be horizontal but it can never fall.

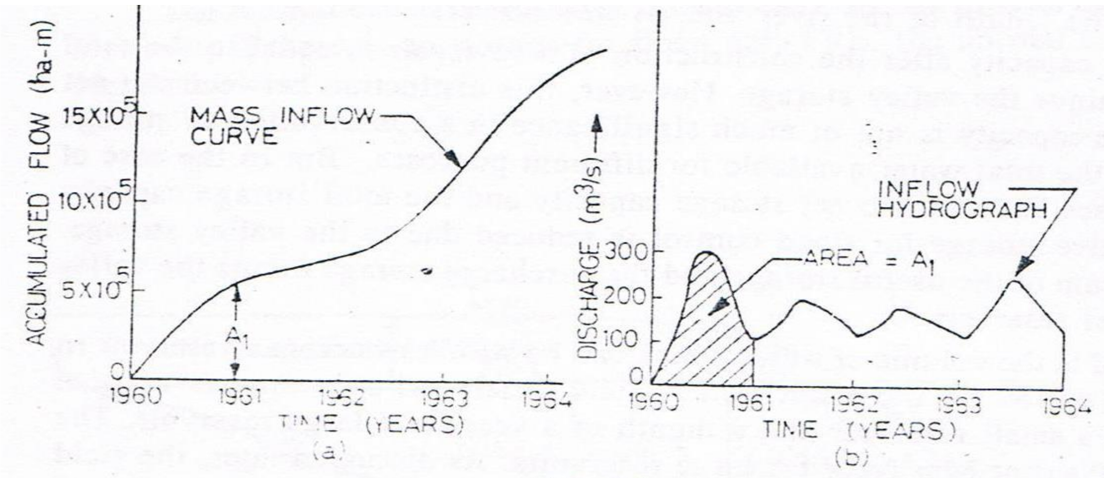


Figure (7.5) M ass Inflow Curve and Hydrograph.

b) **Mass demand curve** let us first define a demand curve. A demand curve is a plot between the demand rate as ordinate and the time as abscissa .The mass demand curve is a plot between the accumulated demand volume as ordinate and the time as abscissa. The mass demand curve is determined from the demand curve. If the demand is uniform, the demand curve is horizontal line (Figure 7.6), and the corresponding mass demand curve is straight line .If the demand is variable. The mass demand curve is arising curve. The mass demand curve is obtained from the demand curve after finding out of the area of the demand curve.

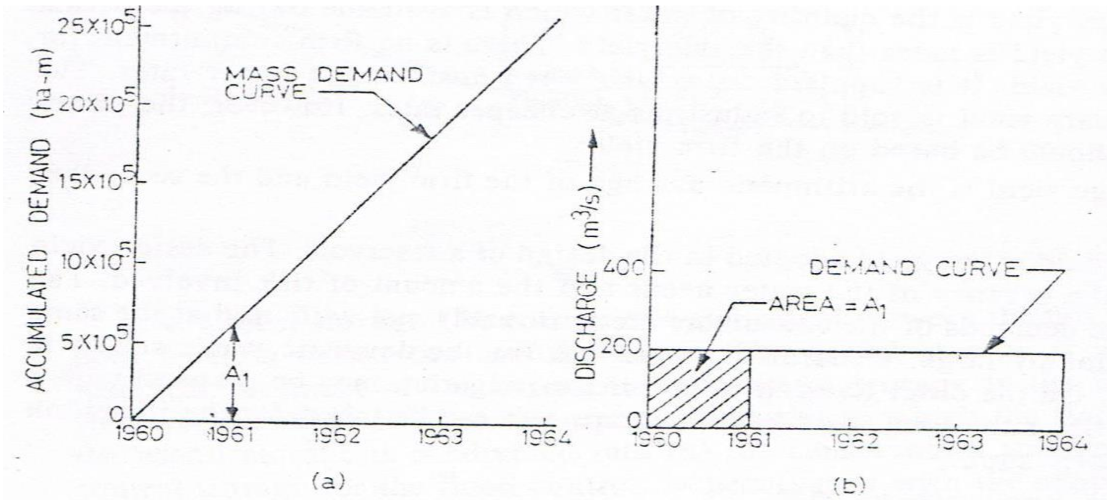


Figure (7.6) Mass Demand and Demand Curve.

7.9. Analytical Method For Determination of Storage Capacity:

The capacity of the reservoir is determined from the net inflow and demand. The storage is required when the demand exceeds the net inflow. The total storage required is equal to the sum of the storage required during the various periods. The following procedure is used for the determination of storage:

1. Collect the stream flow data at the reservoir site during the critical dry period. Generally, the monthly inflow rates are required. However. for very large reservoirs, the annual inflow rates may be used.
2. Determine the direct precipitation volume falling on the reservoir during the month.

3. Estimate the evaporation losses which would occur from the reservoir.
4. Ascertain the demand during .various months.
5. Determine the average inflow during different months as follows:

Average inflow = Stream inflow + Precipitation - Evaporation - Downstream Discharge.

6. Compute the storage capacity for each months.

Storage required = average inflow – Demand.

7. Determine the total storage capacity of the reservoir by adding the storages required found in Step 6.

Example:

The monthly inflow and monthly pan-evaporation during a critical dry year at the site of a proposed reservoir are given below. The net increase in pool area is 500 ha and the prior rights require the release of the full stream flow or 10 ha-m, whichever is less. Assume that 40% of the precipitation that has fallen on the submerged area reached the stream earlier and 60% of that directly falls on the reservoir. Determine the storage capacity. Take pan coefficient as 0.80.

Month	Jan	Feb	Mar	April	May	June	July	Aug	Sep	Oct	Nov	Dec
Inflow (ha-m)	10	10	4	2	1	200	2000	4000	1500	100	15	10
Pan evaporation (cm)	8	10	10	12	15	20	15	15	15	12	10	8
Precipitation (cm)	2	0	0	0	0	30	40	45	40	10	0	2
Demand (ha-m)	150	150	50	50	50	50	50	50	150	150	150	150

Solution:

Month (1)	Inflow (ha-m) (2)	Pan Evaporation (cm) (3)	Precipitation (cm) (4)	Demand (ha-m) (5)	Water right (ha-m) (6)	Evaporation (ha-m) (7)	Precipitation (ha-m) (8)	Adjusted inflow (ha-m) (9)	Require storage (ha-m) (10)
Jan	10	8	2	150	10	32	6	-26	-176
Feb	10	10	0	150	10	40	0	-40	-190
Mar	4	10	0	50	4	40	0	-40	-90
April	2	12	0	50	2	48	0	-48	-98
May	1	15	0	50	1	60	0	-60	-110
June	200	20	30	50	10	80	90	200	-
July	2000	15	40	50	10	60	120	2050	-
Aug	4000	15	45	50	10	60	135	4065	-
Sep	1500	15	40	150	10	60	120	1550	-
Oct	100	12	10	150	10	48	30	72	-78
Nov	15	10	0	150	10	40	0	-35	-185
Dec	10	8	2	150	10	32	6	-26	-176
									∑= -1103

Explanation:

Columns (1) to (6) are the given data

Column (7) = (Column (3) x 500 x 0·8)/100 = 4.0 x colm. (3)

Column (8) = Column (4)x500x 0.6/100 = 3·0 x colm. (4)

Column (9) = Columns (2) + (8) - (6) - (7)

Column (l 0) = Columns (9) - Column (5), whenever negative

Required capacity = total of column (10) = 1103 ha-m

7.10. Operation Plan of Multipurpose Reservoir:

An essential feature of the design of a multipurpose reservoir is to develop an operation plan which would be ideal for the utilization of the available storage for various purposes. For optimal planning of such a reservoir, a schedule which gives the maximum total benefit for different purposes of operation, without encroaching upon the lower or upper limits of storage in the reservoirs, is selected for initial adoption. The selected

schedule is put into operation, and if necessary its modified on the basis of the particle experience and future needs.

Let us consider the operation plan of a typical north Indian reservoir located on a snow-fed river. The reservoir is designed as a flood control reservoir for lowering the flood stage downstream and as a conservation reservoir to meet the demands for irrigation and hydropower. The basic requirements for operation of such a reservoir are as follows:

i. Reservation of a certain storage at all times for provision for flood control.

ii. Provision for a certain storage for irrigation of (summer) crops during April, May and June.

iii. Provision for a certain storage for irrigation of (winter) crops from the middle of November to the middle of February.

iv. Provision for a steady discharge for hydropower to meet the firm yield requirements.

7.11. Storage for flood control:

From the above operational plan, it is obvious that because the pattern of floods is distinctly seasonal only a very small proportion of the total reservoir capacity is specifically reserved for flood control to take care of unusual floods. Generally, before the flood season, almost the entire conservation storage is available for flood control. In countries where severe floods may occur at any time during a year, a large proportion of the total reservoir capacity has to be reserved for flood control.

It may be noted that the most of the conservation storage between the FRL and the minimum pool level is used for double purpose of irrigation and hydropower. For most of the time, the water released for irrigation is also

used for hydropower, except during the monsoon season when the water is used only for hydropower and not for irrigation. However, it does not affect the useful storage because excess flood water is available during this period. As discussed earlier, the planning of a multipurpose is carried out by trying a large. number of tentative schedules prepared after considering the hydrological data and the frequency of occurrence of high floods for the past 3 0 to 40 years.

7.12. Reservoir Losses:

Losses in a reservoir occur because of evaporation, absorption and percolation (seepage). Because of these losses, some of the stored water is lost and is not available for useful purposes. While planning a reservoir for conservation, it is essential to consider these losses. Moreover, various measures should be adopted to reduce these losses as far as possible.

1. **Evaporation losses:** evaporation depends upon a number of factors such as temperature, wind velocity and relative humidity. The greater the surface area of the reservoir . the greater will be the evaporation loss. Evaporation loss is usually expressed as depth of water lost in cm (or m). Evaporation loss from a reservoir is generally estimated by measuring the evaporation in an evaporation pan and multiplying the same by a suitable. pan coefficient. Thus:

   Volume of water lost = mean surface area * pan evaporation * pan coefficient.

   If the evaporation-pan data are not available, the observed evaporation data of the existing reservoirs may be used for the design of new reservoirs in the region. after making suitable adjustments due to the difference in the meteorological and other conditions.

2. **Absorption Losses:** The absorption losses occur due to absorption of water by the soil forming the reservoir basin. The absorption losses depend mainly on the type of soil. These losses are comparatively large in the beginning when the soil is dry. For most of the reservoirs, the absorption losses are quite small and are neglected.
3. Percolation Losses The percolation (or seepage) losses occur due to continuous flow of water under pressure from the reservoir to the adjoining strata. The percolation losses are usually small but they may be quite significant in some reservoirs if there are continuous seams of porous strata or caverns and fissures in rocks. To prevent percolation losses. the proposed reservoir basin should be thorough investigated by a geologist and checked for water tightness. If necessary suitable measures. such grouting of the basin, should be adopted to reduce seepage.
4. **Total losses:** The sum of losses due to evaporation, absorption and percolation is equal to the total loss. The total losses from a reservoir during a given period can be determined by applying the water budget equation. Thus

   Total losses= inflow volume - Outflow volume - Change in storage volume.

   From the inflow and outflow rates. the inflow volume and outflow volume can be computed. The change in storage is found by measuring the drop in the reservoir level and multiplying it by the average surface area during the period. If there is direct precipitation on the reservoir. it should also be accounted for by increasing the inflow volume by that amount. The value of the total losses determined from the study of the existing reservoirs is quite used for planning of the future reservoirs.

7.13. Measures Adopted To Reduce Evaporation Losses From Reservoirs:

The following measures are usually adopted to reduce evaporation losses from reservoirs:

1. Constructing Deep Reservoirs: As far as possible, deep reservoirs should be construct. evaporation losses are less for deep reservoirs as compared to those from shallow reservoirs. because in the former for the same storage the surface area is less.
2. Growing tall trees: Evaporation depends upon the wind velocity. If tall trees are grown on windward side of the reservoir, they act as wind breakers and hence reduce evaporation.
3. Removing weeds and water plants: If the weeds and water plants are removed from the reservoir periphery and surface. The evaporation (and also transpiration) losses are reduced.
4. Covering the reservoir: The evaporation from a reservoir can be reduced by covering it with polythene sheet. However, this method can be used only for small reservoirs.

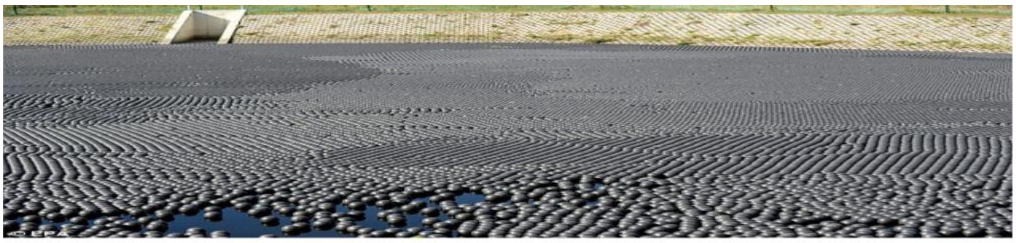


Figure (7.7) Covering the reservoir by polythene sheet.

5. **Spraying Chemicals:** When certain chemicals or fatty acids are sprayed over the water surface a thin film is formed which reduces the evaporation. Generally, acetyl alcohol (called hexadecanol) is used for reducing evaporation losses. When sprayed over water, about 0.015 micron in thickness (1 micron= 10^{-6} m) over the water surface. The layer is invisible, non-toxic, and it permits free passage of rain, oxygen and sunlight. The layer is quite effective ,in reducing evaporation. If the continuity of the film is maintained at all times, the reduction of evaporation losses may be about 50 to 60% of the natural evaporation. However, the method is effective only when the wind velocity is low. The method is quite suitable for small and medium reservoirs.

6.14. Useful Life of Reservoir:

All reservoirs ultimately get filled with sediments. The river carries sediments to the reservoir which are deposited in the reservoir. The deposition of sediments gradually decreases the available storage capacity of the reservoir. As more and more sediments are deposited in the reservoir, a stage comes when the reservoir is not able to serve its intended purpose and its useful life is over. If the annual sediment inflow is large compared with the reservoir capacity, the useful life of the reservoir would be very short. While planning a reservoir, it is essential to consider the rate of sedimentation to know whether the useful life of the proposed reservoir will be sufficiently long to justify the expenditure on its construction. The rate of sedimentation in the reservoir depends on the trap efficiency. The trap efficiency is defined as the percent of the total inflow sediment which is retained in the reservoir over total sediment. Thus:

$$\boldsymbol{Trap\ efficiency\ (n_t) = \left(\frac{Sediment\ retained}{Total\ sediment}\right) * 100}$$

From the observations of the rate of sedimentation of existing reservoirs, it has been found that the trap efficiency of a reservoir depends upon the capacity/inflow ratio. Brune (1948) gave the curves relating the trap efficiency and the capacity - inflow ratio on the basis of sedimentation data of existing reservoirs (Figure 7.8). In general, the greater the capacity-inflow ratio, the greater is the trap efficiency. In other words, the sedimentation rate is high in relatively larger reservoirs. The trap efficiency decreases with the age of the reservoir because the available capacity gradually decreases due to sedimentation. However, complete filling of the reservoir by the sediments may take very long time. The useful life of a reservoir is over when the capacity occupied by the sediment is large enough to prevent the reservoir from serving its intended purpose. Generally, the useful life of a reservoir is considered to be over when its capacity is reduced to about 20% of the designed capacity.

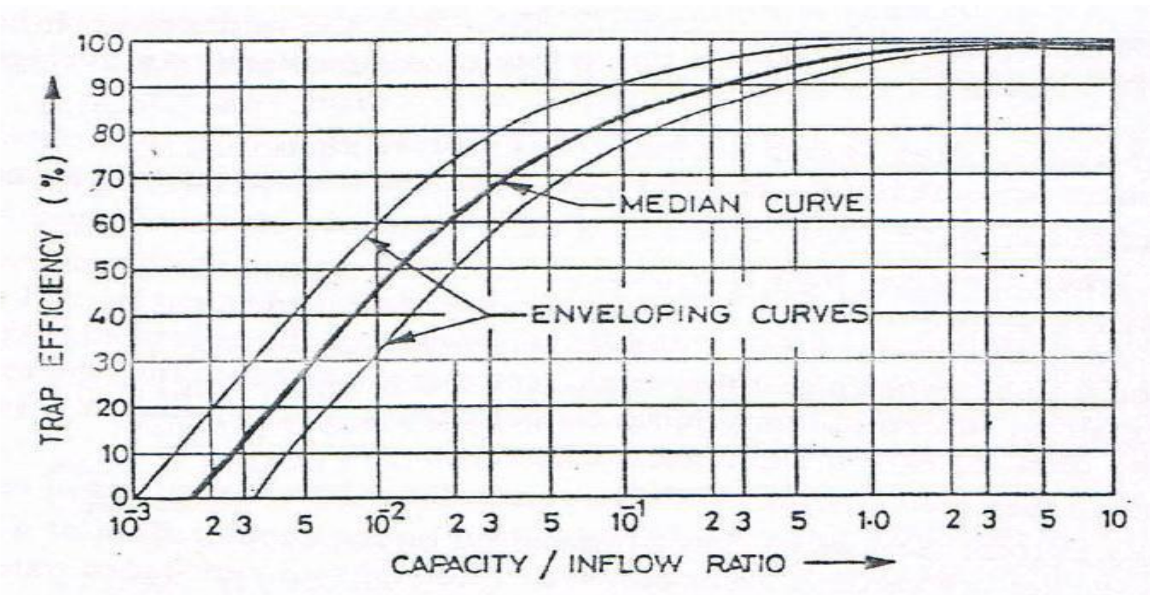


Figure (7.8) Determine (Trap efficiency).

To decrease the rate of sedimentation, the reservoir is sometimes built in stages. Initially the height of the dam is kept low and it is later raised in stages. When the reservoir of small capacity is constructed on a river having large inflow, the capacity-inflow ratio is low and the corresponding trap efficiency is also small. Most of the inflow is quickly discharged to the downstream and the suspended sediments are not able to settle fully. On the other hand, if a reservoir of large capacity is constructed, the capacity inflow ratio is large and the trap efficiency is high. In this case, the inflow is not discharged to the downstream quickly. and. therefore. the water is retained for a longer duration and most of the sediments get deposited.

The following procedure is used to estimate the useful life of a reservoir. It is assumed that the capacity, the annual inflow, the annual sediment load, the trap efficiency and the mass density of the deposited sediments are known.

1. Determine the capacity- inflow ratio from the annual inflow volume and the initial capacity of the reservoir.
2. Find the trap efficiency from Figure (7.8) for the capacity-inflow ratio found in step (1).
3. Divide the available storage in different portions. Generally its divided into 10 portions. Compute the capacity- inflow ratio when the capacity is reduced to (say) 90% of the initial capacity.
4. Determine the trap efficiency for the capacity-inflow ratio found in step (3).

5. Find the average of the trap efficiencies found in steps (2) and (4). This is the trap efficiency with which the sedimentation takes place when the capacity is reduced by first 10%.
6. Multiply the average annual sediment load by the average trap efficiency to determine the amount of sediment deposited annually.
7. Convert the deposited sediment from mass (or weight) units to the volume units by dividing the mass (or weight) of the sediment by the mass density (or specific weight).
8. Determine the number of years required for filling the 10% capacity of the reservoir by dividing this 10% capacity by the annual volume of sediment found in step (7).
9. Repeat the above steps and find out the number of years required for filling the successive 10%, of the capacity till only 20% of the capacity is left. i.e. 80% of its is silted.
10. Calculate the sum of the number of years required to fill successive 10% of the capacity. This sum total is equal to the useful life of the reservoir.

Example:

A reservoir has a capacity of 3.6 Mha-m up to the level of the spillway crest. The average annual inflow is 1.5 Mha-m of water. If the average annual sediment inflow is $3 *(10^{11})$ kg, determine the following:

a) The useful life of the reservoir, assuming that the usefulness of the reservoir is terminated when two -thirds of the total capacity is filled with sediments.
b) The storage capacity at the end of 30 years.

c) The period in which the reservoir will be completely filled up to the crest level. Assume mass density of sediments as 1100 kg/m3

Consider only 6 divisions of the total storage capacity.

Solution:

Annual sediments in flow = 3 *10^{11}kg

= (3 *10^{11}/1100) = 272.73 Mm^3 = 0.0273 Mha.m

Annual inflow of water = 1.5 Mha.m

Initial capacity - inflow ratio = 3.6/1.5 = 2.4

Storage capacity at the end of useful life = 1/3 x 3.6 =1.2 Mha.m

Capacity - inflow ratio =1.2/1.8 = 0.8

Volume of sediment deposited in 30 years = 30 x 0·0273 = 0.82 Mha.m
Storage capacity at the end of 30 years = 3.6 - 0·82 = 2.78 Mha.m
Capacity- inflow ratio = 2.78/1.5 = 1.85

When the reservoir is completely filled with the sediment. Capacity - inflow ratio = 0·0

The calculations are shown in the table below. The values of the trap efficiency are obtained from Figure (7.8).

S. No. (1)	Capacity (Mha-m) (2)	Capacity inflow ratio (3)	Trap efficiency		Annual trapped sediment		Incremental volume Mha-m (8)	No. of years required (9)
			Point efficiency (%) (4)	Average efficiency (%) (5)	mass (kg) (6)	Volume (Mha-m) (7)		
1.	3·60	2·40	98·2					
2.	3·00	2·00	98·0	98·1	$2{\cdot}943 \times 10^{11}$	0·0268	0·60	22·4
3.	2·40	1·60	97·5	97·75	$2{\cdot}933 \times 10^{11}$	0·0266	0·60	22·6
4.	1·80	1·20	97·0	97·25	$2{\cdot}918 \times 10^{11}$	0·0265	0·60	22·7
5.	1·20	0·80	96·0	96·5	$2{\cdot}895 \times 10^{11}$	0·0263	0·60	22·8
6.	0·60	0·40	95·0	95·5	$2{\cdot}865 \times 10^{11}$	0·0260	0·60	23·1
7.	0	0	0·0	47·5	$1{\cdot}425 \times 10^{11}$	0·0130	0·60	46·2
								159·8 years

Ans. Useful life of reservoir = 22.4 + 22.6 + 22.7 + 22.8 + 23.1 + 46.2

= 159.8 years.

7.15. Measures To Control Reservoir Sedimentation:

In order to increase the useful life of a reservoir, the rate of deposition of sediments in the reservoir should be decreased. The following measures are usually taken to reduce the reservoir sedimentation.

1. **Selection of suitable site:** The reservoir should be at the location where the sediment inflow is low. It sho uld exclude the runoff from an easily erodible part of catchment. The reservoir should not be built on the downstream of the confluence of a tributary carrying a large quantity of sediment.
2. **Proper design:** the rate of sedimentation is low in the reservoirs of small capacity. The reservoir may be designed in such a way that its capacity increases in stages. Initially, a reservoir of smaller capacity is created by constructing the dam to a lower height. After a portion of the reservoir gets filled up with sediments, the height of the dam is then increased.

3. **Provision of sluices:** An adequate number of sluices should be provided at different levels in the dam to discharge sediment-laden water to the downstream.
4. **Creating large reservoirs:** As far as possible, large reservoirs should be created. Although the trap efficiency of the large reservoirs is high, it does not increase linearly with an increase in capacity. Therefore, the useful life of a large reservoir is longer than that of a small reservoir. if all other factors remain constant. Of course, the cost of larger reservoir will also be more.
5. **Soil conservation:** Soil conservation methods commonly used are terraces. Strips cropping and contour farming. These methods reduce the velocity of flow of water and hence erosion. Other methods used are a forestation. control of deforestation, re grassing, control of grazing. checking gully formation by small check dams. etc. All these methods of soil conservation are effective for controlling sedimentation of reservoir to some extent but are quite expensive. Moreover, these are long-term measures and show encouraging results only after a long time.

CHAPTER EIGHT

DAMS OUTLET

Dam outlet

8.1. Introduction:

An outlet is a closed conduit formed in the body of the dam. It may also be in the form of a pipe or tunnel that passes through the hill side at one end of the dam. A major portion of the storage volume in the reservoir on the upstream of a dam is below the spillway crest level. Dam outlets are provided in the body of the dam or its abutments below the crest level of the spillway so that the water can be withdrawn from the reservoir. Sluiceways are special type of outlets provided in the body of a concrete (or masonry) dam figure 8.1

Outlets are required for releasing the impounded water as and when needed for various purposes such as hydropower, irrigation, municipal water supply and pollution control on the downstream. Outlets are also used for diverting water into canals or pipe lines. Sometimes outlets are designed to pass a part of the design flood to the downstream, as a supplement to the spillway. The water released by an outlet may be used for a combination of multipurpose requirements.

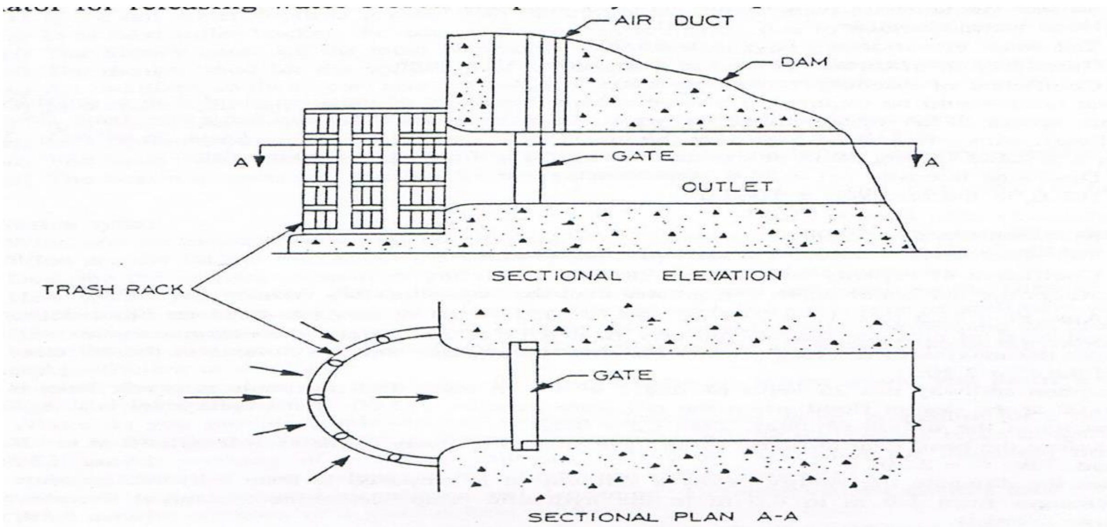


Figure (8.1): Section of outlet.

An outlet work may also act as a flood control regulator for releasing water stored temporarily in the space reserved for flood control. The outlets may also be used to empty the reservoir to make needed repairs or to maintain the upstream face of the dam or other structures.

8.2. LOCATION OF DAM OUTLETS:

For a concrete (or masonry) dam, the outlets pass through the body of the dam and are called sluiceways.

For the earth and rockfill dams, the outlets are generally placed outside the limits of the dam. However, for small earth dams, sometimes the outlet conduits are permitted to pass through the body of the dam. However, to prevent piping, adequate measures should be adopted. Generally, projecting collars are provided on the conduits to reduce the seepage along the outside is that the of the conduit.

A common rule is that the collars should increase the seepage length by at least 25 percent. Thus in Fig. 8.2,

$$\mathbf{2Nx \geq 0.25L \ldots\ldots\ldots\ldots 8.1}$$

Where: N is the number of collars, x is the projection of the collar measured from the outer surface of the conduit and L is the length of the conduit.

Generally, there are more than one outlet in a dam. If the outlet discharge considerably, it is always better to provide a number of small outlets than one large outlet.

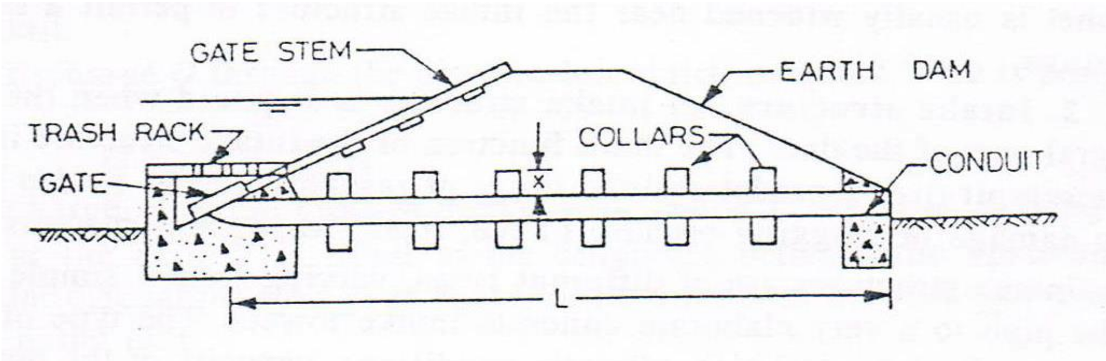


Figure (8.2) location of dam outlets

8.3. CLASSIFICATION OF OUTLETS:

The outlets may be classified as follows:

1. **Classification based on purpose:**

   Based on the purpose served, the outlets are classified as follows:

   i. River outlet: It is an outlet which discharges directly into the river downstream.

   ii. Canal outlet: It is an outlet which discharges into a canal taking off from the dam.

   iii. Pipe outlet: The outlet which delivers water into a closed pipe is called a pipe outlet.

2. **Classification based on physical and structural arrangement:**

   i. Open channel conduit: This type of conduit is in the form of an open channel.

   ii. Closed conduit: It is a conduit which is in the form of a pipe. It may also be a cut-and-cover type conduit or a tunnel.

3. **Classification based on shape:**

   i. Circular conduits: These are usually pipe conduits of circular section.

   ii. Rectangular conduits: These are generally open channel conduits or cut-and cover conduits.

iii. Other shapes: Conduits of other shapes are also sometimes used, especially in the tunnel outlets.

4. **Classification based on hydraulic action:**
   i. Open channel flow: These outlets have open channel flow. Closed conduits having a free surface also have open channel flow.
   ii. Pipe flow: These outlets have pipe flow and are under pressure.
5. **Classification based on gates (or valves):**
   i. Gated outlets: These outlets are provided with gates (or valves).
   ii. Ungated outlets: These outlets are not provided with any gate (or valve).

8.4. COMPONENT PARTS OF OUTLET WORKS:

Outlet works usually consist of the following component parts:

1. Entrance channel.
2. Intake structure.
3. Waterway.
4. Control device.
5. Terminal structure.
6. Exit channel.

1. **Entrance channel:** An entrance channel the is sometimes required when the entrance of the conduit is not the dam. The entrance channel leads the water from the reservoir to the outlet works (or intake).The entrance channel should be excavated to suitable slopes and dimensions. It should be designed for nonscouring velocities and the side slopes of the channel should be stable. The channel is

usually widened near the intake structure to permit a smooth, uniform flow in trash-racks to take opening.

2. **Intake structure:** An intake structure is required when the entrance to the conduits is not an integral part of dam. The main function of the intake structure is to permit with drawl of water from the reservoir over a pre determine derange of reservoir levels. It also protects the entrance of the conduit from damage or clogging because of ice, trash, waves and currents.

   Intake structures are of different types, varying from a simple concrete block supporting the inlet of the pipe to a very elaborate concrete intake towers. The type of intake is selected depending upon of the reservoir characteristics, climatic conditions, capacity of the outlets and number of other factors.

3. **Waterway:** The waterway is the passage which is used to carry the water from to the entrance to the exit of outlet. It usually consists of a closed conduit.

4. **Control device:** Once the type and size of the waterway has been designed, the method of control can be easily selected. The following types of control devices are generally used.

   a) Entrance gates: Most intakes and sluiceways are provided with some type of gate or valve at their entrance. On small dams with head less than 30 m, entrance gates may also be used as flow a regulators. However, with greater heads, the entrance gates are used only as emergency gates to permit inspections and repairs of the conduit.

   b) Interior gate valves: These are located downstream of the conduit entrance. For sluiceways in a gravity dam, the valve operating mechanism is generally located in a gallery inside the dam. For heads less than 30 m, interior gate valves are generally used to regulate flow. However, for greater heads, they are kept eithers in the fully open or fully closed position.

c) Flow regulation: On high-head installations, needle valves and tube valves, etc. are used for flow regulation.

5. **Terminal structure:** The discharge from an outlet emerges at a high velocity. It is usually in a nearly horizontal direction. The following types of terminal structures are provided:

   a) Deflector: For a free-flow (open channel) flow conduit, usually a deflector device is provided to direct the high-velocity flow away from the outlet structure if a strong bed rock exists in the downstream channel. However, if the rock is soft, a suitable energy dissipating device is provided to dissipate the energy of flow. These structures are essentially the same as those for spillways.

   b) Plunge basin: The flow from a valve at the end of the outlet is generally in the form of a jet it can be discharged into a plunge-basin downstream of the outlet or into a hydraulic-jump type stilling basin.

   c) Stilling well: When the exit of the outlet is submerged, a sing well dissipator is usually provided to dissipate the energy. It consists of a vertical well filled with water in which energy dissipation occurs by turbulence or by diffusion of the incoming flow.

6. **Exit channel:** An exit channel is required when the outlet does not discharge directly into the river downstream. It carries the discharge from the outlet to the river. The exit channel should be excavated to suitable slopes and dimensions to provide a safe, non-scouring velocity. The side slopes should be safe. The channel dimensions will depend upon the nature of the material through which the

channel is excavated. The channel may need lining or rip rap protection if the velocity is high.

8.5. DISCHARGE THROUGH AN OUTLET:

The outlet should be designed to have the required discharge capacity, as The discharge through the outlet can be determined as explained below separately for the pipe conduit outlet and the open-channel conduit.

(a) Pipe conduit outlet: The discharge through the pipe conduit outlet, in which there is pipe flow is determined from the relation:

$$Q = C_d A \sqrt{2gH} \quad \dots\dots\dots\dots 8.2$$

Where: C_d is the coefficient of discharge, A is the cross sectional area, and H is the head causing flow. If the outlet is submerged at the exit, **H** is equal to the difference between the upstream and downstream levels. When the outlet discharges free, **H** is equal to the difference between the upstream water level and the center of the outlet exit The value of coefficient C_d depends upon a number of factors, such as the design and shape of e conduit, type of the gate (or valve), various losses, etc.

In an outlet works, the following hydraulic losses should be considered and evaluated:

1. **Trash rack loss (h1):** The head loss occurs when the flow takes place through the trash rack. The head loss depends upon the design of the trash rack and the velocity through it. The trash rack loss is usually small because the velocity through the net area is usually limited to 0.3 to 0.6 m/s.

The trash rack loss may be taken 0.03, 0.09 and 0.15 m for the velocities of 0.3, 0.45 and 0.60 m/s, respectively.

2. **Entrance loss (h_2):** The head loss at the entrance to the conduit depends upon the size and shape of entrance and the velocity. The entrance is generally of the elliptical shape for high-velocity conduits . Depending upon the shape of the entrance, the following values for the entrance loss may be assumed.

$$\boldsymbol{a)\ Circular\ antrance:\ h_2 = 0.05\left(\frac{V^2}{2g}\right) \ldots\ldots\ldots\ldots 8.3a}$$

$$\boldsymbol{b)\ Rectangular\ antrance:\ h_2 = 0.1\left(\frac{V^2}{2g}\right) \ldots\ldots\ldots\ldots 8.3b}$$

Where: V is the velocity in the just d/s of the entrance.

3. **Friction (h3):** The friction loss depends upon the surface of the conduit. its size and length. and velocity of flow (V). Any standard formula may be used for calculation of the pipe-friction loss. Generally, the Darcy-Weisbach equation is used, after selecting a proper value of the friction factor f. Thus

$$\boldsymbol{h_3 = f\,\frac{L}{D}\frac{V^2}{2g} \ldots\ldots\ldots\ldots 8.4}$$

Where: L is the length and D is the diameter of the conduit.

4. **Transition losses (h4):** Transition losses occur wherever there is a change in the cross-section. These losses are usually expressed as:

$$\boldsymbol{h_4 = k\left(\frac{{V_1}^2}{2g} - \frac{{V_2}^2}{2g}\right) \ldots\ldots\ldots\ldots 8.5}$$

Where: $\boldsymbol{k}$ is a coefficient, and $\boldsymbol{V_1}$ and $\boldsymbol{V_2}$ the velocities before and after the transition.

The value of k is 0 .10 for a gradual contraction and 0.20 for a gradual expansion .For an abrupt change, the value of $\boldsymbol{k}$ is about 0.50.

For sudden enlargements, the following equation is normally used.

$$h_4 = \left(\frac{V_1{}^2}{2g} - \frac{V_2{}^2}{2g}\right) \ldots\ldots\ldots\ldots 8.5a$$

5. **Bend losses (h5):** The bend losses depend upon the bend radius, conduit diameter and the angle of the bend. The loss is usually expressed as:

$$h_5 = k\left(\frac{V^2}{2g}\right) \ldots\ldots\ldots\ldots 8.6$$

Where: k is the coefficient of bend, the value of which varies between 0.20 to 0.10.

6. **Gate and valve losses (h6):** The gate and valve losses depend upon their design, shape and size. The loss is usually expressed as:

$$h_6 = \left(\frac{1}{C_D{}^2} - 1\right)\frac{V^2}{2g} \ldots\ldots\ldots\ldots 8.7$$

Where: C_D is the free-discharge coefficient of the gate or valve, based on the total head just u/s of the gate. Table 8.1 gives the valves of C_D for various types of gates and valves.

Table 8.1 The valves of C_D for various types of gates and valves.

S. No.	Type of gate or valve	C_D
1.	Slide gate	0·95-0·97
2.	Ring follower gate	About 1·00
3.	Cylinder gate	0·80-0·90
4.	Needle valve	0·45-0·60
5.	Tube valve	0·50-0·55
6.	Howell-Burger valve	0·85
7.	Butterfly valve	0·60-0·85
8.	Sphere valve	About 1·00

7. **Exit loss (h7):** The exit loss is usually taken as:

$$h_7 = \frac{V_e{}^2}{2g} \ldots\ldots\ldots\ldots 8.8$$

Where: V_e is the exit velocity. Eq. 8.8 is based on the assumption of head at exit. If there is a discharge tube at exit, the loss is reduced. Applying Bernolli's equation to a point on the reservoir and at the exit of the outlet (Figure 8.3).

$$H = h_L + \frac{V_e^{\,2}}{2g} \dots\dots\dots\dots 8.9$$

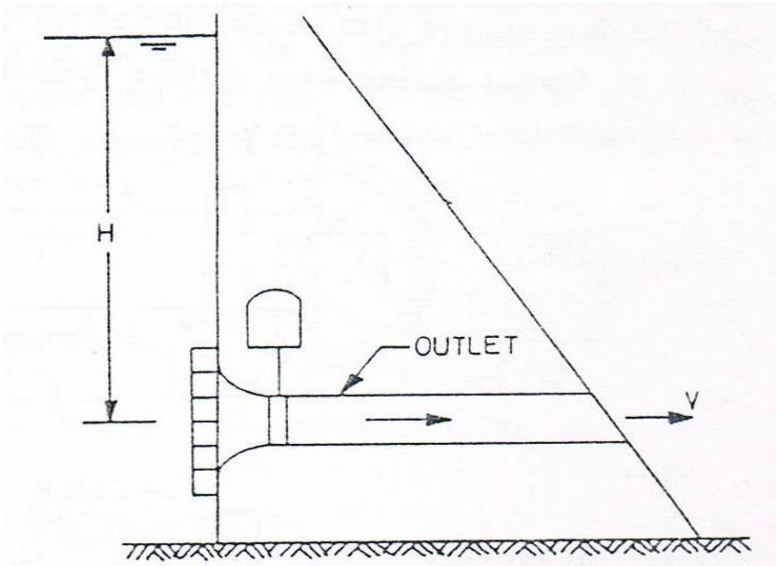


Figure (8.3) dam outlet

Where: $h_L = h_1 + h_2 + h_3 + h_4 + h_5 + h_6$ = total loss in system Of course, V_e is the velocity at the exit.

All the velocities V_1, V_2, etc. can be expressed in terms of the velocity V_e, at the exit by the continuity equation. Thus Eq. 8.9 may be written as:

$$H = k_L \frac{V_e^{\,2}}{2g} \dots\dots\dots\dots 8.10$$

Where: k_L is the composite coefficient of all the losses. Thus:

$$V_e = \sqrt{\frac{2gH}{k_L}}$$

Therefore, the discharge:

$$Q = AV_e = A\sqrt{\frac{2gH}{k_L}} \;\ldots\ldots\ldots\ldots\; \mathbf{8.11}$$

For a given discharge **Q**, if H is increased, the cross sectional area of the conduit is decreased. In other words, the conduit size can be reduced if the minimum pool level, at which the outlet is locate, is kept lower. For a given head **H** and the discharge **Q**, the conduit size can also be reduced by decreasing various losses and hence the value of $\boldsymbol{k_L}$.

b) Open – channel conduit: Flow in an open channel outlet is similar to that in an open channel (i.e. chute) spillway.

i. The discharge through the outlet when the gates are fully open is given by the weir formula:

$$Q = C\,L\,H^{3/2} \;\ldots\ldots\ldots\ldots\; \mathbf{8.12}$$

Where: C is the coefficient of discharge, L is the length of crest and is the head over the crest.

ii. When the open channel-outlet is controlled by a partly closed gate, the discharge given by the large orifice formula:

$$Q = \left(\frac{2}{3}C\sqrt{2g}\right) L\left({H_1}^{3/2} - {H_2}^{3/2}\right) \ldots\ldots\ldots\ldots\; \mathbf{8.13}$$

Where: $\boldsymbol{H_1}$ is the total head, including the head due to the velocity of approach over the bottom edge of the orifice, and $\boldsymbol{H_2}$ is the total head over the top edge of the coefficient of discharge, which depends upon the type of gate and the crest.

iii. When the gate opening is partly or entirely submerged due to high tail water level, the discharge is given by the submerged orifice formula:

$$Q = C_d\sqrt{2gH} \;\ldots\ldots\ldots\ldots\; \mathbf{8.14}$$

Where: H is the difference between the u/s and d/s water levels and A is the area of opening. C_d is the coefficient of discharge.

8.6. REQUIRED CAPACITY OF AN OUTLET:

The required capacity of an outlet is determined water needs, flood control regulation, storage considerations or legal requirements The required capacity of outlets for different purposes is fixed as follows:

1. **Irrigation outlets:** The required capacity of irrigation outlets is usually determine from the farm needs. It depends upon the area to be irrigated, the consumptive use of water (or duty), the type of crops.
2. **Municipal water supply outlets:** The capacity of municipal water supply outlets is established as in the case of an irrigation outlet. It depends upon the daily water requirements and the population served.
3. **Prior rights outlets:** Releases of water to satisfy prior rights should be included in other needed releases, as far as possible. The capacity of the outlets should also be adequate to satisfy the minimum downstream flows for pollution abatement, fish preservation and other needs.
4. **Flood control outlets:** Allocated flood control storage space of a of reservoir can be evacuated through a gated spillway at the higher levels or through an outlet at the lower levels. The capacity of the flood control outlets depends upon the required time of evacuation and the storage space to be evacuated. However, the combined flood control releases and irrigation releases into the river downstream must not exceed the safe channel capacity of the river downstream to avoid flooding.

5. **Outlet acting as a service spillway:** If the outlet is to be designed as a service spillway for discharging the normal flood water its capacity has to be fixed considering the design flood.
6. **Outlet for emptying:** The outlet capacity is sometimes decided considering the volume of water to evacuated and the permitted period for emptying the reservoir for inspection or repair.
7. **Outlet for diversion of flow during construction:** Sometimes an outlet works of the cut-and-cover type or tunnel type is also utilized for diverting the flow during the construction of the dam.

8.7. GENERAL CONSIDERATIONS FOR GATE AND VALVES:

The following points should be kept in mind while planning and selecting gates and valves for outlet works for a dam.

1. For outlets passing through the earth and rockfill dams. It is a good practice to install a gate at the upstream end of the conduit so that the conduit is under pressure only when the gate is open, as shown in Figure 10.2. On low-head installations, these entrance gates may also be used as regulating gates, but for high-head installations, these are used only as guard gates.
2. Guard gates should always be provided on the upstream of high-pressure regulating valves to permit inspection or repair of the Valves. The guard gate may be located near the upstream face of the dam or immediately upstream of the regulating valve in the same operating chamber. On large projects, two guard gates are sometimes provided in tandem so that at least one guard gate can be operated if the other becomes inoperative (Figure 8.4).
3. To minimize negative pressure and cavitation, an air inlet valve should be provided downstream of the high pressure gates and valve which do not discharge into the atmosphere.

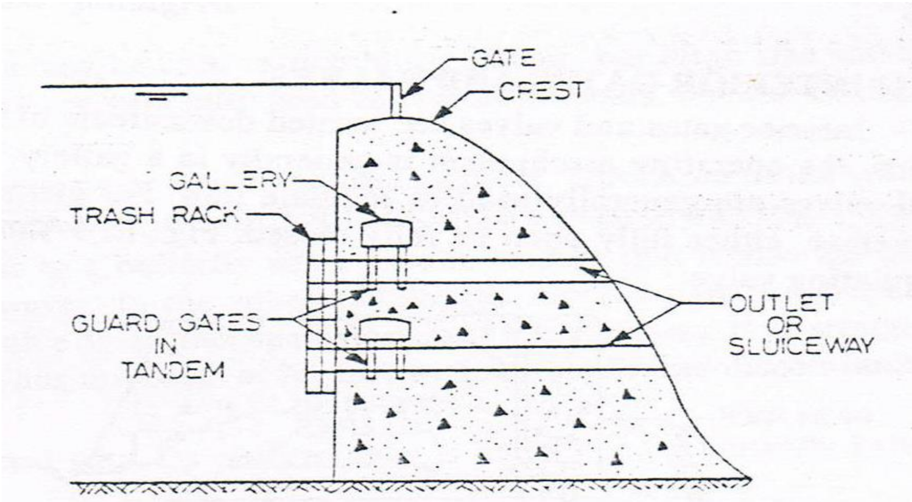


Figure 8.4 guard gate

4. For low-head installations, commercially available gates and valves or relatively simple gates can be used instead of the special devices which are quite expensive.
5. For high–head installation, especially designed gates and valves are generally required. These can be provided at the upstream end, at intermediate points as in line installation or at the downstream end of the conduit, depending upon their design.
6. The control of upstream gates is generally placed in a tower structure extending above the maximum water level.
7. Where a free -flow (open-channel) conduit is provided downstream of the control device, an access shaft is provided directly over the control. The access shaft for tunnel outlet can be sunk into the hillside.
8. The gates which are designed to operate under the conditions of balanced pressure on both sides, the conduit on downstream side of the gate is filled through by pass pipe through connected to the reservoir upstream. An air vent should be provided to drive out the entrapped air.

APPENDICES

Appendices

1. Dams in Iraq

1.1. Introduction:

Water is a vital, multi-dimensional, scarce and unevenly distributed resource. Water sustains various forms of life and is a key driver for various ecological and economical processes such as food and energy production. Shorter of water resources in middle east is one of the most problems since the last century due to increase in population and increase water demand and also the change in climate. Iraq is located in the eastern part of the MENA region [MENA: Middle East and North African countries]. It is surrounded by Iran in the east, Turkey to the north, Syria and Jordan to the west, Saudi Arabia and Kuwait to the south and the Gulf to the southeast .The total area of Iraq is 438,320 km^2 of which 924 km^2 of inland water. About 25% of the in-habitants live in rural areas. The population density ranges from 5 to 170 inhabitants/km^2 in western desertic and the central part from the country respectively. This rate had dropped since 1989 due to severe economic hard- ship . Iraq is shaped like a basin containing the great Mesopotamian plain of the Tigris and Euphrates rivers. The climate is mainly of a continental, subtropical semi-arid type with the north and nor- the astern mountainous regions having a Mediterranean climate . The temperature during summer is usually over 430°C during and drops down to 20°C and 160°C during the day and night re- spectively in winter time . Iraq was considered rich in water resources and water supply due to presence of Euphrates and Tigris River until 1970s .after this year, Turkey and Syria started to constructed dams on the Euphrates river and decrease of its flow due to impounding of water in some of the new reservoirs. This make Iraqi government to speed up construct as much as can from the planed irrigation structures. From 1970s

to 1990s was the best period of the development of Iraqi water system and that development stopped in 1990s due to the first Gulf war. during this period (1970 – 1999) there was change in supply and demand situation. In 1970s Turkish government started to building GAP project on the Tigris and Euphrates river in the South-eastern Anatolia. The project includes 22 multipurpose dams and 19 hydraulic power plants which are to irrigate 17103 km^2 of land with a total storage capacity of 100 km^3 which is three times more than the overall capacity of Iraq and Syrian reservoirs . Eight of these dams are to be constructed on the River Tigris, only three were built (two in 1997 and one in 1998). The irrigation projects within the GAP will consume about 22.5 $km^3 \cdot year^{-1}$ after completion . The reduction of flow in the Tigris and Euphrates Rivers in Iraq is considered to be a national crisis and will have severe negative consequences on health, environmental, industrial and economic development. In view of the above, it became necessary to know the water resources trends in the Tigris-Euphrates rivers basin within Iraq.

1.2. Water Resources of Iraq:

1.2.1. River Tigris:

The Tigris River is one of two major sources of surface water in Iraq, with 1900 km length, 1415 km of which are in Iraq. The River Tigris rises in the southeastern part of Turkey on the southern slopes of the Touros mountain range and drains an area of 472,606 Km^2 which is shared by Turkey , Syria and Iraq. River Tigris has three major tributaries is [Butman SU , Karzan and Raznk] join the Tigris before it reaches the Turkish/Iraqi border. Tigris enters Iraq at fiesh khabur where the khabur tributary joins the main river at the small distance to the south. The river Tigris flows towards the south and reaches the first major city (Mosul). Its mean discharge at Mosul reaches 630 m^3/s. The river of Greater Zab joins

the Tigris about 60 km south of Mosul city . The confluence of the two rivers is situated midway between Mosul and Sharkat cities. This tributary drains an area of 25,810 km^2 of which about 62% lies in Iraq. This tributary is one of the largest with a mean annual flow of 418 m^3/s. Further south, the Lesser Zab tributary joins the Tigris at Fatha. This tributary drains an area of 21,476 km^2 (25% in Iran) with a mean annual flow of 227 m^3/s whiles the mean annual flow of the Tigris reaches1340 m^3/s down-stream of this confluence. South of Fatha, the Adhaim tributary joins the Tigris. This tributary drains an area of 13,000 km^2 and lies totally in Iraq . The mean annual flow of this river reaches 25.5 km^3. This tributary runs dry between June and November each year. Furher to the south, the last major tributary, the Diyala River joins the Tigris south of Baghdad. The Diyala basin is 31,846 km^2 of which about 20% lie in Iran. The mean daily flow of this tributary is 182 m^3/s. No major tributary joins the River Tigris south of Baghdad . Few canals draw water from the Tigris in this region for irrigation purposes. For this reason, the mean annual daily flow of the river falls Baghdad (1140 m^3/s) in Kut and Amara cities at the south .The Tigris River mean discharge at Mosul city prior to 1984 was 701 m^3/s and dropped to 596 m^3/s afterward . This implies a 15% decrease of the river discharge. Characteristics of the Tigris River basin shown in the (Table 1).

Table (1) Characteristics of the Tigris River basin.

Tigris river	Turkey	Iraq	Syria	Iran	total
Discharge $Km^3.year^{-1}$	33.5	6.8	0.0	11.2	51.5
Discharge (%)	65.0	13.2	0.0	21.8	100
Drainage area Km^2	45,000	292,000	1000	37,000	375,000
Drainage area(%)	12.0	54.0	0.2	33.80	100
River length (km)	400	1318	44	-	1862
River length (%)	21.0	77.0	2.0	-	100

1.2.2. River Euphrates:

Euphrates is the longest river in western Asia. The majority of the water resources of the Euphrates are located in the Turkish territories of Anatolia. The river rises near Mount Ararat at heights of around 4500 m near Lake Van, the Euphrates is formed from two tributaries, the Murat-Su and the Kara-Sue (or Frat-Sue). The River flows southward to 160 km of the Mediterranean with average slope 2 meters per kilometre before it turns left into Syria to continue in a south-east direction, almost straight towards Shatt Al-Arab River. After the river enters the Syria’s borders at Jarablis, within Syrian territories by two small tributaries join the river. They are Balikh and the Khabur Rivers that contribute with small amount of water to the Euphrates River. Euphrates enters Iraq at Hasaibah. Its annual flow at the Iraqi border is of the order of 28 to 30 $km^{3}\cdot$ year−1. In Iraq, 360 km from the border, the river reaches a giant alluvial delta at Ramadi where the elevation is only 53 m. From that point onward, the river traverses the deserted regions of Iraq, losing part of its waters into a series of desert depressions and distributaries, both natural and man-made. Euphrates has number of small tributaries in the central and southern parts of Iraq for irrigation purposes. No tributary contributes water into the river within Iraqi territories stics of Euphrates basin tabulated in Table (2).

Table 2: Characteristics of the Euphrates River basin.

Euphrates River	Turkey	Iraq	Syria	Total
Discharge (km^3. $Year^{-1}$)	32.5	0	0.5	32.5
Discharge (%)	98.5	0	1.5	100
Drainage area (km^2)	125000	177000	76000	444000
Drainage area (%)	28	40	17	85
River length (km)	1230	1060	710	3000
River length (%)	41	35	24	100

1.2.3. River Shatt Al-Arab:

Shatt Al-Arab River is formed after the confluence of Tigris and Euphrates Rivers at Qurnah in Iraq. Its total length is 192 km and its drainage area is 80,800 km^2. Its width is about 300 m near Qurnah and increases downstream to 700 m near Basra city and to about 850 m near its mouth at the gulf area. Karun and Karkha Rivers usually contributes 24.5 and 5.8 billion cubic meters (BCM) annually respectively (Figure 1). This forms about 41% of the water of Shatt Al-Arab. Its annual discharge at Fao city reaches 35.2×10^9 m^3. Shatt Al-Arab River is characterized by its high sediments which resulted in the formation of large number of islands during its course.

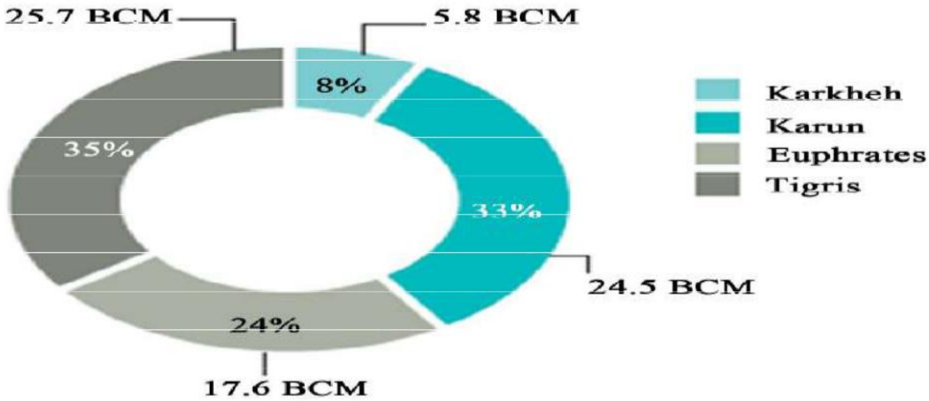


Figure 1: Water contribution to Shatt Al-Arab River.

1.3. Dams and Reservoirs in Iraq:

The dams built on rivers, waterways and valleys are of great benefit as they arise to control floods and use water stored to meet water needs throughout the year for the sectors used for all water, especially the agricultural sector and for drinking, as well as electricity generation and tourism development by providing attractive and convenient sites for residents and citizens. Wishing to be present for tourism and comfort, and contribute dams to improve the environment and climate in the areas surrounding the lakes as well as to improve the living conditions of the

population through employment opportunities prepared in the project in addition to Exploit The wealth of fish and improve the agricultural reality.

The Ministry of Water Resources in Iraq achieved this activity and worked on developing it and continuing to build dams on the rivers and their tributaries and on the waterways in order to achieve the mentioned objectives, especially after the water resources that are returned to Iraq in the Tigris and Euphrates Rivers and their tributaries have been reduced due to the establishment of the riparian countries Turkey and Syria Iran and the establishment of dams on the rivers within its territory as affected the water received to Iraq quantitatively and qualitatively. The cases of scarcity and drought are indicative of the importance and benefits of dams and the possibility of being controlled by floods, water imports and redundancies in certain seasons and launch them at other times, especially the seasons of scarcity, so it is permissible for Iraq to have enough dams to store the quantities of water needed. In addition to the drought is the early movement of neighboring countries that hold water inside its territory and operate in order to achieve its internal objectives and this leads to deterioration of water revenues received to Iraq in quantity and quality, and all this calls for thinking to create reservoirs on the watercourses seasonal and seasonal water, This is what has been done by the Ministry of Water Resources to develop a strategy for the construction of large and small dams in different parts of the country to work on the reservation and contain any available amount of water and employ them properly. As explained below:

1.3.1. Dams in the Euphrates basin:

The following is a list of dams and reservoirs sorted according to location in the Euphrates river basin. As in figure (2).

Table 3: dams and reservoirs location in the Euphrates river basin.

No.	Name of dam	Type of dam	Storage volume	Height (m)	Length (m)	Opening data	Number of gate	Design discharge m^3/s	Power Station	Governorate
1	Haditha	Earth fill	8.3 km^2	57	9064	1987	-	-	660 MW	Al - Anbar
2	Fallujah Barrage	-	For control	-	-	1985	16 (52x28)ft	3600	-	Al - Anbar
3	Hindiya Barrage	Building blocks	For control	-	250	1913	36 (5 m wide)	-	-	Babil
4	Ramadi Barrage	Concrete	For control	-	209	1955	24 (6 m wide)	3600	-	Al - Anbar
5	Kufa Barrage	-	For control	-	-	1988	7 (12x6.3)m	1400	-	Babil
6	Shameya Barrage	-	For control	-	-	1988	6 (12x6.3)m	1100	-	Al-Diwanya

1.3.2. Dams in the Tigris basin:

The following is a list of dams and reservoirs sorted according to location in the Tigris river basin. As in figure (2).

Table 4: dams and reservoirs location in the Tigris river basin.

No	Name of dam	Type of dam	Storage volume m^3	Height (m)	Length (m)	Opening date	spillway	spillway discharge m^3/s	Power Station	Governorate
1	Dukan	concrete arch dam	$6970x10^6$	116.5	360	1959	2(tunnel) 3 gates	2450	400 MW	Al-Sulaymaniyah
2	Darbandikhan	Embankment; rock-fill, central clay core	$3x10^9$	128	445	1961	Spillway on right bank (3 gates)	11400	249 MW	Al-Sulaymaniyah
3	Hemrin	Embankment; rock-fill, central clay core	$2040x10^6$	53	3360	1981	-	-	50 MW	Diyala
4	Mosul	Embankment, earth fill clay core	$11100x10^6$	113	3400	1986	controlled chute	13.000	187.5 MW	Ninawa
5	Duhok	earth-fill embankment	$52x10^6$	60	600	1988	Bell-mouth	81	-	Dohuk
6	Adhaim	Embankment, zoned earth-fill	$1.5x10^6$	76.5	3500	2000	-	1150	27 MW	Diyala
7	Lund	earth-fill embankment	$38x10^6$	24	1342	2013	-	-	-	Diyala
8	Kut Barrage	concrete	-	10.5	516	1939	56 gates	6000	-	Wasit
9	Samarra	-	$150x10^6$	65	-	1955	-	7000	84 MW	Salah ad Din
10	Dibis	gravel-alluvial fill embankment	$15x10^6$	23.75	376	1965	2	4000	-	Kirkuk
11	Diyala	-	-	-	-	1969	-	-	-	Diyala

1.3.3. Dams under construction that are planned for construction:

1. Badoush dam on the Tigris (under construction).
2. Bakhmah Dam on the Great Zab River (under construction).
3. Mundawah Dam on the Great Zab River.
4. Bakerman Dam on the Khazar River.
5. Khalikan dam on the Khazar River.
6. Dam Taq on the Lower Zab River.
7. A dam on a branch of the Great River.
8. Al-Baghdadi Dam in Ramadi.
9. Tulsaq dam in Diyala.

1.3.4. Regulators:

The regulations, whether major, subsidiary, conclusive or marginal, of the important establishments within the irrigation projects, are established on the irrigation channels in their issuance or in different locations, or at the head of the dams on the rivers, their tributaries and their various ramifications.

The regulator shall be designed according to the discharge through it. It shall be constructed of concrete or reinforced concrete and its floor shall be calculated according to the pressure of the water flowing through the openings. The joints shall be separated from the openings called the supports of the regulator based on the passage road which is above the regulator and usually from reinforced concrete or concrete to withstand the stressed water pressure They are in different forms in terms of facing the water passing through the regulator.

The regulator operates according to the quantities of water required and then distributed to the sub-channels according to the water needs of the various irrigation projects. The total number of regulators in irrigation

projects in Iraq is (2351), distributed between all governorates and different types (main, sub-branches, seagulls and seagulls), in addition to (49) large branch organizations of the Tigris and Euphrates rivers and their branches. Such as the Tharathar on the Samarra and the Diyala dams on the Diyala River, the Kut dam and the Amara dam on the Tigris River, Ramadi, Fallujah and Hindi on the Euphrates River. As in figure (2).

1.3.5. Dams in waterways connecting the Tigris and Euphrates basins:

1. Badaa Head Regulator, on the Shatt al-Hayy.
2. Gharraf Head regulator, on the Shatt al-Hayy.
3. Gharraf Regulators, on the Shatt al-Hayy.
4. Shallala Weir, on a canal from Lake Tharthar to the Tigris.
5. Tharthar Diversion Structure, regulating the flow from Lake Tharthar.
6. Tharthar Canal Control Structure, regulating the flow from Lake Tharthar toward either the Euphrates or the Tigris.

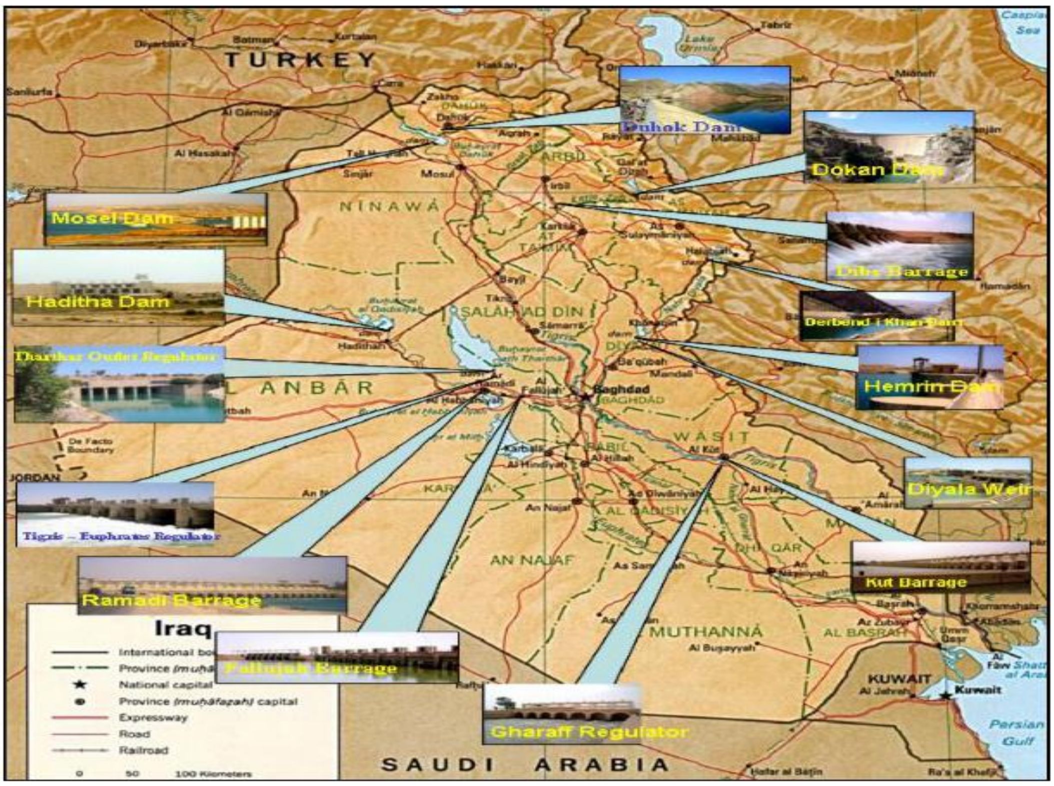


Figure (2): Dams and Reservoirs in Iraq.

1.4. References:

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Haditha Dam

Fallujah Barrage

Ramadi Barrage Hindiya Barrage

Hemrin Dam Kut Barrage

Adhaim Dam

Dibis Dam Samarra Dam

Diyala Dam

Dukan Dam

Duhok Dam

Darbandikhan Dam

Mosul Dam

Kufa Barrage

2. Health effects of dams

2.1. Introduction:

Water, today, is considered as one of the fundamental basis of development, economic prosperity, and social well-being and its shortage will cause many limitations. Water shortages directly and indirectly effect on sectors such as the control, storage and water supply, distribution types, transmission, factor productivity, planning, conservation management and .... In this regard, it is considered to be constructed large and small dams and small irrigation networks of rivers in different countries to take advantage of the water potential to meet the needs and objectives such as drinking, agriculture, industry, generating non-polluting hydropower energy, flood control and improved water quality and quantity. In additions, dams provide water during a year; they can be used for boating and recreational opportunities and tourist purposes.

Dams have one of the most important roles in utilizing water resources. They were constructed long years before gaining present information about hydrology and hydromechanics. Dam projects, which are useful in meeting the demand for water in desired times and in regulating stream regimes, have undertaken an important function in the development of civilization. Dams have been constructed in order to prevent floods, to supply drinking and domestic water, to generate energy and for irrigation purposes since the old-times. The first dam was The Quatinah Barrage or Lake Homs Dam, located in Syria, is the oldest operational dam in the world. The dam was constructed during the reign of the Egyptian Pharaoh Sethi between 1319-1304 BC, and was expanded during the Roman period and between 1934 and 1938. The masonry gravity dam impounds the Orontes River and creates Lake Homs, supplying water for the city of Homs through canals. Dams have a great deal of positive and negative effects on the environment

besides their benefits like controlling stream regimes, consequently preventing floods, obtaining domestic and irrigation water from the stored water and generating energy. Dams hold possibilities of considerable harm for living beings in addition to their advantages such as meeting basic requirements of the society and increasing living standards. Nearly 700 dams were built every ten years up to 1950s. This number grew rapidly after 1950s. While the dams were built and completed it was observed that there was something missing and detrimental. Although the effects of water on human life and the development of civilizations are well known all over the world, it is claimed that the economical benefits expected from the projects designed to utilize water resources could not be gained and also necessary precautions to decrease the environmental, economical and social losses were not taken. Because of this, in the sustainable management of the water, taking into account the economical, social and cultural development and the environmental impacts which came out as a result of the mentioned studies, has gained an increasing importance. Therefore, it is essential that these water resources development studies should have a legal background to ensure sustainable development.

2.2. EFFECTS OF DAMS ON HYDRAULIC SYSTEM:

The main hydraulic effect is the discharge of the collection basin to a stationary reservoir instead of a stream bed. Therefore, an instant change will start downstream; downstream of a stream dries partially or totally whenever the reservoir begins to accumulate water. During this temporary or periodically repeating time interval, the hydrological balance can collapse; Irreversible death, disappearance and structural jumps are observed in the water dependent ecosystem. Decay of dead flora and fauna in the new coming water body speeds up. So, upstream water flows polluted, without oxygen in deeper parts, dark colored for a long time and

usually smells rotten because of hydrogen Sulphur disposal. Although after this process the stream forms a new and healthy ecosystem in this part of it, neither this new aqua balance nor the terrestrial ecosystem and even the sea environment that the stream joins the sea have the chance to join their previous health. Some of the effects on the hydraulic system are mentioned below:

- As the reservoir works like a big settlement basin, turbidity in the water flowing downstream decreases and erosion around the lake deceases slowly.
- Increases in the evaporation losses because of the enlargement in the water surface can be observed.
- The various in the temperature regime of the water environment can be classified in two groups:

1. Thermal variations that may end in seasonal thermal layer formations depending on the water depth in the dams.
2. The variations that happen in the water temperature inside the reservoir related to the water depth that was able to leak trough the downstream gates and the exchange of water with constant temperature.

The river will behave like a cold climate river from chemical and biological qualifications point of view, as the water will be always cold even in summer, if the gate depth lies below the thermocline of the reservoir.

On the contrary, it will behave like a hot climate river if the flowing water is at surface water temperatures. Effects similar to these can continue kilometers along the downstream. Numerous other effects can be added to this list. The most important point that must be considered here is to distinguish the temporary harms from the long term and irreversible harms clearly. It is compulsory that the experts consisting of biologist, engineers,

hydrologists, social scientists and other profession groups prepare the environmental impact assessment studies and that the alternatives do their duty in the estimation of environmental effects. Dams, which contribute to the national economy from many aspects like irrigation, drinking water supply, flood control, electricity generation, fishing, tourism, are also effective in increasing the living and culture level of the region that they were constructed. Meanwhile, the new environment created by the dam also supports the arrival of different species to the area. Dams are not only important in economic growth, but also in overall economical and moral development. In many developed countries, dams have performed a key role in the development of the underdeveloped regions.

2.3. THE EFFECT OF DAMS ON CLIMATE SYSTEM:

Changes in humidity percentage, temperature and air movement because of the great volume of stagnant water will change the climate related to the region of topography. Moreover graded changes of climate are perceived. As an example by building the “Kalghan dam” in the cold region of Iran, great volume of water will be saved behind the dam which will have some effects on weather and climate of the region. Among these changes we can name increase in absorbing solar energy in the region, thermo interchange between dam lake water and adjacent atmosphere and changes in amount of vaporization and fog by increase in vaporization level amount of steaming will grow. Although these changes may not be so harmful for humans’ health, but they are remarkable for many plants and animals and have secondary effects on humans. Moreover, one of the most significant factors for air pollution has been to move diesel and petrol cars in during of “Kalghan dam” constructions. These cars was spread a lot of smog. One of the other bad effects of dams is spreading greenhouse gases

from reservoir because of the spoiling and decay of plant covering and carbon stream from the reservoir.

2.4. EFFECTS OF DAMS ON HUMAN LIFE:

Despite the fact that the dams are an important target for development; they are not easily acceptable for the people whose agricultural areas, houses and the environment they are living in, go under water. For example, when the Volta Lake was created in Ghana in 1969, although a much better settlement area was provided for 80,000 people in another location, these people have returned as 100,000 people and have built their own houses unplanned on the lake shore. Such an unsuccessful experience caused by the social-psychology can be very dangerous for the bio systems in the region and for the reservoir itself. There are changes in the employment and production systems starting before the construction of the dam including expropriation of the land, employment of construction workers and the transport of construction material with the machines to the site. Unqualified workers are employed from the site; however, the technicians and experts come from other places.

Generally, settlement areas, social buildings, hospitals, schools etc. are built for the people coming from outside the site. The more these facilities can be hold open for public usage the more the dam becomes a kind of symbol for development. The new settlements improve by this way and result in second ecological needs and changes.

For example, drinking water, domestic waste water, waste water treatment etc. Moreover, the social life becomes active, trade increases, cultural activities rise. Important alterations are observed in the transportation system. At the same time dams decrease the pollution effect considerably in the downstream part by lowering the pollution load coming from the

source, thanks to their big storing reservoirs. In addition, they decrease the pollution load again by containing water continuously in their beds during dry periods. Dams decrease the flood risk in the downstream, by their storing opportunity in their reservoir. Meanwhile, dams protect the people living downstream from floods. After comparing harms and benefits for a long period of time, a decision can be given about dams. May be the unwanted side effects of dams will be no longer in force because of the benefits in the future. But these big engineering structures should remind us that we are not able to change only a part of the ecosystem. Because whole chains are connected together in the ecosystem.

Large dams have forced some 40-80 million people from their lands in the past six decades, according to the World Commission on Dams. Indigenous, tribal, and peasant communities have been particularly hard hit. These legions of dam refugees have, in the great majority of cases, been economically, culturally and psychologically devastated. Those displaced by reservoirs are only the most visible victims of large dams. Millions more have lost land and homes to the canals, irrigation schemes, roads, power lines and industrial developments that accompany dams. Many more have lost access to clean water, food sources and other natural resources in the dammed area. Millions have suffered from the diseases that dams and large irrigation projects in the tropics bring. And those living downstream of dams have suffered from the hydrological changes dams bring to rivers and ecosystems; an estimated 400-800 million people--roughly 10% of humanity--fall into this category of dam-affected people.

In response to the massive human rights problems and environmental impacts of large dams, affected people and supporting local and international organizations have joined together to fight for change in how and whether dams are planned, designed and built. This movement

includes thousands of environmental, human rights, and social activist groups around the world. International dam-affected people's meetings in Brazil, Thailand and Mexico in recent years have brought together dam-affected peoples and their allies to network and strategize, and call for better planning of water- and energy-supply projects. And every year, groups from around the world show their solidarity with those dispossessed by dams on the International Day of Action for Rivers, a global event to raise awareness about the impacts of dams and the values of free-flowing rivers.

2.5. THE EFFECT OF DAMS ON MARINE SPECIES ECOSYSTEM:

In constructing phase, increase of erosion and sedimentation in lower part of dam site will happen so by growth in water particles and their sediment causes to eliminate some species of marine environments and confuses ecologic balance of these regions. At the beginning, decomposing of living creatures causes to increase food materials in the water for a short period, so water O2 taking (BOD) is rising up and an anaerobic decomposing environment is formed by assistance of still layers in the depths of reservoir that this creates a dark and noisome smell in the lake, so a high increase in phytoplankton are perceived. Besides the plants which cover water surface in the form of a broad layer in dark green color, some big vegetable species macro flora) grow on water surface. This can be dangerous either for the lake life or for people fishing and sailing and even for dam's valves and turbine fans. Sometimes these macro floras act as a disease transmitter, meanwhile increase in water plants results to increase in vaporization and unnatural transpiration. Dams are some barriers to fishes movement from upper hands of the river toward its lower part. Therefore existence of dam means death of species of fishes that spend a

part of their life in springs and or in water uprisings and spend another part of it in the crossing of rivers and seas together. Considering that some marine fishes move toward fresh waters and lakes to spawn and then come back to the sea with young fishes, existence of dam halts this movement and interferes the cycle and leads to decline these fishes. Sometimes for this, siding crossing streams are made.

2.6. EFFECT OF DAMS ON TERRITORIAL BIOLOGICAL SYSTEM:

Biological life of the river changes fast both in the reservoir and in downstream. The parts of the bio-system that are affected from the dam are the watered parts on the shore. During the filling works of the dam, while the lands remain under water the land part of the region decreases. However, the water-land boundary extends. Thus, plant, animal or human being settlement areas change. 9 Forests, agricultural areas may go under water. As the water level differentiate periodically, some species begin to live under water from time to time, in the tide zone. This area may turn to marshy land or reed-bed depending on the soil structure. Water-soil-nutrient relations, which were settled after floods in the downstream of the dam, change in a long period of time. Furthermore, important changes occur in flora, fauna and the agricultural traditions of people in the region. This effect can extend over some kilometers.

2.7. HEALTH EFFECTS OF DAMS:

We can call the hygienic effects of dams in this manner:

- Dams' lake can be source of many contagious diseases such as malaria and blood diseases, this event in some countries including African countries has caused many damages.

- In shallow lakes, it provides an appropriate bed for insects to shed seeds.
- Considering that refilling with water is done at watery season, so less amount of water is transferred to the river and lower part of dam. Considering the stable capacity of wastewater and other entering contaminators in lower parts we expect the added amount of pollution in lower part. Also in dry period the condensation of surface waters contaminators in lower parts increases so that this matter intensifies in lower parts by taking much water.

2.8. EFFECTS OF DAMS ON AQUATIC ECOSYSTEM:

At the beginning, the decomposing organisms cause an increase in the nutrient substances in water in a short period of time because during decomposition process, different products are released: carbon dioxide (CO2), energy, water, plant nutrients and resynthesized organic carbon compounds. Therefore, BOD (Biological Oxygen Demand) value of water rises. An anaerobic decomposition media is performed with the help of the stationary layers along the reservoir's depth. This results in a dark colored lake smelling badly. Afterwards, an enormous increase in phytoplankton feed by the increased amount of nutrients is observed.

Besides the plants covering the water surface as large green-dark colored bodies, macro-flora grows up on water surface. These events can be harmful both for the liv ofanimals living in the lake, and also for the people fishing, taking a boat-trip and even for thedam gates and turbine propellers. Sometimes, macro-flora created here acts like a source fordisease vectors. Separately, this increase in water plants cause more evaporation losses thanit happens by evapotranspiration normally.The dam is a real obstacle for the animals swimming from one end of the river to the other end. The existence of the dam means death for the fish species spending certai parts

oftheir life in the spring or in the flood water and other parts in the cross-section where theriver joins sea. We know that some sea fishes come to fresh water and swim up to the spring in order to lay eggs. Later on, they return to sea with new young fishes. A dam that will be built on this way will interrupt the life cycle of these creatures and cause deaths in a mass. It has seen that by-pass flows are designed for this purpose.The environmental consequences of large dams are numerous and varied, and includes direct impacts to the biological, chemical and physical properties of rivers and riparian (or "stream-side") environments. The dam wall itself blocks fish migrations, which in some cases and with some pieces completely separate spawning habitats from rearing habitats. The dam also traps sediments, which are critical for maintaining physical processes and habitats downstream of the dam (include the maintenance of productive deltas, barrier islands, fertile floodplains and coastal wetlands).

Another significant and obvious impact is the transformation upstream of the dam from a free-flowing river ecosystem to an artificial slack-water reservoir habitat. Changes in temperature, chemical composition, dissolved oxygen levels and the physical properties of a reservoir are often not suitable to the aquatic plants and animals that evolved with a given river system. Indeed, reservoirs often host non native and invasive species (e.g. snails, algae, predatory fish) that further undermine the river's natural communities of plants and animals. The alteration of a river's flow and sediment transport downstream of a dam often causes the greatest sustained environmental impacts. Life in and around a river evolves and is conditioned on the timing and quantities of river flow. Disrupted and altered water flows can be as severe as completely de-watering river reaches and the life they contain. Yet even subtle changes in the quantity and timing of water flows impact aquatic and riparian life, which can

unravel the ecological web of a river system. A dam also holds back sediments that would naturally replenish downstream ecosystems. When a river is deprived of its sediment load, it seeks to recapture it by eroding the downstream river bed and banks (which can undermine bridges and other riverbank structures, as well as riverside woodlands). Riverbeds downstream of dams are typically eroded by several meters within the decade of first closing a dam; the damage can extend for tens or even hundreds of kilometers below a dam. Riverbed deepening (or "incising") will also lower groundwater tables along a river, lowering the water table accessible to plant roots (and to human communities drawing water from wells) . Altering the riverbed also reduces habitat for fish that spawn in river bottoms, and for invertebrates. Large dams have led to the extinction of many fish and other aquatic species, the disappearance of birds in floodplains, huge losses of forest, wetland and farmland, erosion of coastal deltas, and many other immitigable impacts. Some other effects are listed below:

- Loss of wild lands, wetlands and wildlife habitat.
- Effects of stopping the flow of nutrients downstream. Reduced biological activity downstream.
- Anaerobic decomposition of vegetation and production of greenhouse gasses.
- Water-loss due to evaporation.
- Changes in water quality due to the lack of dissolved oxygen near the bottom of reservoirs. This is toxic to fish and can lead to the death of aquatic life. It is also corrosive to turbines.
- Accommodation of amphibians, riparian fauna and birds to a new environment.

- Migration of animals to new areas, where new equilibrium may favor some species over others.
- Blocking fish migration.
- Introducing of new species of fish in the reservoirs.
- Inappropriate reservoir operation with large variations in water levels could threaten fish by drying up shallow-breeding and flood producing areas.

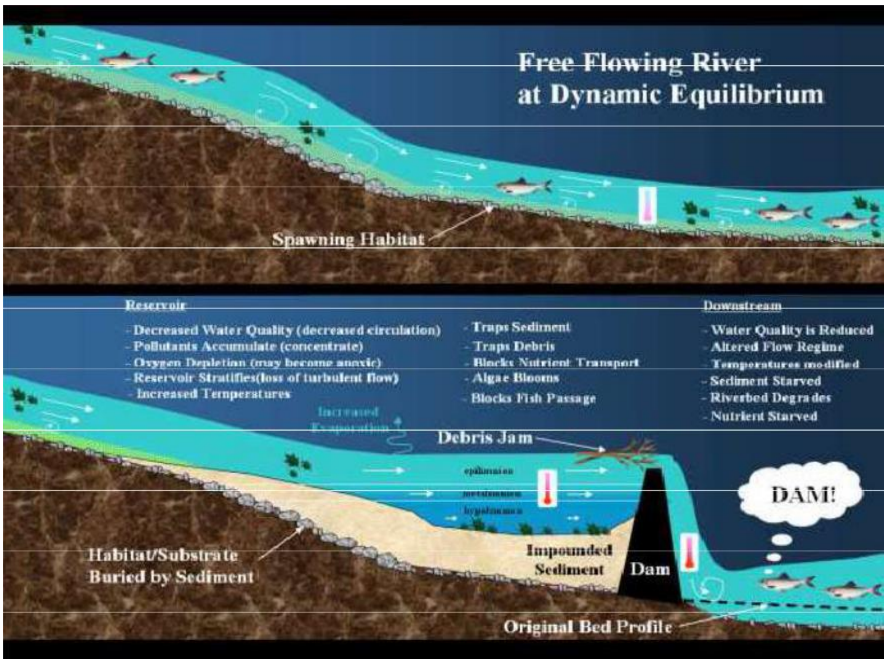

Figure (1) Spawning Habitat with Dam and without Dam.

2.9. PHYSICAL AND CHEMICAL EFFECTS OF DAMS:

Physical and Chemical Effects of Dams are explained as follows:

- Constructing dams as an obstacle against movement and crossing floating objects in the course of rivers.
- Sedimentation of dregs materials in reservoir andlake of the dam that leads to obstructing the gates and outlets. Besides change in natural balance of transferring dregs that intensifies river erosion in lower part of the dam.

- The effect of exit of muddy water containing dregs materials over the lower part regions and regional environment.
- Occurring to much water floods resulting from releasing overflows (spillways) water and drain cocks causes main physical, chemical and biological changes in lower parts of the dams.
- The effect on changes of underground water level.
- Land sliding :at the result of reaction between lake water level and getting wet of the region and different laminated structures beside the reservoir, land sliding happens that its effects lead to lessen of the lake capacity and making high waves and dam overflowing and or destroying it.
- Forming inductive earthquakes: because of the effects that water charge has on the lake floor it may cause inductive earthquakes.
- To salt agricultural lands for running up the underground water levels and draining of lands as a result of surface irrigation.

2.10. EFFECTS OF DAMS ON ATMOSPHERIC SYSTEM:

Variations in moisture percentage, temperature and air body movements of air caused by the big stationary water body differentiate micro-climate related to region topography. In addition, regional scaled climatic changes can be observed. These alterations may seem not very harmful for human health, but they are notable for many plants and animals. Their secondary effects influence human being.

2.11. EFFECTS OF DAMS ON EARTH CRUST:

Even though it is claimed that the dam reservoirs have some seismic effects. it must be stated that this has not been proven scientifically. Till now scientists are researching on this topic.

2.12. IMPACT ON SOUND POLLUTION:

During constructing phase with coming the machineries and equipment’s of building the dam, sound level is intensified in the region severely. Also produced sound by relevant equipment’s and explosions done within dam building causes to disturb privacy in the region and has undesirable effects over wild life in the region of constructing operation. Leaving the nest, migrating to adjacent regions, probability of abortion in mammalian and laying eggs in birds are among the effects and sound pollution consequences from project activities in the constructing phase.

2.13. ENVIRONMENTAL CATASTROPHIC OF DAMS COLLAPS:

Dams are considered "installations containing dangerous forces" under International Humanitarian Law due to the massive impact of a possible destruction on the civilian population and the environment. Dam failures are comparatively rare, but can cause immense damage and loss of life when they occur.

Figure (2) Aftermath of Bento Rodrigues dam disaster.

2.14. ECONMIC AND SOCIAL EFFECTS OF DAMS:

The economic and social destructive effects of dam buildings can be said as following:

- Villages and community in the area of dam lake are drowned, it leads to immigration and increase in cities population and arising false jobs.
- Destroying the roads and power transferring lines as a result of crossing among the lakes, no access to some parts of dam area, old canals, leak stoppers, etc. are among the losses of big dams construction.
- Destroying agricultural lands to provide dam constructing materials or drowning them causes unemployment of so many people.
- Historical and ancient regions and places with special and beautiful topography which are found rarely, will be disappeared by going under water.

2.15. POSITIVE IMPACTS OF DAMS ON THE ENVIRONMENT:

Dams, which contribute to the national economy from many aspects like irrigation, drinking water supply, flood control, electricity generation, fishing, tourism, are also effective in increasing the living and cultural level of the region that they were constructed. Meanwhile, the new environment created by the dam also supports the arrival of different species to the area. Dams are not only important in economic growth, but also in overall economical and moral development. In many developed countries, dams have performed a key role in the development of the underdeveloped regions like:

- **Flood control benefits:** it decreases and remove the flood effects.

- **Land improvement benefits:** are the extra benefits that will occur after an increase in the soil productivity because of drainage and land improvement precautions.
- **Electrical energy benefits:** Hydropower is fueled by water, so it's a clean fuel source, meaning it won't pollute the air like power plants that burn fossil fuels, such as coal or natural gas.
- **Transportation benefits:** are the benefits that will happen in the case of where there is waterway transportation in the project.
- **Providing drinking water and domestic water:** the dams been constructed in most cases provide drinking water and for domestic use.
- **Irrigation benefits:** defines the distinction benefits between dry and irrigated positions which is for agricultural purposes.
- **Recreation:** dams provide recreational opportunities for fishing, boating, swimming, bird watching, etc. This is due to the increased habitat for water fowl, fish, and water sports. These activities support local economies by increasing tourism.

2.16. NEGATIVE IMPACTS OF DAMS ON THE ENVIRONMNET:

While preparing the water resources projects, it is important to make clear what the environmental impacts of the project may be when it is executed. The environmental impacts of the dams have been written down below:

- Archeological and historical places in company with geological and topographical places that are rare with their exceptional beauties disappear after lying under the reservoir.
- Discharge of toxic materials (pesticides, toxic metals etc.)

- Water-soil-nutrient relations, which come into existence, are related to the floods occurring from time to time in a long period. Depending on this fact, compulsory changes come into existence in the agricultural habits of the people living in this region and also in the flora and fauna.
- Dams may cause increases in water sourced illnesses like typhus, typhoid fever, malaria and cholera.
- Dams affect the social, cultural and economical structure of the region considerably. Especially forcing people, to migrate and affect their psychology negatively.
- Rise in evaporation loses may be expected as a result of the increase in the water surface area. This causes increase in humidity of the area.

2.17. Case Study: Farakka Barrage-Problems (The Farakka Barrage: Impact on Bangladesh):

In 1974India built a barrage on the Ganges at Farakka in order to divert water for its own use .The water is diverted to the Hooghly Rivervia a 26-mile long feeder canal. The unilateral and disproportionate diversion of the Ganges caused a dangerous reduction in the amount of sediment and water flow of the Ganges in Bangladesh. Now Bangladesh's delta receives less sediment and inadequate water flow for navigation and irrigation during the summer months.

Groundwater Also dropped below the level of existing pumping capacity. Such conditions lead to significant decreases in food production and curtailment of industrial activities.

Figure (3) The Farakka Barrage (Impact on Bangladesh)

As a result of a lack of adequate freshwater inflow, the coastal rivers experienced saltwater intrusion 100 miles farther inland than normal during the summer months, affecting drinking water in these areas.

The reduction in sediment supply has curtailed delta growth and has led to increased coastal erosion. At the present time the rates of sediment accumulation in the coastal areas are not sufficient to keep pace with the rate of relative sea level rise in the Bay of Bengal.

This reduction in carrying capacity due to river bed aggradations has increased the frequency of severe floods over the last decade, causing enormous property damage and loss of life. If the amount of sediment influx in the coastal areas is further reduced then a relative sea level rise in the Bay of Bengal by 1 meter will severely curtail the delta growth, resulting submergence of about one-third of Bangladesh.

Figure (4) The Farakka Barrage: Impact on Bangladesh).

India has suggested building of 3 dams on Brahmaputra in Assam and excavation of a 209-mile link canal through which 100,000 cusec of water would be diverted from the Brahmaputra to the Ganges during the summer months.Building of the more canal and dams on the Brahmaputra will make Bangladesh more dependent on India for its share of water.

Diversion of water from the Brahmaputra will jeopardize irrigation, navigation, and other components of ecosystem in the Brahmaputra valley. Besides, about 20,000 acres of lands will be lost in Bangladesh to the canal.Bangladesh is suggesting for an alternate artificial canal connecting the Ganges and Barhmaputra between Sirajganj and Bheramarain Kushtia district wouldserve the same purpose for Bangladesh as the 209- mile link canal proposed by India. The length of the proposed Sirajganj-Bheramara link canal would only be 60 miles.

The Government of Bangladesh proposes to augment the Ganges flow by building of numerous storage reservoirs in India, Bhutan, and Nepal to store water during rainy seasons which can be released in summer months.

The potential for electric power that can be generated from such project is huge. A better physical control of the supply, accumulation, and dispersal of sediment and water can ensure growth of the delta.

Annual dredging of rivers and dispersion of the dredged sediments on the flood plains and delta plains will increase the water carrying capacity of the rivers, and reduce flooding propensity. Re-excavation and re-occupation of abandoned distributaries of the major rivers would re-establish the already disrupted equilibrium of the hydrodynamic system due to upstream diversion of the Ganges. An integrated management plan for both surface and groundwater is necessary for Bangladesh to reduce the economic hardship that is being caused by the diversion of the Ganges.

2.18. CONCLUSION:

- The environmental changes coming out of dams are in various amounts and in different importance degrees. It is difficult to consider the relations between these effects beforehand and determine which positive and negative effects will come up.
- There are real and potential benefits obtained from these projects. Industrial development has gained speed; irrigation channels and food production have improved as a result of the increase in electricity generation. Meanwhile, dams protect the people living downstream from floods. After comparing harms and benefits for a long period of time, a decision can be given about dams.
- Dam construction, like a coin, has its two sides. How to exploit favorable conditions and avoid unfavorable ones is the business of dam architects , dam constructors and dam managers.
- We should study the disadvantages caused by dam engineering, and demonstrate the feasibility of dam construction project.
- The degradation of river ecosystems, as a consequence of river regulation, can have profound economic and social implications. In the past failure to take into account the cost of these consequences has resulted in the benefits of many dams being overstated.

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3. Numerical Work

3.1. Introduction:

Over the past decades, numerical modeling techniques have been rapidly developed as computational power enhanced to the point where numerical solutions are now possible for many applications. This development has led to the widespread use of numerical modeling as a standard design tool in many engineering disciplines. Despite the wide range of numerical modeling applications, the fundamental principles, upon which all numerical models are based, is similar for all models. Problems begin with a set of partial differential equations that describe the underlying physics of the particular situation. Some types of numerical method, such as finite element analysis or the finite volume method is then used to formulate a set of algebraic equations that represent the partial differential equations. An approximate solution to those algebraic equations is then obtained through some form of either an iterative or matrix solution. This solution is often a very computationally intensive, which makes the use of modern computational power so important to the use of numerical models. In most cases, the numerical model solutions are verified or calibrated through comparisons to field observations or physical model experiments before being applied in practice. Even after extensive model verification, sound engineering judgment is required to ensure the accuracy of any model output.

3.2. Flow 3D Uses and Benefits:

FLOW 3D is a powerful numerical modeling software capable of solving a wide range of fluid flow problems. Current areas of software application include the aerospace industry, various forms of casting, inkjet printers, and several different aspects of hydro-electric generating stations.

A good selection of different options across the entire FLOW 3D graphical user interface allows the software to be applicable to such a wide variety of situations. FLOW 3D allows either one or two fluid flow, with or without a free surface, and a multitude of available physics options to suit the specific application. Various meshing and geometry options are available including multi-block grids and the ability to draw simple objects in the software or import different forms of more complex geometry or topographic files. A large selection of boundary conditions is also available to properly model for each specific application. Another benefit of FLOW 3D is the ability to select from several different implicit and explicit numerical solver options. All of these model set-up parameters can easily be specified by either encoding selections in the text editor or by making radio button selections in the graphical user interface (Chanel, 2008).

3.3. Theory of Flow:

The governing equations incompressible, viscous fluids are continuity and momentum equations. These equations are known as Navier- Stokes equations. These equations express conservation of mass and momentum mathematically. Also, the two equations turbulence model is an effective numerical simulation method used in recent decade. The continuity equation, momentum equation and equations for (K-e) model are given as Eq. 1 to 4 below:

$$\frac{\partial \rho}{\partial t} + \frac{\partial u_i}{\partial x_i} = 0 \quad \dots\dots\dots.1$$

$$\frac{\partial \rho u_i}{\partial t} + \frac{\partial (\rho u_i \varepsilon)}{\partial x_i} = -\frac{\partial P}{\partial x_i} + \frac{\partial}{\partial x_i}\left((\mu + \mu_i)\left(\frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i}\right)\right) + \rho G_i \dots.2$$

$$\frac{\partial \rho u_i}{\partial t} + \frac{\partial (\rho u_i k)}{\partial x_i} = -\frac{\partial P}{\partial x_i} + \frac{\partial}{\partial x_i}\left(\left(\mu + \frac{\mu_i}{\sigma k}\right)\left(\frac{\partial k}{\partial x_i}\right)\right) + G + \rho \varepsilon \dots\dots.3$$

$$\frac{\partial(\rho\varepsilon)}{\partial t}+\frac{\partial(\rho u_i\varepsilon)}{\partial x_i}=\frac{\partial}{\partial x_i}\left(\left(\mu+\frac{\mu_t}{\sigma\varepsilon}\right)\left(\frac{\partial\varepsilon}{\partial x_i}\right)\right)+C_{1\varepsilon}\frac{\varepsilon}{k}G+C_{2\varepsilon}\rho\frac{\varepsilon^2}{k}\;\ldots\ldots 4$$

In which: t=time, u_i = velocity component; ρ= density, μ=molecular viscosity; $\bar{P}=P+\frac{2\rho k}{3}$

Where: $\bar{P}$ is modified pressure and P is the pressure. The parameter μ_i=turbulence viscosity, which can be calculated by turbulence kinetic energy (k) and turbulent dissipation rate (ε) as Equation 5 (after chen et al., 2002).

$$\mu_i=\rho C_\mu\frac{k^2}{\varepsilon}\;\ldots\ldots\ldots .5$$

Where: C_μ =experimental constant. The turbulence Prandtl number for $(k-\ \varepsilon)$ are σ_k=1 and $\sigma_{\varepsilon=}$1.3, respectively, and $C_{1\varepsilon}$=1.44 and $C_{2\varepsilon}$=1.92 are constants for the (ε) equation. The generation of turbulent kinetic energy (G) due to mean velocity gradients can be defined as Equation 6 (after chen et al., 2002):

$$G=\mu_i\left(\frac{\partial u_i}{\partial x_j}+\frac{\partial u_j}{\partial x_i}\right)\frac{\partial u_i}{\partial x_j}\;\ldots\ldots\ldots .6$$

3.4. Computational Fluid Dynamics (CFD):

In present Study, FLOW 3D version 10.0.1 was used to solve the governing equations of Navier-Stokes equations in Cartesian coordinates staggered grid. The flow region is sub divided into a grid of fixed rectangular cells. Mesh is a subdivision of the flow domain into relatively small regions called cells, in which numerical values such as velocity and pressure are computed. Determining the appropriate mesh domain along with a suitable mesh cell size is a critical part of any numerical model simulation. Mesh and cell size can

affect both the accuracy of the results and the simulation time, so it was important to minimize the amount of cells while including enough resolution to capture the important features of the geometry as well as sufficient flow details. An effective way to determine the critical mesh size was to start with a relatively large mesh and then progressively reduce the mesh size until the desired output no longer changes significantly with any further reductions in mesh size. A useful option in FLOW 3D that made the process of meshing even more effective was the many blocks mesh. This allows the user to use more than one mesh, including mesh size and configuration, before restarting the simulation with the use of information from the last time step of the previous simulation. For each of the spillways that were modeled, this process was utilized beginning with a uniform (0.008 m) mesh cell size. Restart runs with a uniform (0.006 m), (0.005m), (0.004m) and (0.003m) mesh for free overflow simulations where discharge was the main desired model output. A similar process was also used when looking at free overflow water surface profiles and pressure data; however, nested mesh block of (0.004 m) used in this study. A nested mesh block was defined in FLOW 3D as a mesh block that has a smaller mesh size and that lies completely within the boundaries of a surrounding mesh block. The use of this technique would allow the model to more effectively capture the geometry and flow detail over spillway body without overly increasing simulations times (Chanel, 2008).

FLOW 3D mesh generator uses the FAVOR method to handle the complicated geometries in an orthogonal mesh defined in cartesian or cylindrical coordinates. Only the orthogonal mesh was allowed to simplify the process of meshing domain in FLOW 3D as shown in

Figure 1. The obstacles and baffles were embedded in the orthogonal mesh, which allowed separate definition of the mesh and geometry, so the modification of geometry had not any influence on the mesh.

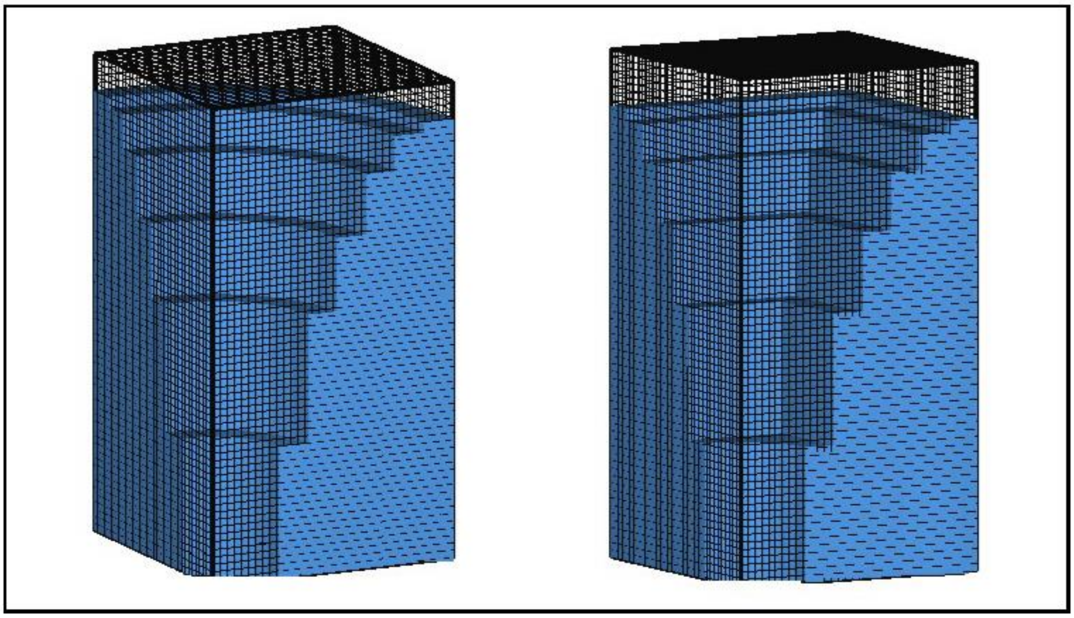

Figure 1: Mesh generations by FAVOR.

3.5. Numerical Approximations:

To construct discrete numerical approximations to the governing equations, control volumes are defined surrounding each dependent variable location. For each control volume, surface fluxes, surface stresses, and body forces can be computed in terms of surrounding variable values. These quantities are then combined to form approximations for the conservation laws expressed by the equations of motion.

Finite Difference Solution Method:

The basic procedure for advancing a solution through one increment in time, δ_t, consists of three steps:

1. Explicit approximations of the momentum equations are used to compute the first guess for new time-level velocities using the initial

conditions or previous time-level values for all advective, pressure, and other accelerations.

2. To satisfy the continuity equation when the implicit option is used, the pressures are iteratively adjusted in each cell and the veloc ity changes induced by each pressure change are added to the velocities computed in Step(1). Iteration is needed because the change in pressure needed in one cell will upset the balance in the six adjacent cells. In explicit calculations, iteration may still be performed within each cell to satisfy the equation-of-state for compressible problems.
3. Finally, when there is a free-surface or fluid interface, it must be updated to give the new fluid configuration. For compressible problems, density and energy must be updated to reflect advective, diffusive, and source processes. Turbulence quantities and wall temperatures are also updated in this step.

Repetition of these steps will advance a solution through any desired time interval. At each step, suitable boundary conditions must be imposed at all mesh, obstacle, and free-boundary surfaces. Details of these steps and boundary conditions are given in the following subsections. A generic form for the finite-difference approximation as in Equations 7, 8 and 9.

$$u_{i,j,k}^{n+1} = u_{i,j,k}^{n} + \delta 1^{n+1}\left[-\frac{P_{i+1,j,k}^{n+1} - P_{i,j,k}^{n+1}}{(\rho\delta x)_{i+\frac{1}{2},j,k}^{n}} + G_x - FUX - FUY - FUZ + VISX - BX - WSX\right] \quad 7$$

$$v_{i,j,k}^{n+1} = v_{i,j,k}^{n} + \delta 1^{n+1}\left[-\frac{P_{i,j+1,k}^{n+1} - P_{i,j,k}^{n+1}}{(\rho\delta y)_{i,j+\frac{1}{2},k}^{n}} R_{i+\frac{1}{2}} + G_y - FVX - FVY - FVZ + VISY - BY - WSY\right] \quad 8$$

$$w_{i,j,k}^{n+1} = w_{i,j,k}^{n} + \delta 1^{n+1}\left[-\frac{P_{i,k,j+1}^{n+1} - P_{i,j,k}^{n+1}}{(\rho\delta y)_{i,j,k+\frac{1}{2}}^{n}} + G_y - FWX - FWY - FWZ + VISZ - BZ - WSZ\right] \quad 9$$

Where: The advective, viscous, and acceleration terms have an obvious meaning, e.g., FUX means the advective flux of u in the x-direction; VISX is the x-component viscous acceleration; BX is the flow loss for a baffle normal to the x-direction; WSX is the viscous wall acceleration in the x-direction; and GX includes gravitational, rotational, and general non-inertial accelerations.

The simplest FLOW -3D finite-difference approximation is first-order accurate in both space and time increments. In this case, the advective and viscous terms are all evaluated using old-time level (n) values for velocities. Wall shear stresses are implicitly evaluated as described below Figure 2. Because the pressures at time level n+1 are generally unknown at the beginning of the cycle, these equations cannot be used directly to evaluate the n +1 level velocities but must be combined with the continuity equation. In the first step of a solution, the pn+1 values in these equations are replaced by pn values to get a first guess for the new velocities. In an explicit approximation the pressure gradient is evaluated at time n, therefore further adjustment to p does not influence the evaluation of u^{n+1}.

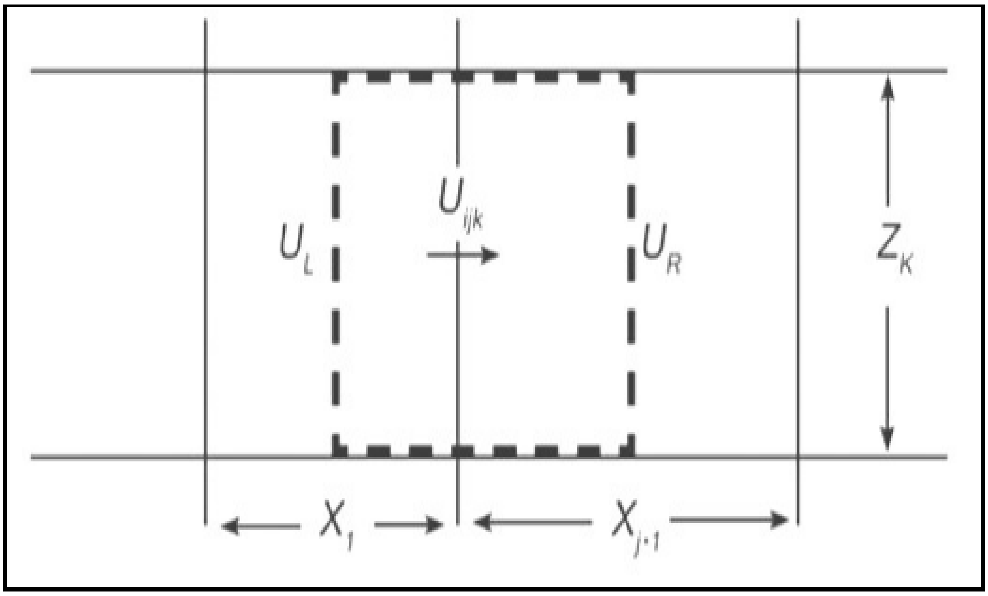

Figure 2: Description of wall shear stresses.

3.6. Numerical Model Set-Up:

The general model set-up for all spillway simulations that were conducted was quite similar. In each case, the global tab was specified with one fluid, incompressible flow, and a free surface or sharp interface being selected. Also, the fluid properties were specified as those for water at 20 degrees Celsius for all simulations. Several other model parameters remained generally constant as well, and will be further discussed in the following sections.

3.6.1. Physics:

Although there are many different physics options available, activation of only two selections was required to obtain accurate simulations of the data that was desired in this study. The gravity option was activated with gravitational acceleration in the vertical or z-direction being set to negative 9.81 m/sec2. The viscosity and turbulence option was also activated with Newtonian viscosity being applied to the flow along with the selection of an appropriate turbulence model. Once the FLOW 3D model was completely prepared, one turbulence model applied in this study, as long as the 2-equation (k-ε) model was selected. The made decision based on comments in FLOW 3D user's manual (2007) that the (k-ε) turbulence model is the most accurate and strong model available in the software for spillways simulations.

3.6.2. Geometry:

Preparation of the numerical model geometry was somewhat different for other spillways that were modeled earlier by FLOW 3D. Depending on the information that was available from the experimental study, the geometry used in the simulations was provided as a stereo lithographic (STL) image drawn in Auto CAD and exported in STL format.

The STL images then directly imported into FLOW 3D where the appropriate mesh can be generated. The spillway geometry of SMGS is shown in Figure 3.

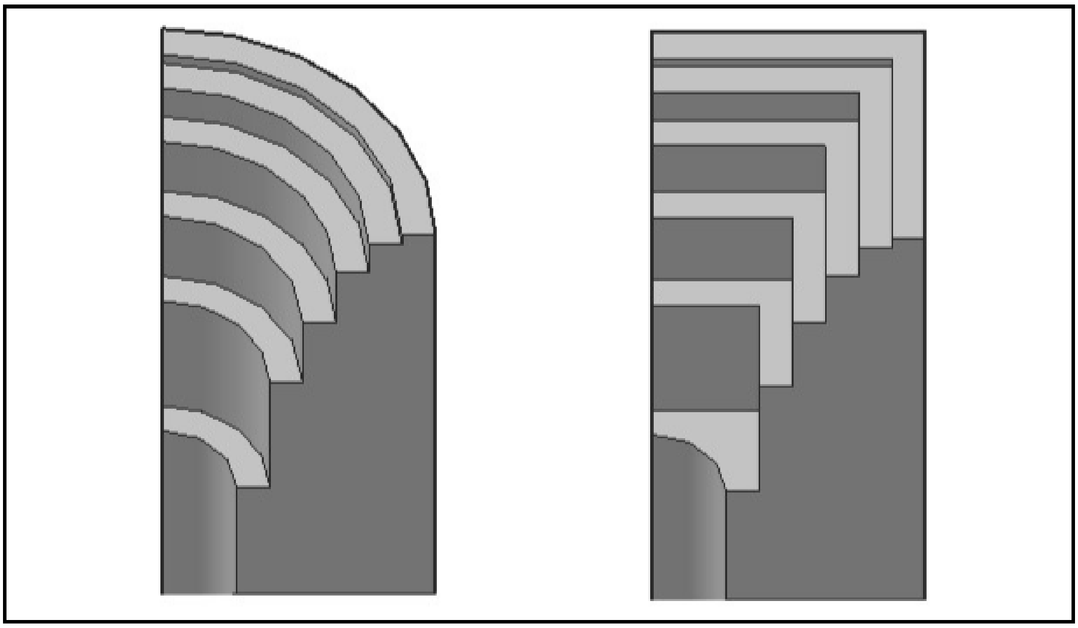

Figure 3: Quarter part of SMGS model imported to FLOW 3 D.

Another geometry option that remained constant for all spillway models completed as a part of this study, the typical concrete roughness value for all SMGS geometry was considered to be neglected and the component of geometry remained at standard option.

3.6.3. Boundary and Initial Conditions Pressure and Outflow Boundary Conditions:

The ability to specify a pressure condition at one or more boundaries of a computational region is an important and useful computational to ol. Pressure boundaries represent such things as confined reservoirs of flui d, ambient laboratory conditions, and applied pressures arising from mechani cal devices. There are typically two ty pes of pressure conditions, referred to as static or stagnation pressure conditio ns. In a static condition, the pressure is more or less continuous across the boundary and

the velocity at the boundary is assigned a value based on a zero normal-derivative condition across the boundary. Better outflow boundary conditions exist for some classes of flow problems. For example, for wave propagation problems, special boundary treatments have been devised that try to determine the speed and direction of waves approaching the boundary and then set boundary conditions in such a way as to allow their continuation through the boundary with a minimum of reflection. An early and useful example of this type of treatment, sometime called a radiation boundary condition, is described by (Orlanski, 1976).

The outflow boundary condition allows users to numerically investigate the effects of wave interactions with structures. This capability permits a reduction in the extent of the computing mesh needed for accurate computations.

Turbulence Model Boundary Conditions:

At all boundaries, except for rigid no-slip boundaries, the turbulence energy and dissipation functions are treated as analogous to other cell-centered quantities such as density. At symmetry boundaries no special conditions are needed, as there are zero velocity derivatives across the boundary, and hence zero turbulence production. Also, there is a zero flow area that automatically ensures no advective or diffusive fluxes. At rigid, no-slip boundaries, special considerations are necessary because numerical resolution is usually too coarse to resolve details of a laminar boundary layer region. The wall shear-stress model based on a turbulent (power law) velocity profile has already been discussed. To be consistent with this, it is necessary to define wall boundary values for the turbulence energy and turbulence dissipation functions.

Within the context of the FAVOR method, in which rigid walls may cut at any angle through a mesh cell, it is not obvious how these boundary conditions are to be satisfied. We have elected to use the following procedure. The turbulence energy and dissipation values are set in all cells having one or more faces partially or wholly blocked by a no-slip, rigid boundary. Wall boundary values are deduced from the assumed velocity profile (logarithmic law approximation) and the assumption of a local equilibrium between turbulent production and decay processes.

Free-Surface Boundaries:

The normal stress, i.e., specified pressure, condition at a free surface is satisfied by the pressure setting scheme described in Incompressible SOR Method. (Free-surface boundary conditions only exist in single fluid problems). Tangential stresses at a free-surface are zero because all velocity derivatives that involve velocity components outside the surface are set to zero. Velocities must be set on every cell boundary between a surface cell and an empty cell, however, to correctly account for fluid advection. This is done in two steps. First, every velocity component on a face adjacent to an empty cell is set equal to the value on the opposite face of the surface cell. In the second step, a virtual pressure adjustment is made to the surface cell that attempts to drive the velocity divergence of the cell toward zero. Only velocities on sides open to empty cells are adjusted in this process. The cell's velocity divergence may not actually be driven to zero because the adjustment is also proportional to the fractional amount of fluid in the cell.

In applying the above boundary conditions, care is taken to keep the flow consistent with internal obstacles. Setting the appropriate boundary conditions has a major impact on whether the numerical model results reflected the actual situation one tried to simulate. In this case flow, data

from free surface flows was desired in mesh block (1) and so the top boundary was set as atmospheric pressure while the bottom boundary was specified as symmetry. For the extent of the mesh in the vertical or z-direction, the bottom boundary was set just below the inlet shape of model geometry in order to capture the channel bed, while the top boundary was set just above the highest water elevation.

To make simulation compatible with the experimental work, the variation of head water levels and flow rates was realized by setting the upstream boundary as a specified pressure with fluid height. In these simulations, it was also decided to set the downstream boundary to outflow, although there were several other boundary options available in the software that could be applied to the downstream side such as specified velocity. The mesh in the ups tream of x-direction and y-direction was adjusted to be symmetry option. The initial condition was defined as the fluid region in the upstream boundary and the pressure was defined at top boundary of mesh block (1) as hydrostatic pressure. Figure 4 show the boundar y and initial condition for SMGS model.

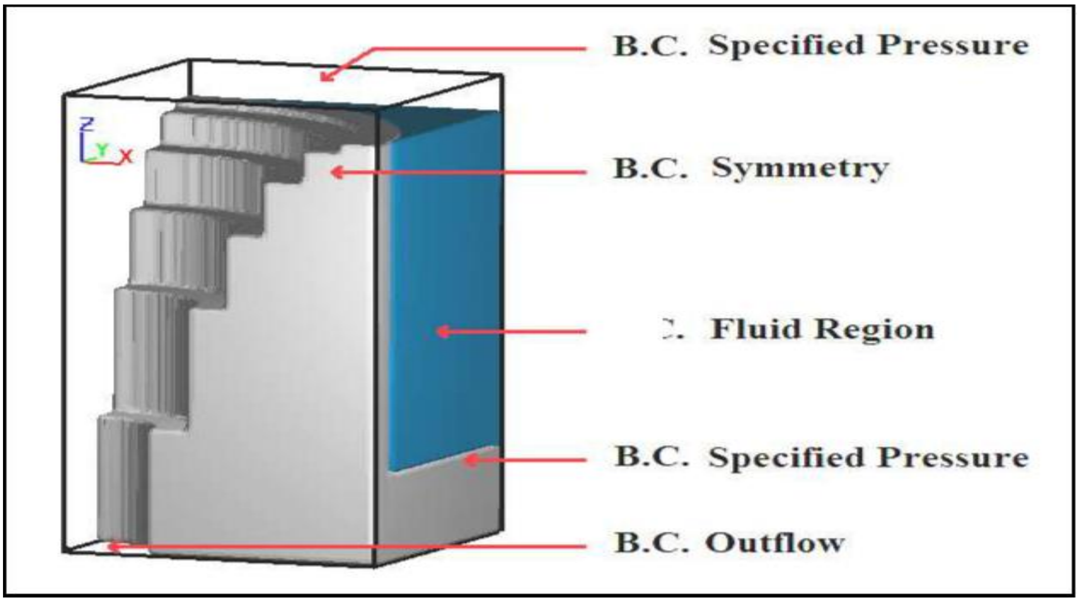

Figure 4: Boundary and initial conditions for SMGS model.

3.6.4. Numerical Simulations Options:

A variety of options are available in the numeric tab of the FLOW 3D model set-up. These options presented modifications to the way the Reynolds-averaged Navier-Stokes (RANS) equations, which are the fundamental underlying equations in FLOW 3D, are solved. In the majority of simulations completed, the default selections were used.

The time step controls were left as default unless the simulation would crash with the provided error message that the time step was smaller than the minimum. In that case, a smaller finish time was sometimes attempted to try and obtain a converging solution. Simulations using the default Generalized Minimum Residual (GMRES) pressure solver options. Simulations were also generally completed using the default explicit solver options, the difference between an explicit and implicit solution is that an explicit solution solved progressively at each computational cell by stepping through time, while the time step is restricted to meet stability criteria. An implicit solution solved in each time step using information from another time step, something that requires more complex iterative or matrix solutions but that does not impose a time step restriction. In the volume of fluids advection section of the Numeric tab, most simulations were run with the selected default automatic button which means the software would automatically select the one-fluid free surface option based on the specifications made in the global tab. Also, all simulations were run while solving both momentum and continuity equations and with first order momentum advection selected according to information found in the FLOW 3D users manual (2007).

3.7. Mesh Sensitivity:

The mesh of FLOW 3D software has a cell of cubic shape and is considered one of the affective factor on the simulation process, Therefore, different cell size was selected to identify the best cell size that satisfy the phenomenon conditions. The flow rating curves showed a comparison between different sizes of cells range (3 to 8) mm as shown in Figure 5.

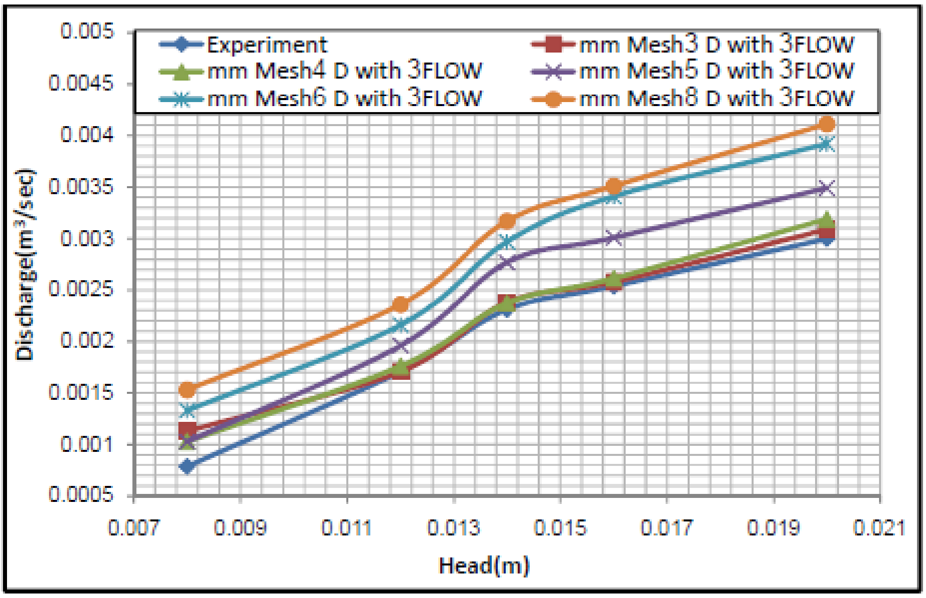

Figure 5: Mesh sensitivity test to cell size.

3.8. Flow 3D Results:

Three Dimensional Outputs:

Figures 6 to 7 show the three dimensional output of FLOW 3D for the studied models. Each Figure shows the water depth, depth averaged velocity and Froude Number distributions along the surface of the inlet structure of SMGS. In three dimensional outputs, iso-surface option selected as fraction of fluid, color variable option selected according to each property output, soled volume option selected as component iso-surface overlay. In all SMGS models, the outputs showed in SI units for

both fluid depth and depth -averaged velocity as (m) and (m/s), respectively.

	Fluid Depth	Depth averaged velocity	Froude Number

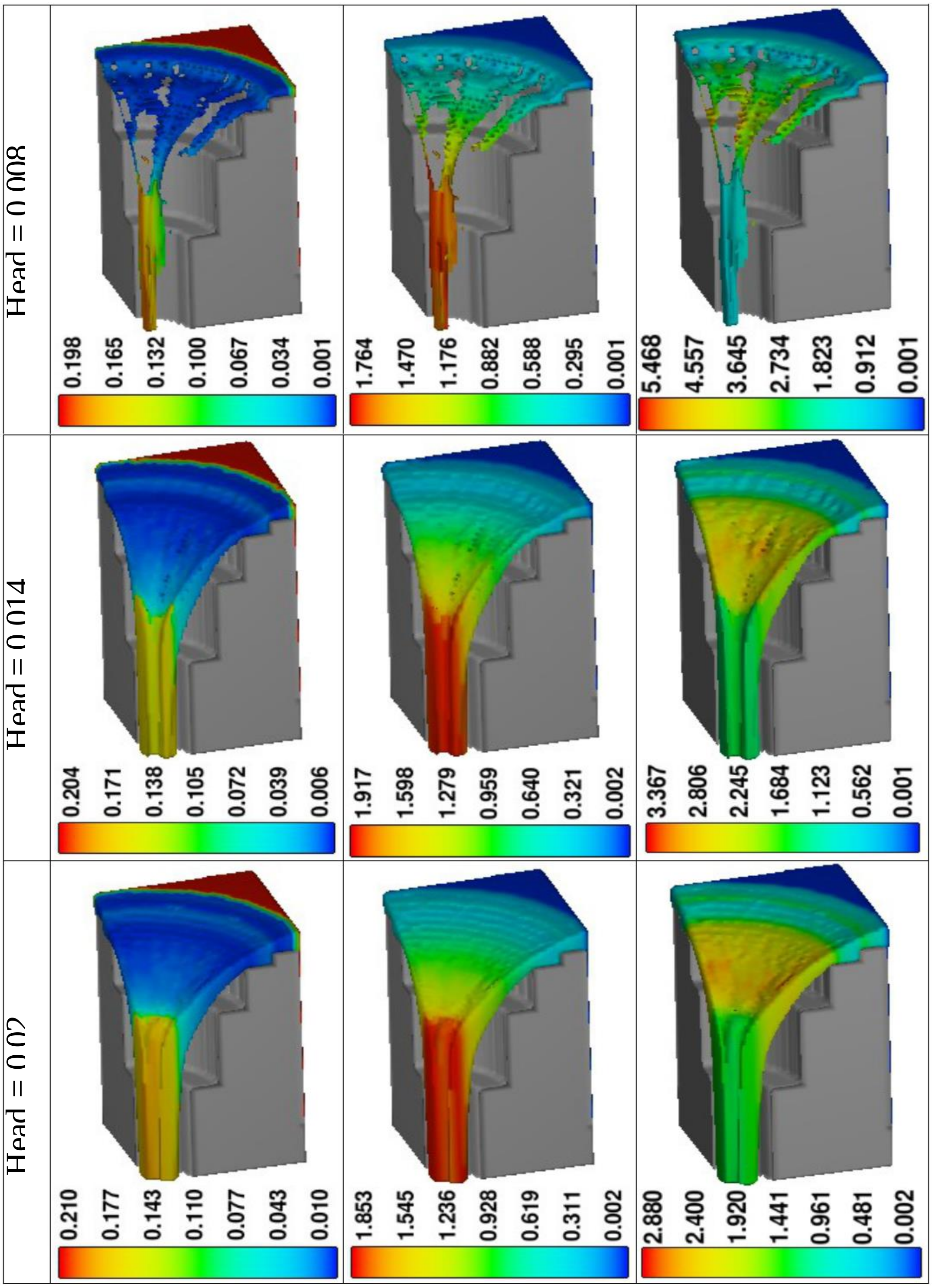

Figure 6: Four steps circular SMGS model results of FLOW 3D simulation.

	Fluid Depth	Depth averaged velocity	Froude Number

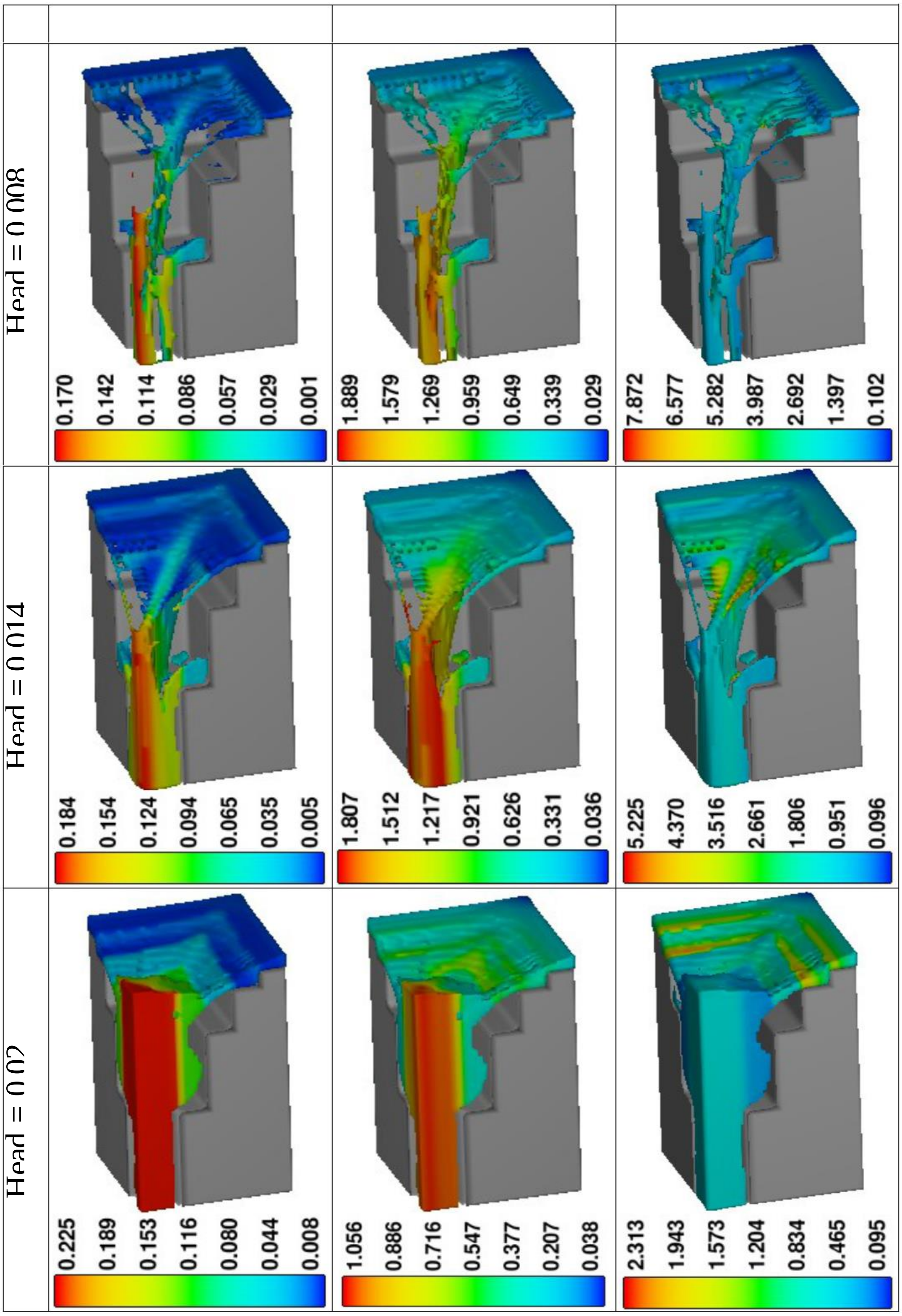

Figure 7: Four steps quadrate SMGS model results of FLOW 3D simulation.