I. TYPES OF DAMS, BASIS FOR THEIR CLASSIFICATION AND DAM SITE INVESTIGATION

1.1. INTRODUCTION

Def: A dam is a barrier constructed across a river or a natural stream to create a reservoir for impounding water (for irrigation, water supply, flood protection), or to facilitate diversion of water from the river, or to retain debris flowing in the river along with water.

The construction of dams ranks the earliest and most fundamental of civil engineering activities. All great civilizations have been identified with the construction of storage reservoirs appropriate to their needs, in the earliest instances to satisfy irrigation demands arising through the development and expansion of irrigated agriculture.

Examples:

1. Dam built at Sadd-el-kafara(Egypt, around ***2600 B.C***.) the oldest known dam
   - Height 14m
   - Construction - Earthfill central core
     - Rock shoulders
     - Rubble masonry face protection
   - Breached - probably due to flood over topping after a relatively short period of service.
2. Marib embankment dam – (Yemen, completed around ***750 B.C***.)
   - Height- 20m
   - Purpose – for Irrigation

   Others were also constructed in Middle and Far East countries

Dams are individually unique structures. Irrespective of size and type they demonstrate great complexity in their load response and in their interactive relationship with site hydrology and geology. In recognition of this and reflecting the relatively intermediate nature of many major design inputs dam engineering is not a stylized and formal science. As practiced it is a highly specialist activity which draws up on many scientific disciplines and balances them with a large element of engineering judgment; dam engineering is a uniquely challenging field of endeavor.

1.2 CLASSIFICATION OF DAMS

1.2.1 Classification based on function (use)

i) ***Storage Dams***

Storage dams are constructed to create a reservoir to store water during the periods when the flow in the river/stream is in excess of the demand, for utilization later on during the period when the demand exceeds the flow in the river/stream.

ii) Detention Dams

Constructed to temporarily detain all or part of flood water of a river and gradually release the stored water at controlled rates so that the entire region on the d/s side of the dam may be safe guarded against the possible damage due to floods. Detention dams are also constructed to trap sediment. Theses are often called debris dams.

iii) Diversion Dams

These small dams are used to raise the river water level in order to feed an off-taking canal and/or some other conveyance systems. They are useful as irrigation development works. A diversion dam is usually called a weir or a barrage.

1.2.2. Classification based on Hydraulic Design

i) Overflow Dams

They are designed to pass the surplus water over their crest. They must be made of materials which will not be eroded by such discharges.E.g. Concrete, masonry etc…

ii) Non-overflow Dams

They are those which are not designed to be overtopped. This type of design extends the choice of materials to include earth fill and rock fill dams.
Many times the types are combined together to form a composite structure.

1.2.3 Classification based on material of construction

i) Rigid dam

It is a dam constructed from rigid materials such as masonry, concrete, etc…Examples are Gravity, arch and buttress dams.

Concrete gravity Dam: Resists the forces exerted up on it by its own weight. Its cross section is approximately triangular in shape.
Arch Dam: Is a curved concrete dam, convex u/s, which resists the forces exerted up on it by arch action. It is structurally more efficient than the gravity or buttress dams, greatly reducing the volume of concrete required.

Buttress dam: It consists of water retaining sloping membrane or deck on the u/s which is supported by a serious of buttresses or counter forts. The sloping membrane is usually R.C.slab. In general the structural behavior of buttress dam is similar to that of gravity dam. It may be considered as a lightened version of gravity dam.

ii) Non Rigid dams

A dam which is constructed from non-rigid materials such as earth, rockfill etc…. are called non-rigid dams. Earthfill and rockfill dams are non-rigid dams. They are usually called embankment dams.

- ❖ Earthfill embankments: An embankment may be categorized as Earthfill dam if compacted soils account for over 50% of the placed volume of material.
- ❖ Rockfill embankments: In rockfill embankment the section includes a discrete impervious element of compacted Earthfill or a slender concrete or bituminous membrane. The designation 'rockfill embankment' is appropriate where over 50% of fill material may be classified as rockfill, i.e. course grained frictional material.

1.3. FACTORS GOVERNING SELECTION OF DAM TYPE

It is rare that for any given site only one type of dam is suitable. It is only in exceptional circumstances that the experienced designer can say that only one type of dam is suitable or most economical. Thus, it would be necessary to prepare preliminary designs and estimates for the several types of dams before one can get the best solution from the point of view of direct costs and all other factors. Some the physical factors which affect the choice of the type of dam are discussed below.

1.3.1. TOPOGRAPHY

Topography dictates the first choice of the type of dam and the most important factor in this respect is the shape of the valley.

i. A narrow V-shaped valley with sound rock in abatements has an arch dam as the first choice. However, for economic arch dam it is preferable to have the top width of the valley less than about four times its height. It is also suitable for rockfill dam.
ii. A narrow or moderately wide U-shaped valley with sound rock foundation is best suited for gravity or buttress dam.
iii. Wide valley with foundation of soil material to a considerable depth (deep over burden) favor Earthfill embankment dam.

1.3.2. GEOLOGY AND FOUNDATION CONDITIONS

The foundations have to carry the weight of the dam. The dam site must be thoroughly surveyed by geologists, so as to detect the thickness of the foundation strata, presence of faults, fissured materials, and their permeability, slop and slip etc…
The common types of foundations encountered are:

i. Solid Rock Foundation

Because of high bearing capacity and resistance to erosion and percolation, any type of dam can be built on such foundations. However, the choice of the type of dam will be governed by economy of materials or overall cost. The removal of disintegrated rock together with the sealing of seams and fractures by grouting will frequently be necessary.

ii. Gravel Foundations (and course sand)

If well compacted such foundations are suitable for earthfill, rockfill and low concrete gravity dams (<15m). As these foundations are frequently subject to water percolation at high rates, special precautions must be taken to provide effective water cut offs or seals.

iii. Silt and Fine Sand Foundations

These foundations suggest the adoption of earth dams or very low gravity dams (up to 8m high), but they are not suitable for rockfill dams. The main problems are settlement, the prevention of piping, excessive percolation losses, and protection of the foundation at the d/s toe from erosion.

iv. Clay Foundations

Clay foundations are can be used to support Earthfill dams after special treatment to consolidate clay. Since there may be considerable settlement, if the clay is unconsolidated and the moisture content is, clay foundations ordinarily are not suitable for the construction of concrete gravity dams, and should not be used for rockfill dams. Tests of the foundation material in its natural state are usually required to determine the consolidation characteristics of the material and its ability to support the supper imposed load.

v. Non Uniform Foundations

At certain places, a uniform foundation of the types described above may not be available. In such a case, a non uniform foundation of rock and soft material may have to be used if the dam is to be built. Such unsatisfactory conditions have to be dealt with by special designs or appropriate foundation treatment.

1.3.3. MATERIALS FOR DAM CONSTRUCTION

Elimination or reduction of transportation expenses for construction materials, particularly those which are used in great quantity, will effect a considerable reduction in the total cost the project. Thus availability of suitable aggregate (i.e. sand and gravel or crushed stone) for concrete is a factor favorable to the construc5tion of concrete dams. On the other hand, if suitable soils are available, the choice may be for an earthfill dam.

1.3.4. SPILL WAY SIZE AND LOCATION

The spillway is a vital appurtenance of dam. Frequently its size, type and natural restrictions in its location will be the controlling factors in the choice of the type of dam. Spillway requirements are dictated primarily by the runoff and stream flow characteristics, independent of site conditions or type of dam. The selection of a specific spillway types will be influenced by the magnitudes of the floods to be bypassed. Thus, it can be seen that, on

streams with large flood potential, the spillway selection of the type of dam could become a secondary consideration.

The cost of constructing a large spill way is frequently a considerable portion of the total cost of the development. In such cases, combining the spillway and dam in to one structure may be desirable, indicating the adoption of a concrete overflow dam.
In certain instances, where excavated material from separate spillway channel may be utilized in dam embankment, an earthfill dam may prove to be advantageous. Small spillway requirements often favor the selection of earthfill or rockfill dams, even in narrow dam sites.

1.3.5 ERATHQUAKE

If the dam lies in area that is subject to earthquake shocks, the design must include provisions for the added loading and increased stresses. Although by including the provisions for the added loading due to earthquake in the design of any type of dam may be adopted in these areas. Earthfill and concrete gravity dams are the best suited types in this respect.

1.4 INVESTIGATION OF DAM SITE

Dam site investigation requires careful planning and a considerable investment of time and resources. Where possible, in situ and field test techniques should be employed to supplement laboratory testing progarmmes. Proper interpretation of geological and geotechnical data demands the closest cooperation between the engineering geologist, the geotechnical specialist and the dam engineer.

Extensive investigations are conducted to confirm that, the site can be developed on the desired scale and at acceptable cost. The nature of soil and rock formations present, critical to foundation integrity must be proved by subsurface exploration. Foundation competence is determined by stability, load carrying capacity, deformability, and effective impermeability. All are assed in relation to the type and size of dam proposed.

In the case of a difficult site, the site evaluation programme can be protracted and expensive. Expenditures may be of the order of 1% up to, exceptionally, 2.5 or 3% of the anticipated cost of the dam. The scope of individual aspects of an investigation reflects circumstances unique to the site.

In parallel with these investigations, extensive and detailed surveys are required to establish the location and extent of potential sources of construction materials in reasonable proximity to the site.

Overall site viability is additionally subject to economic considerations, notably site preparation and construction material costs. It may also be influenced by seismicity, access development cost or other local constraints, including environmental considerations.

Generally site investigation may be broadly classified under three categories, viz

i) Reconnaissance
ii) Preliminary investigation
iii) Final investigation

Reconnaissance: Involves visiting all available sites which have a possibility of being utilized and gathering information which will be useful for planning the detailed surveys and investigations. The information to be collected may include geological data without any kind of subsurface exploration, approximate estimate of stream flow data, storage capacity and head available, etc….

Preliminary Investigation: Sufficiently precise data is collected at several sites selected during reconnaissance to determine the most economical and suitable site among these. Preliminary investigation usually requires the following items.

a) Less precise site survey with the resulting topographic site map
b) Some investigation of the overburden
c) Few borings, say from 6 to 50, according to the size of the dam
d) Preliminary geologic investigation and corresponding report
e) Investigations of construction materials, e.g. earth, gravel, concrete aggregate etc…
f) Determination of public utilities such as road, telephone lines etc… that may be affected by the construction of the dam
g) Hydrologic studies
h) Determination of sediment load of the stream
i) Checking of high water marks for their use in determining spillway capacity requirements.

Final Investigation: One of the several possible dam sites investigated in preliminary investigation is elected for final, precise investigation. Final investigation involves the following items.

a) Sufficiently precise site survey and preparation of topographic maps to serve all purposes of design and construction of the dam
b) Accomplishment of necessary borings, test pits subsurface explorations, geologic studies and tests on the materials in foundation and in the proposed borrow lands.
c) Determination of the type of dam to be constructed
d) Planning for the foundation treatment on the basis of subsurface investigation
e) Determination of the extent of land which would be submerged in the reservoir and the arrangements for rehabilitation of the residents of that area.
f) Obtaining sufficient information for accurate estimate of cost
g) Determination of the final location of the dam, construction equipment, labor and other staff members, probable source of construction materials and all other information needed to the construction Engineer.

It may, however, be mentioned that there is no distinct line of demarcation between the preliminary and the final investigations of dam sites.

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II GRAVITY DAM

A concrete gravity dam is entirely dependent up on its own weight (mass) for stability. The gravity profile is essentially triangular to ensure stability and to avoid over-stressing of the dam or its foundation. A gravity dam is mostly straight in plan and thus known as straight gravity dam. However, in some cases it may be curved in plan (slightly).

In plan the axis of the dam is defined as the horizontal trace of the u/s edge of the top of the dam and it is also called the *BASE LINE OF THE DAM*. In the cross section of dam the vertical line passing through the u/s edge of the top of the dam is considered as the axis of the dam.

The length of the dam is the length measured along the axis of the dam at the top of the dam from one abutment to the other abutment.

The maximum base width of the dam is the horizontal distance the outer points of the heel and the toe of the cross section of the dam.

The maximum height of the dam or structural height of dam is the vertical distance between the lowest point in the foundation and the top of the dam.

2.1. FORCES ACTING ON GRAVITY DAM

The first consideration in designing a dam is the determination of the forces acting on the structure. These forces may be considered as consisting of the following:

I. **PRIMARY FORCES**: These are of major importance to all dams, irrespective of type. They are:

    1. Water pressure
    2. Self weight of dam
    3. Uplift(seepage) pressure

II. **SECONDARY FORCES (or LOADS)**: are universally applicable although of lesser magnitude, or alternatively, are of major importance only to certain types of dams (e.g. thermal effects with in concrete dams). They include:

    1. Sediment (or silt) pressure
    2. Wave pressure
    3. Ice pressure
    4. Wind pressure

III. **EXCEPTIONAL FORCES (or LOADS)**: They have limited general applicability or have a low probability of occurrence. They are:

    1. Earthquake(or seismic) forces

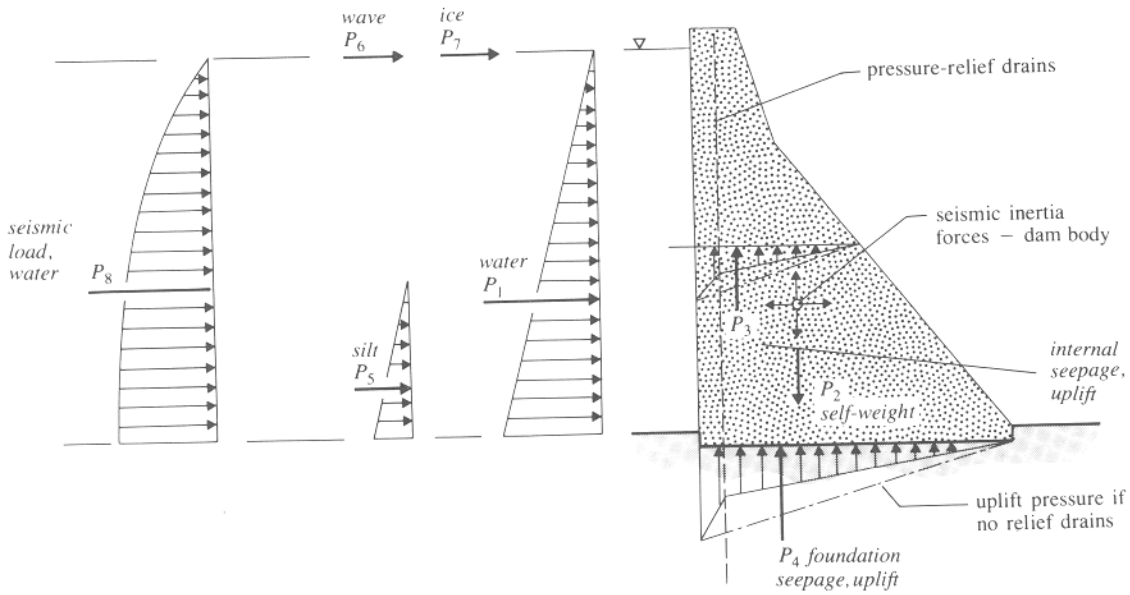


Fig 1 Schematic of principal forces on gravity dams

WATER PRESSURE: Water pressure is the major external force acting on gravity dam. When the u/s face is vertical its intensity is zero at the water surface and equal to γ_wH at the base. The resultant force due to this pressure is

P = $\frac{1}{2}\gamma_w H^2$ and acts at H/3 from the base.

When the u/s face is partly vertical and partly inclined the resultant water force is resolved in to horizontal component P_H, and vertical component P_V.

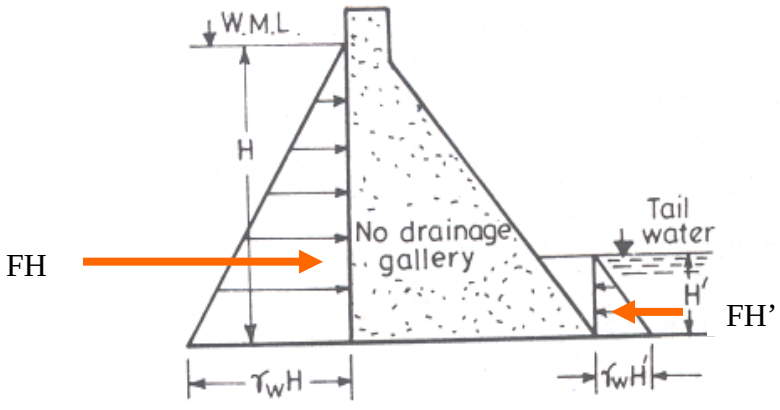


Fig 2 Water pressure on gravity dam

SELF WEIGHT OF DAM: The weight per unit length of the dam is given by the product of the area of cross section of the dam and the unit weight of the construction material, i.e., concrete or stone masonry, and it acts vertically downwards at the centeroid of the cross sectional area, Ap, of the dam profile.

$$Pm = \gamma_c \, Ap \quad [kN/m]$$

γ_c = Unit weight of concrete, assumed as 23.5 KN/m^3

For a gravity dam the weight of the dam is the main stabilizing force, and hence the construction material should be as heavy as possible. Thus in order to get heavier, the course aggregate should have greater specific gravity.

UPLIFT (OR SEEPAGE) FORCE: Is the force exerted by the water penetrating through the pores, cracks and seams with in the body of the dam, at the contact between the dam and its foundation, and with in the foundation.

It acts vertically upwards at any horizontal section of the dam as well as its foundation and hence causes a reduction in the effective weight of the portion of the structure lying above this section.

The computation of uplift pressure involves the consideration of two constituent elements, viz. (i) the area over which the up lift pressure acts and (ii) the intensity of the uplift pressure at various points.

The percentage of area on which the uplift pressure acts is defined as the area factor, η. Several investigations have been made and some of the earlier investigators recommended, for both concrete and rock, a value of area factor ranging from 1/3 to 2/3, i.e. only 1/3 to 2/3 of the area may be considered as effective area over which the uplift pressure acts.

$$A'_h = \eta A_h \quad (A'_h = \text{effective area})$$

However, Terzaghi and Leliavsky, have indicated that, for both concrete and rock, the value of area factor is nearly unity. As such the present practice in dam design is that the up lift pressure is assumed to act over 100% of the area with in the body of the dam and its foundation (i.e., $\eta = 1$).

Uplift pressure can be reduced by forming drains through the concrete of the dam and by drilling drainage holes in to the foundation rock. In modern dams internal up lift is controlled by the provision of vertical relief drains close behind the u/s face. Formed drains raise the full height of the dam from an inspection gallery located as low as practicable in relation to the tail water level.

At the line of drains, in the body of the dam as well as the contact plane between the dam, and its foundation and with in the foundation, uplift pressure is assumed to have an intensity, P_{du}, equal to

$$P_{du} = \gamma H' + \frac{1}{3}[H - H']$$

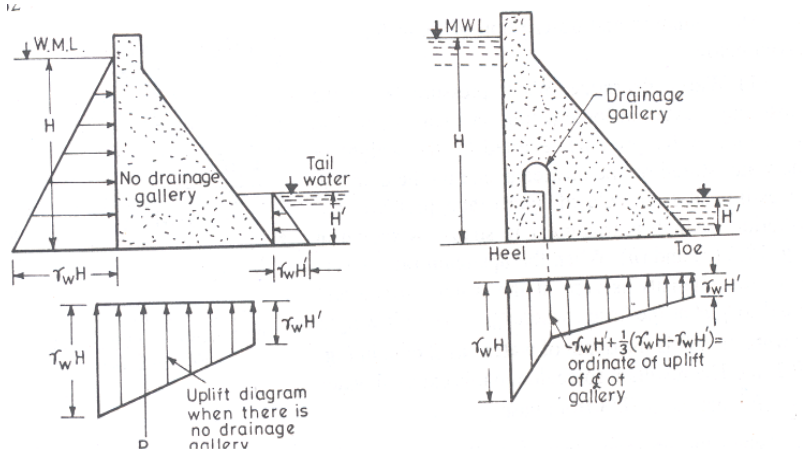


Fig 3 Internal up lift and assumed pressure envelopes

The uplift pressure at the contact of the dam with the foundation may also be reduced by constructing a cut off wall or grout curtain close to the u/s face of the dam and extending it for considerable depth in the foundation. The reduction in the up lift pressure intensity due to the provision of the cut off wall is expressed in terms of intensity factor. The intensity factor is the ratio of the actual intensity of uplift pressure developed when cut off wall is provided to the intensity of up lift pressure, which would be developed with out cut off wall. The values of intensity factor (φ) given in table 1 may be used with judgment. However, in designs of dams theses reduction factor are generally disregarded or are considered to be unity. This is because:

$$\varphi = \frac{P_u \ when \ cutoff \ wall \ is \ provided}{P_u \ when \ cutoff \ wall \ is \ not \ provided}$$

a) It is not possible to determine quantitatively by the effectiveness of the cutoff wall in the reduction of the uplift pressure (by reducing under flow of water through foundation).
b) A cutoff wall is considered to be an additional factor of safety.

Table 1 Uplift intensity factor (φ) Ref. Creager, et al

Height of Dam**	Type of rock foundation	Grouting and Drainage	φ
Moderate	Horizontally stratified	None	1.00
-Do-	Fair, Horizontally stratified	Yes	0.67
High	- Do-	-Do-	0.75

Moderate	Good, Horizontally stratified	-Do-	0.5
High	-Do-	-Do-	0.67
Moderate	Fair, massive	None	0.67
-Do-	-Do-	Yes	0.50
High	-Do-	-Do-	0.67
Moderate	Good, massive	None	0.50
-Do-	-Do-	Yes	0.50*
High	-Do-	-Do-	0.50

**Moderate represents dams up to about 60m
High represents dams above about 60m
* A minimum limit

SEDIMENT (or SILT) PRESSURE: The gradual accumulation of significant deposits of fine sediment notably silts, against the face of the dam generates a resultant horizontal force, P_s. The magnitude of P_s, which is additional to water load P, is a function of the sediment depth h, the submerged unit weight γ'_s and the active lateral pressure coefficient, K_a.

$$P_s = \frac{1}{2} K_a \gamma'_s h^2$$

and acts at h/3 from the bottom of deposit. $\gamma'_s = \gamma_s - \gamma_w$ where γ_s is the sediment saturated unit wt, and $K_a = \frac{1-\sin\phi_s}{1+\sin\phi_s}$ where ϕ_s is the angle of shearing resistance of the sediment 9 angle of internal friction).

Values of γ_s 18-20 Kn/m3 and ϕ_s 30o. Accurate prediction of h is inhibited by major uncertainties 9 function of sediment concentration, reservoir characteristics, river hydrograph etc…), but sediment load is seldom critical in design other than for smaller flood control dams and its introduction is not universal.

WAVE PRESSURE AND HEIGHTS: The upper portions of dams are subject to the impact of waves. The magnitude of the wave pressure mainly depends on the dimensions of waves, which in turn depend on the extent, configuration of the water surface, the velocity of wind and the depth of water in the reservoir. Knowledge of wave height is important if overtopping wave splash is to be avoided. The most significant dimension of wave is the height of the wave in terms of which the wave pressure exerted on the dam can be expressed.

Wave height may be determined (Moliter) by the following formula:

$$h_w = 0.032\sqrt{VF} + 0.763 - 0.271\sqrt[4]{F} \quad for\ F < 32km$$
$$and\ h_w = 0.032\sqrt{VF} \quad for\ F > 32km$$

Where, h_w = Height of the wave from trough to crest, m.

F = the fetch or straight length of water expanse normal the dam axis subject to wind action, km

V = velocity of wind on the water surface, km/h

The maximum pressure intensity occurs at about 1/8 h_w above the still water level and is approximately:

$$P_w = 2.4\,\gamma_w\, h_w$$
$$= 23.544\, h_w \qquad [kN/m^2]$$

The wave pressure diagram is of a curvilinear form which for present purposes may be approximately represented by the triangle 1-2-3 in fig 4.

The total pressure Pw per unit length of the dam is given by the area of the triangle 1-2-3 and given by

$$P_w = \frac{1}{2} \times 2.4 \times \gamma_w h_w \times \frac{5}{3} h_w$$
$$P_w = 2\gamma_w h^2{}_w = 19.62 h^2{}_w \qquad [kN/m]$$

Its center of application is above the still water surface at height $3/8\, h_w$ (or $0.375\, h_w$)

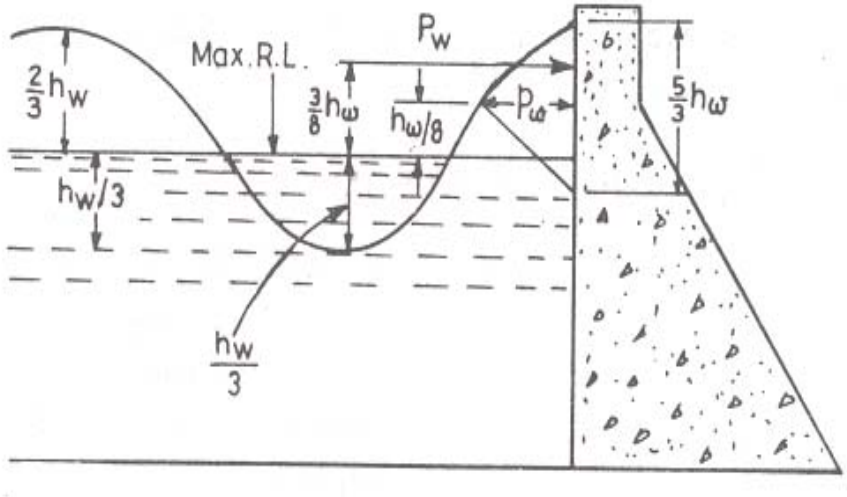


Fig 4 Wave configuration and wave pressure on gravity dam

ICE PRESSURE: Ice load can be introduced in circumstances where ice sheets form to appreciable thickness and persist for lengthy periods. In such situations, ice pressure may generate a considerable horizontal thrust near crest level.

An acceptable initial provision for ice load, P_{ice}, where considered necessary is given by $P_{ice} = 145kN/m^2$ for ice thickness in excess of 0.6m (USBR, 1976). Where ice thicknesses are unlikely to exceed 0.4m and/or will be subject to little restraint, as on sloping face , ice load may be neglected.(Contraction and expansion due to change in temperature causes ice pressure).

WIND PRESSURE: Wind pressure is seldom a factor on the design of dams. Such structures are usually in sheltered locations. Even in exposed locations, the wind has access to only the d/s face of a loaded dam. The maximum possible pressures are small when compared to the loads for which the dam is designed, and it acts against the water load. The superstructure of dams carrying very large sluice gets may need to be proportioned to resist wind loads of 1 kN/m^2 to 1.5 kN/m^2.

EARTHQUAKE (or SEISMIC) FORCES: Dynamic loads generated by seismic disturbances must be considered in the design of concrete dams situated in recognized seismic 'high risk' regions. The possibility of seismic activity should also be considered for dams located outside those regions, particularly where sited in close proximity to potentially active geological faults.

Seismicity is accessed through a specialist review of regional and local geology in conjunction with historical evidence when a risk of seismic activity is confirmed; estimates of probable maximum intensity provide the bases for selecting seismic design parameters.

Seismic activity is associated with complex oscillating patterns of accelerations and ground motions, which generate transient loads due to the inertia of the dam and the retained body of water. Horizontal and vertical accelerations are not equal, the former being of greater intensity. For design purposes both should be considered operative in the sense least favorable to stability of the dam. Horizontal accelerations are therefore assumed to operate normal to the axis of the dam.

Effect of Horizontal Earthquake Acceleration

Due to the horizontal acceleration imparted to the foundation and dam the following two forces act on the dam.

i. Inertia force
ii. Hydrodynamic pressure

(i) Inertia force:

Under reservoir full conditions the most adverse seismic loading will occur when the ground shock is associated with foundation acceleration operating upstream, i.e. form d/s to u/s. This will cause the inertia force to act on the dam in the direction from u/s to d/s. However, for reservoir empty conditions the acceleration in the direction from u/s to d/s would produce the worst combination. In general, the inertia force is equal to the product of weight of the dam and seismic coefficient.

$$P_{em} = \pm \alpha_h gm = \pm \alpha_h W \qquad \text{[kN/m]}$$

And the inertial force is considered to operate through the centeroid of the dam section.

(ii) Hydrodynamic Pressure (Water Reaction)

Due to horizontal acceleration the foundation and dam are accelerated either towards the reservoir or away from it depending on the direction of the acceleration being from u/s to d/s or vice versa. However, this movement of the dam is restricted by the water in the reservoir owing to its inertia, on account of which there is an instantaneous hydrodynamic pressure (or suction) exerted on the dam. The direction of hydrodynamic pressure is opposite to the direction of earthquake acceleration.

For dams with vertical or sloping u/s faces, the hydrodynamic pressure, p_e, in kN/m^2, at any elevation y below the water surface is given by the following equation. (Ref Fig 5)

$$p_e = C\alpha_h \gamma_w h \qquad [Zangar, 1952]$$

Where α_h = horizontal seismic coefficient $= \dfrac{earthquake\ acceleration}{acceleration\ due\ to\ gravity} = \dfrac{a}{g}$

Where

γ_w = unit weight of water, kN/m^3

h = Total depth of reservoir at section of dam considered

C = a dimensionless coefficient which depends on the shape of the dam and depth of reservoir.

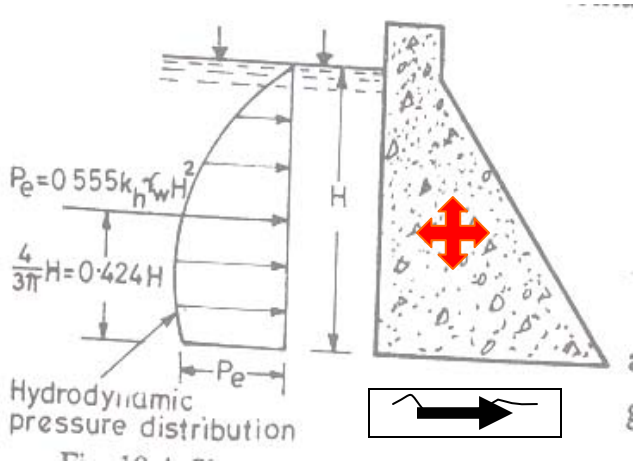


Fig 5 Hydrodynamic pressure and inertia forces

The value of the coefficient C may be obtained for dams with u/s face either vertical or having constant slope for the entire height by the following expression:

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

Where C_m = maximum value of C for a given constant slope of the face of the dam

y = depth of horizontal section under consideration below the water surface in the reservoir, m

Value of C_m may be obtained from a plot of C_m Vs ϕ, where ϕ is the angle (^{o}) that the face of the dam makes with the vertical. However, approximate value of C_m may be calculated from:

$$C_m = 0.735\left(1-\frac{\phi}{90}\right)$$

For dams with u/s face partly vertical and partly sloping the value of C may be obtained as follows: (Ref Fig 6).

a) If the height of the vertical portion of the u/s face of the dam is equal to or greater than half of the total height of the dam, then the entire face is considered as vertical.
b) If the height of the vertical portion of the u/s face of the dam is less than half of the total height of the dam, then the slope of the u/s face is considered to be equal to the slope of the line joining the point of intersection of the u/s face of the dam and the water surface in the reservoir; and the point of intersection of the u/s face of the dam and foundation (i.e. the heel of the dam).

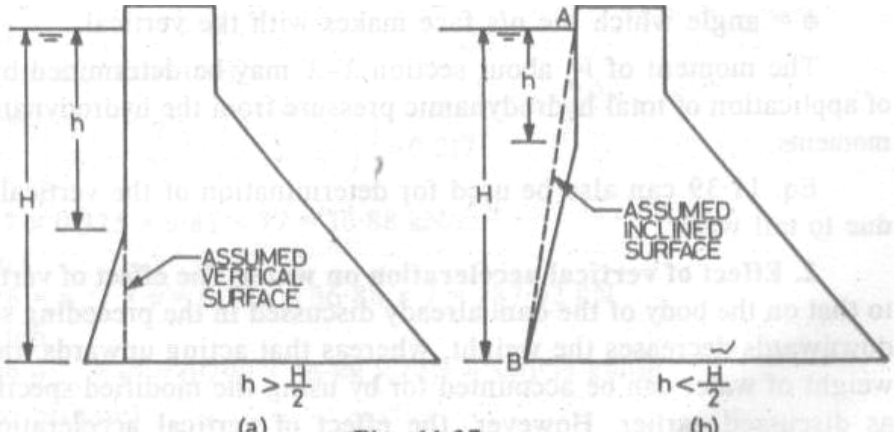


Fig 6

The total hydrodynamic force, P_e, above any elevation y distance below the reservoir surface, and the resulting total overturning moment, M_e, above the elevation are given by:

$$P_e = 0.726\, p_e y$$

$$M_e = 0.299\, p_e y^2$$

The corresponding values of Pe and Me for the dam above its base may be obtained by substituting h for y as

$$P_e = 0.726\, p_e h$$
$$M_e = 0.299\, p_e h^2$$

The hydrodynamic pressure acts normal to the face of dam. As such if the u/s face of the dam is sloping, then it will have horizontal and vertical components. The horizontal component is given by equation:

$$P_e = 0.726\, p_e y$$

The vertical component for part of the dam above a horizontal section x-x at a depth y below the water surface is given by:

$$V_e = (P_{e2} - P_{e1}) \tan\phi$$

Where V_e = Vertical component of total hydrodynamic pressure for a part of dam up to a horizontal section being considered at depth y below the water surface;

P_{e2} = horizontal component of total hydrodynamic pressure for a part of dam up to a horizontal section being considered at depth y below the water surface;

P_{e1} = horizontal component of total hydrodynamic pressure for a part of the dam at a horizontal section at which the slope of the face commences;

ϕ = Angle between face of dam and vertical

Effect of Vertical Earthquake acceleration

Under reservoir full condition the most adverse seismic loading will occur when the ground shock is associated with vertical ***acceleration operating downwards***. Foundation acceleration downwards will effectively reduce the mass, and hence the stability of the structure. The inertia force is exerted on the dam as well as the water in the reservoir in the direction opposite to that of acceleration.

If W is the weight per unit length of the dam and α_v is the vertical seismic coefficient then the inertia force exerted per unit length of the dam is given by:

$$P_{emv} = \pm\alpha_v W$$

Thus for an upward acceleration the inertia force P_{emv} would be acting downwards and hence it would result in an increase in the weight of the dam from W to W ($1+\alpha_v$). On the other hand, if the acceleration is acting downwards, the inertia force P_{emv} would be acting upwards which would result in reduction of the weight of the weight of the dam from W to W ($1-\alpha_v$). Similar expressions may be obtained for water in the reservoir.

It is commonly assumed that $\alpha_h = (1.5 - 2.0)\ \alpha_v$ for the purpose of analysis.

Representative seismic coefficients, α_h, applied in design are listed in table 2.

Table 2 Seismic acceleration coefficients, α_h

Coefficient α_h	*Modified Mercalli Scale*	*General damage Level*
0.0	-	Nil
0.05	VI	Minor
0.10	VII	Moderate
0.15	VIII-IX	Major
0.20		Great

Note that uplift pressure is normally assumed to be unaltered by seismic shock. This is so because the duration of the earthquake is too short to permit the building up of pore pressure in the concrete and rock foundation.

2.2 LOAD COMBINATIONS

A concrete gravity dam should be desired with regard to the most rigorous adverse groupings or combinations, which include transitory loads of remote probability, and therefore have a negligible livelihood of occurrence in service, are not considered a valid basis for design. Such combinations may be investigated when verifying the design of the most important dams, but are generally discounted in the analysis of lesser structures.

The design of gravity dam should be checked for two cases, i.e.,

i. When reservoir is full, and
ii. When reservoir is empty

Case I. Full Reservoir

USBR has classified the 'Normal Load Combinations' and 'Extreme Load Combinations' as below.

a) Normal Load Combinations

1. Normal water surface elevation, ice pressure (if applicable), silt pressure and normal uplift → Taken when ice pressure is serious
2. Normal water surface elevation, earthquake force, silt pressure and normal up lift
3. Maximum water surface elevation, silt pressure and normal uplift pressure

b) Extreme load combination

Maximum flood water elevation, silt pressure, and extreme uplift (with no drains) in operation to release to uplift.

Case II Empty Reservoir

1. Empty Reservoir with out earthquake has to be analyzed to determine bending moment diagrams, etc; for reinforcement design, for grouting studies or other purposes.
2. Empty reservoir with horizontal earthquake force towards the u/s has to be checked or non development of tension at toe.

2.3. STABILITY REQUREMENTS OF GRAVITY DAM

A concrete gravity dam must be designed to resist, with ample factor of safety. There are three tendencies of destruction.

1. **Rotation or overturning**
2. **Translation or sliding**
3. **Overstressing or material failure**

OVERTURNING STABILITY

The overturning of the dam section takes place, if tension is ignored; when the resultant force at any section cuts the base of the dam d/s of the toe. The factor of safety with respect to overturning can be expressed in terms of the moments about the d/s toe of any horizontal plane. The factor of safety against overturning is defined as the ratio of the summation of all restoring forces/moments (i.e. positive) to the summation of all overturning moments (i.e. negative).

$$F.S. = \frac{\sum \text{Re}\, storing\ moments}{\sum Overturning\ moments} = \frac{\sum M_R}{\sum M_O}$$

Values of F.S. in excess of 1.25 may generally be regarded as acceptable, but F.S. should not be less than 1.5 (i.e. F.S.$\geq$ 1.5 is desirable.)

SLIDING STABILITY

A dam will fail in sliding at its base, or any other level, if the horizontal force causing sliding are more than the resistance available to it at that level. The resistance against sliding may be due to friction alone, or due to friction and shear strength of the joint. The planes of weaknesses are the necessary horizontal construction joints, including the joint at the base. The shearing and frictional resistance of the joint must be sufficient to with stand the tendency to slide.

If the shear strength is not taken in to account, the factor of safety is known as *factor of safety against sliding. If* $\sum H$ is the summation of all the horizontal forces causing the sliding and $\sum V$ are sum of the all vertical forces, factor of safety against sliding is given by:

$$F.S. = \frac{\mu \sum V}{\sum H} \text{ or } F.S. = \frac{f \sum V}{\sum H}$$

If f represents the coefficient of static friction of the material above and below the joint, $f \sum V$ will be frictional resistance to sliding. For equilibrium, $f \sum V \geq \sum H$ or $\frac{\sum H}{\sum V} = \tan\theta \leq f$ where θ is the angle between the vertical and the resultant. Values of f for masonry on masonry and masonry on rock foundation varies b/n 0.6 and 0.75; F.S. >1.

It is considered that a low gravity dam should be safe against sliding considering friction alone. However in large dams, shear strength of the joints should also be considered for an economical design. The factor of safety in this case is, therefore, known as Shear Friction Factor (S.F.F.) and is defined by the equation

$$F.S.S. = \frac{f \sum V + rsA}{\sum H}$$

Where s = unit shearing strength of the material

r = an averaging factor = ratio of the average to the maximum shearing stress on the joint ≈ 0.5.

A = Area of the joint = 1XB

Shear strength s may be determined by tests. It is necessary to known the shear strength of both the foundation and the concrete, smaller value being used. The factor of safety against sliding when friction alone considered is relatively small. Low values are permissible because of the added safety due to the neglected shearing strength. When shear is included, S.F.F should approach 4 or 5. According to USBR recommendation minimum S.F.F should be greater than 5 during the most sever conditions of reservoir load combined with maximum horizontal and vertical earthquake accelerations.

OVERSTRESSING

A dam may fail if any of its part is overstressed and hence the stresses in the dam should be with in the specified limit (allowable working stresses) for the dam body and in the foundation.

The stresses at any point at the base of the dam or with in the dam body can be obtained from the following equation.

$$\sigma_{max/\,min} = \frac{\sum v}{B}\left[1 \pm \frac{6e}{B}\right]$$

- Positive sign is used to calculate normal stress at the toe.
- Negative sign is used for calculating normal stress at the heel.

If σ_{max} exceeds the allowable compressive stress of dam material (for concrete 30 kg/cm^2) the dam may crush and fail by crushing. For reservoir full condition, maximum compressive stress (σ_{max}) is produced at the toe.

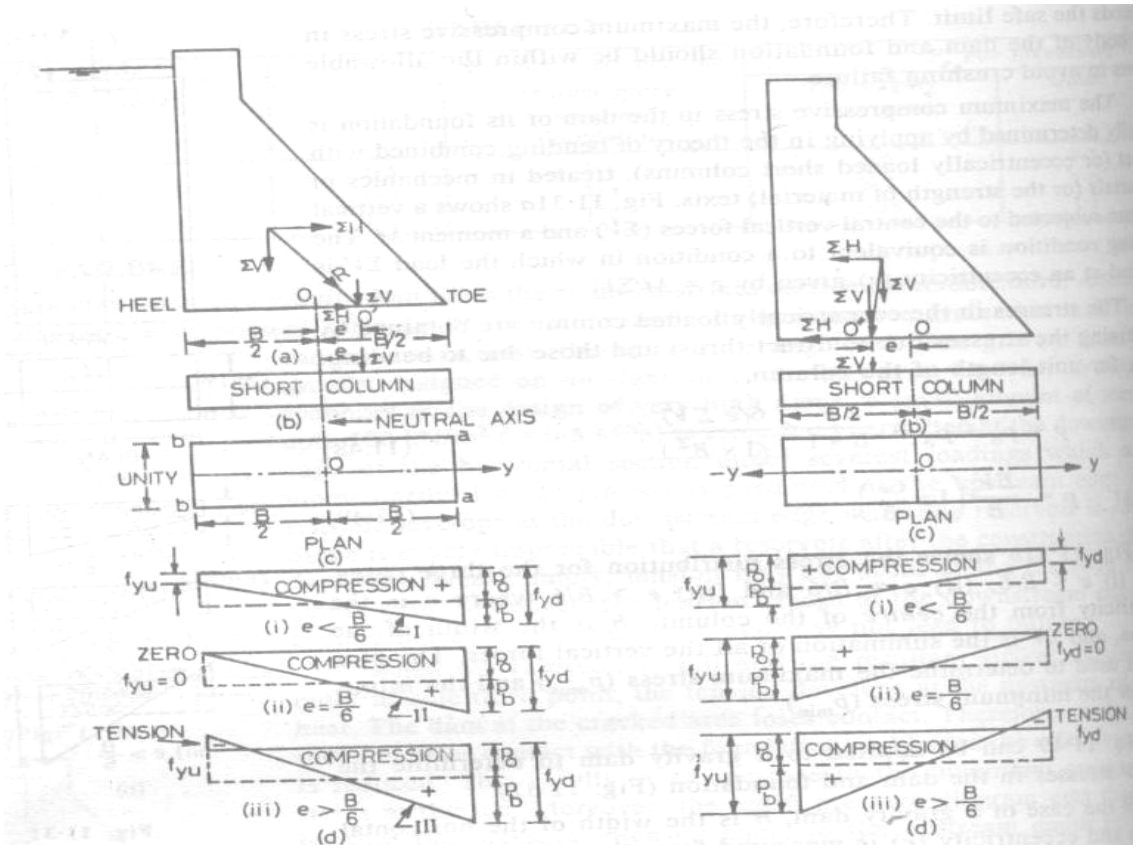


Fig 7 Normal stress distribution at the base

Evidently, the maximum compressive stress occurs at the toe and for safety this should not be grater than allowable compressive stress (σ_{all}) all the foundation material; i.e.

$$\frac{\sum v}{B}\left[1 \pm \frac{6e}{B}\right] \le \sigma_{all}$$

Concrete gravity dams are usually designed in such a way that no tension is developed anywhere; since concrete cannot withstand sustained tensile stresses.

In order to ensure that no tension is developed anywhere, we must ensure that $\sigma_{\min}$ is at most equal to zero.

$$\sigma_{\max/\min} = \frac{\sum v}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$\sigma_{\min} = \frac{\sum v}{B}\left[1 \pm \frac{6e}{B}\right] = 0$$

$$\left[1 - \frac{6e}{B}\right] = 0 \Rightarrow e = \frac{B}{6}$$

Hence, the maximum value of eccentricity that can be permitted on either side of the center is equal to B/6; which leads to the famous statement:

'THE RESULTANT MUST LIE WITH IN THE MIDDILE THIRD'_
the middle third rule.

PRINCIPAL and SHEAR STRESSES: The vertical stresses $\sigma_{\max/\min}$ determined above are not maximum direct stresses produced anywhere in the dam. The maximum normal stress will be the major principal stress that will be generated on the major principal plane. Consider a small element near the toe of the dam.

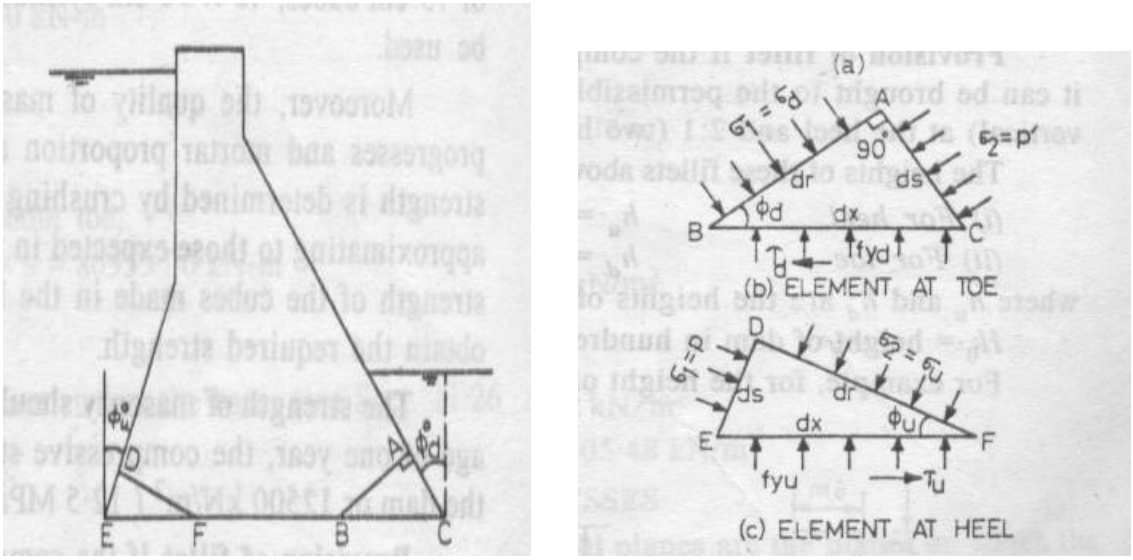


Fig 8 Principal stresses at the toe and heel of gravity dam.

The stress intensities are assumed to be uniform on the faces of the element.

Since the tail water pressure σ_n acts normal to the face and there is no shear stress, BA is a principal plane. Since principal planes are at right angles to each other BC is perpendicular to AB.

Resolving all forces in the vertical direction, we get

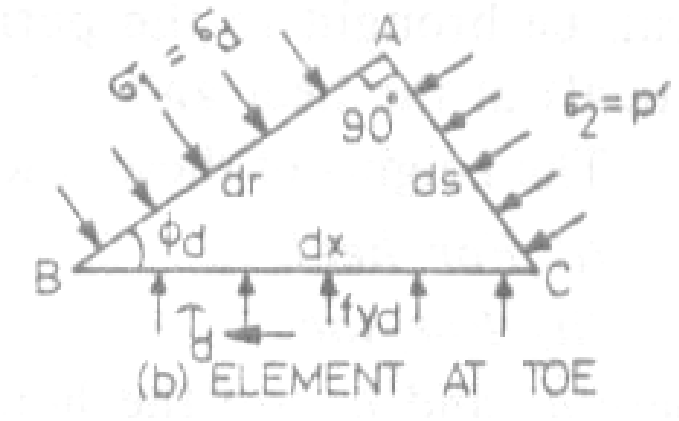


Fig 9 Enlarged view of Element ABC

$$\sum F_v = \sigma_n.ds.\sin\phi + dr.\sigma_1\cos\phi = \sigma_z db$$

Also

$$\frac{ds}{db} = \sin\phi \Rightarrow ds = db.\sin\phi$$

$$\frac{dr}{db} = \cos\phi \Rightarrow dr = db.\cos\phi$$

$$\therefore \quad \sigma_n.(db.\sin\phi).\sin\phi + (db.\cos\phi).\sigma_1\cos\phi = \sigma_z db$$

$$\sigma_n.\sin^2\phi + \sigma_1\cos^2\phi = \sigma_z$$

$$\sigma_1 = \frac{\sigma_z - \sigma_n.\sin^2\phi}{\cos^2\phi}$$

$$\underline{\underline{\sigma_1 = \left[\frac{\sigma_z \sec^2\phi - \sigma_n.\tan^2\phi}{1}\right]}}$$

For σ_1 to be maximum, σ_n should be zero, i.e. no tail water and

$$\sigma_1 = \sigma_z \sec^2\phi$$

Since $\sec^2\phi$ is always more than 1 σ_1 will be greater than σ_z

Therefore, this maximum normal stress produced anywhere in the body of the dam must be calculated and compared with the allowable compressive stress.

Similarly the equation at the heel is given by

$$\sigma_1 = \sigma_{at\ heel} = \sigma_z \sec^2 \phi^1 - (P + P_e) \tan^2 \phi^1$$

Where, ϕ^1 is the angle the u/s face makes with the vertical.
P and P_e are water pressure and hydrodynamic pressure respectively

But at the heel, water p[pressure P is always more than σ_1 and hence P will be the major principal stress and σ_1 is the minor principal stress.

A shear stress also acts on plane AC. No shear stress on AB and BC (principal planes). Resolving all forces in the horizontal direction, we get

$$\tau_o = (\sigma_z - \sigma_{n)} \tan \phi$$

Neglecting tail water,

$$\tau_o = \sigma_z \tan \phi$$

2.4. ELEMENTARY PROFILE OF A GRAVITY DAM

The elementary profile of a gravity dam, subjected only to the external water pressure on the u/s side will be a right angled triangle, having zero width at top water level and a base width B at the bottom.

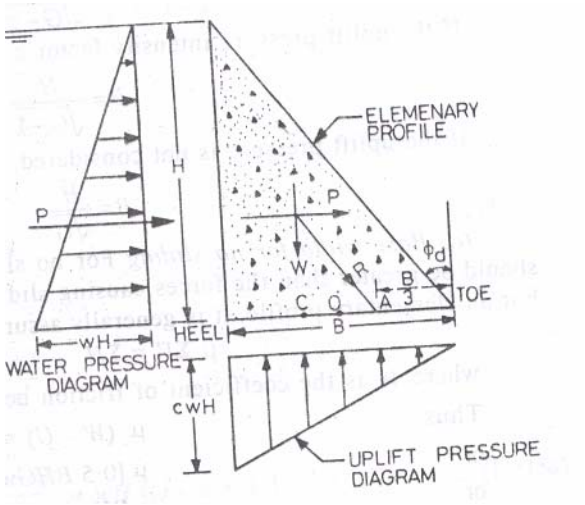


Fig 10 Elementary profile of a gravity dam

For ***reservoir empty condition*** the elementary profile provide the maximum possible stabilizing force against overturning w/o causing tension in the base. This is so

because the weight of the dam (the only force) acts at a distance B/3 from u/s face.

Vertical stress distribution (reservoir empty) at base is

$$\sigma_{max/\,min} = \frac{\sum v}{B}\left[1 \pm \frac{6e}{B}\right] \quad \sum v = W \text{ ; e = B/3*1/2 = B/6 (at u/s middle third point)}$$

$$\sigma_{max} = \frac{W}{B}\left[1 \pm \frac{6e}{B}\right] = \frac{W}{B} \times 2 = \frac{2W}{B} \; at\ heel$$

$$\sigma \min = \frac{W}{B}\left[1 - \frac{6}{B} \times \frac{B}{6}\right] = 0 \; at\ toe$$

Considering the elementary profile of the gravity dam (fig. 10) of height H and base width B;

i. Weight of dam, $W = \frac{1}{2}\gamma_c BH = \frac{1}{2}\gamma SBH = \frac{1}{2}BHS\gamma$

Where S = Specific gravity of dam material

γ = unit weight of water γ_c Unit wt of concrete

ii. Water Pressure, $P = \frac{1}{2}\gamma H^2$

iii. Uplift Pressure, $P_u = \frac{1}{2}\varphi\gamma HB$

φ = Up lift intensity factor

Base Width of Elementary Profile for No Tension

For no tension at any point in the base of the dam, the resultant must pass through the middle third of the base.

Thus for reservoir full condition the outer middle third point A is the limiting position of the point at which the resultant may meet the base for no tension at any point in the base of the dam. Thus taking moments of all forces about A, and equating it to zero, we get;

$$W(B/3) - P(H/3) - Pu(b/3) = 0$$

$$\frac{1}{2}BHS\gamma(B/3) - \frac{1}{2}\gamma H^2(H/3) - \frac{1}{2}\varphi\gamma HB(B/3) = 0$$

$$\frac{1}{6}B^2HS\gamma - \frac{1}{6}\gamma H^3 - \frac{1}{6}\varphi\gamma HB^2 \Rightarrow B^2S - H^2 - \varphi B^2 = 0 \quad \text{……..(a)}$$

$$B2(S-\varphi) = H2 \Rightarrow B = \frac{H}{\sqrt{S-\varphi}}$$

Hence, if B is taken equal to or greater than $\frac{H}{\sqrt{S-\varphi}}$, then no tension will develop at the heel (full reservoir).

When φ = 1 (usually $\varphi \le 1$) ……. Full uplift is considered.

$B = \frac{H}{\sqrt{S-1}}$ is the worst case. But if uplift is not considered, $B = \frac{H}{\sqrt{S}}$

Base width of elementary profile for No sliding

For no sliding to occur, the force resisting sliding must exceed the force causing sliding and in the limiting case these forces must be equal. Assuming sliding is resisted only by friction, and then the force resisting sliding is equal to f(W-P_u) and the force causing sliding is equal to P and in the limiting case we have;

$$f(W - P_u) = P$$

$$f\left[(\frac{1}{2}BHS\gamma) - \frac{1}{2}\varphi\gamma HB\right] = \frac{1}{2}\gamma H^2$$

$$f\left[BS - \varphi B\right] = H \qquad \text{……(b)}$$

$$Bf\left[S-\varphi\right] = H \Rightarrow B = \frac{H}{f(S-\varphi)}$$

Where f coefficient of static friction

Therefore, the minimum base width to be provided for the elementary profile of a gravity dam should be the greater of the base widths given by equations (a) and (b).

Limiting Height of Elementary Profile of Gravity Dam- High and Low Gravity Dams

The normal stress at te toe and heel is given by

$$\sigma_z = \frac{\sum v}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$\sum v = (W - Pu) = \frac{1}{2}(BHS\gamma - \varphi\gamma BH)$$

In the case of elementary profile, $= \frac{1}{2} BH\gamma(S - \varphi)$

$$\therefore \sigma z = \frac{1}{2}\gamma H(S - \varphi)\left[1 \pm \frac{6e}{B}\right]$$

In the limiting case of no tension at any point in the base of the dam, e = B/6, and hence

$$\sigma_z = \frac{1}{2}\gamma H(S - \varphi)[1+1] = \gamma H(S - \varphi)\ at\ toe\ **$$

$$and\ \sigma_z = 0\ at\ heel$$

The principal stress at the toe of the dam is given by

$$\sigma_1 = \sigma_z \sec^2 \phi \text{ (for no tail water)}$$

Substituting the value of σ_z for elementary profile from equation (**), we get;

$$\sigma_1 = \gamma H(S - \varphi)\sec^2 \phi$$

$$but\ for\ elementary\ profile, \sec^2 \phi = \left\{1 + \frac{B^2}{H^2}\right\}$$

Therefore, $\sigma_1 = \gamma H(S - \varphi)\left[1 + \frac{B^2}{H^2}\right]$ ……(c)

From equation (a), $\frac{B}{H} = \frac{1}{\sqrt{S - \varphi}}$

By substituting this value in equation (c), we get

$\sigma_1 = \gamma H(S - \varphi + 1)$ ……..(d)

Equation (d) shows that the value of σ_1 varies only with H as all the other terms are constant for any dam. In order to avoid failure of the dam, due to crushing the value of σ_1 (at toe) should not exceed the allowable working stress f_{all} for the dam material and the limiting case.

$$\sigma_{allaw} = fall = \sigma 1 = \gamma H(S - \varphi + 1)$$

$$From\ which\ the\ height, H, is\ given\ by$$

$$H = \frac{f_{allawable}}{\gamma(S - \varphi + 1)}$$

Thus, this value of H is the maximum height which may be provided for an elementary profile of a dam w/o exceeding the allowable working stress for the dam material.

However, a lower value of H will be obtained when uplift pressure is not considered (i.e. φ =0). Hence to be on the safer side to determine the limiting height of a dam having elementary profile, the uplift pressure is neglected. This height is given by:

$$H = \frac{f_{allawable}}{\gamma(S+1)} \quad \text{.....(e)}$$

If the height of the dam is more than given by equation (e),the maximum compressive stress will exceed the permissible stress. This condition is undesirable.

Equation (e) defines the distinction b/n a low and high gravity dam. Low gravity dam is one in which H is less than that given by equation (e) and maximum compressive stress is not more than the allowable stress.

If the height of the dam to be constructed is more than that given by equation (e), the dam is a high gravity dam. For such a dam, the section has to be given extra slopes at the u/s and d/s sides, below the limiting height to bring the compressive stress with limits.

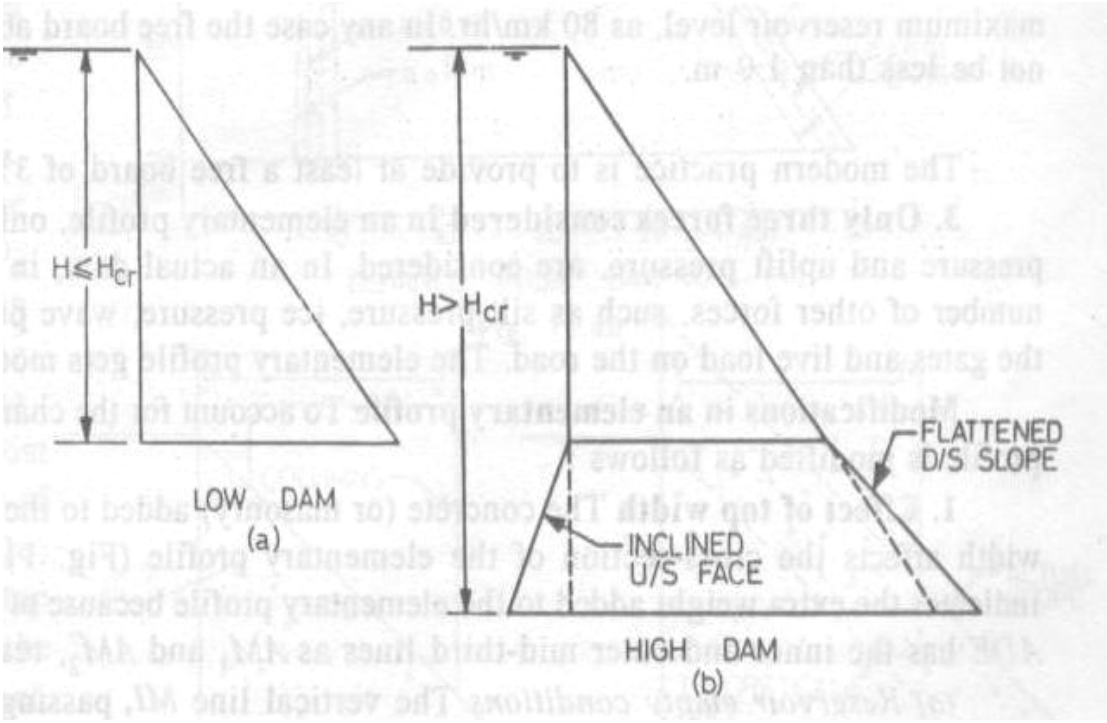


Fig 11 Low and High gravity dams

2.5. PRACTICAL PROFILE OF A GRAVITY DAM

The elementary profile of a gravity dam is only a theoretical profile. Certain changes will gave to be made in this profile in order to cater to the practical needs. These needs are:

i. Providing a straight top width, for road construction over the top of the dam and/or at least to resist the impact action of the floating debris
ii. Providing a free board above the top water surface so that water may not spill over the top of the dam due to wave action, etc…

The addition of these two provisions will cause the resultant force to shift towards the heel. For reservoir empty condition, the resultant shift more towards to the heel, crossing the inner middle third point, and consequently, tension will be developed at the toe. In order to avoid the development of this tension some concrete have to be added on the u/s side of the dam.

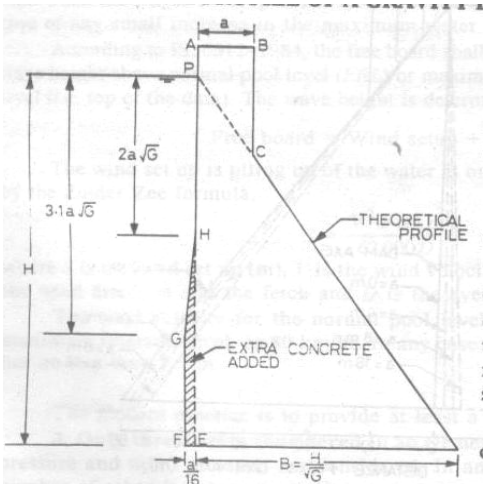


Fig 12 practical profile of a gravity dam

Top Width: The concrete added to provide the top width affects the cross section of the dam.

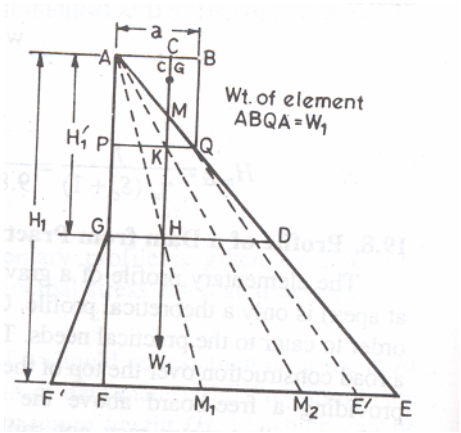


Fig 13 Effect of top width on the profile of a gravity dam

In fig(13a) M_1 and M_2 are the inner and outer third points on the base. Thus AM_1 and AM_2 are the inner and outer third point lines, respectively. Let ADE be concrete added to provide top width with AD = a and MI will be the line passing through the centeroid of the added triangle ADE. This line when produced intersects AM_1 and AM_2 at points G and J respectively.

For ***Reservoir Empty Condition***, due to the additional concrete ADE, the resultant force will intersect at a point to the right of AM1 outside, the middle third for all the sections above plane FGH and on the left of AM1 outside the middle third for all the sections below plane FGH. Hence tension will develop at the d/s face of the dam below plane FGH. Therefore, to avoid this tension, concrete will have to be added at the lower levels on the u/s side of the dam by providing u/s batter FC_1shown in fig (13a).

This would result in increasing the total volume of concrete in the dam.

To find the depth h' of the plane FGH below which u/s batter is required, we have

FG = AM = 2/3 a
FH = 3FG = 3x(2/3)a = 2a

$$B = \frac{H}{\sqrt{S-\varphi}} \;\&\; H = B\sqrt{S-\varphi}$$

But $Hence\ h' = FH\sqrt{S-\varphi}\ \ which\ willresult\ in$

$$h' = 2a\sqrt{S-\varphi}$$

For reservoir full condition, due to the additional concrete ADE, for all sections below plane KJE, the resultant force will intersect at a point on the left of AM2within the middle third (because of added wt, since before ADE is added R was at outer middle third).

However, for the sake of economy the resultant force must intersect at the outer third point line at all sections. As such in order to make the resultant force to intersect at the outer third point at all sections the d/s face of the dam may be shifted from EB to EB1, resulting in the reduction of concrete. Thus, due to the provision of some top width the modified dam section will be ADEB1C1F shown in fig 13b.

Thus an increase in top width, will increases the concrete in the added element and also increase it on u/s face, but shall reduce it on the d/s face. It has been further observed that within limits the concrete added for providing the top width decreases rather than increase the total concrete volume in the dam. Thus it is so b/c the increased volume of concrete in the upper portion of the dam is compensated by reduction in the lower levels.

Hence, the most economical top width is a function of height of dam. Without considering earthquake forces, the most economical top width, has been found by

Creager to be equal to 14% of the dam height. However, for low dams the top width is usually greater than that provided on the bases of economy, b/c of minimum roadway requirement & capacity of top width to withstand shock of floating objects. Its useful value varies b/n 6m and 10 m.

FREE BOARD: Free board is generally provided equal to $3/2h_w$ where h_w is wave height. However modern practice is to provide a maximum free board equal to 3 to 4% of the height of dam, though free board equal to 5% or more might prove economical.

DESIGN OF GRAVITY DAMS

Two of the various methods used in the gravity dams are:-

1. Multiple step method (or zone method)
2. Single step method

MULTIPLE STEP (OR ZONE) METHOD OF DESIGN

In this method the section of the dam is considered to be divided into a number of zones. (The face slopes are altered at suitable interval). Design commences from crest level, and descends through profile stages corresponding to predetermined elevations. Each zone is designed in such away that all requirements of stability are satisfied (i.e. stress levels are maintained with in acceptable limits e.g. no tension under any condition of loading).

Figure 14 below shows a typical dam section with seven zones in a non-overflow gravity dam.

ZONE I: This is the portion above the maximum water surface (1-2-3-4) or if there is ice, it is the one above the bottom of the ice sheet.

If there is no ice, the height of zone-I is controlled by free board requirements and the width is determined by practical consideration or economy for the section as a whole. In case of ice sheets the height of zone I is fixed on the consideration of sliding of the zone due to ice pressure.

ZONE II: For a limited distance below the bottom of zone I, the resultants, reservoir full and empty, lie well with in the middle third (or kern). Both u/s and d/s faces, therefore, may remain vertical until, at some plane 5-6, the resultant, reservoir full, intersects the joint at the exact extremity of the middle third. That portion of the dam b/n the bottom of zone I and the plane 5-6 constitutes zone II.

ZONE III: Below the bottom of zone II, the u/s face continues to be vertical while the d/s face must begin to batter.

The line of the resultant continues to coincide with the d/s extremity of the middle third when the reservoir is full. The resultant, reservoir empty, still being with in the middle

third, the u/s face may remain vertical until at some plane 7-8, where it intersects at the u/s extremity of the middle third. Therefore, zone III is the portion b/n 5-6 and 7-8, determined by tension criteria, reservoir full.

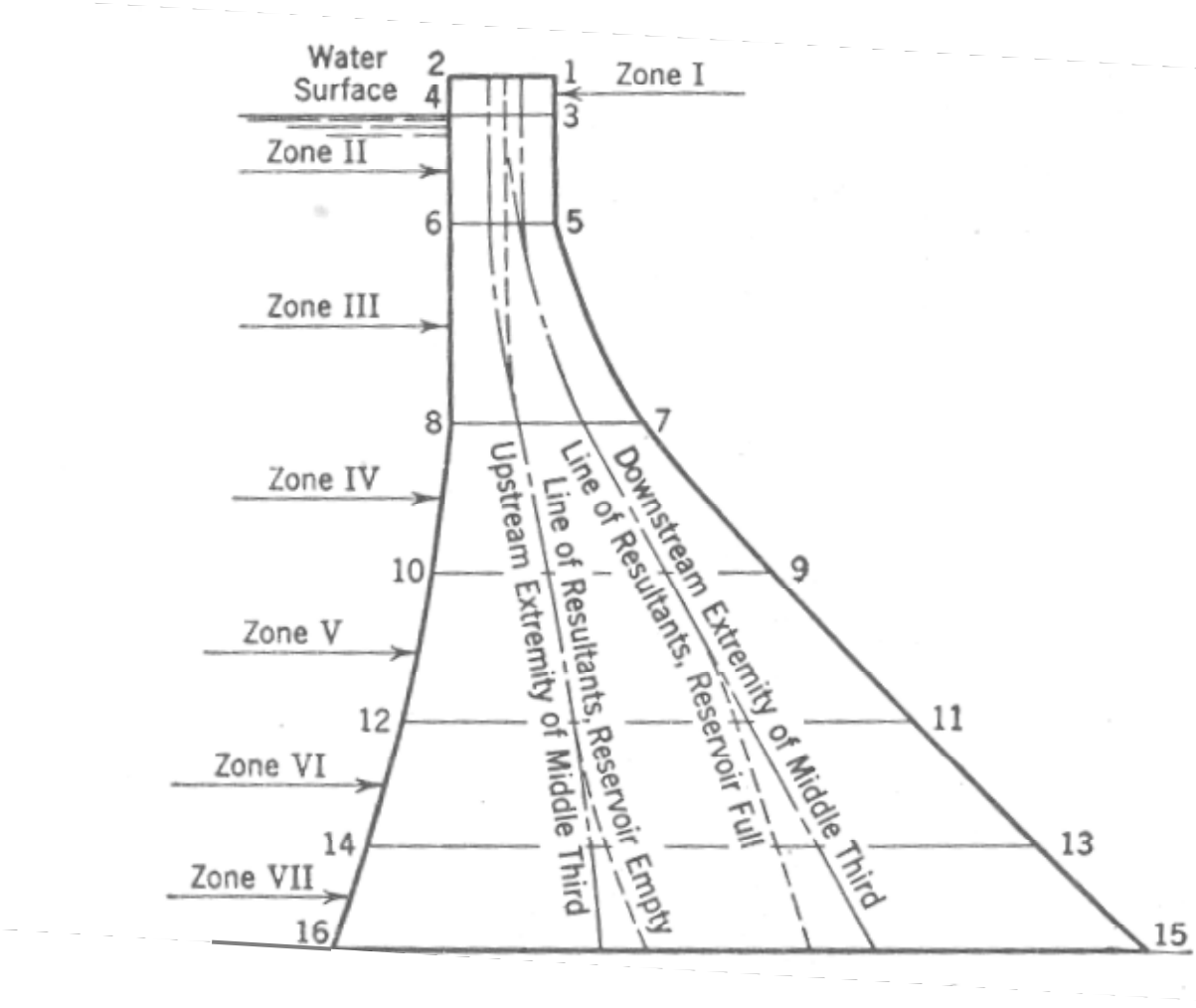


Fig 14 zones for Non-overflow section of Gravity dam

ZONE IV: In this zone, the u/s face also begins to batter so that the lines of the resultants for reservoir empty lie along the corresponding extremities of middle-third. The position of lower limit plane 9-10 of this zone is governed by the criterion that the maximum inclined stress (principal stress) at the d/s toe, for reservoir full condition is just equal to the allowable limit, (which thus fixes the lower limit of zone IV).

The design of zone IV especially the height and d/s and u/s slopes, are determined by trial, by dividing zone IV into a number of convenient blocks till the bottom of zone IV is reached. It should be noted that Low Dams lie with in the limits of zone IV. Zones V, VI and VII are applicable only for high dams.

ZONE V: - Below the plane 9-10 for some distance the inclination of the d/s face of the dam will have to be so adjusted that for reservoir full condition the principal stress at this face does not exceed the max allowable limit. In this case the resultant for reservoir full condition will remain well within the middle third. On the other hand the inclination of the u/s face is so adjusted that for reservoir empty condition the resultant continues to intersect just at the u/s extremity of the middle third. However, at some plane 11-12 for reservoir empty condition the principal stress at the u/s face may reach the maximum allowable limit, which thus fixes the limit of zone V. (B/n 9-10 & 11-12).

ZONE VI: In this zone the conditions of the design are determined by the maximum pressures (principal stresses) at both u/s and d/s faces under reservoir empty and reservoir full conditions, respectively (the inclinations are adjusted that δ_1 should not to exceed the limit). The line of the resultants under both the conditions lies well with in the middle third. The position of the bottom plane 13-14 is reached when the inclined pressure at d/s toe just reaches its maximum value.

ZONE VII: As the height of the dam increases, the batter of both u/s and d/s faces increases. Consequently, at some plane the value of $\sec^2\phi$ (for d/s face) may become so great that the principal stress at the d/s face may exceed the maximum allowable limit. The portion of the dam in which this condition prevails constitutes zone VII. However, this zone should be eliminated by revision of the entire design.

If the height of the dam is so large that it is more than the position of plane 13-14 of zone VI, various changes are made in the upper zones so that the height of the dam lies with in zone VI. If this is not possible, then the height of the dam is reduced or superior material is utilized so that the height is accommodated with in zone VI.

SINGLE STEP METHOD OF DESIGN

For high dams, going beyond zone IV, it is found that the shape of u/s and d/s slopes is sometimes unusual shape. The u/s face has a relatively flat slope while the d/s face has outward convex shape. A convexed face under compression, whether smoothly covered or polygonal, may be subject to tensile stress on surface parallel to the face. Such a shape for the d/s face of the dam is not desirable b/c the outer layer of such a section tends to buckle outward and unless the buckling forces is cancelled by the weight component normal to the face, tension results. The dam section will have to be then redesigned so that such a curvature for the d/s face of the dam is avoided. Alternatively the dam section may be designed by **SINGLE STEP METHOD**.

In the single step method of design a section of the dam is suitably assumed and the entire dam is considered as a single zone.

In the assumed section of the dam the u/s face is kept vertical for some depth to be determined by trial. As a first trial, the vertical portion of the u/s face may be assumed to be extended up to the plane FGH shown in fig 13 (a), below which it is given some slope. The slope given to the u/s and d/s face of the dam are so adjusted that the principal

stresses at the u/s face for reservoir empty condition and at the d/s face for the reservoir full condition reach their maximum values simultaneously. This may be accomplished by trial and error. The dam section is then checked for all stability requirements for reservoir empty and full conditions, which will normally be satisfied at all the points above the base of the dam when the slopes for the u/s and d/s faces are adjusted as indicated above.

Fig 15 shows the sections of a high dam designed both by single-step method and multiple step- method. It is quite clear from the two sections that the multiple-step design is more economical for the upper portion of the dam. The single step method section is under stressed at all the points except at the base. Hence, if the height of the dam is less, multiple step design method would give substantial saving in material.

From the above discussion we draw the following conclusions.

i) Dams of lesser heights can be designed economically only by multiple-step design method.
ii) It may be economical to increase the concrete (or masonry) strength through the use of expensive materials, thus keeping out of zones V and VI by dividing into four zones (for high dams).
iii) High dams beyond zone IV are designed by single step method so that convex curvature of d/s face is avoided.

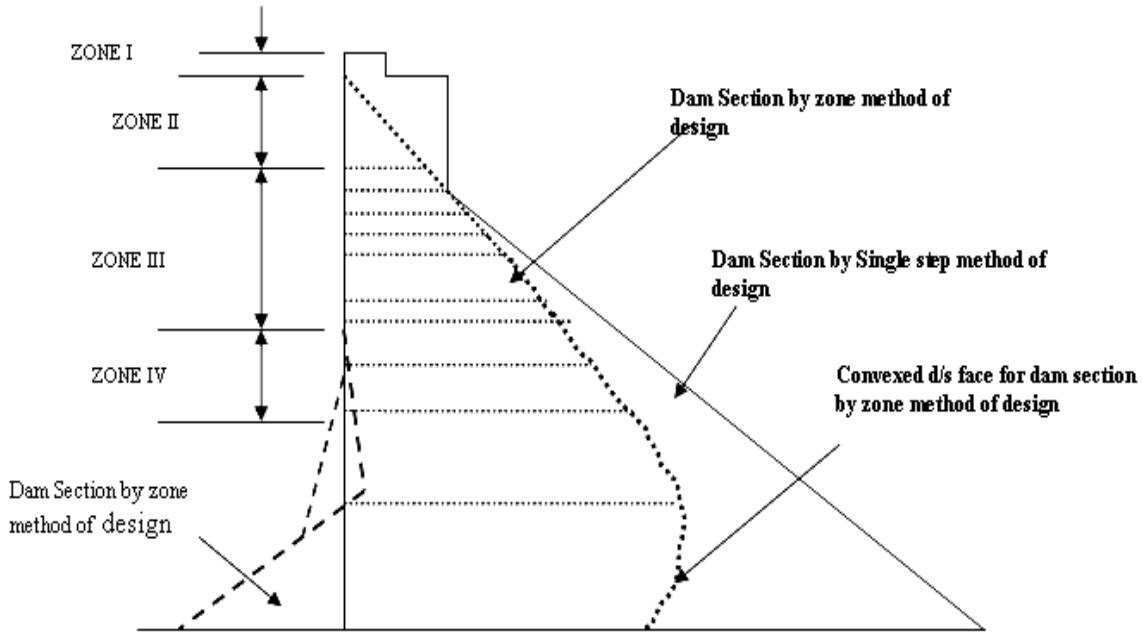


Fig 15 Comparison of zone method (multiple-step) and single step for design of high dams

JOINTS IN GRAVITY DAMS

Depending upon the location and purpose served, joints provided in dams may be classified under two heads:

a) Construction joint (or horizontal joints)
b) Contraction joints.

Construction joints: usually called horizontal joints are necessary since the entire work of concreting the whole dam cannot be completed in one stretch. They are the joints introduced b/n successive lifts. (The lift is the height by which the dam is raised in one continuous operation by pouring concrete (or laying masonry).

For solid gravity dam lifts of about 1.5m is adopted. The concrete of the next lift is placed after sufficient time is allowed for the previously placed concrete to cool and attain its initial set and become hard. As such horizontal joints are provided to allow sufficient cooling b/n the successive lifts of concrete.
Prior to the placement of concrete of the next lift, the surface of the previous placed concrete is thoroughly cleaned by the use of high velocity jet of water and air as well as wet- sand blasting process. Further, immediately before the concrete placing of the next lift begins, a 12.5mm thick layer of mortar should be applied to permit a proper bond between the previously placed concrete and the new concrete. Such treatment of the surface and the use of good concrete create tight horizontal joints for which no provisions to ensure water tightness, such as water stops, keys, etc, are necessary.

Contraction Joints: are mainly provided to avoid cracks caused by shrinkage of concrete due to temperature changes. They are vertical joints. The cracks may be the result of tensile stresses produced when the volumetric changes of concrete is restrained.

Volumetric change results in shrinkage from drying concrete & from temperature variations. The major change, however, results from temperature variation. Surface cracks may develop due to high daily temperature difference b/n the surface and near the surface areas of the dam. Cracks adversely affect water tightness, durability, appearance and internal stresses of concrete.

Contraction joints are of two types:

i) Transverse joint
ii) Longitudinal joints depending upon the plane in which they lie

Transverse Joints: They are provided normal to the axis of the dam. They extend vertically from the foundation to the top of the dam and are continuous from the u/s to the d/s face of the dam, thus divide the dam into separate blocks. They are provided to allow contraction of concrete on the two sides and thus prevent development of transverse cracks in the dam.
The spacing of transverse joints is governed by physical factors of the dam site; details of the structures associated with the dam such as spillway gates, outlets, penstocks,

spillway bridge piers, etc, results of temperature studies and probable plant capacity. For concrete dams these are usually spaced 15 to 20m.

Longitudinal Joints: are provided parallel to the axis of the dam. They are provided to prevent longitudinal cracks.

Longitudinal joints run b/n two adjacent transverse joints and are thus not continuous all along the length of the dam. Spacing of these joints varies b/n 15 and 30m.

The longitudinal joints subdivide each block formed by transverse joints in to several smaller blocks, but since each block must be monolithic block, these joints are provided with horizontal keys over the entire surface of the joint ground. These joints are usually provided in the core of high dams b/c as the height of the dam increases the base width also increases & it approaches limit beyond which vertical cracks parallel to the axis of the dam may develop if longitudinal joints are not provided. Thus these joints serve the same purpose in each block of the dam as the transverse joints serve in the dam as a whole.

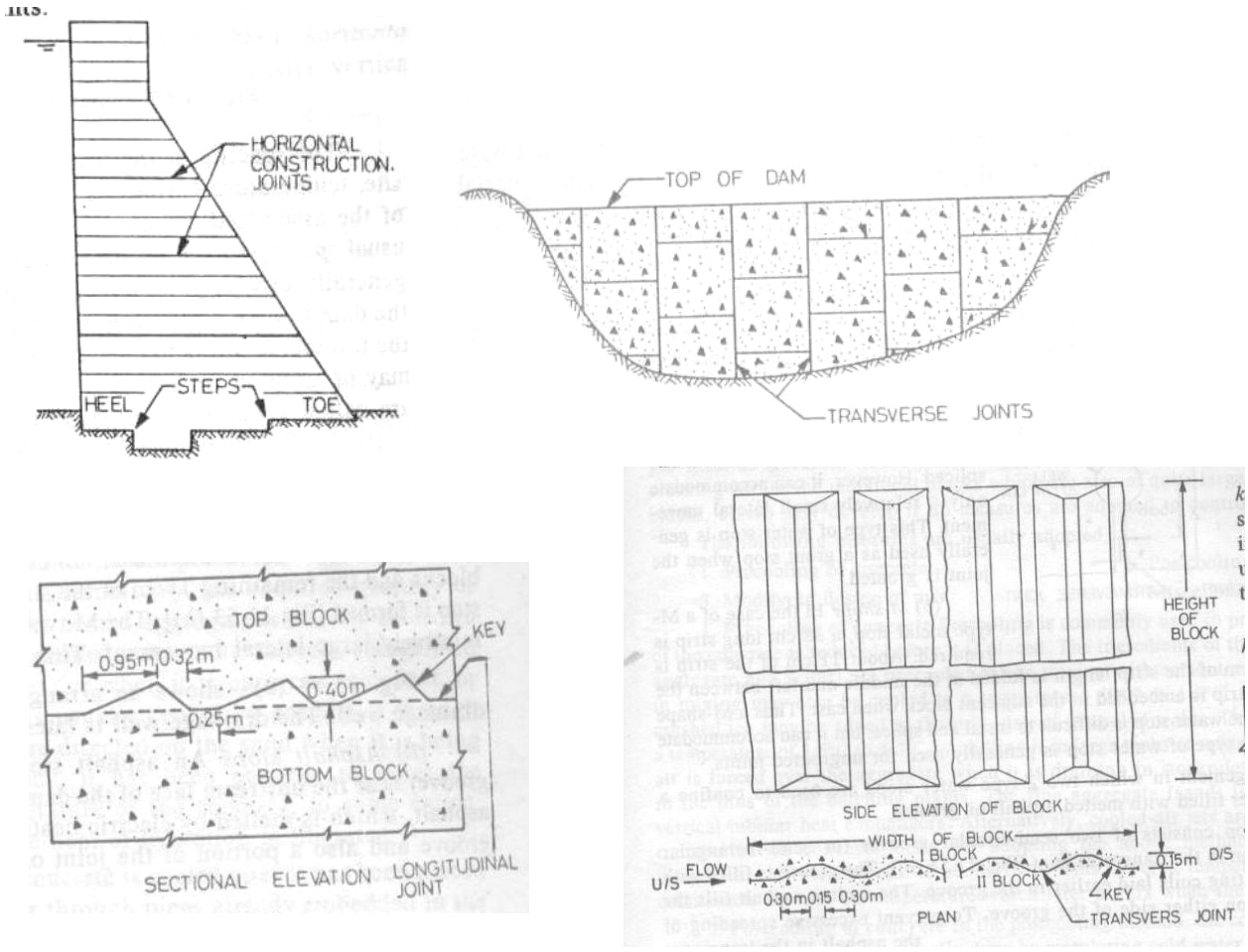


Fig 16 Joints in Gravity Dam

Keys: are the regular projections provided in contraction joints of a dam for transfer of load from one block to the other through shear. In transverse joints vertical keys are provided which are designed to transfer horizontal shear and to assist in minimizing leakage of water through the joints.

In longitudinal joints horizontal keys are provided which are designed to take vertical shear.

Water Stops (or water seals): - Provided across transverse joints, adjacent to the u/s face of the dam, to prevent the leakage of water from the reservoir through the transverse joints if the transverse joints are not grouted.

Water stops can be one of the following three types.

- Metal water stops. Most common types
- Asphalt water stops or
- Rubber/PVC water stops

Openings in Dams: Based on their purpose they are divided in to two: -

i) Water ways
ii) Gallery system

Water ways: - are the openings provided for obtaining water from the reservoir for various purposes such as irrigation, water supply, power generation, etc as well as for allowing the excess water which cannot be stored in the reservoir during floods to over flow to the d/s side. These include outlets, penstocks, spillways etc.

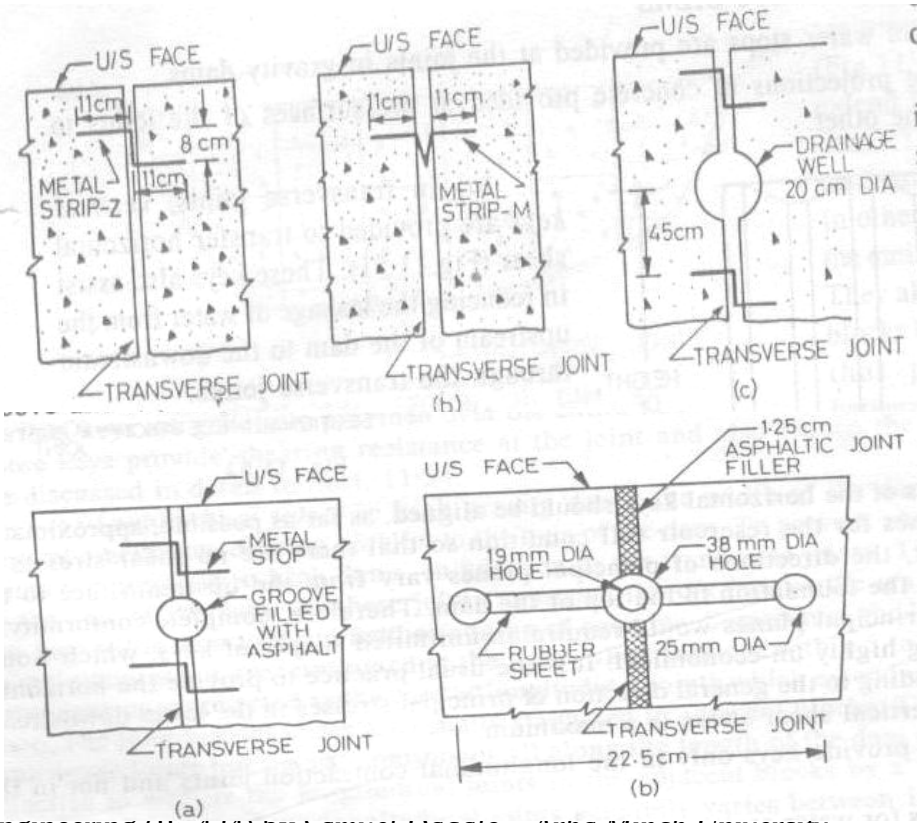


Fig 17 Water stops

GALLERIES

A gallery is an opening provided for obtaining access to the interior of the dam for various purposes. It may run in transverse or longitudinal direction and may run horizontally or on a slope.

Shape: - Usually rectangular with its top and bottom either flat or semi-circular. For flat top and bottom all corners are rounded to reduce stress concentration.

Size: Large enough to provide working space and access for equipment for normal maintenance. Its size in general varies from 1.5m to 1.8m in width and 2.2 to 2.4m in height.

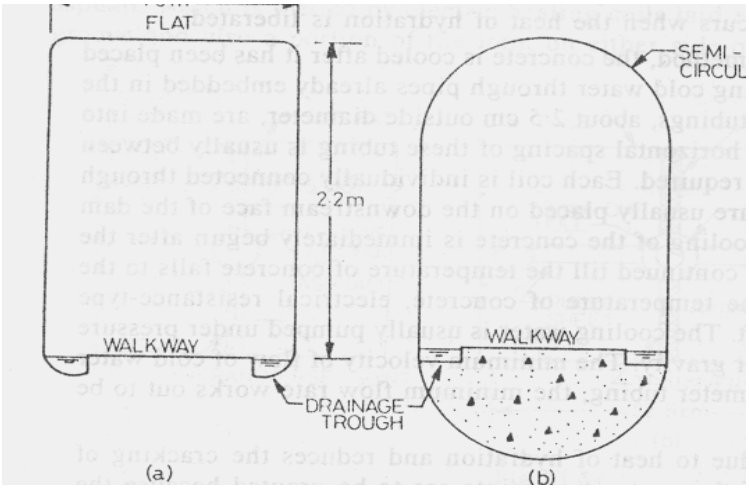


Fig 18 galleries

Purposes of a Gallery

1. To provide drainage of the dam section. Water seeping from u/s face of the dam is drained off through galleries.
2. To provide facilities for drilling and grouting operations for foundation of the dam.
3. To provide space for the piping system and equipment used for grouting of the contraction joints as well as for the post cooling of concrete blocks.
4. To provide access to the interior of the dam for inspection and also for installing various instruments to study structural behavior of the dam after completion.
5. To provide access to, and room for mechanical and electrical equipment required for the operation of gates for outlet conduits, power penstocks and spillway crest.
6. To provide access routes for visitors; to provide access through the dam for control cables and/or power cables.

Depending upon the purposes served the galleries can be classified as follow:

i) Foundation gallery
ii) Drainage gallery
iii) Grouting gallery
iv) Inspection gallery
v) Gate gallery

Foundation Gallery: Provided at the foundation level and follows the configuration of the valley. It extends over the entire length of the dam and it normally runs parallel to the axis of the dam. Always it is provided very close to the u/s face of the dam. Minimum distance from u/s face (for structural safety) should be either 10%of the maximum head water on u/s side or 6m which ever is more. There should be a minimum of 1.5m concrete b/n foundation and floor of the gallery. Its purpose is drainage of water seeping (percolating) through u/s face of the dam. Hence it is also called drainage gallery.

They are also used to drill and grout the holes for grout curtain on the u/s edge of the gallery.

Control of cracking in concrete dams

If proper temperature control is not exercised, the large concrete blocks b/n the joints may crack due to high temperature gradient b/n the interior and the surface. These large blocks of concrete are subject to deep as well as surface cracking. The cracks in the interior of the blocks are produced due to heat of hydration liberated by cement thus giving rise to high temperature gradient. The surface cracks may appear due to daily variations of temperature at the surface. The surface cracks are more harmful, since the disintegration starts through them by wedge action. Water inters these surface cracks, accumulates there and then solidifies; at 4^{o}c it expands resulting in widening and deepening of cracks.

Following methods are employed to check or minimize the development of the cracks in mass concrete.

1. ***Pre-cooling of concrete:*** The concrete is pre-cooled before it is placed in the dam. This is accomplished by cooling the aggregate by refrigerated water, blowing air through them, cooling of sand and using refrigerated water for the manufacture of concrete. The low temperature of concrete counteracts the heat of hydration of the cement, and thus controls the rise in temperature of the concrete. It involves high cost.

2. ***Post Cooling of concrete*****:** Achieved by circulating refrigerated water through pipes embedded in concrete in each lift. The cooling is begun immediately after a block is laid and is continued till the mass temperature falls to the mean annual temperature of the locality. The system consists of pipe or tubing placed in a grid like coils over then top surface of each lift of concrete after the concrete has hardened. Thus coils are formed by joining thin walled metal pipes. They are commonly 25mm in diameter, and their horizontal spacing varies from 0.5m to 2m depending on the extent of cooling required.

The number of coils in a block depends on the size of the block & horizontal spacing of pipes. The velocity of flow is kept > 60cm/sec. Thermometers (resistance type) are embedded in the concrete to ascertain the temperature.

3. ***using Low heat Cement:*** low heat of hydration in concrete

4. ***Using Lower percentage of cement*** in concrete for interior of the blocks, say about 80% of that of the exterior.

5. ***restricting the height of the lift***, say 1.5m

6. ***Allowing considerable time b/n lying of two successive vertical lifts***. The usual time is about 4 days.

7. ***Use of water curing***

Foundation Treatment: - The foundation must be strong enough to withstand the entire load acting up on it.

Defects In foundation Rock: - Geological conditions which constitute defects in dam foundation are primarily features such as bedding planes, unconformity, shale beds, etc… which are developed during the rock formation and secondary features such as faults, joints, cracks, cavities, seams, etc, which result from various stresses, deformation and weathering. Some of these defects are common to all rocks; others are restricted to certain rock types. Thus bedding planes are found in sedimentary rocks which may also have unconformity. Faults, joints, cracks are found in all consolidated rocks; solution cavities are found in limestone and related carbonate rocks; seams and zones of decayed material are found in siliceous, argillaceous and other non-calcareous rocks and flow contracts and flow tunnel in lava.

Methods of Treatment: First surface has to be prepared. This is done by removing all loose overburden of weathered rock as well as any decayed or weak rock till sound bed rock is exposed. In this process the underlying sound rock should not be damaged.

Further, the final rock surface should be excavated to develop stepped or benched foundation to increase the sliding resistance of the dam. The surface should also be thoroughly cleaned with wet sand blasting and washing before concrete is laid.

In order to rectify the deficiencies in deep foundation as well as the water tightness of the foundation, the following methods of treatment are employed.

i) Excavation of seams of decayed or weak rock by tunneling and back filling tunnels with concrete.
ii) Excavating for and making concrete cutoff walls across leakage channels in the dam foundation where the water channels are too large or too wet for mining and grouting.
iii) Grouting the foundation to increase its strength and tender it impervious.

Foundation Grouting: This consists of injecting under pressure a mixture of cement and water (called grout) in to holes drilled in the foundation rock for the purpose of sealing seams, cracks and fissures or filling voids present in the rock. Foundation grouting classified on the basis of pressure used for grouting may be either (i) low pressure grouting (ii) high pressure grouting and when classified on the basis of the purpose served by the grouting, it may be (i) consolidation grouting or (ii) curtain grouting. Consolidation grouting is low pressure grouting and curtain grouting is a high pressure grouting.

Consolidation Grouting: Done to consolidate the entire rock foundation and thus to increase its strength. Shallow holes of depth ranging from 3 to 15m (also called B-holes) are drilled through the foundation rock on a grid pattern at spacing of 6 to 30m. Before grouting operation starts the holes are thoroughly washed with water and compressed air to remove all loose material and drill cuttings. Then the holes are tested for tightness by injecting water under pressure, which helps to decide on the consistency of the grout to be used and to locate seams and other openings in the rock which are to be plugged. The grout is then injected in to these holes at a relatively low pressure which is usually less than about $392.4KN/m^2$ ($4kg\ f/cm^2$). This grouting results in the consolidation of the foundation in to more or less monolithic mass by bonding together the jointed rock. Moreover this grouting serves as cutoff against the leakage of the grout of the high pressure grouting which may be required to be done later.

Curtain Grouting: - Done to create a deep curtain or a barrier in the foundation rock at the heel of the dam which will prevent leakage from the reservoir and thus reduce the uplift pressure on the dam. Deep holes (called A hole) which are either vertical or inclined are drilled. Inclined holes are drilled to intercept the principal joints in the rock to develop an effective curtain, the primary series of holes being spaced 4.5 to 7.5m depending on the nature of the rock. A second series comprises an equal number of intermediate holes. This one is repeated for a third series but reducing the spacing of holes to quarter of that of the first series (i.e. spacing 1.2 to 1.5m). The depths of holes depend on nature of the rock and in general vary b/n 30-40% of the head water depth on dense foundation to 70% of head water depth for poor foundations.

3. ARCH DAM

3.1 General

An arch dam may be defined as a solid wall curved in plan standing a cross the entire width of the river, in a single span. It is one type of concrete dam.
Unlike a concrete gravity dam which carries the entire load by its self weight an arch dam obtains its stability by both the self weight and to a great extent by transiting the imposed loads by arch action in to the valley walls. The geometry of the dam site is therefore the most basic consideration in the selection of an arch dam. As a general rule an arch dam requires a site with abutments of sufficient strength to support the arch trust. Concrete saving in such dams relative to gravity dam is 50-85% & abutment stability is critical to the structural integrity & safety of an arch dam.
The concepts of overturning and sliding stability applicable to gravity or buttress analysis have little relevance to the arch. An arch represents a stable structural form and given that the integrity of the supporting abutments is assured, failure can occur only as a result of overstress. Arch dam design is therefore centered largely up on stress analysis & the definition of an arch geometry which avoids local tensile stress concentrations and /or excessive compressive stress. In a achieving this objective it is frequently necessary to adopt varying curvatures & thicknesses b/n arch crown & abutments & also from crest level to base.
The horizontal component of Arch trust must be transferred in to the abutment at a safe angle, i.e., one which will not promote abutment yielding or instability. At any elevation the arch trust may be considered to enter the abutment as shown in fig 1.

Fig 1 Abutment Entry angle geometry for Arch dams

3.2 Valleys suited for arch dams

Narrow gorges provide the most natural solution for an arch dam construction, the usually recommended ratio of crest length to dam height being 5 or less.
Sarkaria has proposed a Canyon Shape Factor (C.S.F.) which would indicate the suitability or otherwise of the site for an arch dam. The Canyon Shape Factor is Given By:

$$C.S.F = \frac{b + H(Sec\psi_1 + Sec\psi_2)}{H}$$

The usual values of C.S.F = 2 to 5; the lower the value, the thinner the section.

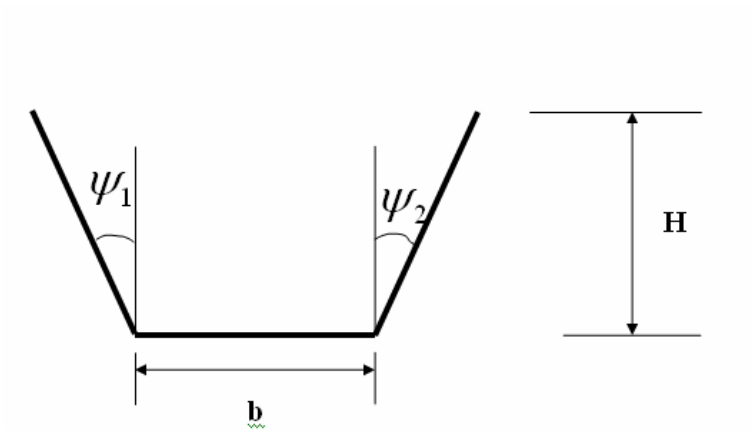


Fig 2 Valley shape and Dimensions

3.3 Types of Arch Dams

Arch dams may be divided in to two types via, the massive arch where a single curved wall, usually vertical or nearly so, spans the full width b/n abutments and the multiple arches, consisting of a number of smaller arches usually inclined, supported on piers or buttresses.

Massive arch dams may be further divided in to

i) Constant radius arch dams
ii) Constant angle arch dam
iii) Variable radius arch dam

Constant radius arch dams: - It has simple geometry**.** A constant radius of arch dam is the dam in which the radius of extrados (outside curved surface of the arch) is constant, most of the time , all elevations from the top of the dam to its base. However, the radius of the d/s face (or intrados) gradually decreases as the depth below the crest is increases to provide increased concrete thickness towards the base to account the hydrostatic water pressure. Thus, in this case, the u/s face of the dam is vertical and the dam is triangular in cross section with increased width at crest.

In constant radius Arch dam, the arch center for the u/s face d/s face and the centerline of horizontal arch rings, at various elevations lie on a straight vertical line that passes through the center of the horizontal arch rings of the u/s face at the crest. Hence it is also known as ***Constant center arch dam***.

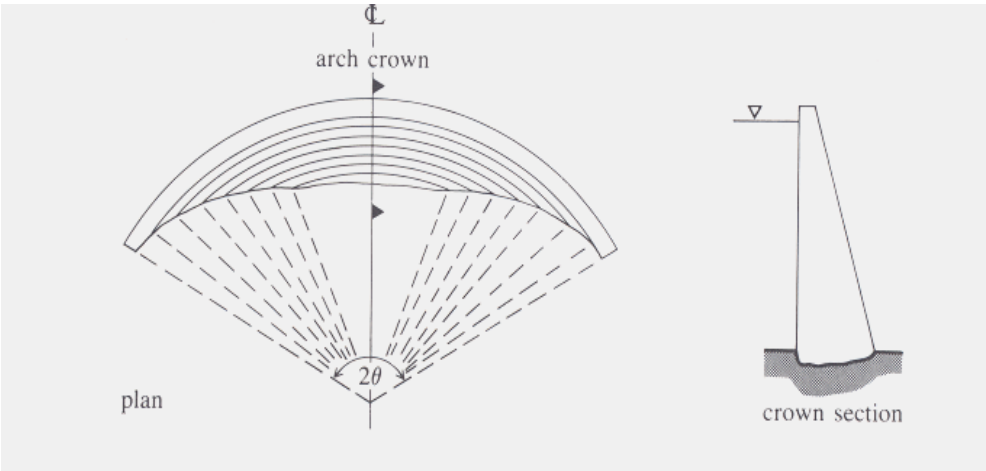


Fig. 3 Constant Radius Arch Dam

In this type of dam the central angles of arch rings of the intrados vary at various elevations, the maximum being at the top of the dam and a certain minimum at the bottom of the dam. Due to reduction in central angle and the central rise of arch rings at the lower elevations near the base of the dam, considerably low arch action will be developed for these arch rings. Hence relatively large proportion of the water load at the lower elevations will be carried by cantilever action for which the arch rings at the lower elevations will have to be made considerably thick.

This will result in uneconomical design. For U–shaped valleys the constant radius arch dams are found suitable since there is a relatively less reduction in the central angle and the central rise of the arch rings at the lower elevations. Reduction at the central angle at the lower elevations would be considerable in the case of narrow V-shaped valleys and would result in uneconomical design for such types of dams.

Constant angle arch dam: - The central angle of the horizontal arch rings is of the same magnitude at all elevations. Theoretically, for economical design of constant angle arch dams central angle of arch rings equal to 133^0 34' may be adopted. However, the practical range of central angle from 100^0 to 150^0 depending on the valley shapes. Due to large central angle greater arch action will be developed for the arch rings at all elevations. Hence water load at all elevations is carried by arch action. In general, the thickness of constant angle arch dam is smaller than that of constant radius arch dam. As a result, under same condition it requires only about 70% of concrete as compared to constant radius arch dam.

The radii of arch rings of constant angle arch dam decreases as the depth below the crest increases since the central angle at all elevations has the same magnitude. Moreover, the curvature of each arc ring will be different and the various arch rings may be arranged to have the crowns of either extrados or intrados of all arch rings flushing. These arrangements will result in an overhang of the u/s face near the

abutments and that of the d/s face near the crown of the arch. Hence, the formwork required is complicated and costly than that of constant radius arch dam.

Constant angle arch dams are suited to narrow V-shaped valleys with large central angle fairly large arch action will be developed even at lower elevations & hence the design will be economical.

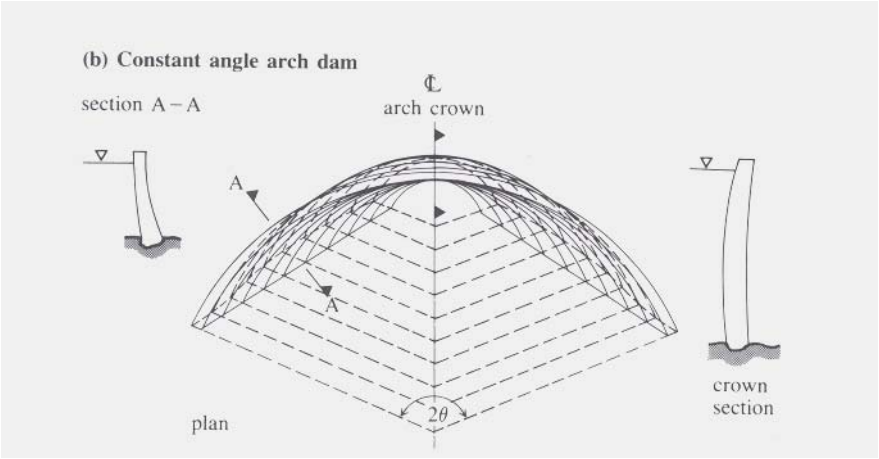


Fig 4 Constant angle Arch dam

Variable Radius Arch Dam: Neither the radius nor the central angle is constant. Radius is maximum at the top & minimum at the base. The central angle of the arch rings usually vary b/n 80 to 150^0 & these are so adjusted that large arch action is developed even for arch rings at lower elevations. The radii at various elevations are also so adjusted that the over hangs are avoided or minimized. Further in this dam, centers of the arch rings at various elevations do not lie on the same vertical line.
This type of arch dam is suited to u-shaped valleys as well as for narrow V-shaped Valley b/c by adjusting the radii & the central angles a fairly large arch action may be developed even for arch rings at lower elevations.

This type of arch dam is suited to U-shaped valleys as well as V-shaped valleys since large arch action will be developed at lower elevations for arch rings having smaller spans by adjusting the radii and the central angles.

As compared to the constant angle arch dam a variable radius arch dam is less economical since under the same conditions a variable arch dam requires about 17% more concrete than constant radius arch dam since under the same conditions the concrete required for a variable radius arch dam is only about 82% of that required for a constant radius arch dam. Hence it is a compromise b/n constant radius & constant angle arch dams.

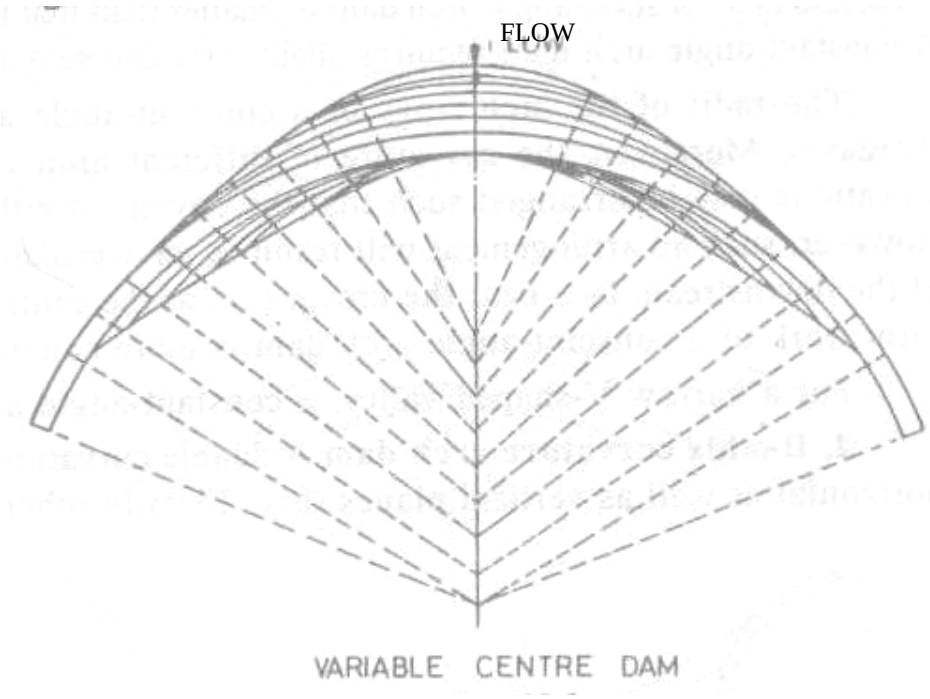


Fig 5 Variable Radius Arch dam

3.4. Methods of Design of Arch Dams

Commonly used methods of design or arch dams are

i) Thin cylinder theory
ii) Thick cylinder theory
iii) Elastic arch theory
iv) Trial load analysis

Thin cylinder theory: - In this theory the horizontal section of the arch dam is assumed to behave as part of thin cylinder ring at the corresponding elevation. Further it is assumed that the horizontal water load is carried entirely by arch action. The pressure on the arch ring equal to the hydrostatic pressure at the corresponding elevation and the stress distribution across the arch ring is uniform.

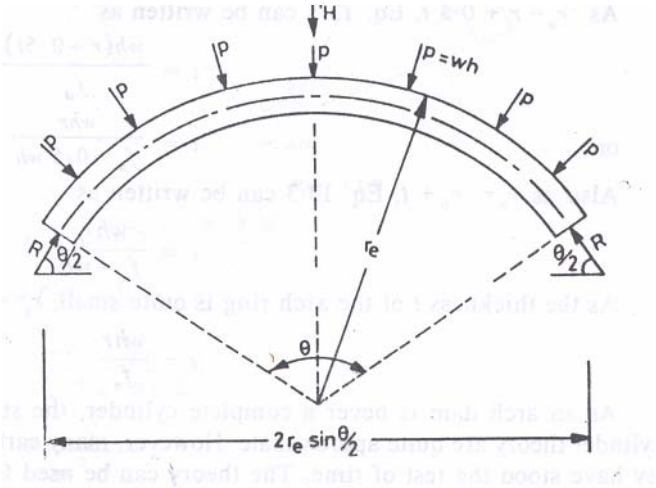


Fig 6 Thin Cylinder Theory definition sketch

The figure shows a section of a thin arch ring having extrados radius r_e, subtended angle of arch ring θ and thickness t

If the arch ring is at a depth of h below reservoir water surface, then P = γh (the hydrostatic pressure acting in the radial) direction.

And the total hydrostatic pressure force p is given by:

$$P = \gamma h \times 2\, r_e \sin\theta/2$$

$$P = 2\gamma h\, r_e \sin\theta/2$$

If R is the abutment reaction its component in the upstream direction which resists the pressure force P is equal to R sin $\theta/2$

$$2R\sin\theta/2 = 2\gamma h\, r_e \sin\theta/2$$

$$R = \gamma h\, r_e \quad \text{(Abutment Reaction)}$$

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.

If σ is the compressive stress developed in the arch ring at the abutments then

$$\sigma = \frac{\gamma h r_e}{t*1} \qquad \text{(Average Stress)} \quad \left(\sigma = \frac{R}{A}\right)$$

If *f* is the allowable compressive stress for the arch material then in the limiting case $\sigma = f$ and

$$t = \frac{\gamma h r_e}{f}$$

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, r_i and mean radius r, can be derived as follows.

$$r_i = r_e - t \Rightarrow r_e = r_i + t$$

$$t = \frac{\gamma h r_i}{f - \gamma h}$$

$$r = r_e - 1/2\ t \Rightarrow re = r + t/2$$

$$t = \frac{\gamma hr}{f - 0.5\gamma h}$$

Most Economical central angle

The concrete volume of any given arch is proportional to the product of the arch thickness and the length of the center line arc. For a fixed combination of span, loading and permissible cylinder theory stress, the follwing can be shown.

The Volume of Concrete, V, per unit depth of Arch Ring is given by

$$V = r(\theta)A$$

Where A = tx1 = cross sectional area of arch ring for unit height

θ = in radians

$$\therefore V = r\theta t$$

Substituting the expression for $t = \frac{\gamma hr}{f - 0.5\gamma h}$, we get

$$V = r\theta\left(\frac{\gamma hr}{f - 0.5\gamma h}\right) = \frac{\gamma h\theta r^2}{f - 05\gamma h}$$

If L is the span of arch ring, then

$L/2 = r\sin\theta/2 \Rightarrow L = 2r\sin\theta/2 \Rightarrow r = L/(2\sin\theta/2)$ ----------(a)

And

$$V = \left(\frac{L}{2\sin\theta/2}\right)^2\left(\frac{\gamma h\theta}{f - 0.5\gamma h}\right) = \frac{\gamma hL2}{2(f - 0.5\gamma h)}\left(\frac{\theta}{\sin^2\theta}\right)$$

By differentiating V with respect to θ and equating it to zero,

$$\frac{dV}{d\theta} = \frac{\gamma hL^2}{2(f - 0.5\gamma h)}\left(\frac{\sin^2\theta - 2\theta\sin\theta\cos\theta}{\sin^4\theta}\right) = 0$$

$\Rightarrow \tan\ \theta/2 = 2\theta$ which gives (by trial and error)

$\theta = 133^0 34'$ ----------(b)

Thus, on the basis of thin cylinder theory, the volume of concrete required for arch dam would be minimum if the central angle is $133^0 34'$. By substituting $\theta/2$ in equation (a) we get

$$r = \frac{L}{2\sin\left(\frac{133^0 34'}{2}\right)} = 0.544L$$ ----------(c)

L = the arch span and the radius may be r_e,r or r_i if L is the corresponding span. In a site with a variable span length, a constant radius dam can have the correct central angle only at one elevation.

The smallest concrete volume for the whole dam, with constant radius, is obtained by increasing the top angle to get the best average angle. The top angle which gives the best average can be found by trial and topography seldom permits value greater than 150^0 for θ at the top of the dam.

3.4.2 Thick Cylinder Theory

Improvement in thin cylinder theory was made by considering the arch as thick cylinder. The stresses at the extrados and intrados are given by

$$\sigma_e = P\left(\frac{r^2{}_e + r^2{}_i}{r^2{}_e - r^2{}_i}\right) \quad and \quad \sigma_i = \frac{2\rho_o r^2{}_e}{r^2{}_e - r^2{}_i}$$

The thickness is given by the relation

$$t = r_e\left(1 - \sqrt{1 - \frac{2p}{\sigma}}\right)$$

$$p = \gamma h$$

4. BUTTRESS DAMS

Efforts have been made from time to time to innovate methods for affecting economy in the use of concrete, by cutting down concrete from concrete gravity dam's portion 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 in which solid wall of specified thickness & section are constructed parallel to the flow at some suitable intervals called buttress & include slabs which are supported on u/s side on these buttresses.

4.1 Component parts of Buttress Dams

A buttress dams consists of the following component parts.

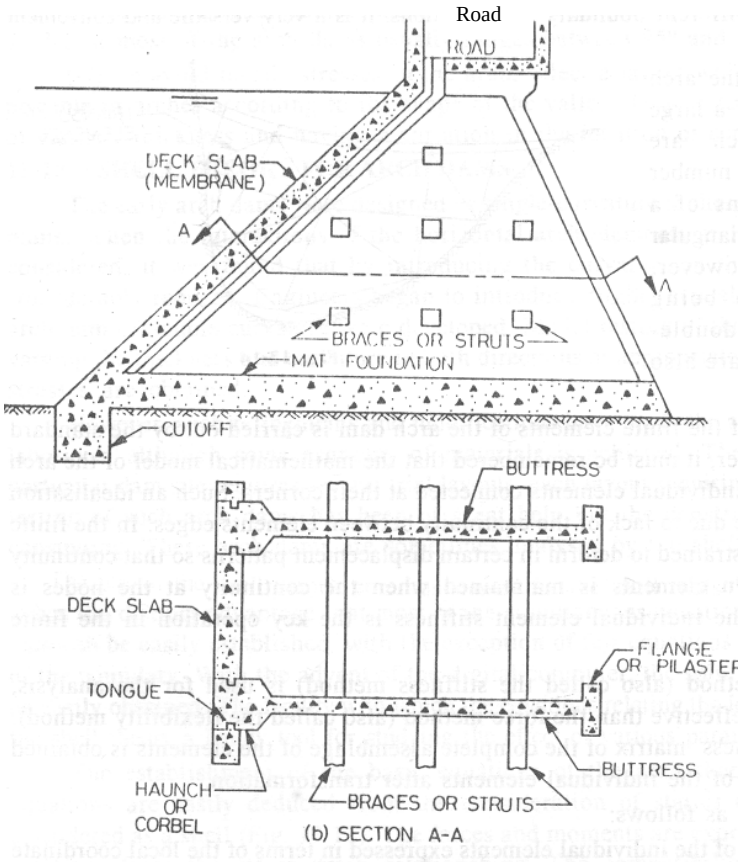


Fig 1 Deck slab or flat slab or Amberson Type buttress dam

1) Sloping membrane: - The sloping membrane or u/s deck supports the water and transmits the water load to a series of buttresses at right angle to the dam axis.

2) Buttresses: - are constructed at right angles to the axis of the dam at certain intervals. They support the slopping membranes & transfer the load to the foundation. They are thin walls of triangular profile with sloping u/s face.

3) Mat foundation or Footings: - Footings are required for the buttresses to transfer the loads to the foundation. If the foundation is relatively strong, spread footings are provided but for relatively weak foundation mat footing is required.

4) Lateral Braces (strut):- are provided b/n adjacent buttresses at right angles to them to reduce unsupported length & thus provide lateral stiffness and resistance to buckling of buttresses.

5) Haunches or Corbels: - The wide upstream end of buttresses which help in transiting the load from the u/s deck to the buttresses.

6) Cutoff (Concrete): - is provided at the upstream end of the dam to reduce/prevent the seepage & up lift.

4.2. Types of Buttress Dams

They may be classified in the following two ways

1. on the basis of the sloping membrane
2. on the basis of joint b/n the sloping membrane & buttresses

Classification Based on the Type of Sloping Membrane

This is further classified in the following categories

i. Deck slab or flat slab or Ambursen type buttress dams
ii. Multiple -arch type buttress dams
iii. Multiple -dome type buttress dams
iv. Massive head type buttress dams or bulk head type buttress dams

i. Deck slab/Flat Slab or Amburesn type Buttress Dam

In this type of buttress dam the u/s sloping deck slab consists of a reinforced concrete slab supported by as series of buttresses. The slab is separated by buttress tounge and supported by reinforced haunches which are constructed monolithically with the buttress. The inclination of the deck slab is kept b/n 35^0 and 45^0 with the horizontal.

The corbel or haunch is always reinforced since tensile stress could develop. The rest of the buttress may or may not be reinforced. Struts are of R.C.

Deck slab may Further Sub divided into:-

a) Simple deck slab type buttress dams (Simply Supported Slab)

The deck slab is not rigidly connected to the buttresses but it is in the form of simply supported slab. In order to provide a wide support for the slab, the upstream end of the buttress where it joints the slab is made wider by providing haunch or corbel. The joint b/n the slab & the buttress tongue is filled with bituminous mastic or some flexible joint compound.

This permits each slab to act independently and allows free expansion of the slab in the direction parallel to the base line of the dam. Reinforcement is placed only at the down stream face (since it is simply supported). But it requires thicker slab. These types of buttress dams are suitable for wide valleys where a long dam is required & the foundation is weak. The main disadvantage of this type is that it requires a thicker slab as compared to the fixed (continues) slab.

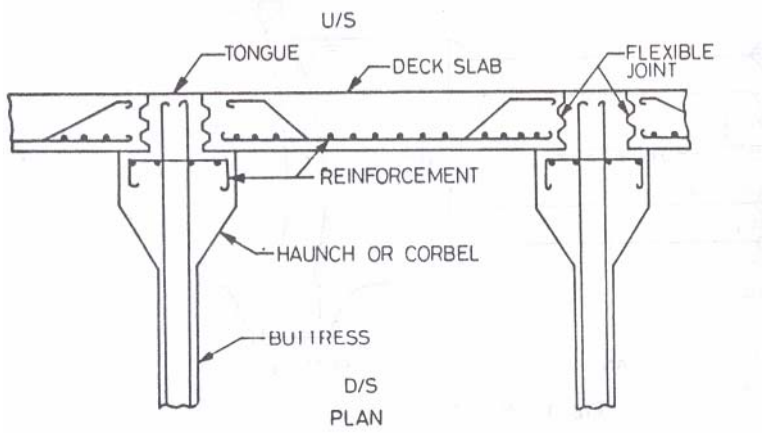


Fig 2

b) Fixed (or continuous) Deck Slab Type Buttress Dams

The deck slab is cast monolithic with the buttresses and it acts as a continuous slab. Reinforcement is provided both on upstream & down stream faces of the slab. This type of deck can be used only when the foundation is strong; where it will be more economical than the freely supported slab b/c the deck slab is thinner.

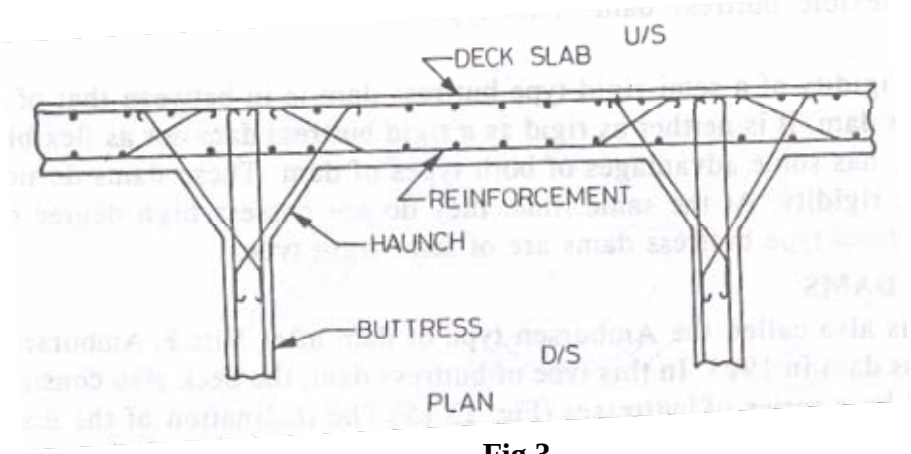


Fig 3

Cantilever Deck slab type Buttress Dams

The deck slab is cast monolithically with the buttresses in such a manner that it over hangs on either side of the buttress and acts as cantilever.

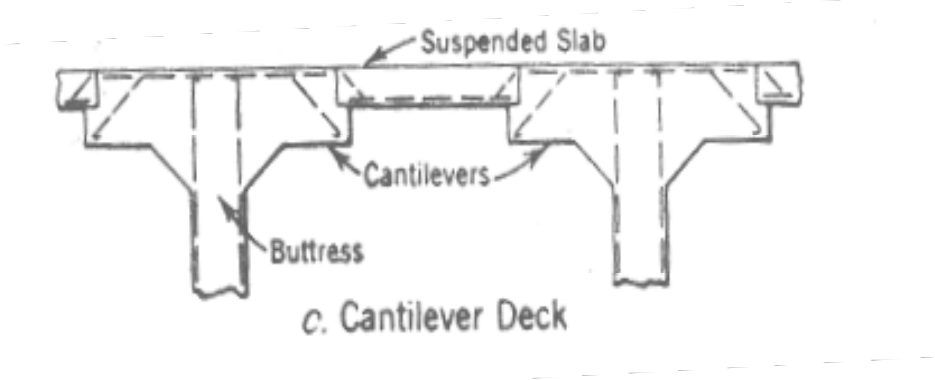


Fig 4

Multiple Arch Types Buttress Dams

The sloping membrane or deck consists of a series of R.C arches supported by a number of buttresses. The u/s face of the damn is usually inclined at 45^0. The arches are cast monolithic with the buttresses.

Multiple -Dome Type Buttress Dams:-

Sloping membrane or deck consists of a series of R.C domes supported by a number of buttresses.

Massive Head Type Buttress Dams:-

No Separate water retaining member (Deck) is provided but the water retaining member is formed by enlarging the upstream end of buttress. The dam is thus made of

a series of buttresses with massive heads placed side by side. Buttress heads are so shaped that the tensile stress are not developed in any portion of the buttress heads and hence the buttress heads are not reinforced.

These types of buttress dams may be further of the buttress heads and hence the buttress heads are not reinforced. These types of buttress dams may be further sub-divided on the basis of the buttress head into:-

A) Round head buttress dams
B) Diamond head buttress dams
C) Tee head buttress dams

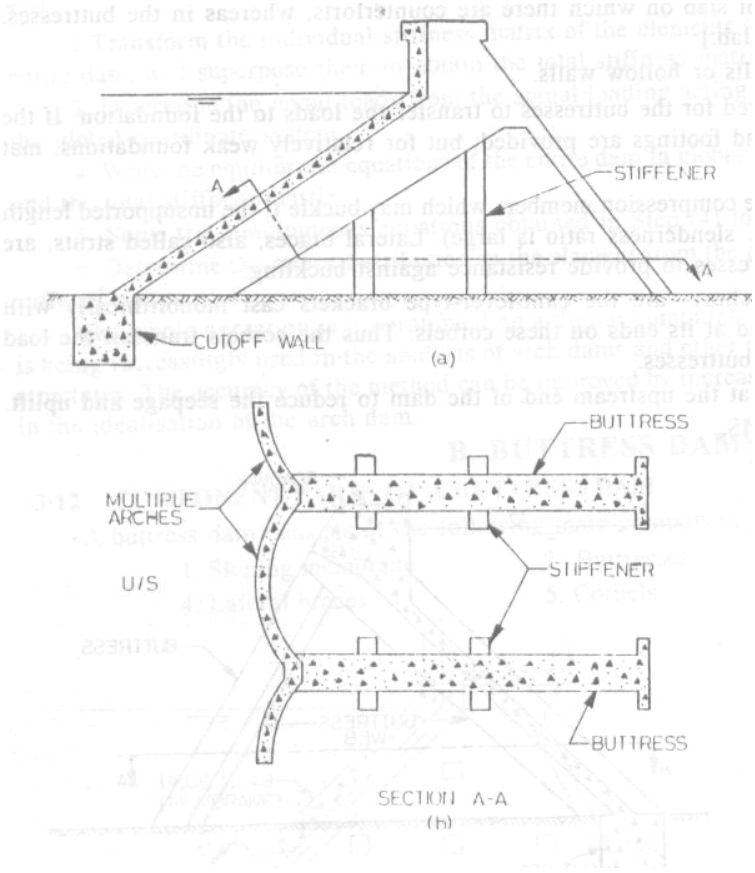


Fig 5 Multiple Arch Type Buttress Dam

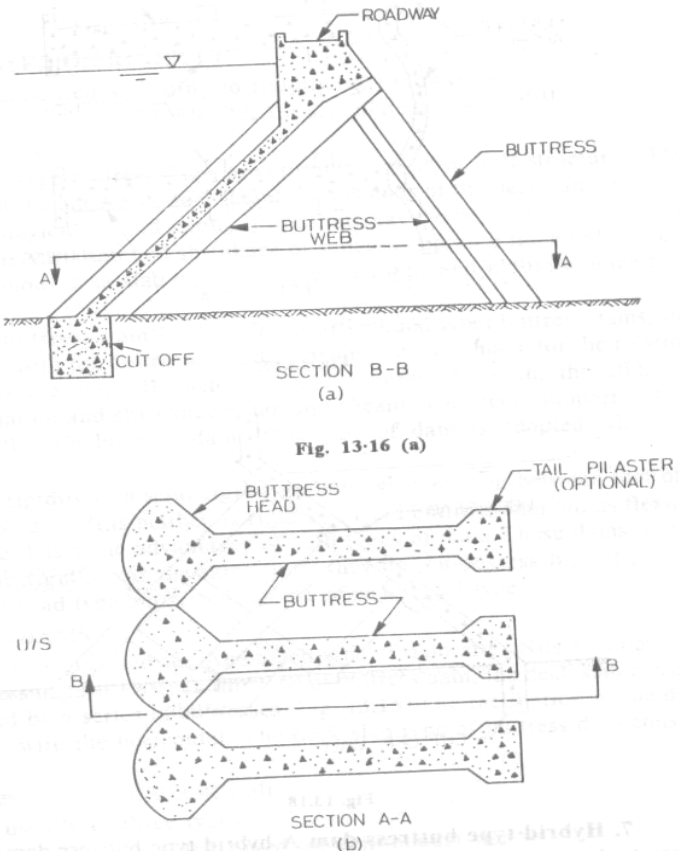


Fig 6 Massive Head Type Buttress Type Dam

Classification Based on the joint b/n the sloping membrane Buttresses

Under this we have following categories

1) Rigid buttress dams: - The up stream deck is cast monolithically with the buttresses and hence the ends of the deck can not move.
   E.g. continuous deck slab type, multiple arch type & multiple dome type buttress dams
2) Flexible (or articulated) Buttress Dams: sloping membrane or deck is not constructed monolithic with the buttresses. as such these dams are flexible. E.g. simple deck slab type.
3) Semi - rigid buttress Dams: - It is neither as rigid as rigid buttress dam nor as flexible as a flexible buttress dam. It is therefore in b/n that of rigid buttress dam & a flexible buttress dam.
   E.g. The massive head type buttress dam

4.3. FORCES ON BUTTRESS DAMS

They are the same as those on gravity dams. Uplift pressure acting on a buttress dam depends on the type of the dam and type of foundation provided for buttresses. If separate spread footings are used for each buttress then the uplift pressure is relieved by the gap b/n the buttresses.

Buttress dams on rock foundation are subject to the uplift pressure due to head water is usually neglected and only the uplift pressure due to TW (if any) is considered for the entire footing. However, in the same case if the foundation rock is such that uplift pressure due to head water cannot be neglected, then under the deck and haunch section full uplift pressure due to head water is considered and the remaining portion of the footing full uplift pressure due to tail water is considered. If a mat foundation is provided for all buttresses, uplift pressure may be computed in a similar manner as in the case of gravity dam.

Buttress dams should full fill stability requirements as that of gravity dams. Further in the case of a buttress dam the total force acting per buttress unit must be considered instead of force per unit length of dam as in the case of gravity dam.

4.4 DESIGN PROCEDURE OF FLAT-SLAB TYPE BUTTRESS DAMS

The design involves the following steps:-

i) Determination of economic buttress spacing & upstream slope

ii) Design of deck slab &other details

iii) Preliminary design of buttress & check for overall stability

4.4.1 Economic Buttress Spacing

The spacing of the buttresses is governed by economy. If the spans are short, face slabs may be thin with a small volume of concrete and reinforcement steel in deck but increase in cost of form work. Wider buttress spacing result in reduction of cost of form work but volume of concrete & reinforcing steel for the deck slab increase

considerably. Thus economic buttress spacing is that which gives economic balance of concrete reinforcing steel and formwork & satisfy design requirement.
The most economical spacing depends up on the following factors

i) The mean height of dam
ii) Type of footing (spread or mat foundation)
iii) Unusual foundation and side hill conditions
iv) Upstream slope of dam

Height of dam:-Economic buttress spacing increases with the height of the dam; whereas for low dams the buttress spacing will be proportionally less.

For buttress dams on sound rock foundations the most economical buttress spacing for deck slab buttress dams are given below.

Height of dam (m)	**Economical spacing c/c of buttress (m)**
15 to 30	5 to 6
30 to 45	9 to 12
Above 45	12 to 15

Type of Footing: it is not economical to have very large spacing of buttresses b/c the quantities of material required for spread footing (or mat foundation) & for the apron of spillways increase with an increase in the buttress spacing and hence it will be economical to have a smaller spacing for buttresses.

Upstream slope: Varies from 35^0 to 45^0 and is governed by the requirements of sliding factor, i.e., $\sum H/\sum v$ resistance against sliding is achieved from the vertical component of the water pressure since self weight of the buttress dam is relatively small. Vertical component of the water pressure varies with the upstream slope.
Further, for constant buttress spacing, small variation in the u/s slope results in an appreciable change in the quantity of concrete and hence it affects economy. Likewise for constant u/s slop the variation in the buttress spacing also results in change in the quantity of concrete and hence affects economy.
Thus for each value of u/s slop there would be an economical buttress spacing which may be determined from a ***master curve*** which may be prepared as indicated below. Further from the same curve, a combination of the most economical buttress spacing and the u/s slope may also be determined.
∗**A master curve** is extremely useful for determining the most economical spacing for d/t upstream slopes.

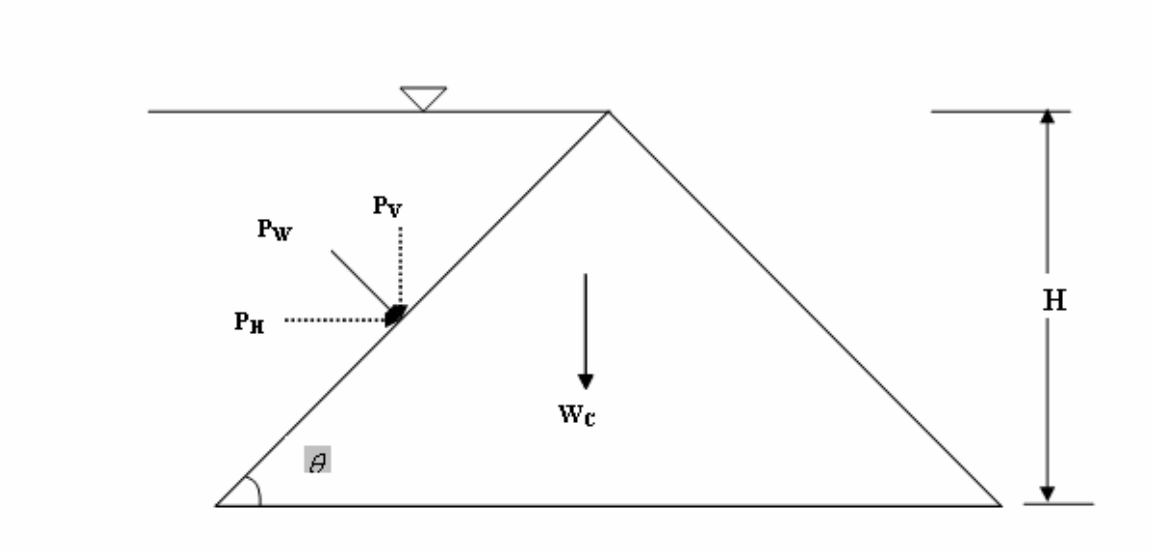


Fig 7 Simplified diagram of buttress dam

Let H = height of dam in meters

P_V = Vertical component of water pressure per meter length of dam

P_H = Horizontal component of water pressure per meter length of dam

W_C = Weight of dam per meter length of dam

Cq = Quantity of concrete (m3) per meter length of dam

f = sliding factor

γ_C = Unit weight of concrete

$$P_H = \frac{\gamma_w H^2}{2}; P_V = \frac{\gamma_w H^2}{2}\cot\theta;$$

$$W_C = \gamma_C Cq$$

Taking

$\gamma_w = 9.81 KN/m^3$ *and*

$\gamma_c = 24 KN/m^3$

$$f = \frac{\sum H}{\sum V} = \frac{P_H}{P_V + W_C}$$

$$f = \frac{\frac{\gamma_w H 2}{2}}{\left(\frac{\gamma_w H^2}{2}\cot\theta + \gamma_C Cq\right)}$$

$$Cq = 0.2083H^2\left(\frac{1}{f} - c\right.$$

The equation shows that for a given height h and a given value of f the quantity of concrete depends up on $\cot\theta$. Thus for a given value of h and f the quantity of concrete Cq, obtained are independent of the buttress spacing and hence this will plot as vertical lines on the plot of buttress spacing vs quantity of concrete. Next choosing certain value of θ, the dam is designed by considering a number of buttress spacing and the quantity of concrete required per unit length of the dam for each buttress spacing is determined. The value of the quantity of concrete so obtained are plotted against the corresponding buttress spacing on the same graph to obtain a curve for the chosen value of θ. By repeating the same procedure for d/t values of θ a series of such curves are obtained as shown by joining the points of intersections of the curves and the vertical lines corresponding to each value of θ which is shown by a solid line. The master curve gives the value of economical buttress spacing for each value of θ for the given height of dam. Further, the apex of the master curve gives the value of the most economical buttress spacing as well as the corresponding value of θ and the quantity of concrete per meter length of dam.

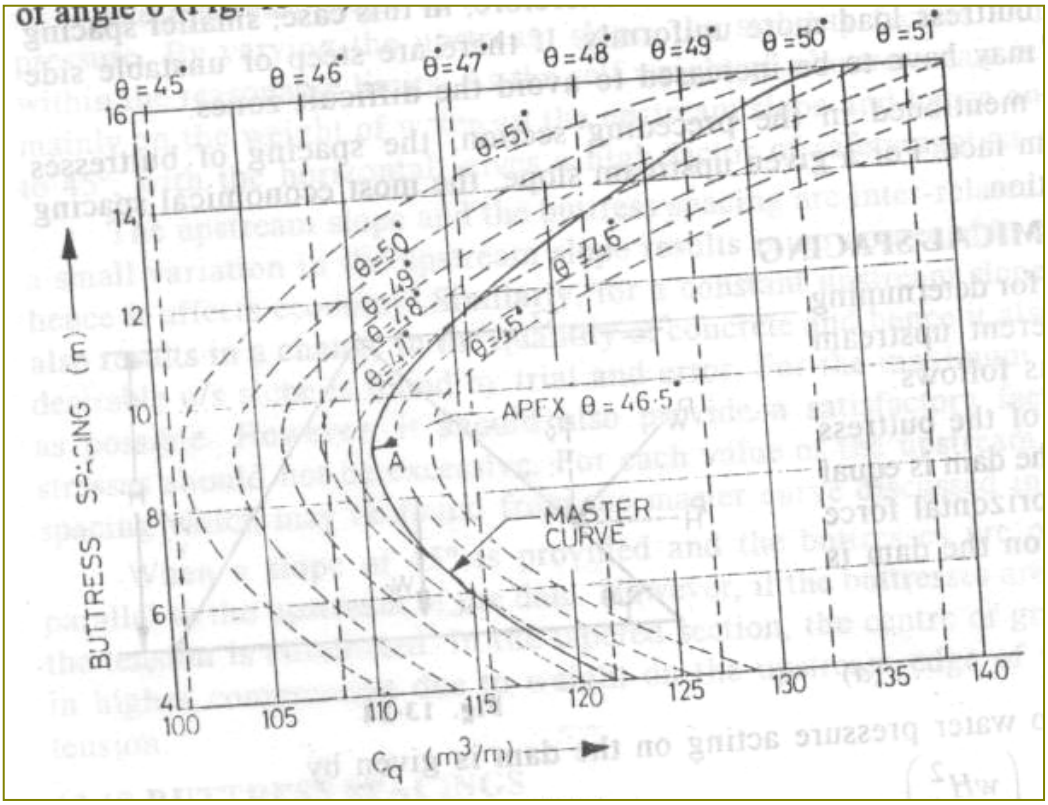


Fig 8 MASTER CURVES

Design of Deck Slab

The deck slab is designed as a simply supported slab by using the standard methods of R.C.C/ design. The slab thickness is controlled by bending moments at higher levels and shear at lower levels. (Refer any R.C.C. text)

Design of A Haunch

The width of the bearing face of the haunch is determined from the permissible bearing pressure and the assumed load distribution on the face. The load distribution on the bearing face is usually assumed to be triangular with the maximum at the outer edge and zero at the inside corner of the haunch.

a) *Width* (B). The actual load distribution on the bearing surface is indeterminate. However the triangular distribution is usually assumed, with the maximum pressure p at the outer edge and zero pressure at the inner edge of the haunch. Thus the total reaction R is Given by

$$R = 0.5pB$$

B = the width of the bearing surface and p = the maximum pressure.

b) *Depth* The depth dm and ds are as per the design requirements respectively, for bending moment and shear. These depths are so proportioned that the bracket profile adequately envelopes the theoretical minimum depth profile for moment and shear.
The depths D_1 and D_2 are the actual depths provided.

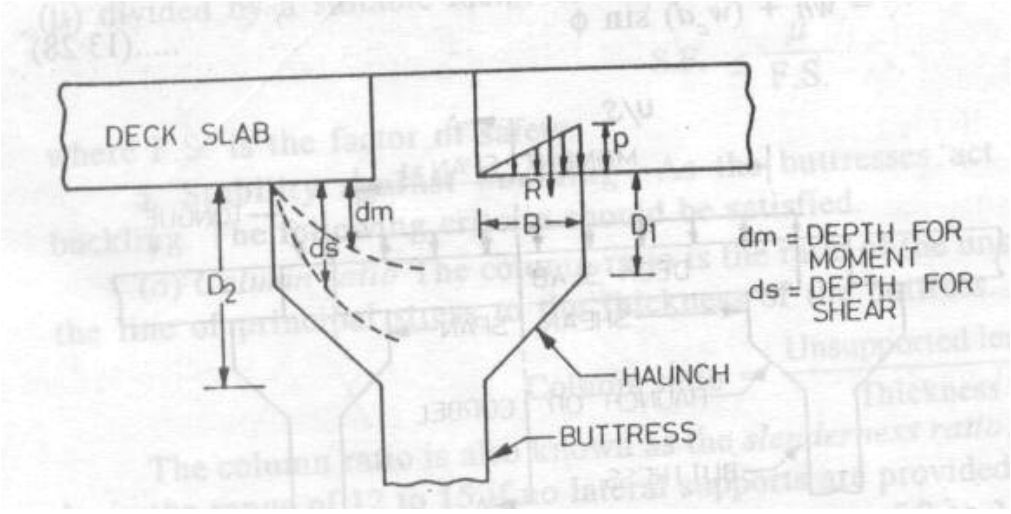


Fig 9

Though the conventional formulae of the design of RCC beams and cantilevers are not strictly applicable for trapezoidal brackets of the haunch, approximate depth profile can be determined from the conventional RCC design formula (see any text on RCC DESIGN).

Resisting moment, $M = Qbd^2$

Resisting Shear = sbjd

Preliminary design of Buttress and Check for Overall Stability

For preliminary design of buttress only water pressure on the deck slab, weight of deck slab and buttress are considered. The final design is checked for other forces such as earthquake, uplift pressure, wave pressure, silt pressure etc…

Preliminary design of buttress is carried out on the bases of *Unit column theory*, in which the buttress is assumed to be made of a number of unit columns.

A unit column is a curved column of unit width.

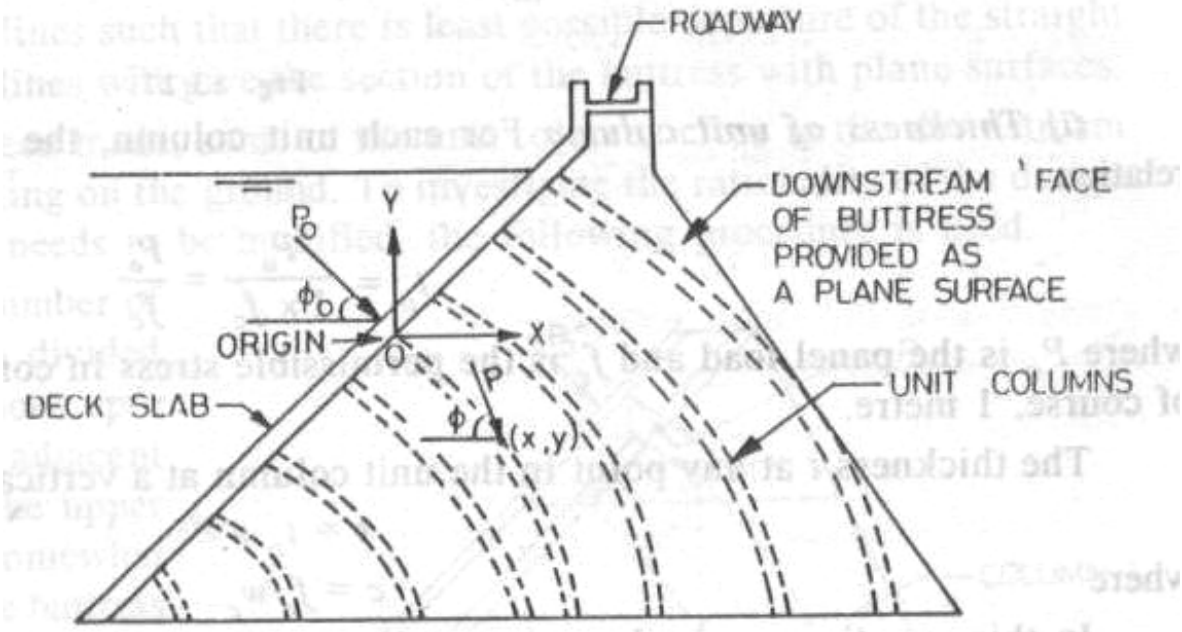


Fig 10 Unit columns and resultant forces for frictionless joints

Assumptions:

1) Each column transfers the load from the top of the column to the foundation independently of the adjacent columns above or below it.
2) Each column is so proportioned that at every section of the column uniform compressive stress is developed and it is curved to avoid eccentric loading.

The magnitude of the initial load Po depends on the type of joint b/n deck slab and the buttress. Thus if the joint is frictionless then $P_o = P_w + W_{dn}$ (total water pressure on the deck and component of the weight of the deck slab normal to the buttress face (fig 10). On the other hand if the joint is monolithic then Po is equal to the resultant of the total water pressure on the deck slab P_w and the weight of deck slab W_d. In this method in general, five columns distributed over the entire buttress will be adequate.

For each unit column, the value of the initial thickness to is obtained from:

$$t_o = \frac{P_o}{f_c} \quad \text{----------(1)}$$

Where f_c is the permissible compressive stress in concrete

The thickness t at any point in the unit column at a vertical distance y from the origin o is given by $t = t_o e^{-y/c}$ ----------(2)

Where y is negative downwards

c = a constant = f_c/γ_c ; γ_c = unit weight of concrete

t_o is the thickness of buttress near the deck slab.

Since the width of the column is constant (i.e. unity), the thickness of the column increases from the deck to foundation.

Further, the equation of the axis of the unit column is

$$y = c(\log_e \cos\phi - \log_e \cos\phi_o \quad \text{----------(3)}$$

Where $\phi =$ angle made by P_o with horizontal

$\phi_o =$ angle made by the tangent to the column axis at any point on the axis to the horizontal

The angle ϕ may also be expressed as

$$\phi = \frac{x + c\phi_o}{c} \quad \text{----------(4)}$$ ϕ and ϕ_o in radians

By substituting value of ϕ from equation (4) in to equation (3), the coordinates (x,y) of any point on the column axis with respect to origin o may be obtained from

$$y = c(\log_e \cos\frac{x + c\phi_o}{c} - \log_e \cos\phi_o \text{ ----------(5)}$$

The minimum thickness of buttress determined on the bases of unit column theory is usually much less than the minimum thickness required from practical considerations. Thus the minimum thicknesses to be provided are given below.

Buttress spacing	Min. Thickness of buttress
4.5 to 6 m	0.2 to 0.3m
> 6m	0.45 to 1.2m

The design of the unit column should therefore be carried out with actual thickness provided as per the requirements of the minimum thickness (as given above).

Downstream Slope of Buttress: The unit column theory gives a curved d/s face for the buttress. However, a buttress with a curved d/s face is difficult to construct and hence from practical point of view a buttress a plane sloping d/s face is usually provided. The slop of the d/s face should satisfy overall stability requirements. Usually the base length of the buttress varies from 1.2 to 1.5 times the depth below the maximum water level in the reservoir.

After determining the dimensions, the buttress has to be checked for overall stability against overturning g and sliding. Using the same formula as that of the gravity dam analysis, the direct stress sliding factor and S.F.F. are computed.

After the buttress has been found adequate for the axial loads, it should be investigated for column ration (slenderness ratio) and massiveness factor.

Column Ratio: Is the ratio of unsupported length of buttress measured along lines of principal stress to the thickness of the buttress. Low buttresses having no lateral support may have a column ratio of b/n 12 and 14, but for high buttresses which require lateral support the column ratio should be limited to the range 7 to 10.

Massiveness factor: Is the ratio of spacing of buttress to the thickness of the buttress. In general its value varies b/n 2.5 and 3.

Finally after calculating foundation pressure (of the design deck slab and buttresses) spread footings and mat foundations are designed depending on magnitude of foundation pressure.

Advantages of Buttress dams

i) less concrete used compared to a gravity dam of the same height
ii) More safety against overturning and sliding b/c of the larger vertical component of hydrostatic force exerted on the dam.

iii) More equal distribution of stresses at foundation
iv) Less massive than gravity dam hence may be used on weak foundations that are not suitable for gravity dam
v) Decreased uplift pressure (if no spread footing joining the buttresses is used)

Disadvantages

i) Needs reinforcement and expensive shuttering
ii) Additional skilled labor is required to create form work
iii) Threat of deformation of concrete from impounded water is more likely than from a thick gravity section
iv) More susceptible to damage by sabotage.

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5. EMBANKMENT DAMS

The embankment dam can be defined as a dam constructed from natural materials excavated or obtained near by. The materials available are utilized to the best advantage, in relation to their characteristics as bulk fill zones with in the dam section. The natural fill materials are placed and compacted with out the addition of any binding agent, using high capacity mechanical plant.
An embankment dam is therefore a non rigid dam which resists the forces exerted up on it mainly by its shear strength. These dams usually provide the most economical and most satisfactory solution for sites at which suitable foundation at reasonable depth may not be available for a dam of concrete or masonry.
The two main forms of embankment dams are (1) Earth (earth fill) dams made predominantly of earth or soil. (2) Rock fill dams made predominantly of quarried rock. However a composite earth and rock fill type of embankment dams are also being widely used.

5.1. Earth Dams (earth fill dams)

An embankment may be categorized as an earth fill dam if compacted soils account for over 50% of the placed volume of material.
Some of the merits of earth dam are:-

- ❖ Local materials is used which is readily available & easy to handle
- ❖ Can be built on almost all types of foundation
- ❖ Large base width of earth dams is suited for pervious foundation
- ❖ Easier and cheaper to extend up wards
- ❖ The natural appearance blends with the surrounding

The disadvantage of earth dams is in that they require greater maintenance than do concrete gravity dams and also because they require a separate spillway for discharging excess flows.

The design of an earth dam involves both a hydraulic and structural analysis. The hydraulic analysis deals with the determination of the seepage patterns and the magnitude of seepage as well as the internal hydrostatic seepage forces for both the dam body and the foundation. Of particular importance is the investigation for possible removal of fine particles near the toe by emerging seepage water (piping). The structural analysis involves the study of the stability of the embankment under the given conditions of seepage and other forces. Settlement and stability studies of the foundation are also important.

5.1.1 Types of Earth Dams

Earth dams may be classified on the basis of methods of construction.
Accordingly we have:-

i) Rolled-fill earth dam
ii) Hydraulic-fill earth dam
iii) Semi-hydraulic fill earth dams

i) **Rolled fill Dam**

In rolled-fill earth dams the embankment is constructed in successive mechanically compacted layers. The material (sand, clay gravel etc) is transported from the borrow pits to the dam site by truckers or scrapers. It is then spread with in the dam section by bulldozers to form layers of 15 to 45 cm thickness. Each layer is then thoroughly compacted and bonded with the preceding layer by means of power operated rollers of proper design and weight.

ii) **Hydraulic fill dam**

In the case of hydraulic-fill dam the materials are transported from borrow pits to their final position (dam site) placed through the agency of water. Thus in this case, at the borrow pits the material is mixed with water to form a slurry which is transported through flumes or pipes and deposited near the faces of dam. The courser materials of the slurry stay near the faces of the dam while the finer ones move towards the center and get deposited there. This would provide a dam section with shoulders of the course free draining particles and an impervious central core of fine grained material such as clay and silt.

iii) **Semi-hydraulic dam**

In the semi-hydraulic fill dam construction, the material is dumped near the upstream and down stream face of the dam to form rough levees as in the case of rolled fill dam w/o the use of water. Then the space b/n the levees are filled with water and the material placed in or upon the levees is washed to wards the center of the dam. For this jets of water are directed on the dumped fill which cause the finer material from the fill near the faces of the dam to be washed away. The finer material moves towards the central portion of the dam and is deposited there thus forming an impervious central core while course material stays near the faces of the dam. However, in the absence of proper jetting action the dumped fill at the faces of the dam may be more dense and impervious than the material immediately below it on the inside of the dam which may result in the failure of the dam.

Out of these three types, the rolled-fill earth dams are the most common. This is so b/c in the case of other two types of dams lack of control in placing the material may result in the failure of the dam.

Rolled Fill dams are of three types

a) Homogenous type
b) Zoned type
c) Diaphragm type

A) Homogenous type: A purely homogeneous type of dam is composed of a single kind of earth material except for the slope protection. It is used when only a single type of material is economically and locally available. Such a section is used only for low to moderately high dams and for dykes. Large dams are rarely designed as homogenous embankments.

For a completely homogeneous section it is inevitable that seepage will emerge on the down stream slope regardless of its flatness and the impermeability of the soil if the reservoir level is maintained for a sufficiently long time. At the down stream slope up to 1/3 of the height may be saturated if internal drainage arrangement are not provided. Besides larger sections (flat slopes) are required to make it stable and safe against piping. Because of this an internal drainage system such as a horizontal drainage layer and a rock toe is added so as to keep the phreatic line well with in the body of the dam. This permits the use of steeper slopes and thus smaller sections.

The material comprising the dam must be sufficiently impervious to provide an adequate water barrier & the slopes must be relatively flat for stability. To avoid sloughing the upstream slope must be relatively flat if rapid draw down of the reservoir is anticipated.
Although formerly very common in the design of small dams, the purely homogenous section has been replaced by a modified homogeneous section in which small amounts of carefully placed pervious materials control the action of seepage so as to permit much steeper slopes. The modified homogeneous section is the one provided with internal drainage filter system in the form of a horizontal drainage blanket or a rock toe or a combination of both.

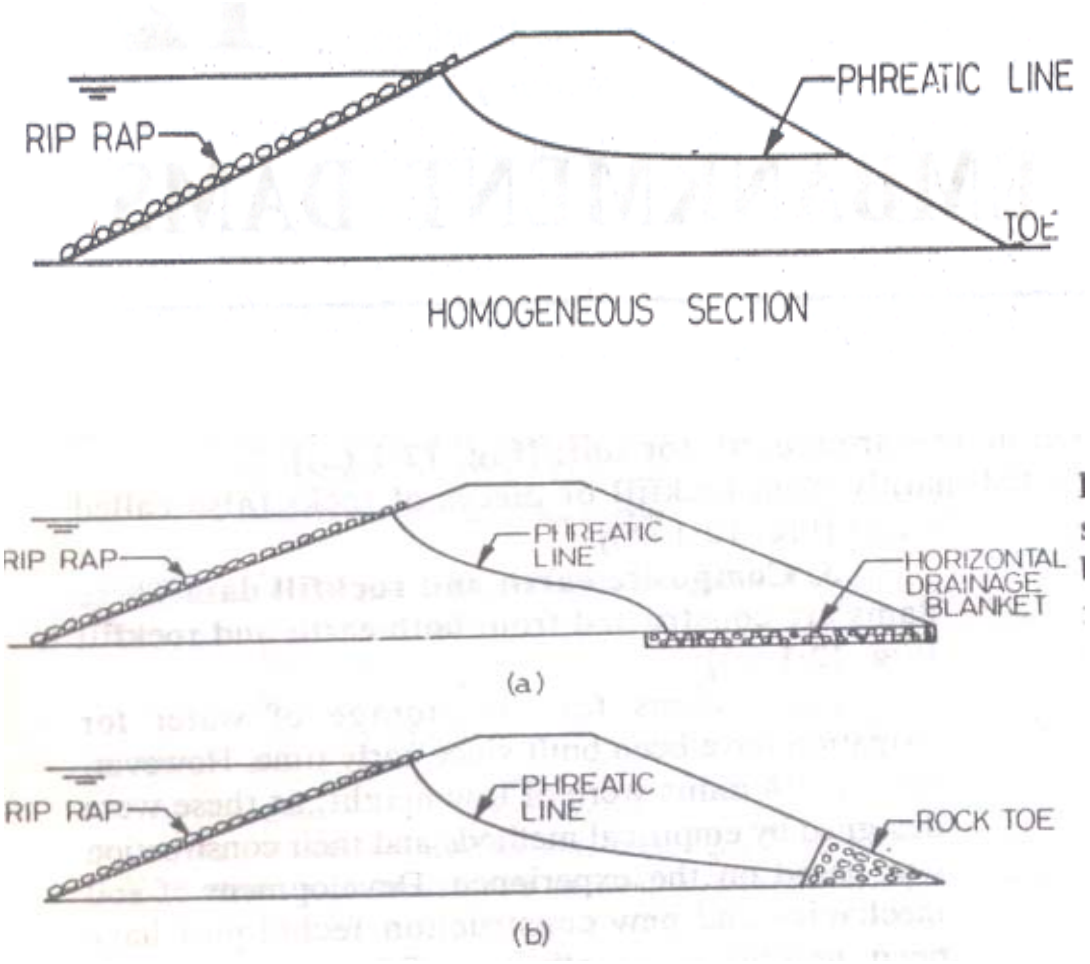


Fig 5.1 Homogeneous dam Section

B. Zoned embankment type: These are the most common for high dams of rolled fill type. They are provided with a central impervious core, covered by a relatively pervious transition filter which is finally surrounded by a more pervious outer zones or shells.
The core thickness should not be less than 3 m or the height of the dam above the section. The central core checks the seepage; the transition filter zone prevents piping through cracks which may develop in the core. The outer zones (shells) provide stability to the core and also distribute the load over a larger foundation area. The core is usually a mixture of clay and sand or gravel or silty clay. Pure clay that shrinks and swells excessively is not suitable .Freely draining materials such as coarse sands and gravels are used as the outer shells. This is necessary b/c the down stream pervious zone should act as a drain to control the line of seepage.
I f a variety of soil are readily available, the choice of type of earth fill dam should always be the zoned embankment type b/c its inherent advantages will lead to economies in cost of construction.

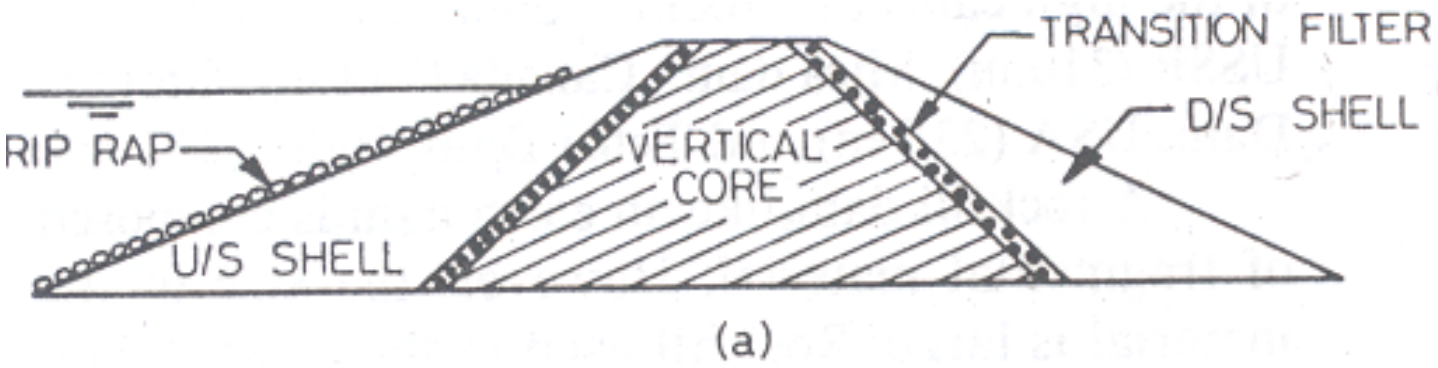


Fig 5.2 Zoned Dam Section

C. Diaphragm type
In this type of section the bulk of embankment is constructed of pervious materials (Sand, grave, or rock) and a thin diaphragm of impermeable material is provided to form the water barrier. The position of this barrier may vary from a blanket on the up stream face to central vertical core. If the diaphragm is provided as an impervious blanket on the u/s face of the dam it needs to be protected against shallow sloughs and slide during draw down and from erosion by wave action. For this the diaphragm is held buried below a thin layer of pervious material over which the up stream slope protection is provided. The diaphragm may be of earth, Portland cement or asphalt concrete or other material.
If the core thickness at any elevation is less than 3m or less than the embankment height above the corresponding section then the dam embankment is considered to be the diaphragm type.

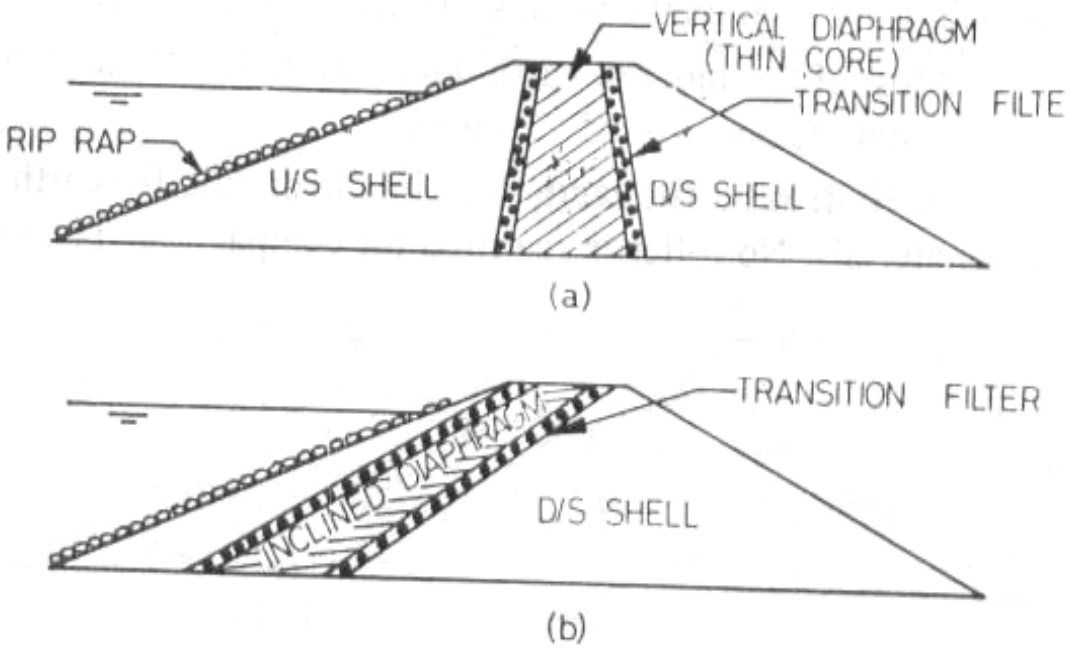


Fig 5.3 Diaphragm type

Foundation Requirements

The essential requirements of a foundation for an earth dam are that it provides support for the embankment under all conditions of saturation and loading and that it provides sufficient resistance to seepage to prevent excessive loss of water. Although the foundation is not actually designed certain provisions for treatment are made in designs to assure that essential requirements will be met.

Foundations are grouped in to three main classes according to their predominant characteristics as rock foundation, Foundation of coarse-grained material (pervious foundation) and foundations of fine grained materials (Impervious foundation).

Impervious foundation Foundations of fine silt and clay are impervious and have very low shear strength. Shear failure may occur in such foundations: If the foundation material is impervious and comparable to the compacted embankment material in structural characteristics, little foundation treatment is required. The minimum treatment for any foundation is stripping of the foundation area to remove the topsoil with high content of organic matter & other unsuitable material which can be disposed of by open excavation. In many cases where the over burden is comparatively shallow the entire foundation is stripped to bed rock.

Rock foundation: - foundations of rock including hard shale do not present any problem of bearing strength for small earth fill dam. The principal considerations are dangerous erosive leakage and the excessive loss of water through joints, crevices, permeable strata and along fault planes. Ordinarily, the design and estimate for a storage dam should provide for the injection of grout under pressure to seal seams, joints & other

opening in the bed rock to a depth equal to the reservoir head above the surface of the bed rock. Grouting is usually done with neat cement and water starting with a ratio of 1:5 pressures usually applied are (0.25 D kg/sq cm) where D is the depth of grouting below the surface.

Pervious foundations: - often the foundations for dams consist of recent alluvial deposits composed of relatively pervious sand and gravel over lying impervious geological formations. Two basic problems are found in pervious foundations. One pertains to the amount of under seepage and the other is concerned with the forces exerted by the seepage.

Quantity of under seepage and seepage forces: - To estimate the volume of under seepage, it is necessary to determine k, by Darcy's formula, the accuracy of which depends on the homogeneity of the foundation and the accuracy with which the coefficient of permeability is determined.

Seepage forces are caused as a result of the friction b/n the percolating water and the walls of the pores of the soil through which it flows. The forces are exerted in the direction of flow and are proportional to the friction loss per unit distance. As the water percolates up ward at the d/s toe of the dam, the seepage force tends to lift the soil resulting in piping.

5.1.2 Design Criteria for earth dams

An earth dam must be safe and stable during all phases of construction and operation of the reservoir. For this the following must be met.

i) The embankment must be safe against over topping during occurrence of the inflow design flood and also by action of wave, by provision of spillway of sufficient capacity and of sufficient free board.
ii) The slope of the embankment must be stable during all stages of construction and under all conditions of operation including rapid draw down in case of storage dam.
iii) The embankment must be designed in such away that it will not impose excessive stress up on the foundation.
iv) Seepage flow through the embankment, foundation and abutments must be controlled so that no internal erosion takes place so that no sloughing takes place where the seepage emerges
v) The upstream slope must be protected against erosion by wave action, and the crest & down stream slope must be protected against erosion due to wind and rain (by growing grass on it and /or putting riprap but not good for aesthetic).
vi) The seepage line should be well with in the down stream face and there should be no opportunity for the free passage of water from the u/s to the d/s face.

5.1.3 Seepage through earth dams

For any dam of homogenous material, seepage will pass trough the dam and appear at the downstream face regardless of the tightness of the material. The line of saturation i.e. the upper boundary of the flow line below which the flow is under hydrostatic pressure is called the phreatic line. If this line of saturation is allowed to intersect the

d/s slope above the toe, a serious sloughing will always occur unless prevented by construction of toe drains or filters or rock fill toe.
Location of the position of the phreatic line is desirable for determination of the amount of through seepage and for construction of the flow nets as it represents one flow boundary.

PHREATIC LINE FOR A HOMGENOUS EARTH DAM WITH A HORIZONTAL DRAINAGE BLANKE

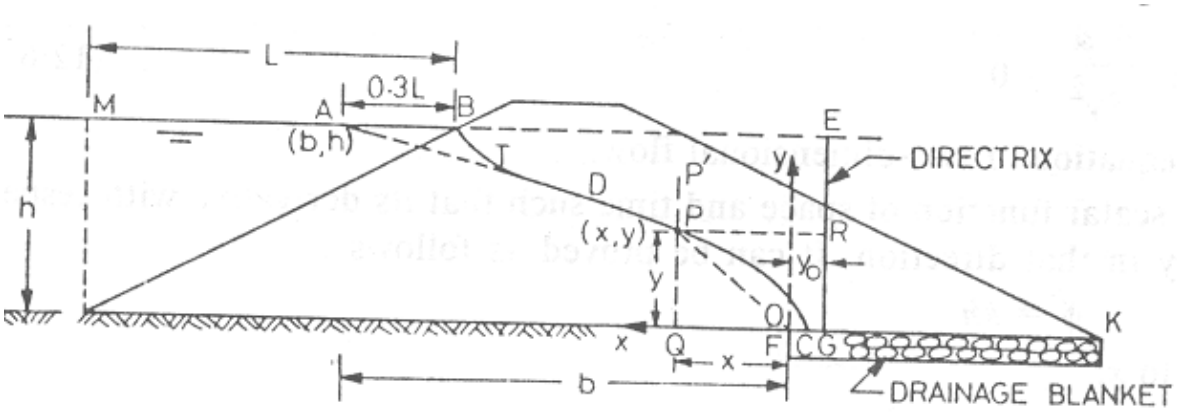


Fig 5.4

Kozeny has shown that the phreatic line in this case coincides with the base parabola ADC except at the entrance. The base parabola has its focus (F) at the starting point of the horizontal drainage blanket & intersects the water surface at A (0.3L from B).
The basic property of parabola which is utilized to draw the base parabola is that the distance of any point P from the focus is equal to the distance of the same point from the directrix.

I) Graphical Method
   i) With center at A & radius AF draw an arc. The arc cuts the line AB when produced at E. Draw a vertical line GE through point E which is the directrix of the base parabola.
   ii) The intermediate points are located by utilizing the above mentioned basic property of the parabola.
   iii) Join all intermediate points by a smooth curve. The last point C on the parabola will be midway b/n F & G as FC = CG

II) Analytical Method

Let us take the origin (O) at the focus with X- positive u/s & Y-positive up ward.

From the basic property of parabola: distance PF = distance PR

$$\sqrt{x^2 + y^2} = x + y_o \quad \text{.....(i)}$$

Squaring both sides & rearranging

$$y = \sqrt{2xy_o + y^2{}_o} \quad \text{.....(ii)}$$

Where y_o is the distance from the focus to the directrix, also called focal distance (f) Thus FG = Y_o. The value of Y_o can be obtained from the known coordinates (X = b, Y = h) of the starting point A.

Substituting the values in equation (i) we have $y_o = \sqrt{b^2 + y^2} - b$

For different values of X the corresponding values of Y can be computed using the parabola equation (II). The intermediate points are plotted from the known coordinates to obtain a smooth curve.

As the actual phreatic line starts from point B and not from point A, a short transition entry correction curve is sketched free hand by eye judgment such that it is perpendicular to the u/s face & meets the basic parabola tangentially.

After the phreatic line is determined, then it is also possible to draw the flow net, considering the phreatic line as the first/top most flow line. And thus, one can compute the quantity of seepage through dam. Discharge through dam can also be quantified using analytical approach for this particular situation.

From Darcy's Law:

$$v = ki$$

k = Coefficient of permeability
i = the hydraulic gradient

The the seepage discharge per unit length can be given by:

$$q = vA = kiA$$

Where A is area of flow per unit length

For steady flow, the discharge through all vertical planes across the dam section will be the same.

Considering the discharge through the vertical section PQ passing through the point P (x.y),

$$q = kiA$$

$$q = k\left(\frac{dy}{dx}\right)(y \times 1)$$

Substituting for the value of y from above,

$$q = k\frac{d}{dy}\left(\sqrt{(2xy_o) + y_o^2}\right)\left(\sqrt{(2xy_o) + y_o^2}\right)$$

$$q = k\left(\frac{y_o}{\sqrt{2xy_o + y_o^2}}\right)\left(\sqrt{(2xy_o) + y_o^2}\right) = q = ky_o$$

PHREATIC LINE FOR A HOMOGENEOUS DAM SITH OUT ANY DRAINAGE SYSTEM

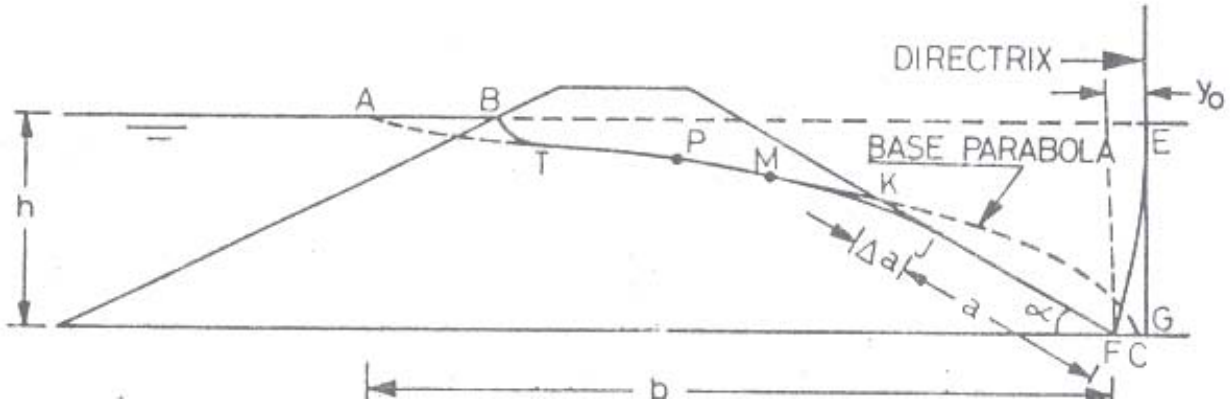


Fig 5.5

In this case the phreatic line cuts the d/s faces at point J above the toe. In addition to the entry correction an exit correction is also required.
The focus (F) of the base parabola is located at the d/s toe of the dam & its starting point A is located at a distance of 0.3L from B.
Casagrande has shown that the exit correction ((Δa) depends up on the slope α of the discharge face and he has given the values $\Delta a/(a+\Delta a)$ for different values of angle α as follows.

$\frac{\Delta a}{a+\Delta a}$	0.36	0.32	0.26	0.18	0.14	0.10	0.00
α	30°	60°	90°	120°	135°	150°	180°

Fig 5.6

The value of (a+ Δa) may be obtained from the figure by measuring FK from the toe after the base parabola has been drawn or by using the following polar equation of the parabola.

$$a + \Delta a = \frac{y_o}{1 + \cos\alpha} \quad \text{Where } y_o = \sqrt{b^2 + h^2} - b$$

Then the point J is marked at a distance Δa from point K by drawing a smooth transition curve by eye judgment such that the phreatic line meets the d/s face tangentially.
To determine discharge, the same concept as above can be adopted.

Variation of the slope angle α α is measured clockwise from the horizontal. The above equation for fig 5.4, cannot be applicable for determining $\Delta a/(a + \Delta a$ for $\alpha <$ $30^{0.}$ Approximate analytical solutions for such case are:

i) Approximate analytical solution for the determination of the distance a for the slope angle $\alpha <$ 30^0

For the slope angle α $< 30^{0,}$ Sceffernak and Van Iterson gave the analytical solution for the determination of the distance a. In this case it is assumed that the hydraulic gradient (i) is equal to the tangent of the angle.

$$\text{Thus} \qquad i = \frac{dy}{dx} = \tan\alpha$$

This assumption is true for small values of the angle α

A vertical line is drawn through the point J to cut the base at J' (Fig). The discharge through the vertical section JJ' per unit length is given by:

$$q = kiA = k\left(\frac{dy}{dx}\right)y$$

$$or \;\; q = k(\tan\alpha)(a\sin\alpha) \quad from\, which\, one\, can\, obtain,$$

$$a\,\sin\alpha\tan\alpha\,dx = y\,dy$$

$$Integrating\, b/n\, the \lim its\,(x = a\cos\alpha,\; and \;\; y = a\sin\alpha)\, to\; (x = b \;\; and \;\; y = h),$$

$$a\,\sin\alpha\tan\alpha\int_{a\cos\alpha}^{b} dx = \int_{a\sin\alpha}^{h} y dy$$

$$a\,\sin\alpha\tan\alpha(b - a\cos\alpha) = 1/2(h^2 - a^2\sin^2\alpha)$$

Simplifying,

$$a^2 - \frac{2ab}{\cos\alpha} + \frac{h^2}{\sin^2\alpha} = 0$$

$$a = \frac{\left(\frac{2b}{\cos\alpha}\right) \pm \sqrt{\left(\frac{2b}{\cos\alpha}\right)^2 - \frac{4h^2}{\sin^2\alpha}}}{2}$$

$$a = \frac{b}{\cos\alpha} - \sqrt{\frac{b^2}{\cos^2\alpha} - \frac{h^2}{\sin^2\alpha}}$$

ii) ***Approximate analytical solution for the determination of the distance a for the slope angle30^0< α < 60^0***

An approximate value of the distance JF (= a) can be found from the approximate analytical solution as given below.

Casagrande suggested that in this case the hydraulic gradient should be taken as $\sin\alpha$ instead of tan α .That is it should be taken as (dy/ds) instead of (dy/dx), where s is the distance measured along the phreatic line.

Thus the discharge per unit length is given by:

$q = kiA = k(dy/ds)(y \times 1)$

$q = k(\sin\alpha)(a\sin\alpha)$

$q = ka\sin^2\alpha$

Thus $ka\sin^2\alpha = k(dy/ds)y$

$a\sin^2\alpha\, ds = ydy$

Integrating b/n the limits $(s = a \text{ and } y = a\sin\alpha)$ *to* $(s = S_o \text{ and } y = h)$

Where So = is the total length of the phreatic line,

$$a\sin^2\alpha \int_a^{S_o} ds = \int_{a\sin\alpha}^{h} ydy$$

$$\text{or } a\sin^2\alpha(S_o - a) = \frac{h^2 - a^2\sin^2\alpha}{2}$$

$$\text{or } a^2 - 2aS_o + h^2/\sin^2\alpha$$

$$a = So - \sqrt{S_o^{\,2} - h^2\cos ec^2\alpha}$$

For a slope angle α up to $60^{0,}$ the length of the phreatic line So can be taken approximately equal to the straight distance FA = $\sqrt{h^2 + b^2}$, thus the above equation becomes

$$a = \sqrt{b^2 + h^2} - \sqrt{(b^2 + h^2) - h^2 \cos ec^2 \alpha}$$

$$a = \sqrt{b^2 + h^2} - \sqrt{(b^2 - h^2(-1 - \cos ec^2 \alpha)}$$

$$a = \sqrt{b^2 + h^2} - \sqrt{(b^2 - h^2(\cot^2 \alpha)}$$

After a is determined, the discharge q can be calculated from above equation.

PHREATIC LINE FOR HOMOGENEOUS ERATH DAM WITH ROCK TOE

The figure below shows an earth dam with rock toe. The u/s face of the rock toe is usually inclined downstream i.e., $\alpha > 90^0$ as shown in figure below (fig 5.7). However, sometimes the u/s face of the rock toe is kept vertical. The drawing procedure for phreatic line is the same as previous cases but the exit correction is somewhat d/t.

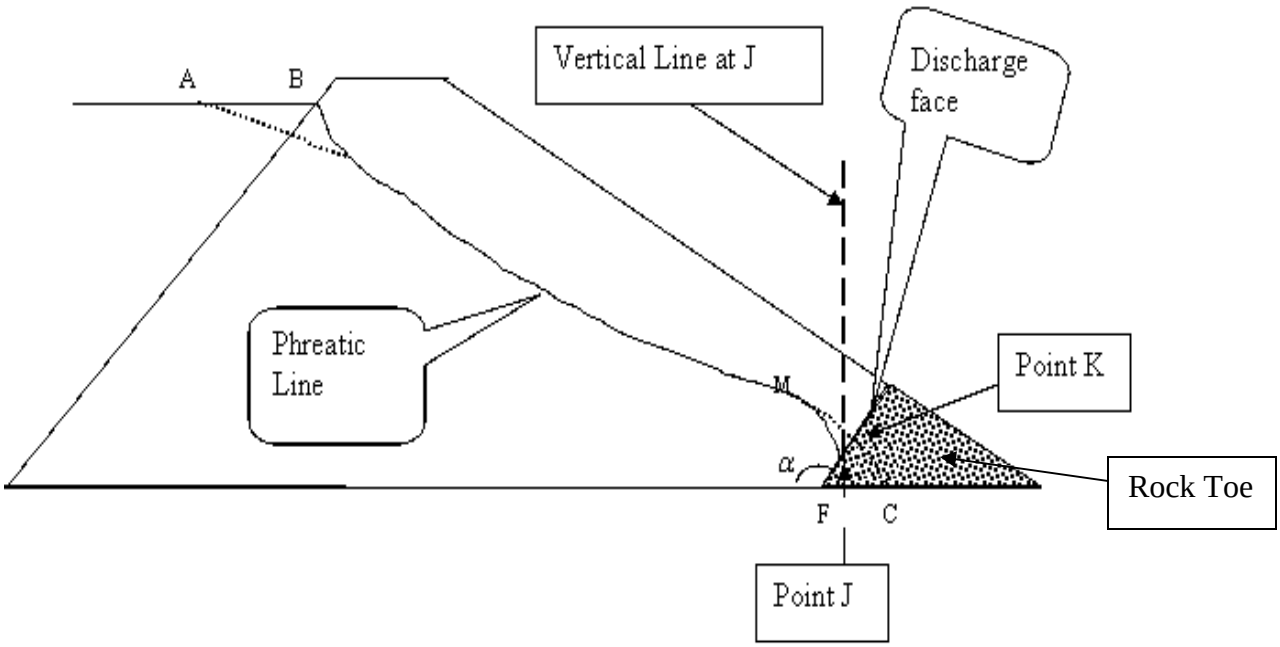


Fig 5.7 Homogeneous Earth Dam with Rock toe

Exit Correction of Phreatic line for earth dam with rock toe

The u/s face of the rock toe acts as the discharge face. It makes an angle α with the horizontal. The base parabola cuts the discharge face at point K. The value of $(a + \Delta a)$ may be obtained by measuring the distance FK or from equation given above for a given value of α and calculated value of y_o

For the known value of α, the value of $\Delta a/(a+\Delta a$ can be obtained from fig 5.6. The exit correction, Δa, is then found from the values of the equations. The joint J is then marked on the u/s face of the rock toe at a distance of Δa from K. The phreatic line is drawn by making a transition curve by eye judgment from the point J to the point M on the base parabola. The transition curve is tangential to the vertical line at J, and it meets the base parabola tangentially at M. It may be noted that the phreatic line drops vertically in to the rock toe.

PHREATIC LINE FOR ZONED ERATH DAM WITH A CENTERAL CORE

For zoned earth dam the phreatic line construction depends mainly on the geometry and thickness of the core section. B/c the permeability of shell material (example sand) is quite large as compared to core material (example clay), the effect of outer shells on the phreatic line in the core is negligible. As such the u/s shell has practically no effect on the position of the phreatic line. The phreatic line can just start from point B where the extension of the water level cuts the core. The d/s shell in this case acts as a drain. Thus the usual practice is to draw the phreatic line for the core section only.

For drawing the phreatic line, the focus F is to be located at the d/s toe of the core and assuming the u/s shell as a reservoir. Then the core section is treated as a homogeneous dam and the same of constructing phreatic line for homogeneous dam is considered.
However, in this case, the phreatic line at the exit end will be slightly above the base of the core so that the seepage water can flow under gravity through the d/s shell which acts a drain (fig 5.8).

Flow net and Flow net construction (assignment to read)

Control of Seepage through earth dam and its foundation

The effects of seepage are:

- Loss of water
- Piping failure
- Reduce slope stability and result in dam failure
- Causes local sloughing

Seepage control measures are divided in to two main categories

A. Measures to reduce quantity of Seepage – lessen loss of water
B. Measures for Safe drainage of seeping water – No piping

Measures to reduce quantity of Seepage

IN DAM:

The only measure is provision of impervious core within the body of the dam called embankment zonation.

Embankment zonation : -for reducing seepage through the body of the dam, a core of impervious material such as silt clay or clayey silt is generally provided. For most effective control of through seepage and seepage during reservoir draw down, the permeability should progressively increase from the core out towards each slope.
Sometimes flat slopes without drains are provided for some dams constructed with impervious soils having flat embankment slopes & infrequent, short duration , high reservoir levels, the phreatic surface may be contained well with in the d/s slope & escape gradients may be sufficiently low to prevent piping failure.

IN FOUNDATION:

If the foundation consists of alluvial deposits of pervious sand and gravel with impervious stratum at a great depth measures to be adopted are cut-off, u/s impervious blanket and d/s berm.

Cutoff trenches: This is the most positive means of controlling the amount of seepage and insuring that no difficulty will be encountered by piping through the foundation or by uplift pressure at the down stream toe. Whenever economically possible, seepage through a pervious foundation should be cut off by a trench extending to bed rock or other impervious stratum.

Partial cutoff trenches The partial cut offs are effective only when they extend down into an intermediate stratum of lower permeability. This stratum must be continuous across the valley foundation to ensure that three- dimensional seepage around discontinues stratum does not negate the effectiveness of the partial cutoff. Such type of measures is effective when the foundation material is stratified alluvial deposit.

Sheet pilling cut offs: - A steel sheet pile cut off consists of interconnected sheet piles to form a continuous impervious barrier. It is relatively expensive and leakage through the interlocks b/n the individual piling is considerable. It is used occasionally

in combination with a partial cutoff trench as a means of increasing the depth of the cutoff. Sheet piling cutoffs are practically limited to use in foundations of silt, Sand and fine gravel.

Slurry trench cutoff: - is an important method of constructing positive cut offs when wet conditions or deep cutoffs in alluvial valleys make conventional constriction methods uneconomical.

Grouting- The stability and impermeability of pervious overburden foundation can be improved by injection a substance which will act as a binder & fill the voids. Cement or cement -clay grouts are usually used to treat foundations over lain by coarse alluvial material.

Upstream blankets: - The path of percolation in pervious foundations can be increased by the construction of a blanket of impervious material connecting with the impervious zone of the dam and extending upstream from the toe. Blankets are usually used when cutoffs to bed rock or to an impervious layer are not practicable b/c of excessive depth; they are also used in conjunction with partial cut off trenches. The length of the blanket will be governed by the desired reduction in the amount of under seepage and its thickness usually varies from 1.5 to 3.0 m.

It may be provided in homogenous dams constructed of relatively impervious soil. The length of the u/s blanket can be obtained from the following formula.

$$l = \frac{khd - pqb}{pq}$$

Where k = mean horizontal permeability coefficient
h = the gross height
p = percentage (stated as decimal) of flow under dam w/o a blanket to which level it is desired to reduce the seepage by meanse of a blanket
b =length of impervious dam material
q = k(h/b)xdx1 seepage flow under the dam (approximate)
d = depth of pervious foundation

q’ = pq

Where q’ is the seepage quantity after provision of u/s blanket

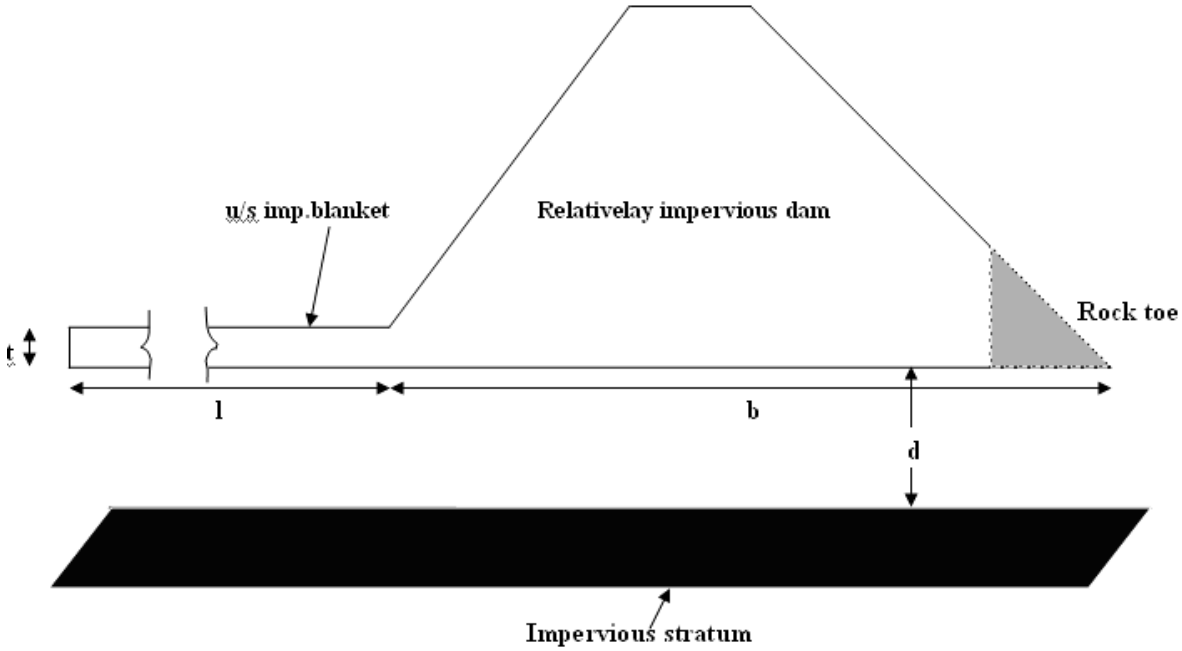


Fig 5.8

Measures for Safe drainage of seeping water – No piping

Some water always seeps through the dam and foundation even after adopting several seepage control measures discussed earlier. The seeping water should be safely drained through the dam and foundation without causing ill effects.

IN DAM:

Horizontal drainage blanket	Rock toe
Chimney drain	Strip drain

i) ***Horizontal drainage blanket***: Vertical (or inclined) and horizontal drains
Because of the often variable characteristics of borrow materials, vertical (or inclined) and horizontal drains with the down stream portion of the embankment are provided to ensure satisfactory seepage control. Also the vertical (or inclined) drain provides the primary line of defense to control concentrated leaks through the core of an earth dam.
Horizontal drainage blankets are commonly used for earth dams of moderate heights. The blanket extends from the d/s toe for a distance of about three times the height of the dam but not longer than 2/3 of the base width. In the case of zoned section it extends up to the core.
The blanket should consist of a pervious material which should quickly drain the seeping water and the layers of the filter should fulfill the filter criteria to avoid migration of materials/ soil particles. The main disadvantage of such a measure is that

it is not effective if the embankment material has stratifications and the horizontal permeability is greater than the vertical permeability.

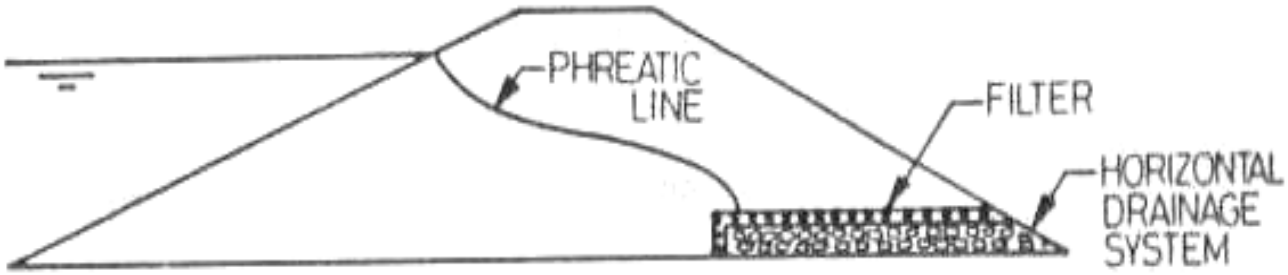


Fig 5.9

ii) ***Rock toe***: The rock toe is provided at the d/s toe of the earth dam and it forms part of the dam. It consists of stones of size varying from 15 to 20 cm. The u/s face of the rock toe may be vertical or inclined. A graded filter may be provided between the rock toe and the soil mass as well as b/n the foundation and the rock toe to avoid migration of materials.
The rock toe is suitable for low to moderate height of dams. The height of the rock toe is generally b/n H/3 to H/4, where H is the height of the dam. Rock toe can also be used in conjunction with horizontal drainage blankets.

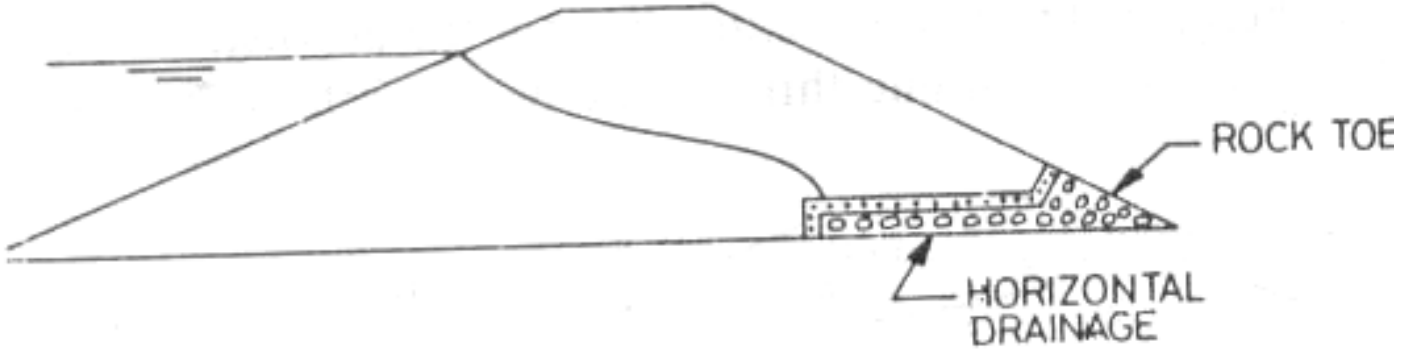


Fig 5.10

iii) ***Strip drain***: A strip drain is provided instead of a horizontal drainage blanket if there is scarcity of pervious materials in the area. Transverse drains are provided to carry water from the strip drain to another parallel drain located at the d/s toe of the dam (refer figure).

As far as possible a horizontal drainage blanket is preferred to a strip drain. If there is chocking of an individual transverse drain in the strip drain system, a significant length of the d/s face of the dam would become un-drained and there may be sloughing.

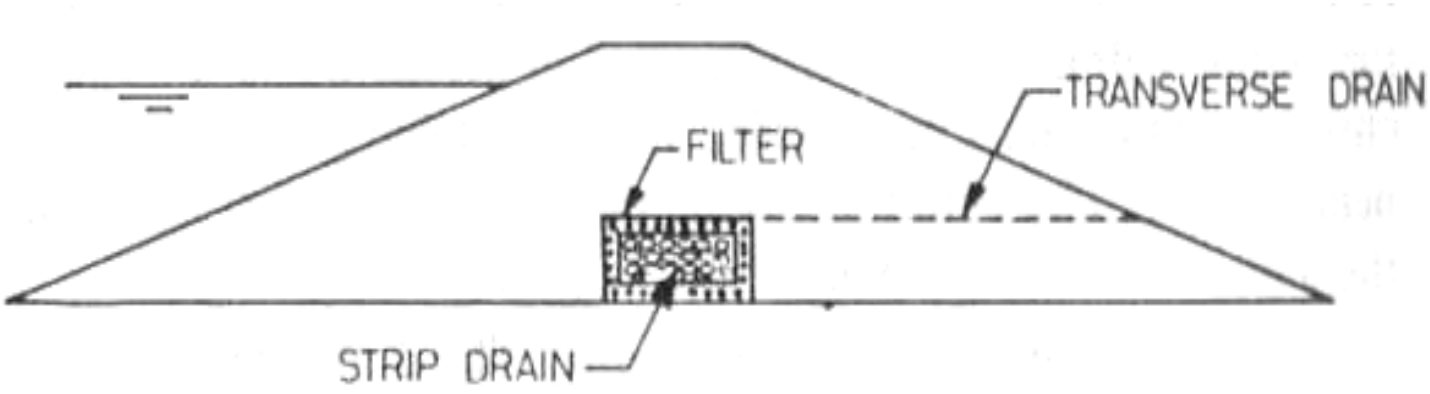


Fig 5.11

iv) ***Chimney drain***: A chimney drain is a vertical/nearly vertical drain located inside the dam so that it intercepts all layers of the dam in the seepage zone. Thus it is helpful in stratified embankments. A chimney drain renders the d/s portion of the dam free from seeping water and it increases the stability of the d/s slope. It also helps in reducing the p[ore water pressure during construction and sudden draw down condition.

Chimney drains are rarely provided in homogeneous dams and they are provided d/s of the impervious core in zoned earthfill dams. From the chimney drain water is carried to d/s by a horizontal drainage blankets. The chimney drain should be accompanied with proper filters.

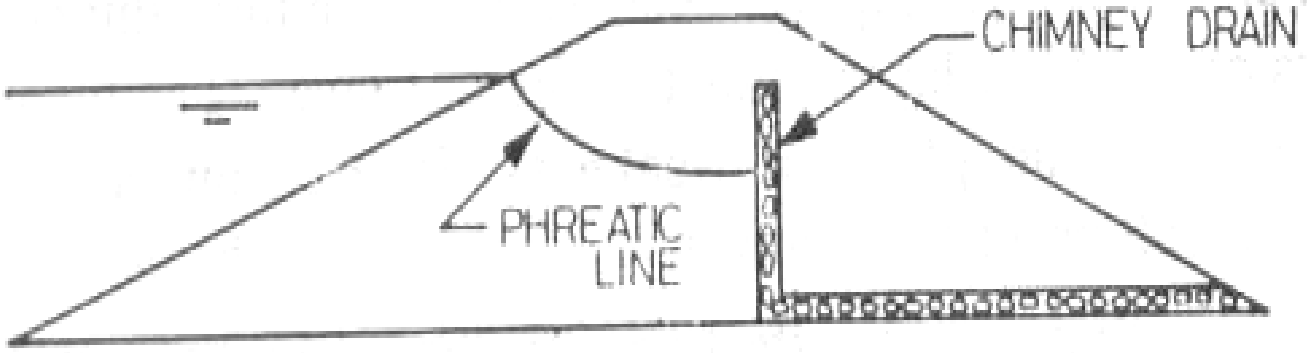


Fig 5.12
Chimney Drain

IN FOUNDATION:

In general, the drainage systems provided for the dam can also serve for the purpose of drainage of foundation. Additional measures such as mentioned below can be taken in to account.

Toe drains	Drainage trenches
Relief Wells	Vertical Sand Drains

i) Toe drain and drainage blanket: The purpose of toe drains is to collect the seepage water from the horizontal drainage blanket (construction in conjunction with it –fig) and foundation to carry it to an outfall pipe which then discharges the water in to the river or spillway stilling basin.

Toe drain pipes are usually of verified clay or perforated asphalt dipped corrugated metal pipes placed in trenches excavated to the required depth below the ground surface to ensure effective interception of seepage flow. Filters are provided as usual.

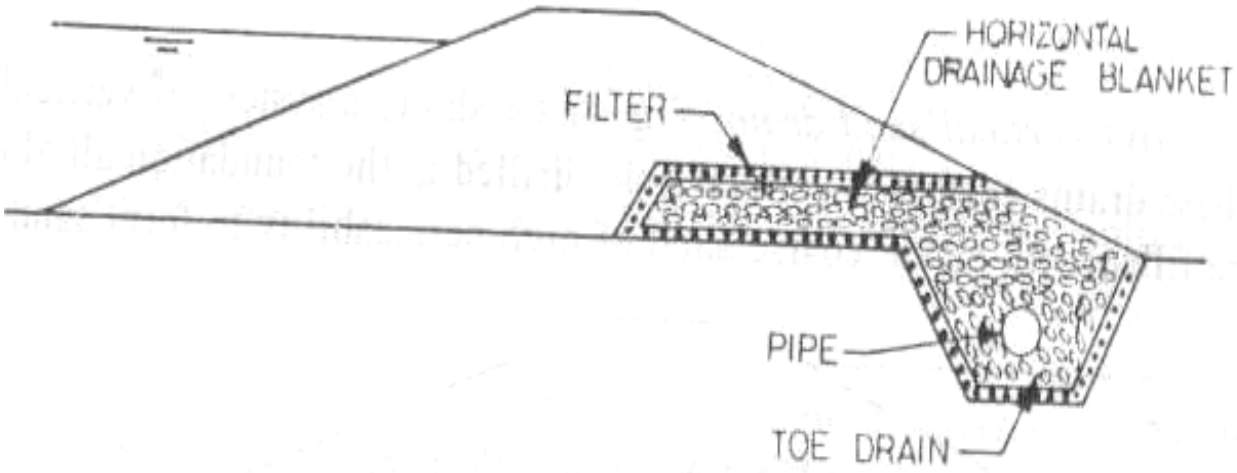


Fig 5.13 Toe drain

ii) Drainage trenches: Drainage trenches are used when a thin impervious top stratum overlies a shallow pervious stratum of the foundation so that the trench can be built to penetrate the pervious stratum substantially. They are similar in arrangement with that of toe drains, but in this case there is no pipe provided to drain. The trenches are excavated to the required depth and backfilled with properly graded layers in accordance with the filter criteria such that the coarser materials are at the inner side. Drainage trenches, however, are not effective if the underlying pervious stratum is quite deep and stratified.

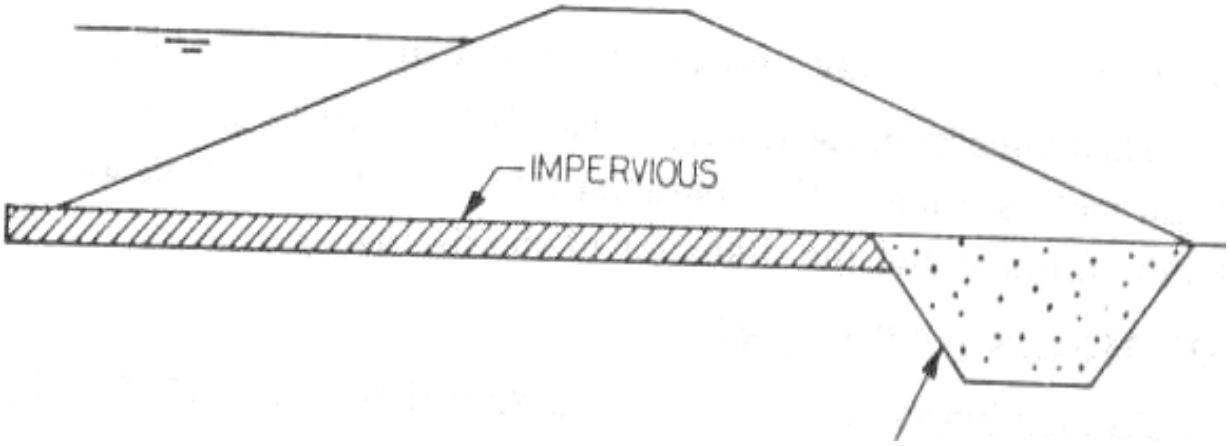


Fig 5.14

iii) Relief Wells: Are generally used for drainage of the foundation if it consists of a deep pervious stratum which is stratified and whose permeability increases with depth. Relief wells are provided at or near the d/s toe of the dam to collect water seeping through the foundation and to reduce the pore pressure in the foundation.

Relief well consists of an interior perforated pipe or a well screen with a minimum diameter of 15 cm. The well screen is surrounded by a small thickness of gravel pack which is properly graded so as to meet the filter criteria for the surrounding soil. (Fig.). Seepage from the relief wells is usually discharged at the toe of the dam in the river channel through the horizontal overflow pipe and a lined drainage ditch. The spacing of the relief wells is usually b/n 15 to 30m.

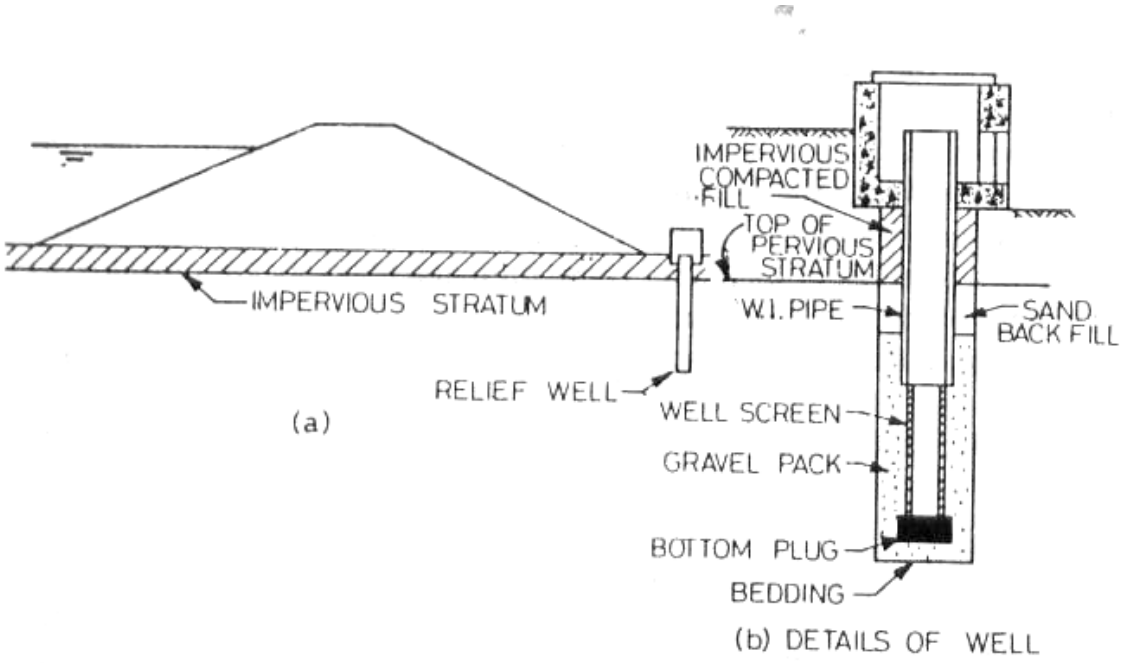


Fig 5.15 Relief Wells

iv) Vertical Sand drains: These drains consist of vertical holes drilled in the foundation all along the base of the dam. These holes are filled with clean, course sand of high permeability to form sand columns (Fig 12). These sand drains reduce the path of drainage in the horizontal direction and help in the drainage of the foundation. They also accelerate consolidation of the foundation soil by providing drainage.

Vertical sand drains are quite effective for the drainage of soft clay foundation which can not be easily drained by other methods. Vertical sand drains can also act as relief wells and thus help in reducing the pore pressure and controlling under seepage.

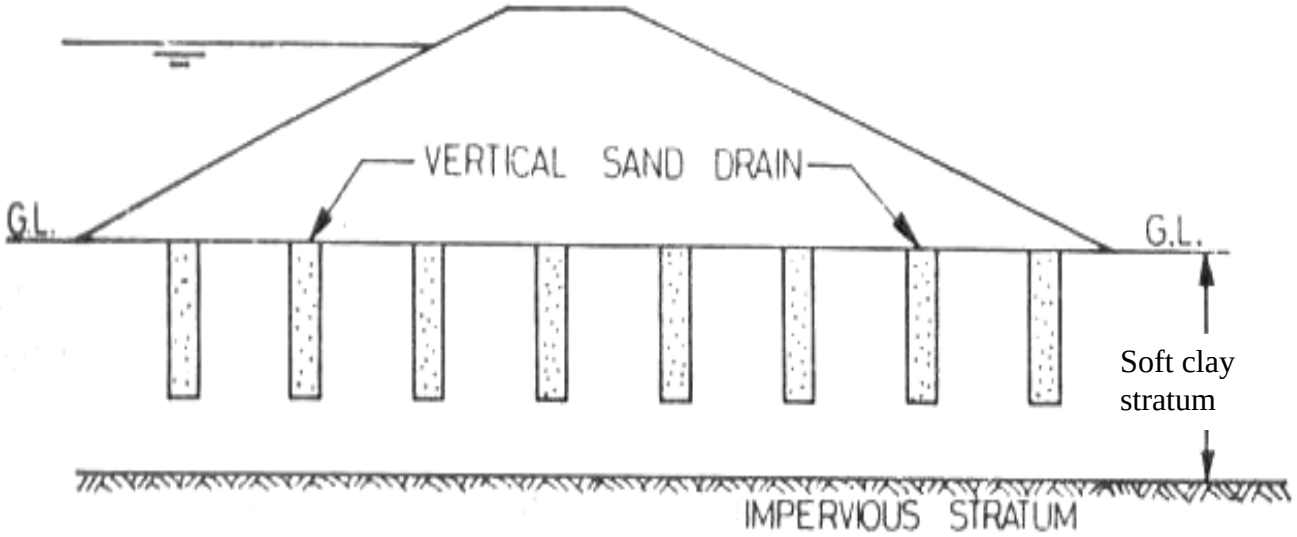


Fig 5.16 Vertical Sand drains

5.1.4 Embankment Design

The design of an earth dam essentially consists of determining such a cross sections of the dam which, when constructed with the available materials, will fulfill its required function with adequate safety.

Thus there are two aspects of the design of an earth dam, viz.

i) To determine the X-section of the dam and
ii) To analyses the stability of the proposed x-section.

For determining the X-section of an earth dam there are no mathematical analyses or formula as in the case of rigid dams.

The design of slopes of earth fill embankments depends on the nature of the materials of construction and the type of dam (i.e. zone, homogeneous & diaphragm). It depends on the nature of the material used for the core and the shells and in the case of zoned embankment on the relative proportion of them.

Details of Earth Dams

i) crest width:- The crest width of an earth fill dam depend on several considerations such as:-

- Sufficiency to keep the top flow line well with in the dam body when the reservoir is full.
- sufficiency to provide the embankment mass for resistance to earth quake shock
- Satisfactory for secondary requirement such as minimum road way width.

Crest width can be calculated using the following formula:

H/5 + 3	for very low dams
$0.55\sqrt{H} + 0.2H$	for dams lower than 30m
$1.65(H+1.5)^{1/3}$	for dams higher than 30m

Where: - H is the height of the dam.

ii) **Freeboard**: Sufficient free board must be provided so that there is no possibility what so ever of the dam being over topped. The necessary free board is calculated by assuming that the maximum flood will occur when the reservoir is full and that the highest possible waves will develop at the same time .The minimum free board shall be 1.5 times the wave height plus a safety factor.

Freeboard = 1.5 h_w + additional safety provision

The additional safety provision generally various from 0.6 to 3m depending up on the size of the reservoir, the height of the dam the reliability of the flood computation etc. The free board should not be less than 2m in any case.

iii) **Embankment slopes**: - The design slopes of an embankment may vary widely depending on the character of the materials available for construction, foundation conditions and the height of the structure. The upstream slope may vary from 1:2 to as flat as 1:4 (V:H) for stability usually it is 1:2.5 or 1:3. Flat upstream slopes are sometimes used in order to eliminate expensive slope protection.

The usual down stream slopes for small earth fill dams are 1:2 where a down stream pervious zone is provided in the embankment and 1:2.5 where the embankment is impervious. Theses slopes are stable for soil types commonly used when drainage is provided in the design so that the down stream slope of the embankment does not be come saturated by seepage.

The table below shows slopes recommended by Terzaghi.

Type of Section	Type of Material	u/s slope (V:H)	D/s slope (V:H)
Homogeneous	Homogeneous, well graded material	1:2.5	1: 2
Do	Homogeneous, coarse silt	1:3	1:2.5
Do	Homogeneous clay or silt clay		
	i) < 15m height	1:2.5	1:2
	ii) > 15m height	1:3	1:2.5
Zoned	Sand or sandy gravel with silty core	1:3	1:2.5
Zoned	Sand or sandy gravel with R.C. core	1:2.5	1:2

Source: Arora (2000), Page 377

iv. Slope protection: - usual types of surface protection for the upstream slope against destructive wave actions are riprap and concrete pavement. The upstream slope protection should extend from the crest of the dam to a safe distance below minimum water level and ordinarily terminate on a supporting berm.

Usual type of surface protection of the upstream slope is stone riprap, either dumped or hand placed. For thin layers, hand placed riprap may be economical than dumped riprap.

V. Surface drainage: - may be necessary to prevent gull eying at the contact of the embankment and valley slopes. This is done by providing a gutter concrete or stone pavement.

Vi. Filters: - filers are always provided between any two dissimilar materials when the difference in their particles is so great that the particles of the finer material can migrate in to the voids of coarser material with seepage water & can cause piping.

There fore filters are provided:-

- B/n the drainage system & the adjoin soils to prevent the migration of the soil particles in to the drains.
- B/n impervious zones (cores) of fine-grained soils and the pervious zones (shells) of the coarse -grained soils for the same purpose.

Criteria for filter design according to USBR

- $\frac{\text{D15 of the filter}}{\text{D15 of base material}} =$ **5 to 40** provided that the filter does not contain more than 5 percent of material finer than 0.074 mm

- $\frac{\text{D15 of the filter}}{\text{D85 of base material}} = 5$ or less
- $\frac{\text{D85 of the filter}}{\text{max. Opening of drain}} = 2$ or more
- The grain size curve of the filter should be roughly parallel to that of the base material

Where D15 is the grain size with 15 percent of the total soil practices are smaller& D85 is the grain size wit 85% of the total soil particles are smaller.

If more than one filter is used, the preceding layer is considered as the base material and the succeed in layer as the filter and the same rules above are applied.

Vii**) Cores:-** the core may be defined as a membrane built with in an embankment dam to form the impermeable barrier, the balance of the dam being provided to ensure stability. It may be of natural materials clay, gravels etc. or prepared materials such as cement or asphaltic concrete or of metal, plastic, rubber etc.

The thickness of the core will depend primarily on the martial available. A general core thickness is one half of the height of the dam depending on materials available. Thin cores may be adequate for impermeability but it is essential to provide well designed filters on either side.

5.1.5 Slope Stability Analysis

Failure of an embankment dam can result from instability of either the upstream or down stream slopes. The failure surface may lie with in the embankment or may pass through the embankment and the foundation soil. The critical stages in u/s slope are at the end of

construction & during rapid draw down. The critical stages for the d/s slope are at the end of construction & during steady seepage when the reservoir is full.
Stability of an embankment is determined by its resistance to shearing stresses that may result from external loads (such as reservoir pressure & earth quake) and internal or body force.
Various methods of slope stability analysis are available but the Swedish slip -circle or slices method is the most common. In this method the factor of safety against sliding is defined as the ratio of resisting moments to actuating moments (or the ratio of resisting forces to actuating forces). The forces are computed taking a trial slip surface and dividing the soil mass above the assumed failure surface in to some number of slices. The potential surface of rupture is taken to be any cylindrical surfaces.

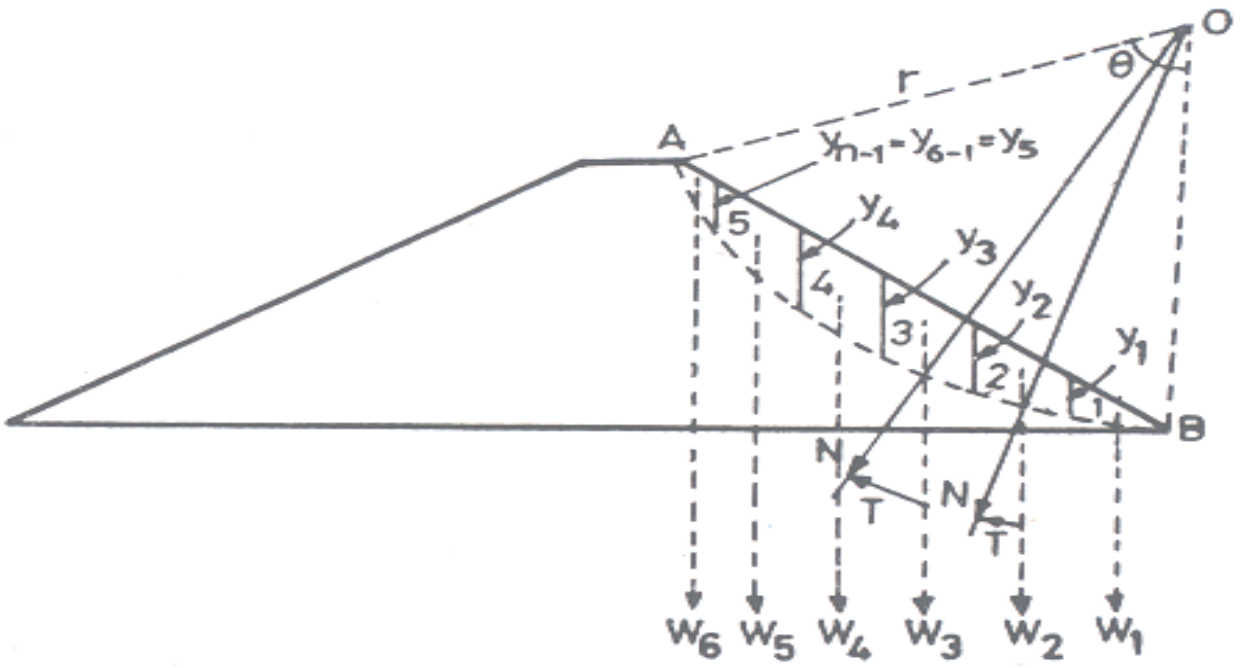


Fig 5.17 Slices (methods of slicing for computing F.S)

The figure shows a trial slip surface AB which is circular; the radius of the circle, r and its center o.
The trial failure wedge is divided in to vertical slices (or strips) by drawing vertical lines. The slices are usually of equal width, but not necessary so. The number of slices is usually kept 5 to 15. The greater the number the more is accuracy. It is convenient to have all the slices of equal width b, but not necessarily. In the case of the zoned section, the slicing should be in such a way that the base of each slice is resting on only one type of the material.

Procedure for Analysis
After proper slicing of an assumed failure surface and let us consider the equilibrium of any one slice and forces acting on it. It is subjected to the following forces (slice 4).

- The driving force is the tangential component of the weight

1. The weight W_4 acting vertically downwards through its center of gravity.
2. Cohesive for C_4 along the curved surface, in the direction opposite to the direction of probable movement of the wedge. The cohesive force C_4 is equal to $c\Delta L$, where c is unit cohesion and ΔL is the length of the base of the strip.
3. Reaction R_4 at the base of the slice, acting along the line inclined at angle ϕ to the normal, where ϕ is the angle of the searing resistance of the soil.It is is assumed that the slippage is imminent and the full shear strength of the soil has been mobilized. Therefore, the inclination of the reaction with the normal is equal to ϕ.
4. The soil reactions E_R and E_L acting on the vertical sides of the slice exerted by the adjacent slices on the right and left slices respectively.

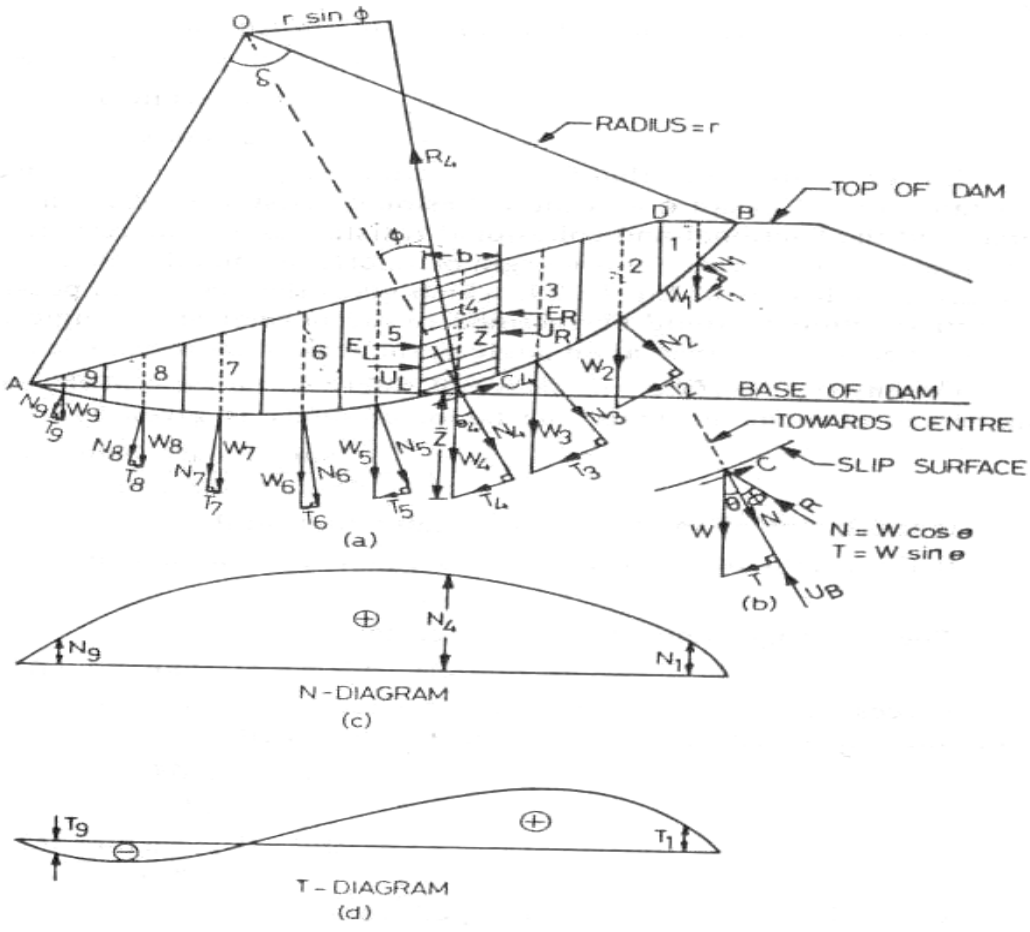


Fig 5.18 Slicing and consideration of forces and the direction of application (SWEEDISH CIRCLE METHOD)

5. The forces due to pore water pressure U_L, U_R and U_B acting on the left face, right face and at the base respectively. The force U_B acts in the normal direction.

In the conventional Swedish circle method, it is assumed that the reactions E_R and E_L are equal and opposite in direction and cancel each other. Like wise the U_L and U_R. Thus there are four forces which are normally considered in the analysis viz, W_4, C_4, R_4 and U_B.

Resolving the Vector W in to its tangential and Normal components by drawing a perpendicular from the tip of the vector W to the normal direction,

$$T = W \sin\theta \quad \text{and} \quad N = W \cos\theta$$

Where θ, is the angle which the normal makes with the vertical

Thus resolving all the forces in the normal and tangential direction,

$N - U_B = R\cos\phi$ In normal (radial) direction
$T - C = R\sin\phi$ In tangential direction

This implies that $T - C = (N - U_B)\sin\phi / \cos\phi$
$T = (N - U_B)\tan\phi + c\Delta L$

T is the driving force which is counterbalanced by the frictional resistance ($N - U_B$) and cohesion resistance ($c\Delta L$). When the forces just balance the failure is imminent. For margin of safety, the soil reaction would be inclined to the normal at angle less than the angle ϕ. As soon as the reaction becomes inclined at angle ϕ, the maximum resistance is developed and the failure is imminent.

On any trial surface, the safety factor against sliding is given by:

$$F.S = \frac{\text{Re}\,sisting\ \ force}{Driving\ \ force}$$

The resisting force is the internal frication resistance plus cohesion, if any
i.e. Resisting force = (N-$\overline{U}$) tanϕ + c Δ L and T is the driving force
Where: - N= Normal force along the arc

$\overline{U}$ = pore water pressure force at the base of each slice = U_B
C and ϕ are cohesion & internal angle of friction of the soil
Δ L = base length of each slice along the arc.

$$F\,S = \frac{\left(N - \overline{U}\right)\tan\phi + C\Delta L}{T}$$

FOR THE TOTAL SLIDING SURFACE;

$$FS = \frac{\sum\left(N - \overline{U}\right)\tan\phi + c\Delta L}{\sum T}$$

For a homogeneous section ϕ is constant

$$Fs = \frac{\tan\phi\sum\left(N - \overline{U}\right) + cLa}{\sum T}$$

Therefore
$$FS = \frac{\tan\phi\sum(W\cos\theta - u\Delta L) + \sum c\Delta L}{\sum W\sin\theta}$$

$$Since\ \Delta L = b\sec\theta,$$

$$FS = \frac{\tan\phi\sum(W\cos\theta - ub\sec\theta) + \sum cb\sec\theta}{\sum W\sin\theta}$$

Where:-La is the length of the entire arc of the slip circle.

$La = \frac{\pi r\delta}{180}$ Where δ is the angle in degrees subtended by the slip surface at the center.

Pore water pressure force ($\overline{U}$) = pore water pressure (u) $\times \Delta L$ u is determined from flow net or other means.

If the soil is dry then the FS CAN BE COMPUTED FROM:

$$FS = \frac{\tan\phi\sum(W\cos\theta) + \sum cb\sec\theta}{\sum W\sin\theta}$$

Various centers and radii are used and those computations are repeated until the arc which gives the minimum safety factor is established. The slip surface which gives minimum factor of safety is the critical. The minimum Fs should be greater than the allowable one (1.5).

Summary of procedure for earth dam stability analysis

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. Thus W

= (b x $\overline{Z}$) γ where $\overline{Z}$ 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 N and T.
4. Determine the pore water pressure, u, at the base of the slice or otherwise. Compute the force U_B due to the pore water pressure as: $U_B = ub\sec\theta$
5. Determine the cohesive force, $C = cb\sec\theta$
6. Determine the factor of safety for slip surface from equations given above.
7. Repeat the procedure for a number of other trail surfaces. The trail surface which gives the minimum factor of safety is the most critical circle. The minimum factors of safety should be greater than the specified safe value.

But how to determine the most critical circle???? Fellenius suggested something……

Location of the most CRITICAL circle

In the course of stability analysis, it is quite cumbersome to take so many trial surfaces and hence Fellenius has shown, to reduce number of trials, suggested a line called Fellenius line (line AB) for a homogeneous slope (see fig below) on which the most critical circle lies.

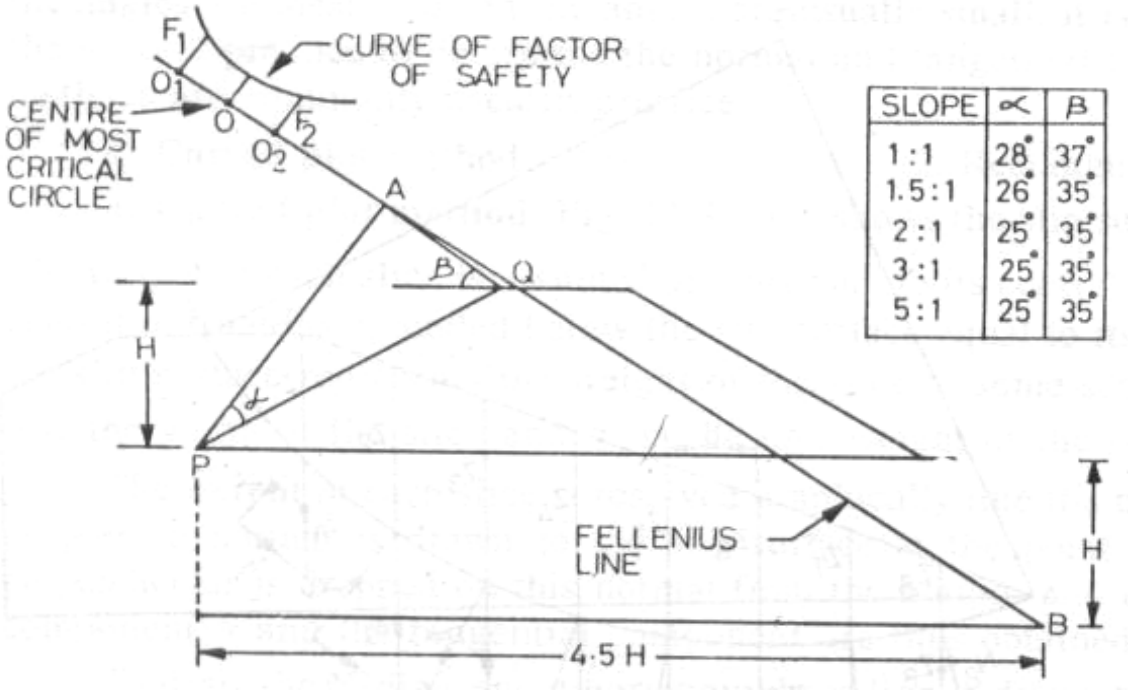


Fig 5:19 Fellenius Line for determining the most critical circle

To draw the Fellenius Line AB, point B is located at a depth of H below the toe and 4.5H from it (Fig 5 :19). Point A is then located by drawing two lines PA and QA. The line PA makes an angle α with the slope line and PQ and the line QA makes an angle β with the horizontal line drawn through the point Q. The values of α and β are given in table along with fig 5: 19 above.

The center of the most critical circle may lie anywhere on the line AB or its extension. However the exact position of critical circle is obtained after conducting the stability analysis for different points. The center O with minimum factor of safety is the center of the most critical circle. The above procedure of drawing Fellenius line is applicable for $C-\phi$ soils. For a purely cohesive soil, the point A itself represents the center of the most critical circle.

For Zoned dams, taking point A and go with successive trials may help at arriving the most critical failure circle. The so called nine point matrix method may help a lot with this regard (*Refer Arora, pp 403-405*).

Critical cases to be considered for Earth dam stability Analysis.

The critical conditions to be analyzed for stability are:

- Stability of downstream slope during steady seepage
- Stability of u/s slope during sudden drawdown condition
- Stability of u/s and d/s slopes during construction

5.1.6. CAUSES OF FAILURE OF EARTH DAMS

Earth dam failures are caused by improper design, frequently based on insufficient investigation and lack of control and maintenance. The various causes may be grouped in to the following three broad categories:-

i) Hydraulic failure
ii) Seepage failure
iii) Structural failure

i) Hydraulic failure: - Caused by surface erosion of the dam by water. They include washout from overtopping, wave erosion of upstream face, scour from the discharge of spillways & erosion of the d/s slope by rain.

ii) Seepage failure: - uncontrolled or concentrated seepage through the dam body or through the foundation may lead to piping and sloughing and subsequent failure of the dam. the following are the common modes of seepage failure:-

- Seepage through pervious foundation: - Presence of strata or lenses of sand or gravel of high permeability or cavities and fissures in the foundation permit concentrated flow of water from reservoir leading to piping.

- ❖ Leakage through embankment: - This is mainly due to poor construction control, in sufficient compaction adjacent to out let conduits, poor bond b/n embankment & foundation or b/n successive layers of the embankment.
- ❖ Conduit leakage: - failure may be either due to contact seepage along the conduit or due to seepage caused by leakage in the conduit.
- ❖ Sloughing: This occurs when the down stream portion of the dam becomes saturated either due to choking or filter toe drain or due to presence of highly pervious layer in the dam body...

iii) Structural failure: - Consists of foundation slide and or embankment slide.

- ➢ Foundation Slide: - When the foundation of soft soil such as fine silt, soft clay etc. the entire dam may slide over the foundation. Partial failures of embankment may also occur over part of the foundation where seams of fissured rock, shale's or soft clay may occur.
- ➢ Embankment Slide: - When the embankment slopes are too steep for the strength of the soil, they may slide causing dam failure. For the upstream slope the critical condition is during sudden draw down and for the down stream slope the full reservoir and steady seepage condition is the most critical.

5.2 ROCKFILL DAMS

The cost producing large quantities of rock for the construction of rock fill dams makes this type of dam economical only in remote areas where the cost of concrete would be high or in areas where there is a scarcity of earth fill materials and the only material for construction of the structure consists of durable hard rock.
A rock fill dam is an embankment which uses variable sizes of rock to provide stability and an impervious membrane to provide water tightness.

The watertight membrane for a rock fill dam should be constructed on the upstream slope where its condition can be inspected when the reservoir is drawdown. And repairs made as necessary. Usually the membrane will consist of Portland cement concrete although steel plates and wood planking have been used to the limited extent of the life of those materials.

Rock fill dams can prove economical when any of the following conditions exist.

1. Large quantities of rocks are available or will be excavated in connection with the project such as
2. Earth fill materials are difficult to obtain or require much processing to be used

3. Short construction season prevails
4. Excessively wet climatic condition limit the placement of large quantities of earth fill materials.

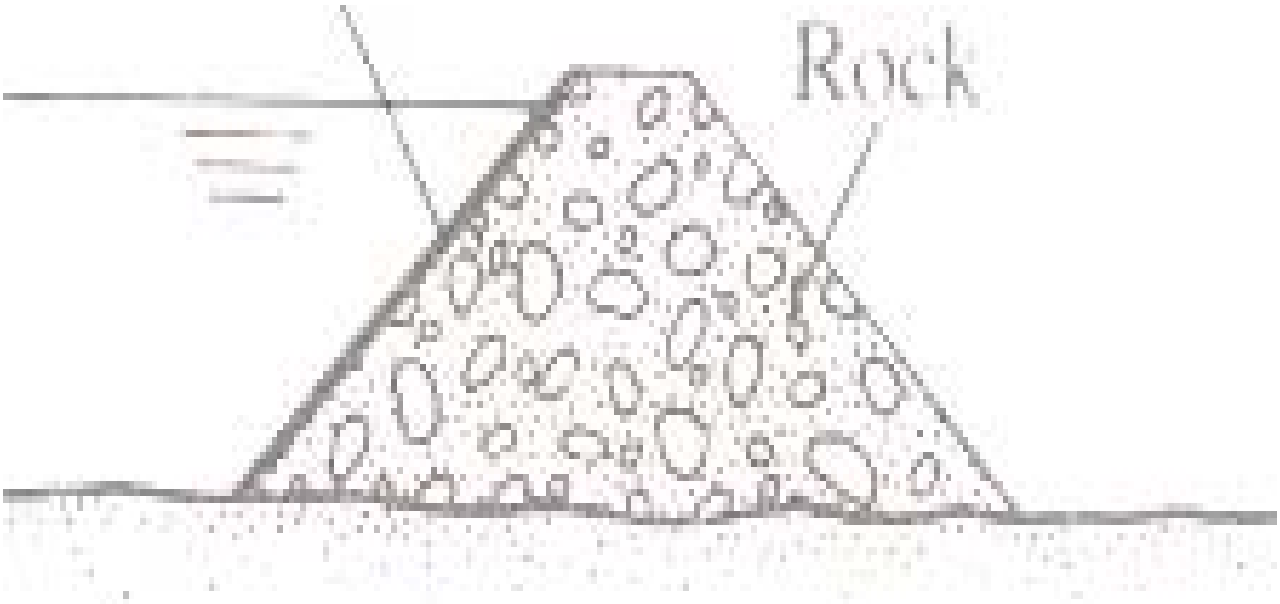


Fig 5:21 Rockfill dam

The major components of Rockfill dam may consist of:

- ❖ Impermeable membrane
- ❖ Rock fill
- ❖ U/S cut-off
- ❖ Rock cushion/rubble masonry

The membrane is usually placed on the u/s face, and in some instances it is placed in the center of the rock fill. The material of the membrane could of concrete, asphalt, steel, timber or impervious soil.

The rock fill usually owns the natural slope at the d/s face. The dry rubble masonry/well compacted rock is provided b/n u/s impervious membrane and the rockfill to make smooth compact bedding for the impervious membrane.

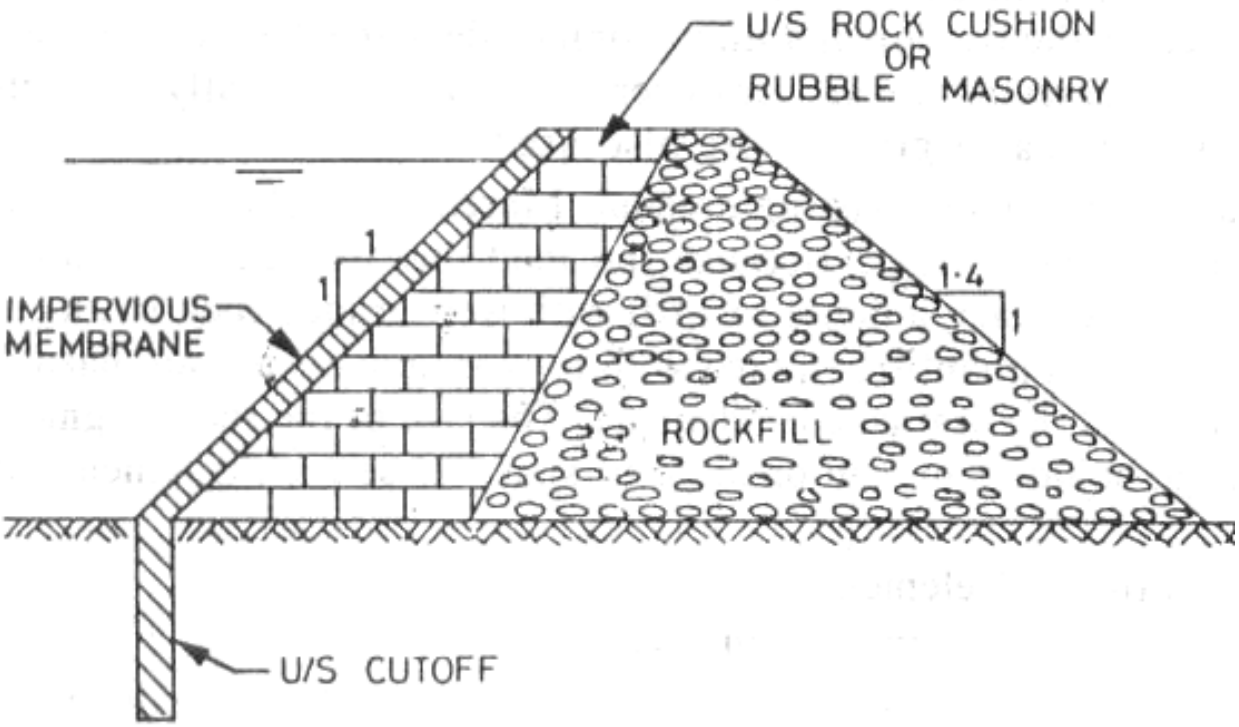


Fig 5. 22 Types of rock fill dams

Depending up on the location of the membrane, rockfill dams may be classified in to three.

i. Central core
ii. Sloping core
iii. U/s membrane or deck

Impervious membrane is used as a water barrier placed either within the embankment or on the u/s slope. Internal membrane constructed of earthen materials should preferably be placed in the central core. For external membranes concrete, asphalt or steel are recommended.

Advantages of Internal membrane:

- Less total area exposed to water
- Shorter grout curtain length
- Protection from effect of weathering and external damage

Disadvantages of Internal membrane:

- Inability to place rockfill material without simultaneous placement of core material and filter
- The dependence on a smaller section of the dam for stability against sliding
- Instability of membrane for damage inspection

Advantages of u/s membrane

- Available for inspection and repair
- Membrane can be constructed after embankment is completed
- A large portion of the dam available for resistance against sliding
- Membrane can be used as a slope protection

Foundation requirement: - The foundation requirement for a rock fill dam is less sever than for a concrete gravity dam. But more sever than earth fill dams. Rock fill dams require foundations which will result in a minimum of settlement.
Generally rock foundations consisting of hard, durable rock which can not be softened or eroded appreciably by percolating water is recommended.
To prevent seepage, the foundation must be grouted.

Cutoff wall: - A watertight seal must be provided along the contact of the impervious membrane with the foundation and abutment at the upstream toe of the dam to prevent seepage under the dam. In existing dams, this seal has been in the form of a concrete cutoff wall which extends from the upstream toe of dam to the bed rock. The cutoff wall must provide adequate support for the weight and thrust of the membrane in addition to its function of preventing under seepage.

Dowell slab cut-offs have the advantage of not requiring extensive excavation in rock, thereby allowing grout operation to begin earlier saving time and reducing design costs.

The depth of penetration of the cutoff wall in to bedrock depends up on the character of the foundation rock. If the rock is sound, the cutoff wall should extend in to the foundation rock not less than 1 meter. A deeper wall or special treatment such as grouting may be required if the rock is not sound or if open joints or broken rock structure exists.

Embankment Design

Selection of embankment material: - The rock must be hard & durable to resist excessive breakdown during the hauling and placing operation and must be located near the dam site for economy .The rock also should with stand disintegration under the action of freezing and thawing.
Un-weathered igneous and metamorphic rocks are of satisfactory quality for rock fill. Sedimentary rocks should be avoided. The rock should be abrasion resistant.
The rock produced in the quarry or obtained from natural sources should be well graded from 0.014 - 0.73 cubic meters in size and should contain less fine than sufficient to fill the voids.

Dam section:- In as much as stability from sliding is not a design consideration in a small rock fill dam b/c of its mass and weight, the determination of the external slopes depends up on the relative cost of dumped rock and rubble masonry. For small rock fill

dams, the downstream slope should be equal to the angle of repose of dumped rock fill (about 1.4 to 1) & the u/s slopes should be 2:1 to facilitate construction of the u/s impervious facing. The u/s and d/s slopes depend up on the type of impervious membrane and its location. Central core/sloping membrane slope ranges from 2:1 to 4:1 u/s and d/s slope. For u/s membrane type rockfill dam, u/s slope ranges from 1.3: 1 to 1.4:1 and d/s slope approximate the natural slope of the rock fill.

Asphalt or concrete faced dams have u/s slopes of range 1.6:1 to 1.7: 1 to facilitate the construction of the membrane whereas steel and concrete faced dams could have slopes in the range of 1.3: 1 to 1.4:1. D/s slopes usually range from 1.3: 1 to 1.4:1 for both above cases.

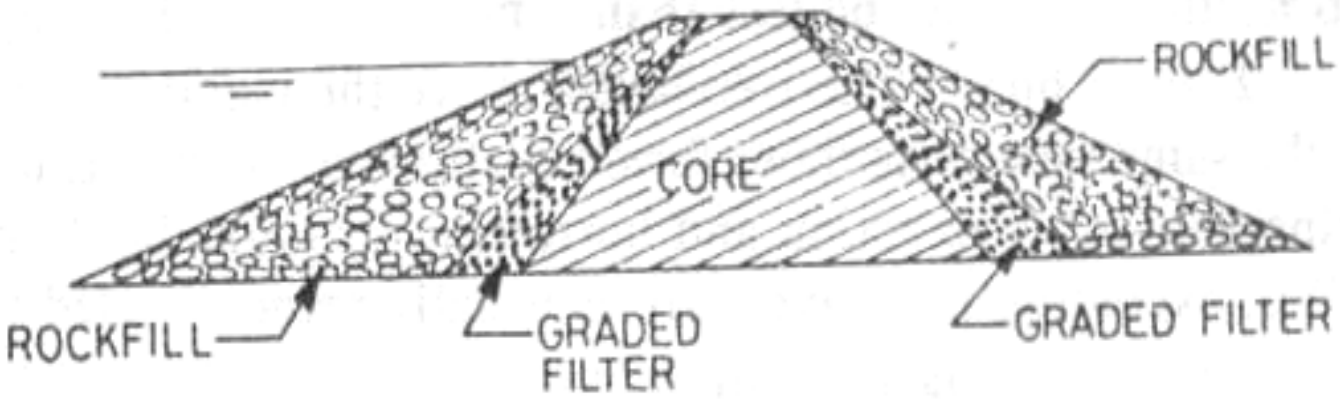


Fig 5.23 Earthfill central core rockfill dam section

Rock fill zone: - The placement of the rock fill is one of the most important operations in the construction of a rock fill dam as it is essential to minimize total settlement and the possibility of damage to the impervious membrane. Settlement of rock fill takes place in two stages. The first major settlement occurs during the construction of the rock fill. This stage of settlement has a minor bearing on the security of the impervious membrane, provided the membrane is not placed concurrently with the rock. The second major stage of settlement occurs as the reservoir fills and the thrust due to water load is transmitted to the rock fill.
For small rock fill dams placement of rock in relatively thin layers is considered to be advisable. The rock should be dumped on the embankment and spread in layer with a maximum thickness of 1 meter. The spreading operation will assure a minimum number of large voids and provide a compact rock fill.

Preparation of upstream facing: - for small dams a zone of graded sand and gravel or quarry fines when well compacted present smooth bedding for the impervious membrane. This zone should have a minimum horizontal width of 1 meter to facilitate compaction. It should be constructed in 30 cm layers thoroughly wetted and compacted. The material

used in this zone should be pervious and well graded from 0.6 cm to 7.5 cm. After placing the upstream face can be dressed smooth to accept any type of membrane.

Design of upstream facing: - Reinforced concrete pavement is the most common type although asphaltic concrete. Steel and timber planking may serve the purpose.
For low dams, a reinforced concrete slab with a minimum thickness of 20 cm should be provided. Horizontal and vertical expansion joints are not required b/c of low reservoir head and minor amount of settlement expected. However, vertical joints may be required to compensate for horizontal expansion on low dams of considerable length.

Reinforcement should be provided: areas of steel equal to 0.5% and 0.7% of the concrete area, vertically and horizontally is considered good practice.
Timber planking is not recommended for general use, although it is often the cheaper type of membrane to construct. The principal objections to this type of construction are the danger of loss by fire at low water and the relatively short life of timber construction when alternately exposed to wetting and drying.

Decked rockfill dam consists of three zones called A, B, C

Zone C: The larger d/s zone of the dam consists of best quality, larger sized compacted rock. This zone provides high stability to the section.

Zone B: Rock of lesser quality than such as excavated from spillway, used to minimize total dam cost.

Zone A: well graded small sized rock gravel; this is used to provide bedding to the u/s membrane and to retard excess water losses when the membrane cracks.

In general, materials in zones B and C should grade from fine rock u/s to course rock d/s with the largest and strongest material placed in the lower d/s portion of zone C.

For central earth core, rockfill dams, the larger and the stronger rock should be placed in the d/s rock fill zone and grade from fine rock next to the filter to course rock near the d/s slope.

The u/s rock fill zone may be rock of lesser quality than the d/s zone and grade from fine at the filter to course at the u/s face.

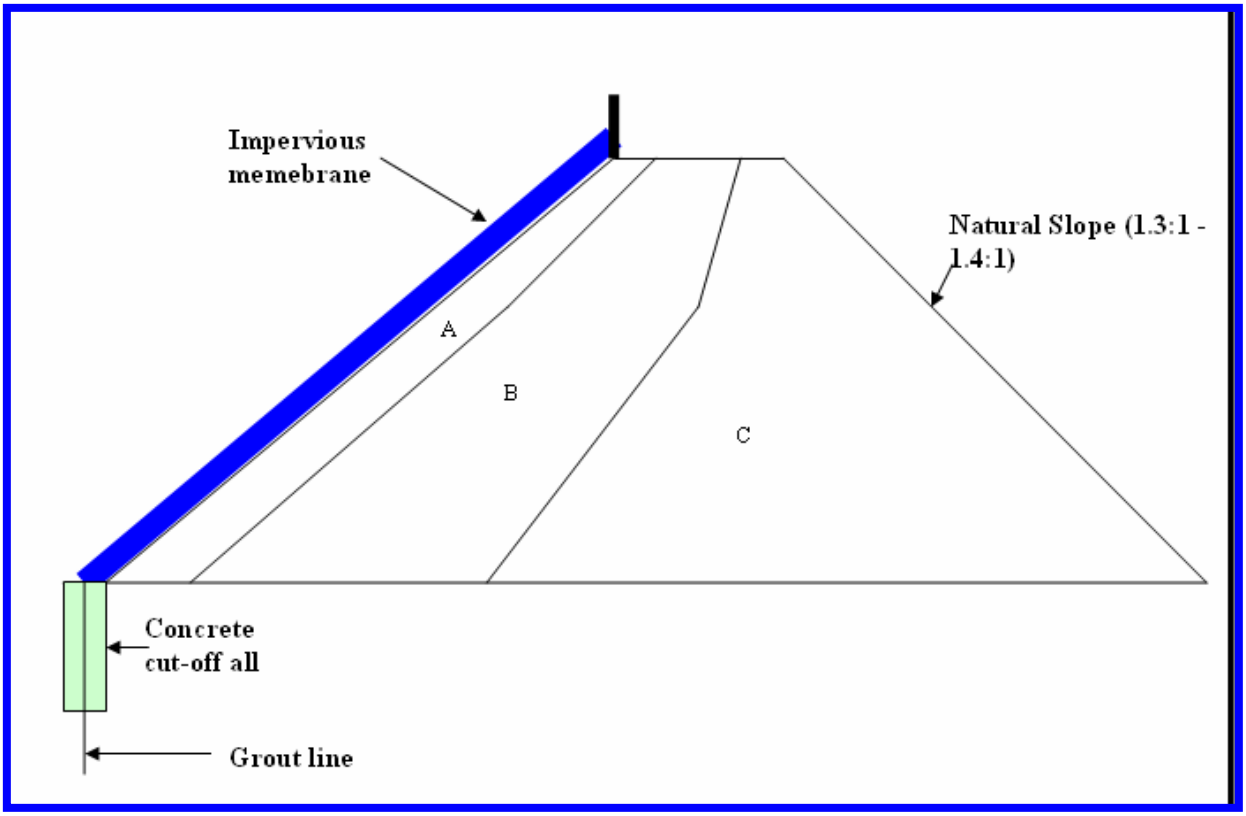


Fig 5.24 Typical section of decked type rockfill dam with zones

VI. SPILLWAYS

6.1 Introduction:

Spillways are provided for storage dams to release surplus or flood water, which cannot be contained in the allotted storage space, and at diversion dams to bypass flows exceeding those, which are turned into the diversion system.

There are several spillway designs. The choice of design is a function of the nature of the site, the type of dam and the overall economics of the scheme. The importance of a safe spillway cannot be overemphasized; many failures of dams have been caused by spillway of insufficient capacity. Ample capacity is of paramount importance for earthfill and rockfill dams, which are likely to be overtopped, whereas concrete dams may be able to withstand moderate overtopping. Usually, increase in cost is not directly proportional to increase in capacity. Very often, the cost of a spillway of ample capacity will be only moderately higher than that of one which is obviously too small.

A spillway may be located either within the body of the dam or at one end of the dam or entirely away from the dam as an independent structure.

6.2 Essential Requirements Of a Spillway

The essential requirements of a spillway are:

i) The spillway must have sufficient capacity;
ii) It must be hydraulically and structurally adequate;
iii) It must be so located that it provides safe disposal of water, i.e. spillway discharge will not erode or undermine the d/s of the dam;
iv) The bounding surfaces of the spillway must be erosion resistant to withstand the high scouring velocities created by the drop from the reservoir surface to the tail water.
v) Some device will be required for dissipation of energy on the d/s side of the spillway.

6.3 Spillway Capacity

The required capacity of a spillway, i.e. the maximum outflow rate through the spillway, may be determined by ***flood routing*** and requires the following data:

i) Inflow hydrograph (plot of rate of inflow vs. time)
ii) Reservoir capacity curve (plot of reservoir storage Vs water surface elevation)
iii) Discharge curve (plot of rate of outflow Vs reservoir water surface elevation).

By flood routing, corresponding to a particular inflow hydrograph, the maximum outflow rate and maximum rise in the water surface may be determined.

However, the required capacity of a spillway depends on the following factors:

i) The inflow flood;
ii) The available storage capacity;
iii) The discharge capacity of other outlet works;
iv) Whether the spillway is gated or ungated;
v) The possible damages if a spillway of adequate capacity is not provided.

The selection of the inflow flood for the spillway design depends on the degree of protection that ought to be provided to the dam, which, in turn, depends on the type of dam, its location, and consequences of failure of the dam.

6.4 Components Of a Spillway

The following are the main components of a spillway:

i) **Control Structure:** Major component, which regulates and controls the outflow from the reservoir. It prevents outflow from a reservoir below a fixed level and allows the flow when the water surface in the reservoir rises above the level. In most of the cases, the control section consists of a weir, which may be sharp crested, ogee, or broad crested. Gates may also be provided on the crest of the control structure to regulate the flow of water from the reservoir.

ii) **Discharge channel (or waterway, or conveyance structure):** Its main function is to convey the water safely from the reservoir downward to the river. Located next to the control structure. The conveyance structure may be the d/s face of the spillway, an open channel excavated along the ground surface, a closed conduit placed through or under the dam, or a tunnel excavated through an abutment.

iii) **Terminal structure or energy dissipator:** Provided to dissipate the high energy of flow from spillway before the flow is returned to the river. It is provided on the downstream of the spillway.

iv) **Entrance or approach channel and outlet channel:** Entrance channels may be required to draw water from the reservoir and convey it to the control structure. Similarly outlet channels may be required to convey the spillway flow from the terminal structure to the river channel below the dam. The entrance and outlet channels are not required where a spillway draws water directly from the reservoir and delivers it directly back into the river; e.g. overflow spillway. However, in the case of spillways placed through abutments or through saddles or ridges, the entrance and outlet channels may be required.

6.5 Types Of Spillway

Spillways may be classified:

1. **According to their function** (or based on the time when the spillway comes into operation) as

(a) Service (or main) spillways: Designed for frequent use in conveying flood releases from the reservoir to a watercourse downstream from a dam. It is designed to pass the entire design flood.

(b) Auxiliary Spillways: - Designed for infrequent use and may sustain limited damages when used. Some damages of the structure from passage of infrequent flood is permissible. It is provided as a supplement to the main spillway and its crest is so located that it comes into operation only after the floods for which the main spillway is designed are exceeded. It is provided in conjunction with the main spillway. The total capacity of the spillway is then equal to the sum of the capacities of the main and auxiliary spillways.

(c) Emergency spillways:- Designed to provide a reserve protection against overtopping of a dam and are intended for use under extreme conditions, such as mis-operation or malfunction of a service spillway or other emergency conditions. Under normal reservoir operation, emergency spillways are never required to function. The control crest is, therefore, placed at or above the designed maximum reservoir water surface.

Some of the situations, which may lead to emergency, are:

a) an enforced shut down of outlet works,
b) a malfunctioning of spillway gates,
c) the necessity for bypassing the regular spillway because of damage or failure of some part of that structure.

2. **According to Mode of Control** as:
   (a) Free (or uncontrolled) spillways,
   (b) Gated (or controlled) spillways.
3. **Based on prominent features pertaining to the various components of the spillway** (or according to hydraulic criteria) as:
   (a) Free overfall or straight drop spillway,
   (b) Overflow or ogee spillway,
   (c) Chute or open channel or Trough spillway,
   (d) Side channel spillway,
   (e) Siphon spillway,
   (f) Shaft or Morning Glory spillway,
   (g) Conduit or tunnel spillway.

6.5.1 Free Over fall Or Straight Drop Spillway

This is the simplest type of spillway, which is constructed in the form of low height weir having d/s face either vertical or nearly vertical. Water drops freely from the crest, and the underside of the falling nappe is ventilated sufficiently to prevent a pulsating, fluctuating, jet. Occasionally, the crest is extended in the form of an overhanging lip to direct the small discharge away from the face of the overfall section.

Since vacuum gets created in the underside portion of the falling jet, sufficient ventilation of the nappe is required in order to avoid pulsating and fluctuating effects of the jet.

If no artificial protection is provided on the d/s side of the over-fall section, the falling jet usually causes the scouring of the stream bed and will form a deep plunge pool.

The free over fall spillway is suitable for thin arch dams and for those dams with nearly vertical downstream face and would permit free fall of water. Free over-fall spillways are used where the hydraulic drops from head pool to TW are not in excess of about 6m.

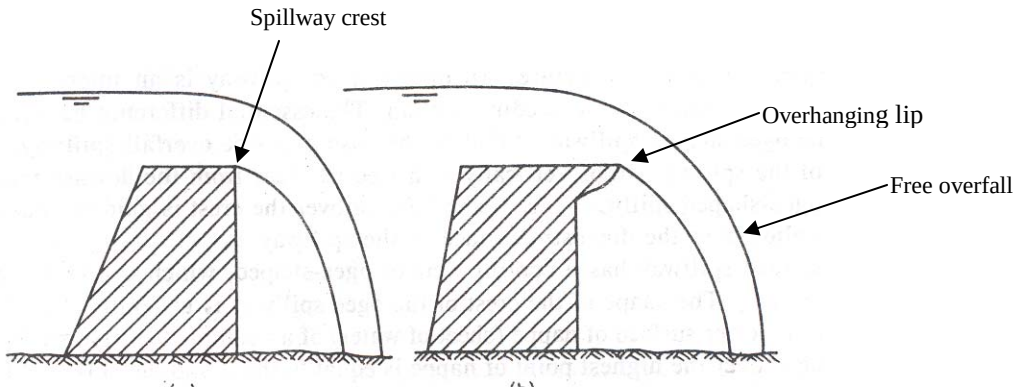


Fig 6.1 Straight drop spillway

6.5.2 Overflow (or Ogee) Spillways

Overflow spillways are by far the most widely adopted. They are mainly used on masonry or concrete dams, and if used with earth fill and need a separate concrete structure.

An overflow spillway is an improvement upon the free overfall spillway. The essential difference between the free overfall spillway and the overflow spillway is that in the case of the former the water flowing over the crest of the spillway drops as a free jet clearly away from the downstream face of the spillway, while in the case of the latter the water is guided smoothly over the crest of the spillway and is made to glide over the downstream face of the spillway.

6.5.2.1 Crest Shape Of Overflow Spillway

The shape of the crest or the upper curve of the ogee profile of this spillway is made to conform closely to the profile of the lower surface of the nappe (or lower nappe) or sheet of water flowing over a ventilated sharp-crested weir when discharging at a head equal to the design head of the spillway.

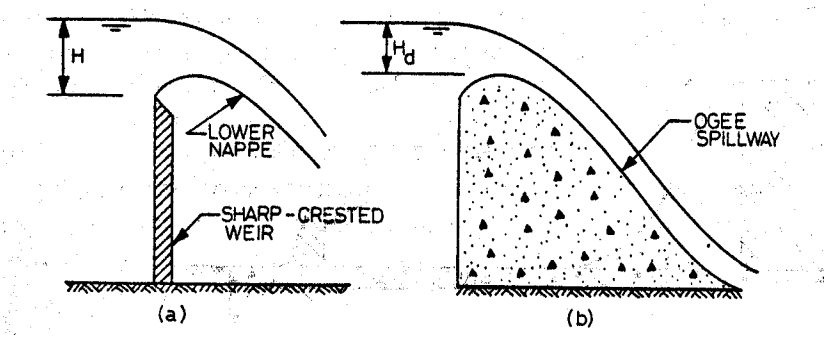


Fig 6.2 Crest shape of overflow spillway

At the design head ($H = H_{d)}$ the water flowing over the crest of the spillway will remain in contact with the surface of the spillway as it glides over it and optimum discharge will occur. In this case no pressure is exerted on the spillway by the flowing water, as there will be atmospheric pressure along the contact surface between the flowing water and the spillway.

At head less than the design head ($H < H_d$) the overflowing water will remain in contact with the surface. The natural trajectory of the nappe falls below the profile of the spillway crest, then there will therefore be positive gage pressures over the crest, as the nappe tends to be depressed. In this case, as the spillway is supporting a sheet of flowing water backwater effect will be created and the discharge will be reduced.

At a head greater than the design head ($H < H_d$), the nappe trajectory is higher than the crest profile, and the overflowing water tends to break contact with the spillway surface and zone of separation will be formed in which negative or suction pressure will be produced. The effect of negative pressure will be to increase the effective head and thereby increase the discharge. This may result in cavitation. However, in practice, this pressure reduction is not normally a serious problem unless $H > 1.5\ H_d$. Indeed recent work suggests that separation will not occur until H approaches $3\ H_{d.}$

6.5.2.2 Design Of Crest Of Ogee Spillway

The shape of the nappe shaped profile depends upon the head, the inclination of the upstream face of the spillway and the height of the spillway above the streambed or the bed of the entrance channel (which influences the velocity of approach to the crest of the spillway).

Several standard ogee shapes have been developed by U.S. Army Corps of Engineers at their Waterways Experimental Station (WES). Such shapes are known as 'WES' standard spillway shapes. The downstream profile can be represented by:

$$X^n = KH_d^{n-1}y \qquad (6.1)$$

Where: x, y = Co-ordinates of the points on the crest profile with the origin at the highest point of the crest called APEX.

H_d = Design head excluding head due to velocity of approach,

K, n = Constants depending on the slope of the upstream face.

The crest equation gives the crest shape downstream from the origin of coordinates. This equation is applicable to positive values of x and y.

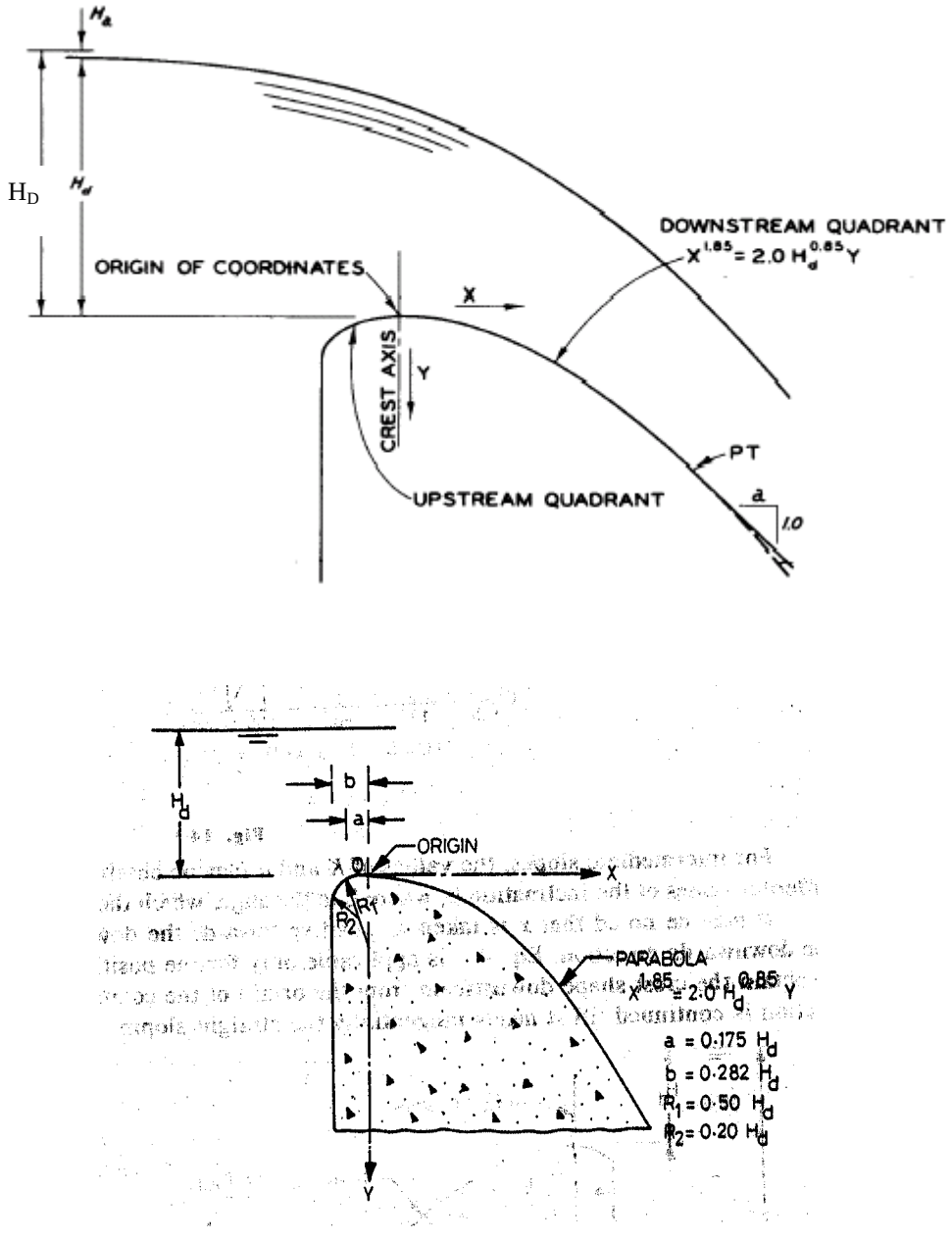


Fig 6.3 WES- standard spillway shape (vertical upstream face)

The following table gives values of K, n and other constants and crest equations

U/s Face slope	K	n	$\frac{a}{H_d}$	$\frac{b}{H_d}$	$\frac{R_1}{H_d}$	$\frac{R_2}{H_d}$	Crest Equation
Vertical	2.000	1.850	0.175	0.282	0.200	0.500	$x^{1.85} = 2H_d^{0.85}y$
1H: 3v	1.936	1.836	0.139	0.237	0.210	0.680	$x^{1.836} = 1.936H_d^{0.836}y$
2H: 3v	1.936	1.810	0.115	0.214	0.220	0.480	$x^{1.810} = 1.939H_d^{0.810}y$
3H: 3v	1.873	1.776	0.000	0.119	∞ (Straight line)	0.450	$x^{1.776} = 1.873H_d^{0.776}y$

According to U.S. Army Corps of Engineers, the u/s curve of the ogee spillway (u/s of origin, though in the form of compound circular curve) having a vertical u/s face, should have the following equation:

$$y = \frac{0.724(x + 0.27H_d)^{1.85}}{H_d^{0.85}} + 0.126H_d - 0.4315H_d^{0.375} \qquad (6.2)$$
$$(x + 0.27H_d)^{0.625}$$

Where the upstream profile extends up to $x = -027H_d$

The corresponding y value is equal to $0.126\ H_d$.

The curved profile of the crest section is continued tangentially along the straight sloping surface, which forms the d/s face of the spillway. The location of the point of tangency (P.T) depends on the slope of the straight portion of the d/s face of the spillway, which in turn depends on the stability requirements and on the features of the stilling basin at toe of the spillway. The slope of the straight portion varies between 1V: 0.6H to 1V: 0.8H. At the end of the sloping surface a curved bucket is provided to create a smooth transition of flow from the spillway to the outlet channel or the river on the d/s side and prevent scoring.

The approximate radius R of the bucket may be obtained from (empirical)

$$R = 10^{(v+6.4H+4.88)/(3.6H+19.52)} \qquad (6.3)$$

V= velocity of flow at to e of spillway [m/s

H = head excluding head due to velocity of approach (m)

Neglecting energy loss over the spillway, velocity of flow v at the toe will be

$$V = \sqrt{2g(Z + Ha - y)} \qquad (6.4)$$

Where Z = the fall, m

H_a = head due to velocity of approach, m

y = depth of flow at the toe, m

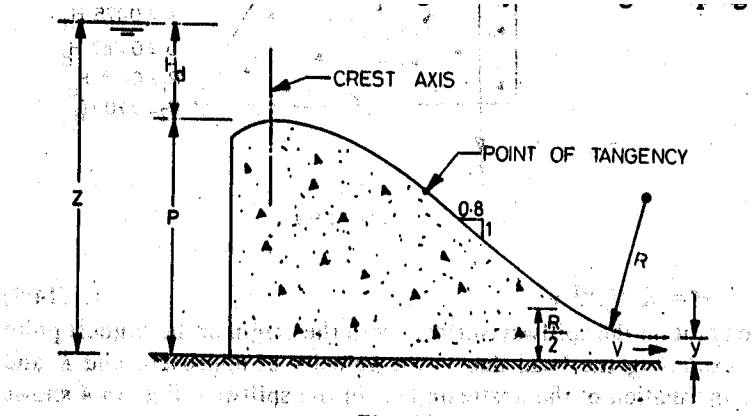


Figure 6.4 Profile of an ogee spillway

6.3 Discharge of Overflow Spillway

The discharge over an overflow spillway is given by

$$Q = CL_e H_D^{3/2} \quad (6.5)$$

Where Q = discharge, m^3/s

C = coefficient of discharge

L_e = effective length of crest of spillway, m

H_D = total head over the crest including that due to velocity of approach.

$H_D = H_d + H_a$

For high ogee spillway H_a is very small, and $H_D \approx H_d$

(i) Coefficient of discharge, C, of Overflow spillway

An overflow spillway has a relatively high coefficient of discharge the maximum value of which may be about 2.2 if no negative or suction pressure is allowed to develop. Its value depends on the following factors:

a) Depth of approach, p
b) Heads differing from design head
c) Upstream face slope
d) Downstream apron interference and downstream submergence

a) Effect of Depth of Approach: With increase in the height of spillway the velocity of approach decreases and the coefficient of discharge increase. Model tests have shown that the effect of approach velocity is negligible when the height of the spillway above the streambed is equal to or greater than 1.33 H_d ($P \geq 1.33\ H_d$) where H_d is the design head excluding the head due to velocity of approach

A plot of C versus P/H_D is shown below, where H_D is the design head including head due to velocity of approach (i.e. $H_D = H_d + H_a$). It may be observed from this plot that there is a marked increase in the value of C till the height of the spillway (P) becomes equal to

twice the design head H_D. With further increase in P there is no much increase in the value of C.

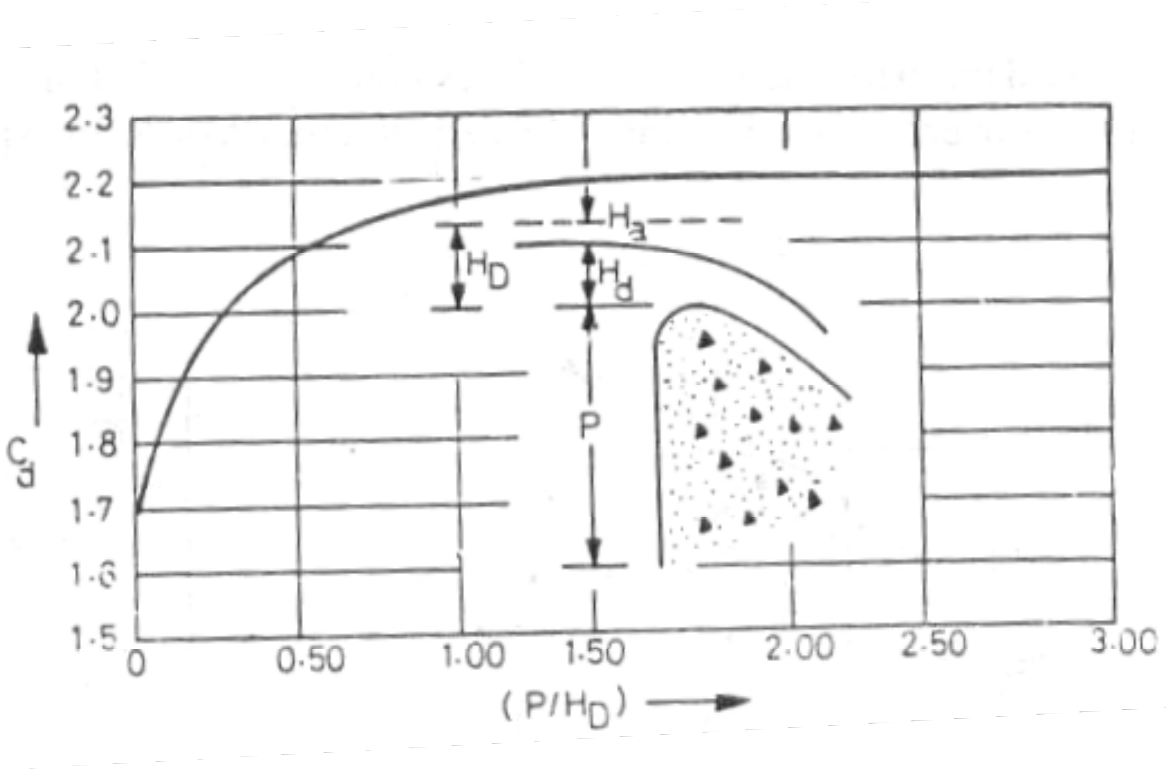


Fig 6.5 Plot of coefficient of discharge versus (P/H_D)

(b) Effect of heads differing from the design head: The plot of (C/C') versus (H_e/H_D) for a spillway of height P above stream bed greater than 1.33 H_d, where C is coefficient of discharge corresponding to the actual head of flow H and C' is the coefficient of discharge corresponding to the design head H_D. It may be observed from this plot that with increase in the value of (H_e/H_D) the value of (C/C') increases. In other words, with increase in the head H the coefficient of discharge increases. However, for $H_e < H_D$, C < C'; and for $H_e > H_D$, C > C'.

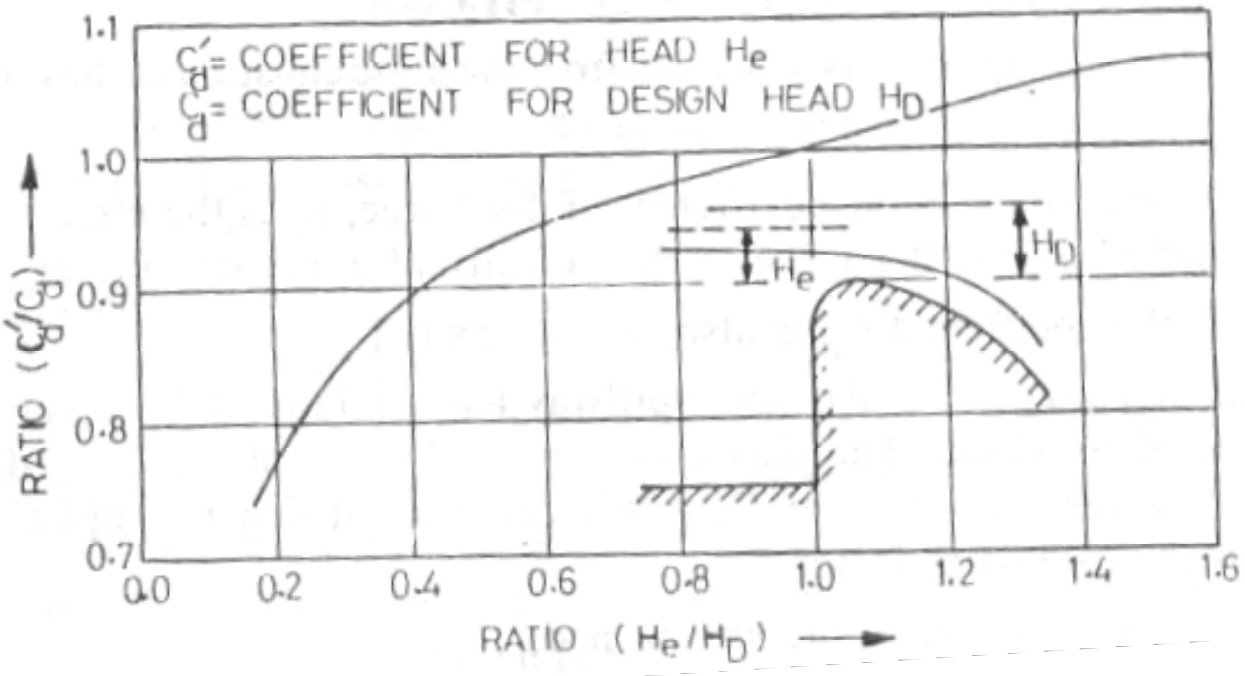


Fig 6.6 plot of C/C' Vs ($H_e/H_{D)}$

Since for heads of flow higher than the design head higher will be the coefficient of discharge, if the spillway crest is designed by assuming a lower design head, for most of the range of heads of flow higher coefficient of discharge will be obtained.

However, the design head should not be less than about 80% of the maximum head in order to avoid the possibility of cavitation.

Model tests have shown that for P > 1.33 H_d the head due to velocity of approach is negligible and when the total head of flow is equal to the design head, i.e. $H_e = H_D$, the coefficient of discharge is equal to 2.2.

When the actual operating head is less than the design head, the prevailing coefficient of discharge, C, tends to reduce, and is given by

$$C = C'\left(\frac{H_e}{H_D}\right)^{0.12} \qquad (6.6)$$

Where H_D = design head including velocity head and C' = 2.2

(c) Effect of upstream face slope: For small values of the ratio (P/H_D) a spillway with sloping upstream face has a higher coefficient of discharge than a spillway with vertical upstream face. However, for large values of the ratio (P/H_D) the coefficient of discharge for spillways with sloping upstream face tends to decrease.

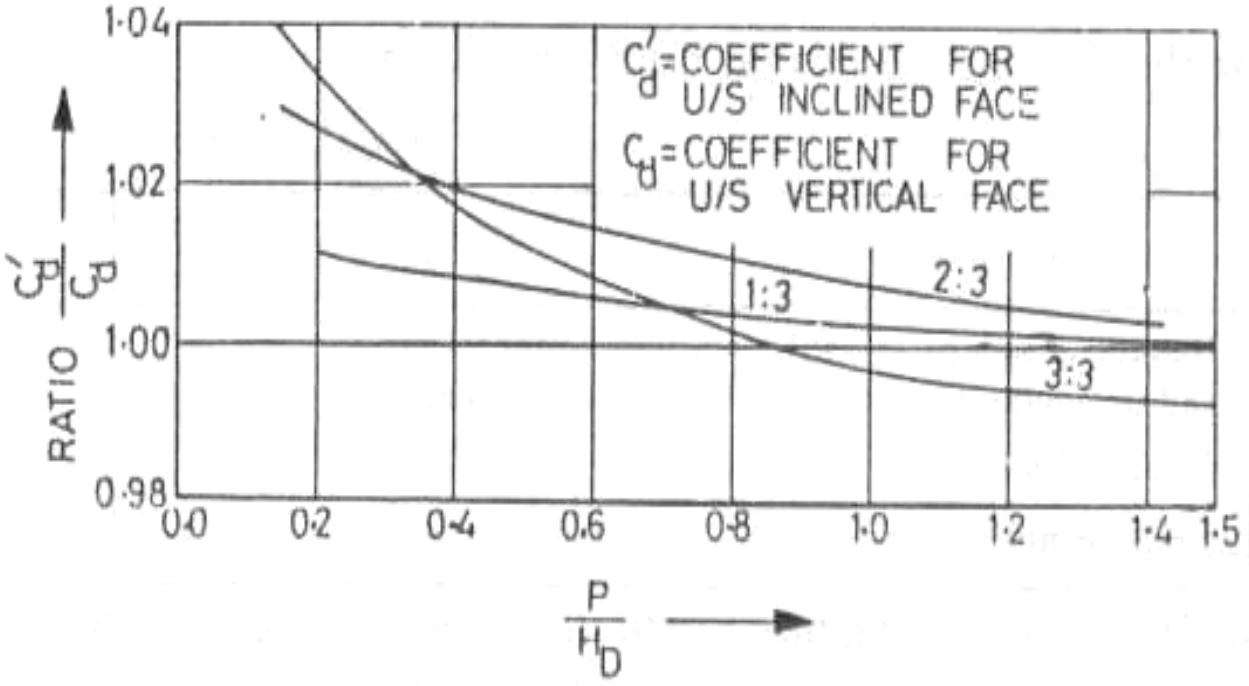


Fig 6.7 Coefficient of discharge for various u/s slopes

(d) Downstream apron interface and submergence effects: The coefficient of discharge is reduced due to submergence. When the tailwater level is such that the top of the weir is covered by it, such that the weir cannot discharge freely; the weir is then said to be submerged weir. Where the hydraulic jump occurs, the coefficient of discharge may decrease due to backpressure effect of the downstream apron and is independent of the submergence effect. When the value of $\frac{h_d + d}{H_D}$ exceeds 1.7, the downstream apron is found to have negligible effect on the coefficient of discharge. But there may be a decrease in C due to tail water submergence.

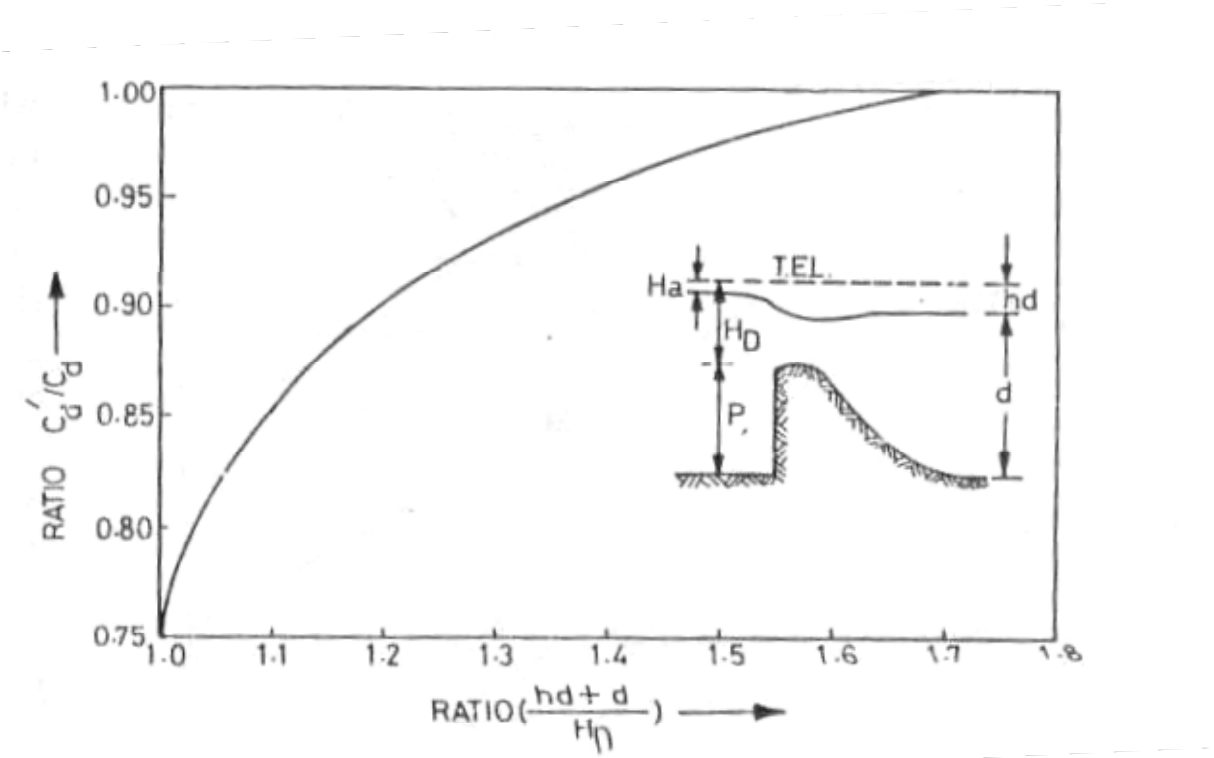


Fig 6.8 Maximum TW depth for a non-submerged weir

(e) Effective Length Of Crest Of Overflow Spillway: The effective length of an overflow spillway is given by

$$Le = L - 2 (NK_p + Ka) H_D \quad (6.7)$$

Where L_e = effective length of crest

L =net length of crest which is equal to the sum of the clear spans of the gate bays between piers

H_D = total head on crest including velocity head

N = number of Piers

K_P = Pier contraction coefficient

K_a = abutment contraction coefficient

The pier contraction coefficient, K_p depends on

i) Shape and location of pier nose;

ii) Thickness of pier;

iii) Velocity of approach; and

iv) Ratio of actual head to design head.

For flow at design head the average values of K_p may be assumed as follows:

Pier coefficients, K_p:

1. Square nosed piers with corners rounded on a radius equal to about 0.1 of pier thickness → K_p =0.02
2. Round-nosed piers → K_p = 0.01
3. Pointed nose piers → K_p = 0.00

The abutment contraction coefficient K_a depends on:

i) Shape of abutment;
ii) Angle between upstream approach wall and axis of flow;
iii) Approach velocity; and
iv) Ratio of actual head to design head

For flow at design head, average value of K_a may be assumed as follows:

Abutment coefficients, K_a:

1. Square abutment with head wall at 90^0 to the direction of flow → K_a= 0.20
2. Rounded abutment with head wall at 90^0 to the direction of flow, when $0.5\ H_d \geq r \geq 0.15\ H_d$ → K_a = 0.10
3. Rounded abutments where $r > 0.5\ H_d$ and headwall is placed not more than 45^0 to the direction of flow → K_a = 0.00.

Where r = radius of abutment rounding
H_d = design head.

VII. ENERGY DISSIPATION

The water flowing over the spillway acquires a lot of kinetic energy by the time it reaches near the toe of the spillway due to the conversion of potential energy into kinetic energy. If arrangements are not made to dissipate this huge kinetic energy of the water, and if the velocity of the water is not reduced, large-scale scour can take place on the downstream side near the toe of the dam and away from it. These arrangements are known as energy dissipation arrangements or energy dissipators.

For the dissipation of the excessive kinetic energy possessed by the water the two common methods adopted are:

i. By converting the supercritical flow into subcritical flow by hydraulic jump.
ii. By using different types of buckets, i.e. by directing the flow of water into air and then making it fall away from the toe of the structure.

7.1 Jump Height and Tailwater Rating Curves

Hydraulic jump can form in a horizontal rectangular channel when the following relation is satisfied between the pre-jump depth (y_1) and post – jump depth (y_2).

$$y_2 = \frac{y_1}{2}\left[-1+\sqrt{1+8\,F_r^{\,2}}\right]$$

Where y_1 = pre-jump (initial) depth
y_2 = post- jump (sequent) depth
F_{r1} = Froude number of the incoming flow

For a given discharge intensity q over a spillway, y_1, will be equal to $q/v_{1;}$ and v_1 (mean velocity of incoming flow) is determined by the drop H_1 $\left(V_1 = \sqrt{2gH_1}\right)$, if head loss is neglected, (see fig. 7.1)

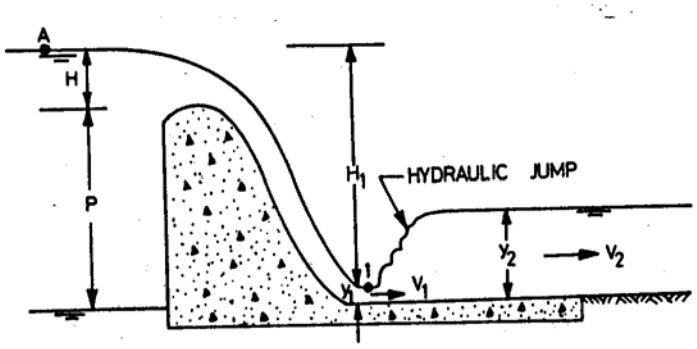


Fig 7.1 Hydraulic jump at the toe of a spillway

Hence, for a given discharge intensity and given height of spillway, y_1 is fixed and thus y_2 is also fixed. But the availability of a depth equal to y_2 in the channel on the downstream cannot be ***guaranteed*** as it depends upon the tail water level y_0 which depends on the hydraulic conditions of the river channel on the downstream side. The values of y_o corresponding to different values of q may be obtained by actual gauge discharge observations and plot of y_o versus q prepared, known as Tailwater Rating curve (T.W.R.C.). The post-jump depth (y_2) for all those discharges, are also computed from equation (7.1) and a plot of y_2 versus q may be made which is known as jump height curve (J.H.C.). If J.H.C. and T.W.R.C. are plotted on the same graph, five possibilities exist regarding the relative positions of these curves.

1. T.W.R.C. ($y_{2'}$) coinciding with y_2 curve for all discharges
2. T.W.R.C. ($y_{2'}$) lying above the y_2 curve for all discharges
3. T.W.R.C. ($y_{2'}$) lying below the y_2 curve for all discharges
4. T.W.R.C ($y_{2'}$) lying below the y_2 curve for smaller discharges and lying above y_2 curve for larger discharges
5. T.W.R.C. ($y_{2'}$) lying above the y_2 curve for smaller discharges and lying below the y_2 curve for larger discharges

The energy dissipation arrangement that can be provided is dependent upon the relative positions of T.W.R.C. and y_2 curve.

Condition 1: In this case for the entire discharges jump will develop close to the toe of the spillway. In such a case, a simple horizontal concrete apron may be provided whose length is equal to the length of the jump corresponding to the maximum discharge over the spillway.

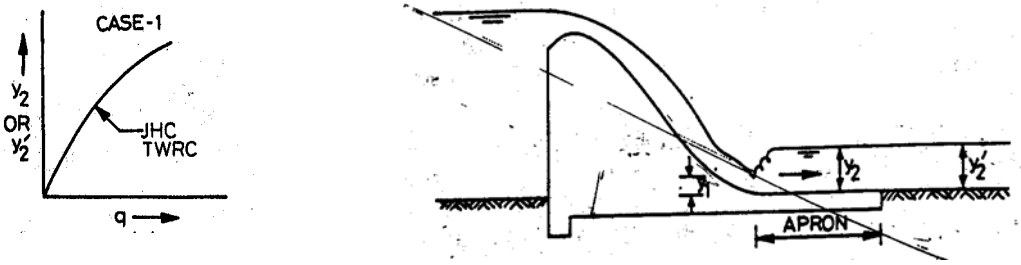


Fig 7.2a Condition 1

Condition 2: The jump forming at toe will be drowned out by tailwater, and little energy will be dissipated.

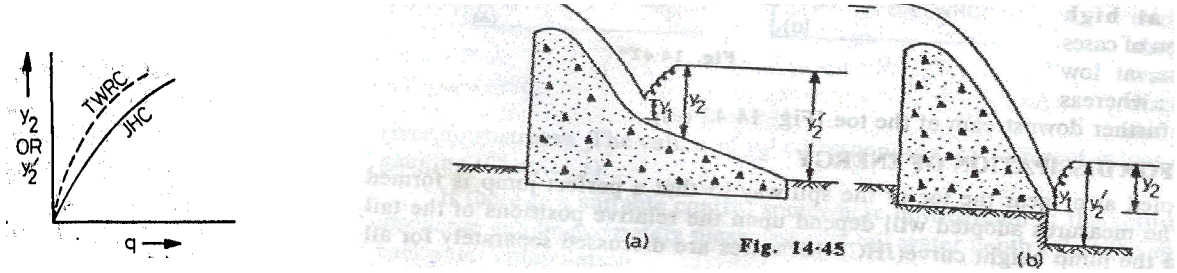


Fig.7.2b Condition 2 **Fig 7.2c** Hydraulic jump on a sloping apron

Water may continue to flow at high velocity along the channel bottom for a considerable distance. The problem can be solved:

i) By constructing a sloping apron over the riverbed extending from the downstream surface of the spillway. The jump will form on the sloping apron where depth equal to y_2 (lesser than the tailwater depth at toe) is available.

The slope of the apron is made in such a way that proper conditions for a jump to occur somewhere on the apron at all discharges.

ii) By providing a roller bucket type energy dissipater. Also a drop provided in the riverbed to lower the TWL can be used to dissipate the energy.(Fig 7.2c).

Condition 3: In this case the jump will develop at a certain section far downstream of the toe of the spillway. This is the most frequent one, and shows that a stilling basin (with a depressed horizontal apron) is required for all discharges in order to produce a jump close to the toe of the spillway.

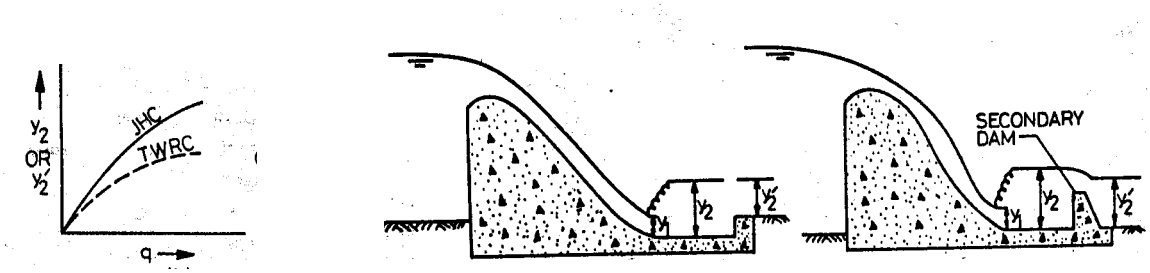


Fig 7.2d Condition 3

If the tailwater is very low, ski-jump type dissipator may be provided. But it needs sound rock at the riverbed, since part of the dissipation takes place by impact; the rest being dissipated by aeration and diffusion in air.

Condition 4*:* In this case the following measures may be taken to develop jump close to the spillway.

i) Provide a stilling basin with an end sill for developing a jump at low discharges and combine the basin with a sloping apron for developing a jump at high discharges.
ii) Provide a sloping apron which lies partly above and partly below the riverbed so that jump will develop at lower portion of the apron at low discharges and at higher portion of the apron at high discharges.

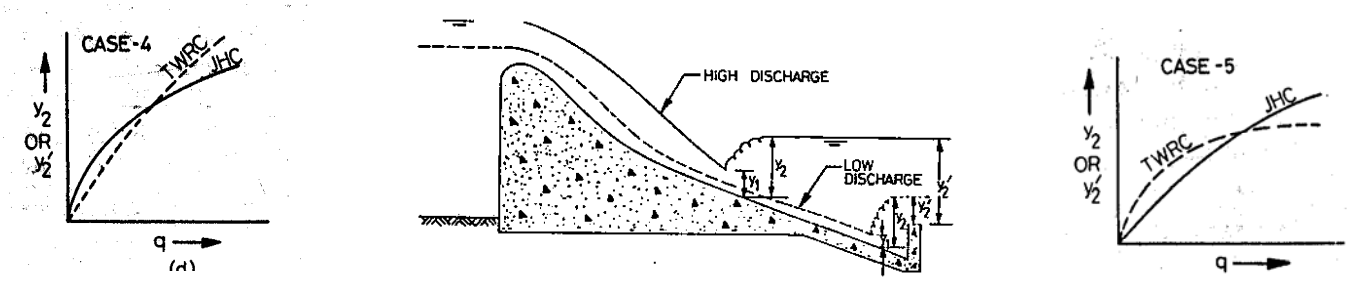


Fig. 7.2e.Condition 4

In this case, the tailwater depth is insufficient at low discharges and is greater at high discharges.

***Condition 5*:** This condition is just the reverse of condition (4) and the same arrangement that was made for condition (4) will serve the purpose. (**Fig 7.2e)**

7.2 Stilling Basin

A stilling basin consists of a short, level apron at the foot of the spillway. It must be constructed of concrete to resist scour. The function of the basin is to decelerate the flow sufficiently to ensure the formation of a hydraulic jump within the basin. The jump dissipates much of the energy, and returns the flow to the subcritical state.

7.2.1 Hydraulic Jump Stilling Basin

The passage of water from a reservoir into the downstream reach involves a number of hydraulic phenomena such as the transition into supercritical flow, supercritical non-aerated and aerated flow on the spillway, entry into the stilling basin with a transition from supercritical to subcritical flow, and echoes of macro-turbulence after the transition into the stream beyond the basin. It is, therefore, possible to consider the energy dissipation process in the following stages, all of which may be combined.

- On the spillway surface
- In the stilling basin
- At the outflow into the river.

7.2.2 Energy Dissipation On Spillway Surface

The energy loss on the spillway surface may be expressed as

$$e = \zeta\ \alpha \frac{V'^2}{2g}$$

Where V' = the (supercritical) velocity at the end of the spillway

α = Coriolis coefficient (energy coefficient)

ζ = head loss coefficient.

The total energy, E, can be expressed as

$$E = \zeta\ \alpha \frac{V'^2}{2g} + \alpha \frac{V'^2}{2g}$$

and taking $$\varphi = \frac{\text{actual velocity}}{\text{theoretical velocity}}$$

Hence, $$\frac{1}{\varphi^2} = 1 + \zeta$$

The ratio of the energy loss, e, to the total energy E (i.e. relative energy loss) is

$$\frac{e}{E} = \frac{\zeta V'^2}{2g} \Bigg/ \left(\frac{V'^2}{2g} + \zeta \frac{V'^2}{2g} \right) = \frac{\zeta}{1+\zeta} = 1 - \varphi^2$$

For the ratio of the height P of the spillway crest above its ending and the overflow head H, with P/H < 30, and smooth spillways (Novak & Cabelka, 1981),

$$\varphi \cong 1 - 0.0155\, P/H$$

For a given P, φ increases as H increases, i.e., if for a given discharge Q the spillway width b decreases and thus q increases.

Thus, for P/H = 5, φ = 0.92 and the relative head loss (e/E) is 15%, where as for P/H = 25, φ = 0.61 and relative head loss is 62 %.

The value of head loss coefficient (ζ) could be increased (and φ decreased) by using a rough spillway or by placing baffles on the spillway surface. However, unless aeration is provided at these protrusions, the increased energy dissipation may be achieved only by providing an opportunity for cavitation damage.

7.3 Energy Dissipation in the Stilling Basin

Referring to the notation is Fig (7.3) and to equations (7.2) and 7.4) we can write

$$E = y_1 + \frac{\alpha q^2}{2g\varphi^2 y_1^2}$$

$$y_2 = \frac{y_1}{2}\left[-1+\left(1+\frac{8\,q^2}{g\,y_1^3}\right)^{1/2}\right]$$

The stilling basin depth is then given by

$$y' = y'_2 - y_0 = \sigma' y_2 - y_0$$

and the length of the stilling basin is given by

$$L = k(y_2 - y_1)$$

where σ´ and k are coefficients derived from laboratory and field experiments.

According to Novak and Cabelka Coefficients σ´ and k can be taken as $1.1 < \sigma' < 1.25$ and
$4.5 < k < 5.5$, where the lower value of k applies for $F_{r1} > 10$ and the higher for $F_{r1} \leq 3$.

Fig. 7.3 Definition sketch for hydraulic jump stilling basin

When applying equations (7-7) –(7-10) we start with a known discharge q and the corresponding downstream depth y_0 and apply the iterative procedure, which follows:

Take the energy (reference) datum at downstream riverbed level, and compute E assuming an initial value of y' = 0;

Choose a suitable value of φ; $(\varphi = 1 - 0.0155p/H)$;

Compute y' for q_{max} from equation (3.7); y_2 from equation (3.8); and y' from equation (3.9) (From a chosen value of safety coefficient, σ');

Compute y_0 (from uniform flow equation – Manning, Chezy) and compare it with y_2

If $y_2 < y_0$, no stilling basin is required; if $y_2 \geq y_0$ stilling basin is required and therefore compute y´ with $1.1 < \sigma' < 1.2$ (≈ 1.2) from equation (3.9);

Take new reference datum at basin bed level; and calculate new E and repeat steps 2-4 to check that $\sigma' \geq 1.1$.

Repeat the above steps at least for one smaller q to check whether the designed stilling basin is adequate for lower discharges as well.

Note: Equations (3.8) and (3.10), and thus the design under discussion, apply to basins with a horizontal floor only.

7.3.1 Additional Considerations in Stilling Basin Design

The hydraulic jump entrains a substantial amount of air additional to any incoming aerated flow. The main significance of the presence of air in the jump region is the requirement of higher stilling basin sidewalls due to higher depth of flow.

The major problems in spillway stilling basin are cavitation, uplift, and abrasion.

The highly turbulent nature of the flow in the hydraulic jump induces large pressure fluctuations and is the cause of cavitation. Cavitation number can be expressed as

$$\sigma = P' \Big/ \left(\frac{1}{2} \rho V_1^2 \right)$$

where P´ is the deviation of the instantaneous pressure P from the time average pressure. If σ falls below a critical value, σ_c, then cavitation occurs.

Another serious structural problem in hydraulic jump stilling basins is the effect of uplift pressures due to the dam drainage system or the tailwater level or the water table in the basin bank, which is aggravated by the macro-turbulent pressure fluctuations underneath the jump. Therefore, it is sensible to design the floor slab for the full downstream uplift pressure applied over the whole area of the floor with the basin empty or the uplift pressure head equal to the root mean square value of pressure fluctuations of the order of $0.12V_1^2/2g$ (V_1= inlet supercritical velocity) applied under the whole full basin. Furthermore, all contraction joints should be sealed, no drain openings should be provided, and the floor slab should be as large as possible and connected by dowels and reinforcement (ICOLD, 1986).

Abrasion of concrete in the basin could take place if this is also used for bottom outlets carrying abrasive sediments (unlikely to happen for V<10m/s), or from sediment drawn into the basin from downstream either by bad design or operation. The basin should be self-cleaning to flush out any trapped sediment.

The prevention of vibration of basin elements (due to turbulence of the flow) also requires massive slabs, pinned to the foundation when possible.

7.2.5 Standard Stilling Basins

Although the stilling basin based purely on a simple hydraulic jump works well and relatively efficiently, in certain conditions other types of basins may produce savings in construction costs. Certain accessories such as chute blocks, baffle blocks (or floor blocks), and end sills (or baffles) are usually provided in the stilling basins to reduce the length of the jump and thus to reduce the length and the cost of the stilling basin. Moreover, these accessories also improve the dissipation action of the basin and stabilize the jump.

The type of stilling basin to be provided depends on the type of jump, which in turn depends on the Froude number F_{r1} of the incoming flow.

7.2.5.1 U.S.B.R. Stilling Basins

7.2.5.1.1 Stilling basin for $1.7 < F_{r1} < 2.5$ (Type I)

Only horizontal apron needs to be provided. The flow does not have much turbulence and hence no accessions are required. However, the apron must be sufficiently long to contain the entire jump. The length of the apron should be the length of the jump (i.e. $5y_2 = L$, and $(L \geq 4y_2)$ where y_2 = sequent depth).

7.2.5.1.2 Stilling Basin for $2.5 < F_{r1} < 4.5$ (Type IV)

Type IV stilling basin is found effective. It is provided with chute blocks and end sill is optional. The length L of the stilling basing may be obtained from the following table.

F_{r1}	2	3	4	5
L/y_2	4.3	5.3	5.8	6

7.2.5.1.3 Stilling Basins for $Fr_1 > 4.5$

True hydraulic jump will form. Depending on the incoming velocity of flow two types of basins are developed:

a) **$V_1 < 15$ m/s**: ***Type III stilling basin may be adopted***. This basin utilizes chute blocks, baffle blocks and end sill (the size, spacing and location of the chute and baffle blocks are shown in the **Figure**). The length of the stilling basin and the

height h_3 and h_4 of the baffle blocks and the end sill may be obtained for different values of Fr_1 as follows:

b)

F_{r1}	5	6	8	10	12	14	16
$\frac{L}{y_2}$	2.3	2.5	2.6	2.7	2.8	2.8	2.8
$\frac{h_3}{y_1}$	1.5	1.7	2.0	2.3	2.7	3.0	3.3
$\frac{h_4}{y_1}$	1.2	1.3	1.5	1.6	1.7	1.8	1.9

The use of chute blocks, impact baffle blocks, and an end sill shortens the jump length and the stilling basin. This basin relies on dissipation of energy by the impact blocks and on the turbulence of the jump phenomena for its effectiveness. Because of the large impact forces to which the baffles are subjected by the impingement of high incoming velocities and because of the possibilities of cavitation along the surfaces of the blocks and floor (due to downstream suction), the use of this basin should be limited to heads where the velocities do not exceed 15 m/s.

c) **V_1 > 15 m/s**: Here impact baffle blocks are not employed and they are designated as ***Type II stilling basin***. Because the dissipation is mainly accomplished by hydraulic jump action, the basin length will be greater than that indicated for type III basin.

However, the chute blocks and dentated end sill (instead of solid end sill) will still be effective in reducing the length from that which would be necessary if they were not used. In this basin baffle blocks are not provided because.

i) due to the high velocities of incoming flows these blocks will be subjected to excessively large impact forces, and
ii) There is a possibility of cavitation along the downstream face of these blocks and the adjacent floor of the basin due to large negative pressure being developed in this region.

The length L of Type II stilling basin may be obtained for different values of F_{r1} from the following table

F_{r1}	5	6	8	10	12	14
L/y_2	3.85	4.0	4.2	4.3	4.3	4.3

7.3 Submerged bucket dissipaters

When the tail water depth is too great for the formation of a hydraulic jump (i.e. when TW depth are too large as compared to the sequent depths required for the formation of

hydraulic jump), dissipation of the high energy of flow can be effected by the use of submerged bucket deflector.
They are of two types, viz.

i) Solid roller bucket
ii) Slotted roller bucket

Solid Roller Bucket: Consists of a bucket like apron with a concave circular profile of large radius and a deflector lip. When water flows over the bucket the entire sheet of water leaving the bucket is deflected upwards by the bucket lip and two rollers are developed. One of the rollers, called bucket roller, moves in counterclockwise direction and is developed on the surface of the bucket. The other roller moving in clockwise direction, called ground roller, is developed on the ground surface immediately downstream of the bucket. The movement of the rollers, also with the intermingling of the incoming flows, causes the dissipation of energy.

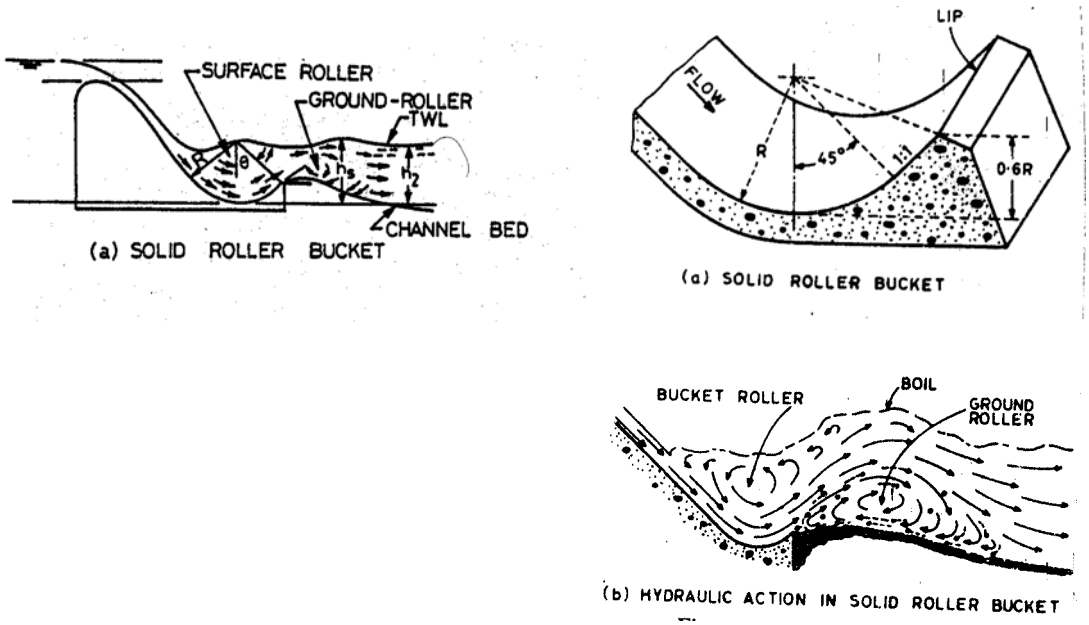


Figure 7.4

7.3.1 Slotted Roller Bucket: Consists of a bucket like apron with a concave circular profile of large radius and a slotted or dentated deflector lip. Its action is, in general, same as solid roller buckets. The two rollers are also developed in this case. However, in this case water leaves the lip at a flatter angle and only a part of it is deflected upwards. Thus surface boil is considerably reduced and less violent ground roller occurs which results in a smoother flow on the downstream side.

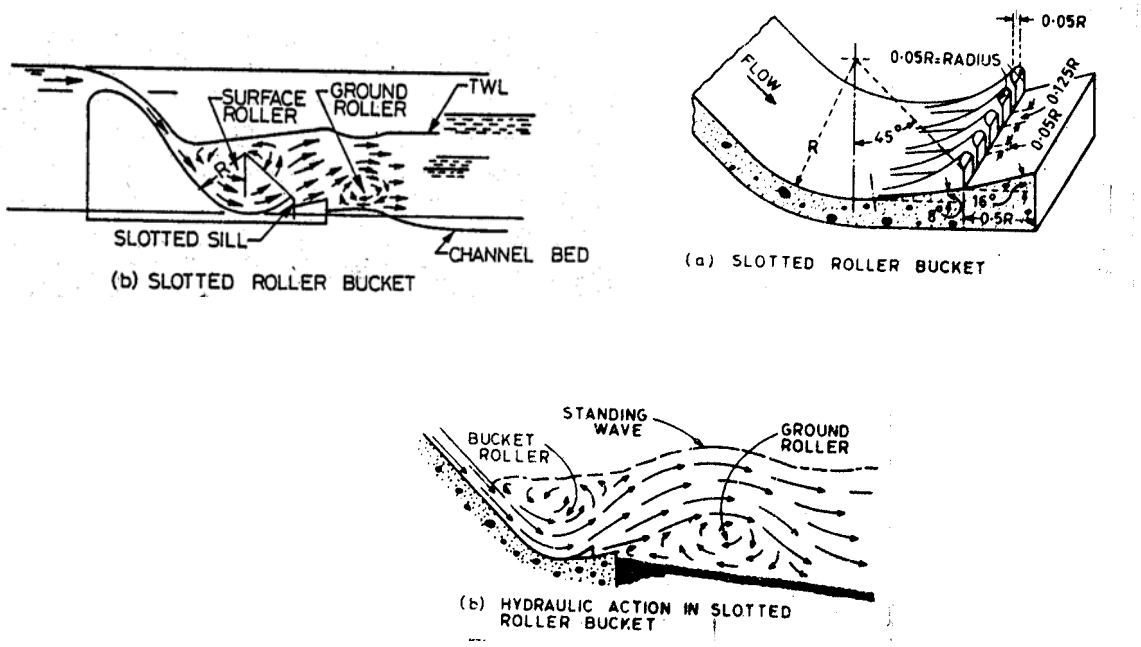


Figure 7.5 Ground and surface rollers

7.4 Ski – Jump /Deflector Bucket

This type of dissipater has a longitudinal profile, which resembles the submerged bucket. However, the deflector is elevated above the tail water level, so a jet of water is thrown clear off the dam and falls into the stream well clear off the toe of the dam. Spillways may be arranged in pairs, and then the jets made to angle inwards so that they converge and collide in mid – air. This breaks up the jets, and is very effective means of energy dissipation.

The ski – jump bucket may be used where the tail water depth is less than the sequent depth required for the formation of hydraulic jump and the riverbed is composed of stiff rock. In this case, the energy is dissipated by air resistance, breaking up of the jet into bubbles and the impact of the falling jet against the riverbed and tail water.

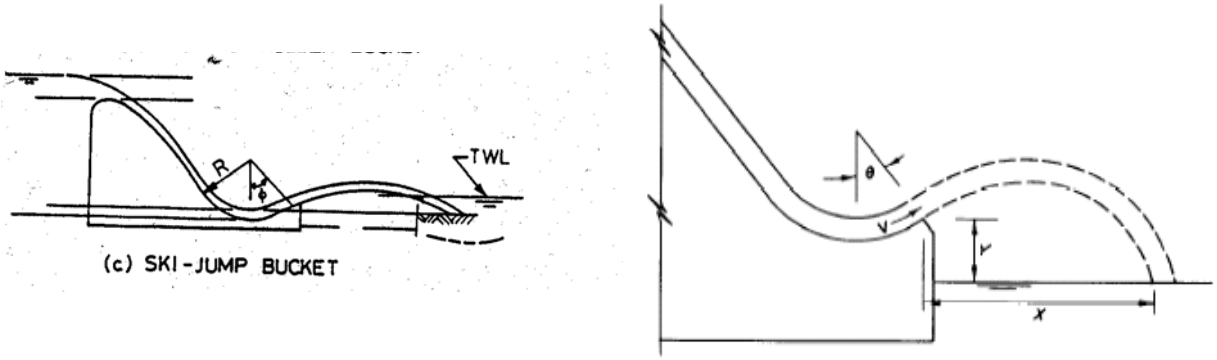


Figure 7.6

The use of ski-jump brings substantial economies where geological and morphological conditions are favorable, and particularly where the spillway can be placed over the power station. The head loss in the jet itself, whether solid or disintegrated, is only about 12% (Novak, 1996). But if the jet is split into several streams, which collide, substantial energy will be dissipated. The main benefit for energy dissipation from jet spillways is in the impact into the downstream pool. The major amount of energy dissipation occurs in the region where the jet plunges into the tailwater.

The key parameters for flip-bucket (ski-jump bucket) design are:

- The approach flow velocity and depth
- The radius R of the bucket, and
- The lip angle, θ.

At low flow, the bucket acts like a stilling basin with water flowing over the lip and the downstream face; the foundation of the bucket has, therefore, to be protected against erosion.

As the flow increases, a 'sweep-out' discharge is attained at which point the flip-bucket starts to operate properly wit a jet. Here, the impact zone of the jet has to be as far away as possible from the bucket to protect the structure against retrogressive erosion. The jet trajectory is hardly affected by air resistance for v < 20 m/s, but for velocities of 40 m/s the throw distance can be reduced by as much as 30% from the theoretical value given by $(V^2/g) \sin 2\theta$.

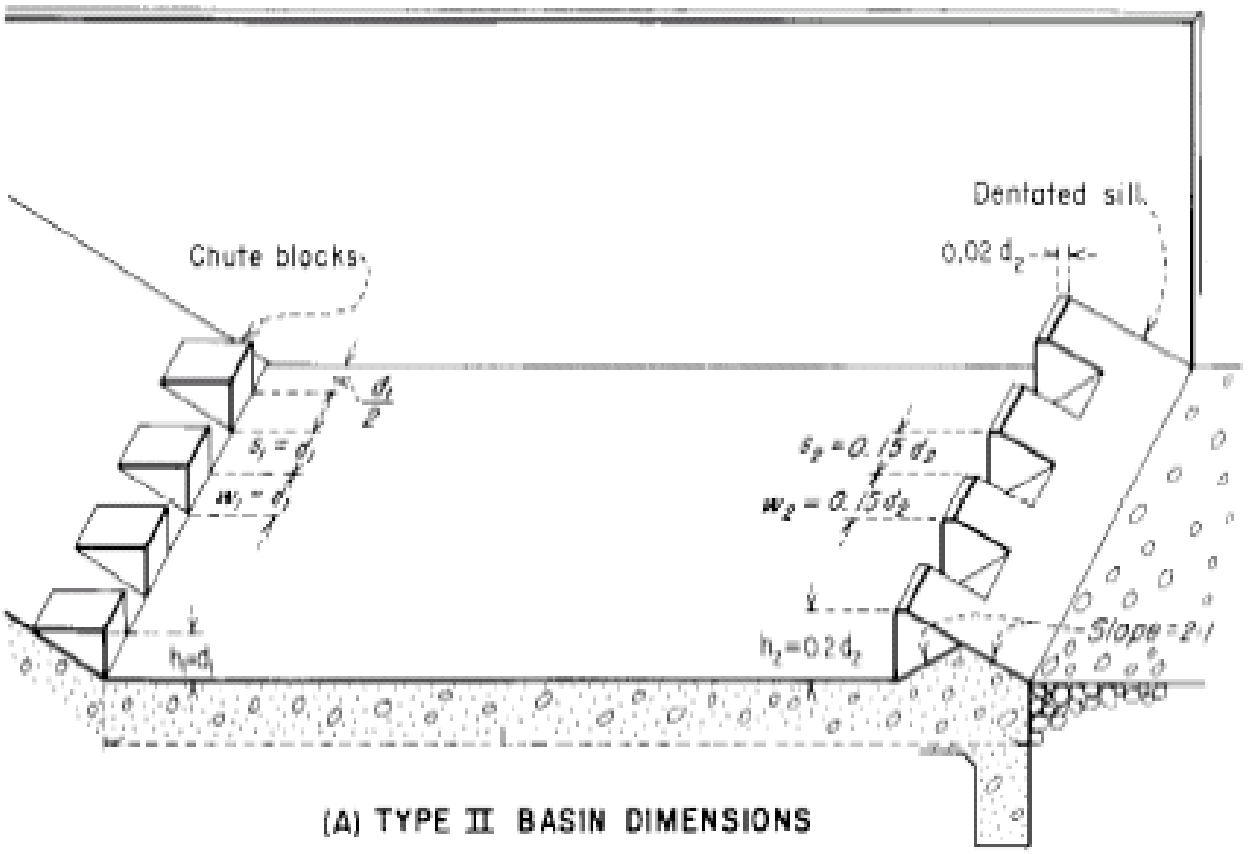
The throw distance x can also be computed from

$$\frac{x}{H_v} = \sin 2\theta + 2\cos\theta\sqrt{\sin^2\theta + \frac{y}{H_v}}$$

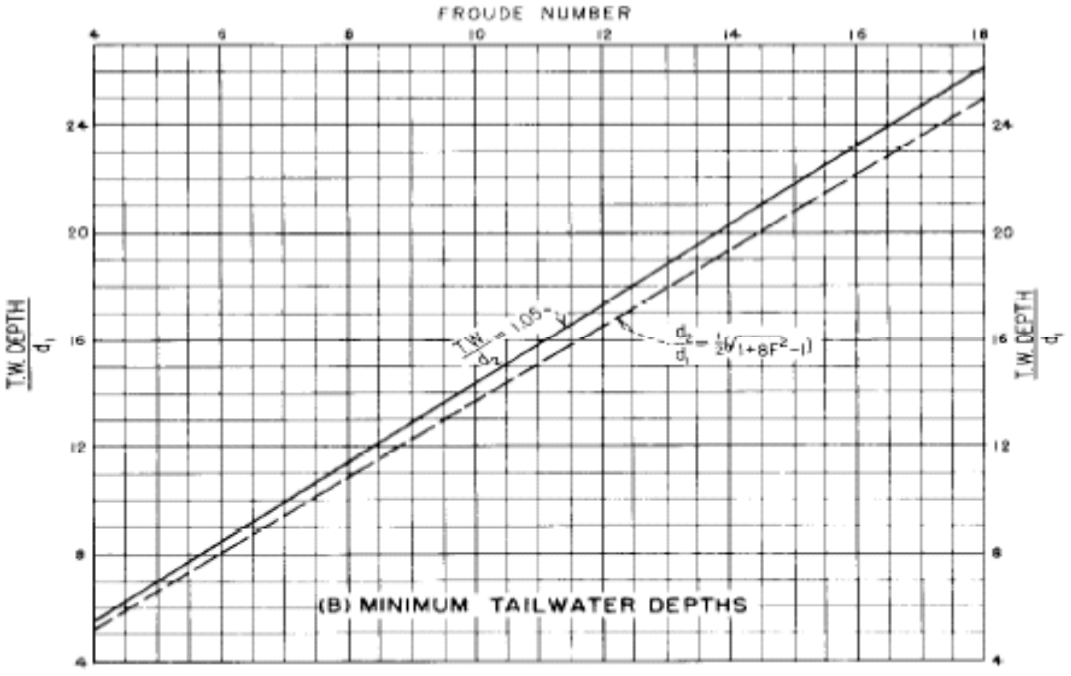
Where x = throw distance, m
y = vertical drop from lip to tail water surface, m
$H_v = V^2/2g$ = velocity head of jet at bucket lip, m
θ = bucket lip angle

Factors affecting horizontal throw distance include:

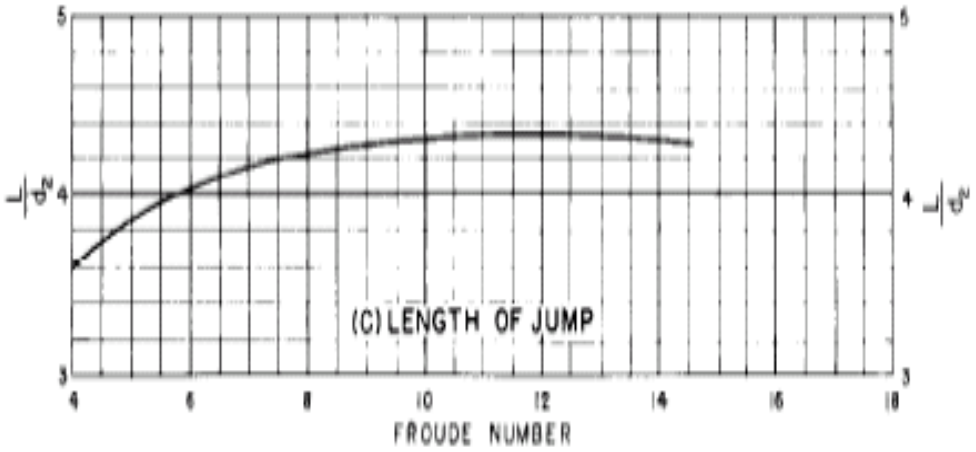
1. Initial velocity of jet
2. Bucket lip angle
3. Difference in elevation between lip and tail water

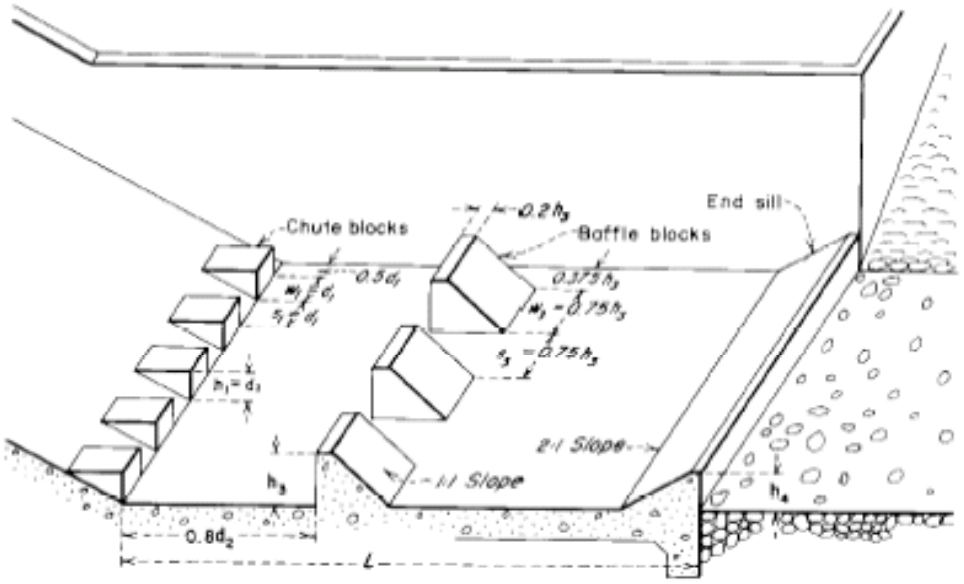


(A) TYPE II BASIN DIMENSIONS

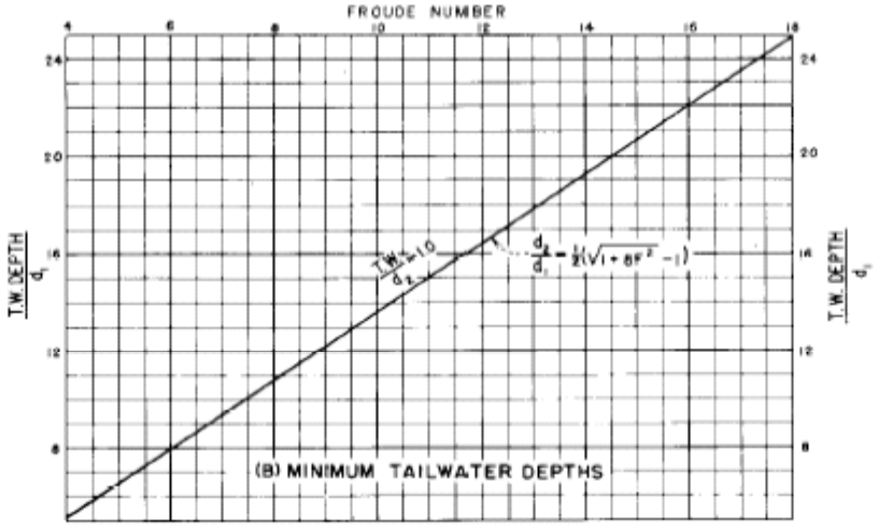


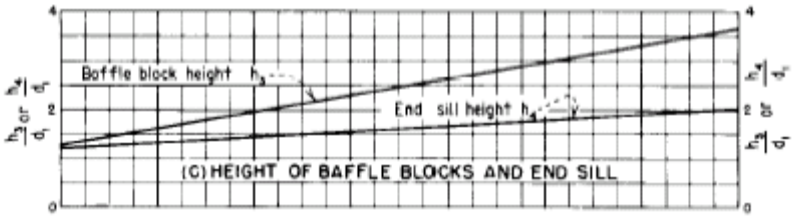
(B) MINIMUM TAILWATER DEPTHS

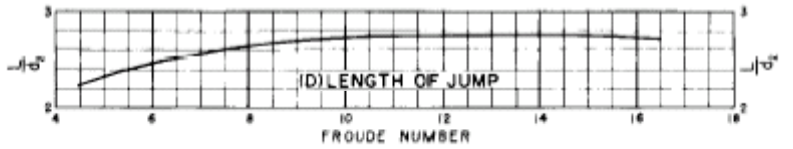


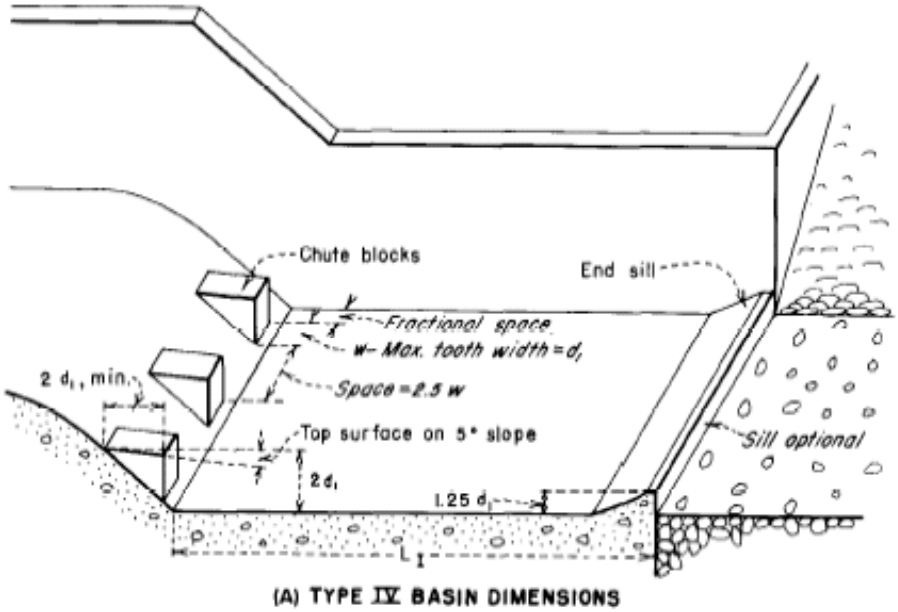


(A) TYPE III BASIN DIMENSIONS

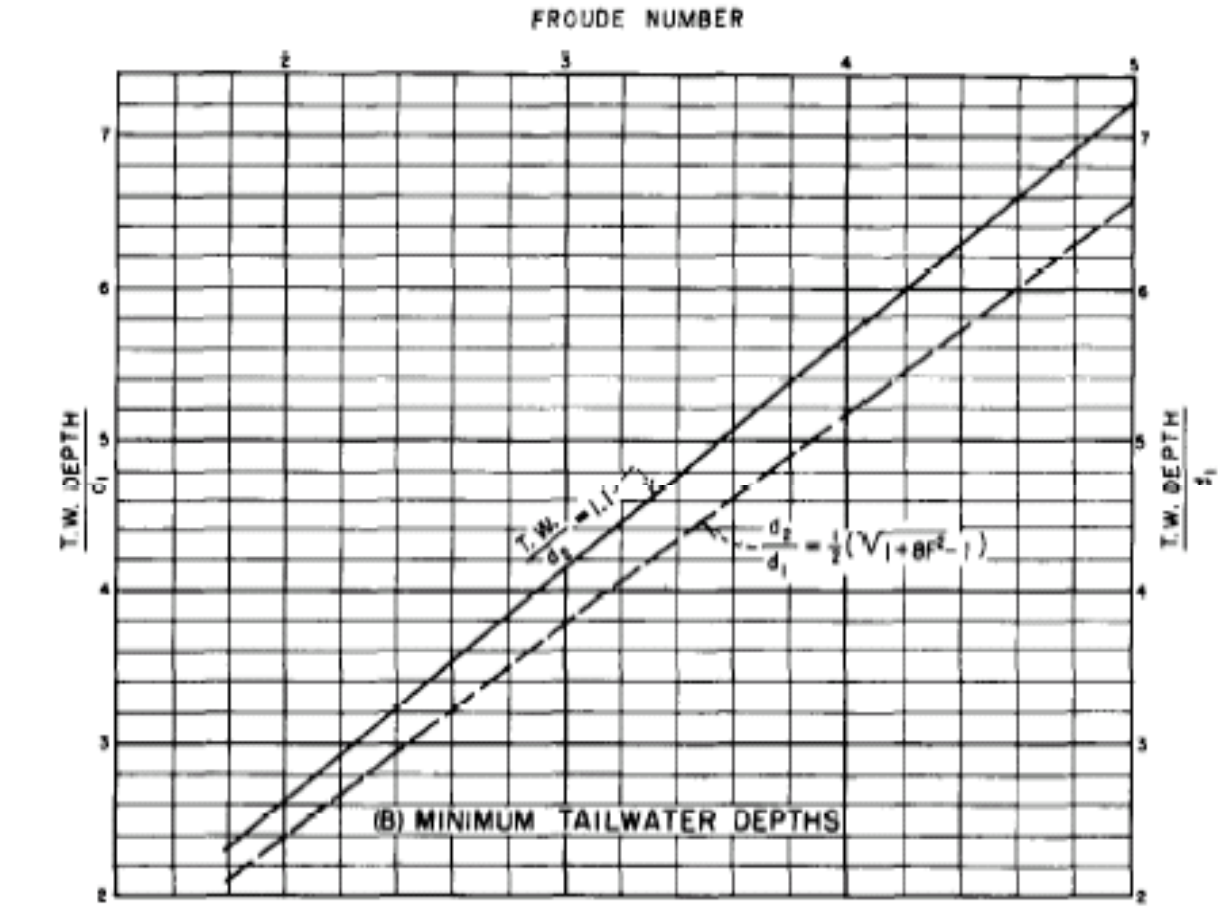


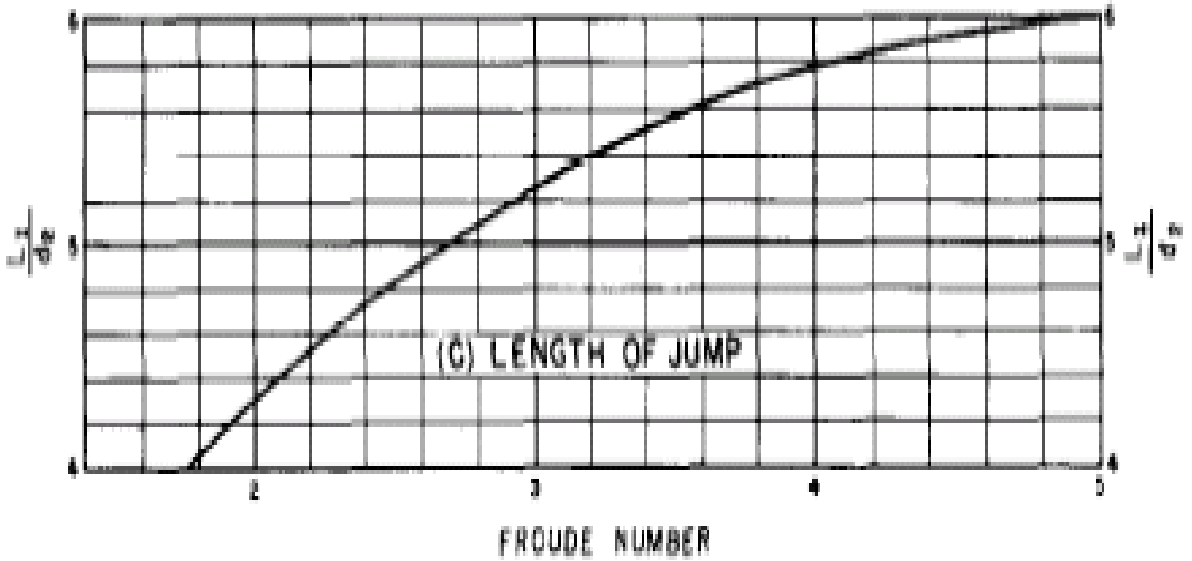






(A) TYPE IV BASIN DIMENSIONS


FROUDE NUMBER
T.W. DEPTH / d_2
T.W. DEPTH / d_1
$\frac{T.W.}{d_2} = 1.1 - \frac{1}{2}$
$\frac{T.W.}{d_1} = \frac{d_2}{d_1} = \frac{1}{2}(\sqrt{1+8F^2}-1)$
(B) MINIMUM TAILWATER DEPTHS


$\frac{L_j}{d_2}$
$\frac{L_j}{d_2}$
(C) LENGTH OF JUMP
FROUDE NUMBER

8. Outlet Works through Dams and River Intakes

8.1 Dam outlets or sluices ways

Dam outlets are provided in the body of the dam or its abutments below the crest level of the spillways so that water can be released for the various purposes for which the dam was constructed.(i.e dam outlets are opening for withdrawal of water from the dam)

The outlet of most of the dams consists of one or more sluice ways with their inlets at about minimum pool level. In most of the cases, a number of outlets are generally provided at different levels: as a single large capacity outlet may be structurally unsatisfactory or difficult to construct. More over by having more number of smaller capacity outlets, greater control on discharge can be obtained, which can be varied as & when desired. Hence, when wider fluctuations in the demand are anticipated, it is always, advisable to go in for more number of small capacity sluiceways, although it may prove to be some what costlier as compared to a few large capacity sluiceways.

Location of dam outlets

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. The function of an outlet is to discharge the stored water in to the channel d/s. For a concrete (or masonry) dam, the outlets pass through the body of the dam and are called sluice ways.

For earthen dams, it is preferred to place the outlets outside the limits of the embankments. But if no adjacent hill site is available & there is no alternative left but to pass the sluiceways through the dam, projecting collars, must be provided so as to reduce seepage along the outside of the conduit. The seepage is thus, reduced by increasing the length of the seepage path by at least 25 percent.

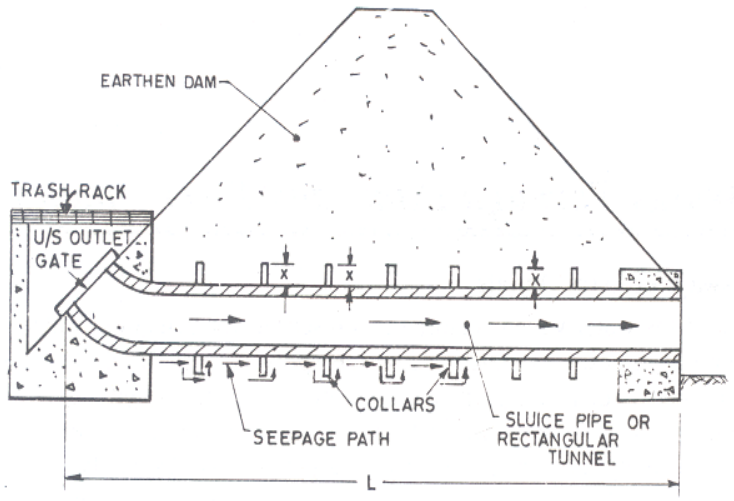


Fig. Typical outlet arrangement through an earthen dam

If L is the total length of sluiceways from u/s to d/s, the length of the seepage path will be given as

$$L + (2x)N \quad \text{or} \quad 2NX \geq 0.25L$$

Where N is the number of collars, X is the projection of the collar measured from the outer face of the conduit & L is the length of the conduit.

Attention should be given to the design of entrance of outlet. In a square-edge entrance, separation may occur. To reduce the loss at entry bell mouth entrance is usually provided for high velocity outlet works.

$$Q = CdA\sqrt{2gH}$$

Where Q= discharge

A= area of outlet sluice

H= differential head causing flow i.e the difference of u/s & d/s water level

Cd= coefficient of discharge whose value depends up on various factors such as the type of gate & trash rack provided, conduit friction, transitions etc...

In an outlet work the following hydraulic losses should be considered:-

i) Iterance loss:- depends up on the shape & size of the entrance

It may be taken as $0.5\frac{V^2}{2g}$ for square edged entrance and equal to $0.04\frac{V^2}{2g}$ for bell mouthed iterance

Where:- V is the flow velocity through the conduit.

ii) Friction Loss: - depends on the surface of the conduit, its size & length and velocity

$$hl = f\frac{L}{D}\frac{V^2}{2g}$$

Where: - f= friction factor

L = length

D= diameter

ii) The head loss through the gate : depends up on the type of gate & valve provided

- A loss of about $0.2\frac{V^2}{2g}$ may be taken for fully open gate & butterfly valve
- Head loss is taken as nil for ring follower gates

iii) The head loss through the trash rack depends up on the design of the trash rack and the velocity through it

Velocity in trash rack(m/s)	Head loss in meters
0.15	0.006
0.3	0.03
0.45	0.09
0.62	0.15

The net effective head which is responsible for flow, should be taken as H_{eff} = differential head (H) – head loss

Then the discharge is calculated as

$$Q = A\sqrt{2gH_{eff}}$$

8.2 Intake structure

An intake structure is required at the entrance of an outlet conduit when the outlet is not an integral part of the dam. An independent intake structure is constructed through which the water is withdrawn from the river (or the reservoir). The primary function of the intake structure is to permit withdrawal of water from the reservoir (or river) over a predetermined range of reservoir levels and thus to protect the conduit from being damaged or clogged by ice, trash, debris, waves etc.

An intake structure may vary from a simple concrete blocks supporting the end of the conduit pipe to huge concrete towers, depending up on the various factors such as reservoir characters tics, capacity and discharge requirements, climatic conditions etc.

Types of intake structures

i) Simple submerged intakes it consists of a simple concrete blocks or a rock filled timber crib supporting the starting end of the withdrawal pipes as shown below

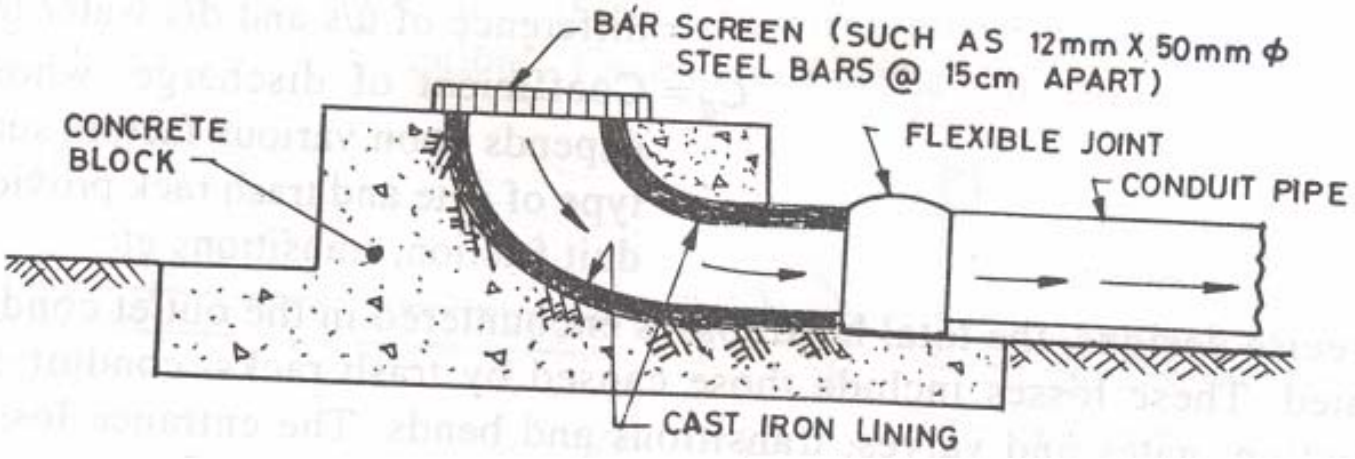


Fig Simple concrete block submerged intake

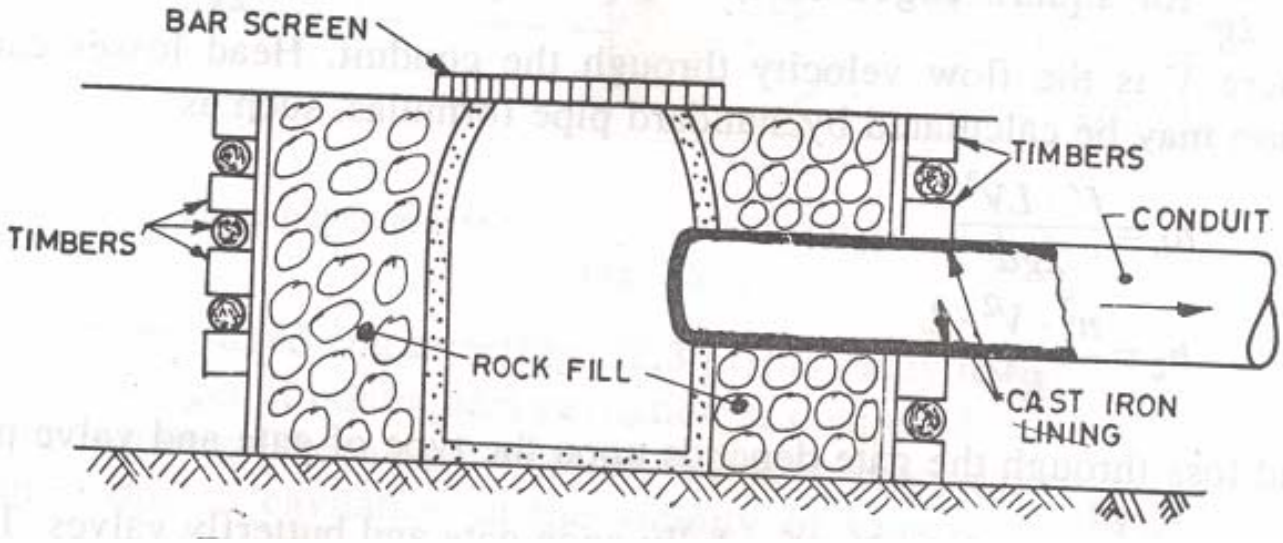


Fig Rock filled timber crib submerged intake

Such intake structures should be in the river or the reservoir at a place where they may not get buried under the sediment. If the capacity of a project is not much and if it is constructed for a single purpose, then such type of intake devices may be used to justify the cost benefit analysis. This type of intake is particularly suitable for water supply intake from small rivers. They are not used on bigger projects as their main disadvantage is the fact that they are not easily accessible for repairing of their gates etc

ii) **Intake Towers**

Intake towers are generally used on large projects and where there are large fluctuations of water level. Openings at various levels called ports

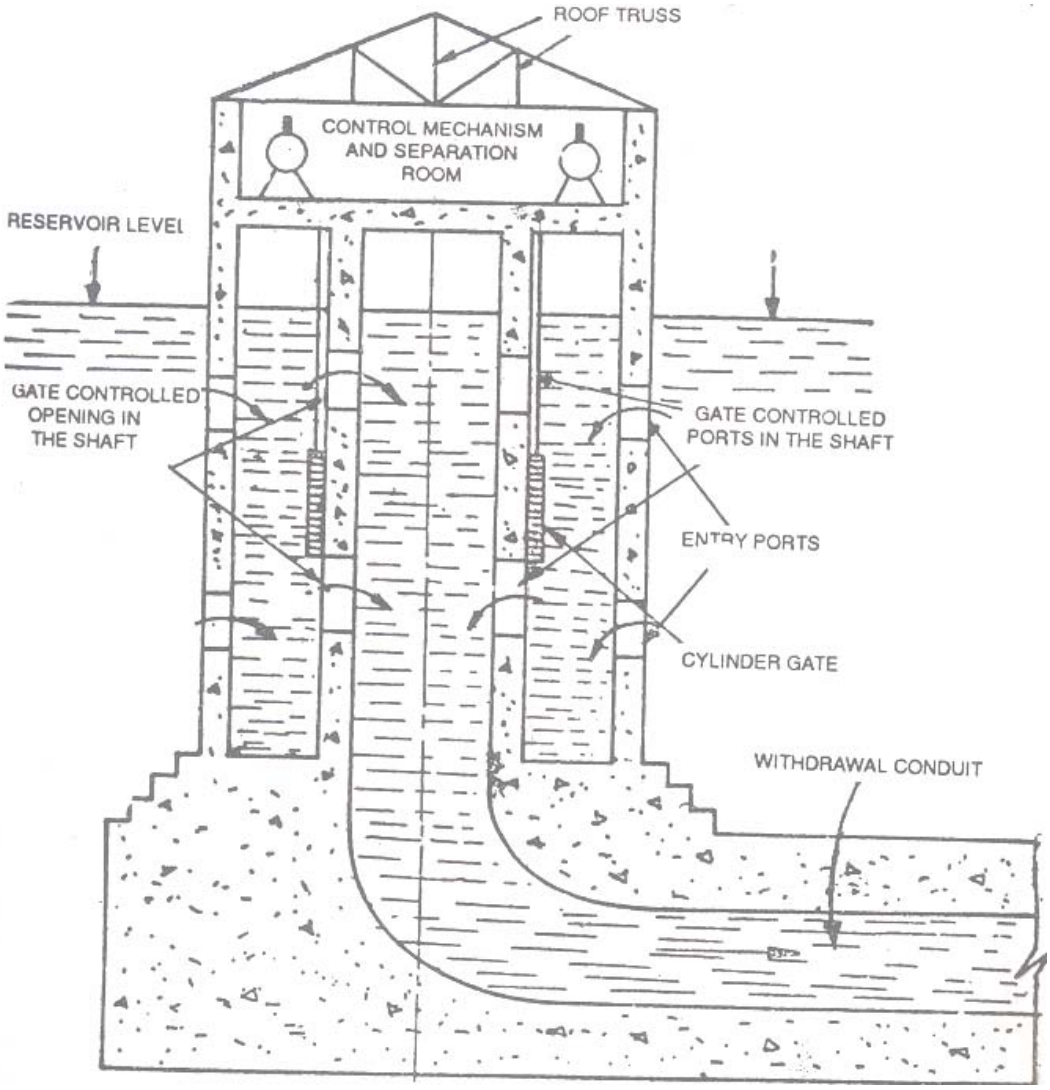
are generally provided in these concrete towers, which may help in regulating the flow through the towers and permits some selection of the quantity of water to be withdrawn.

There are two major types of intake towers, viz.

i) Wet intake towers

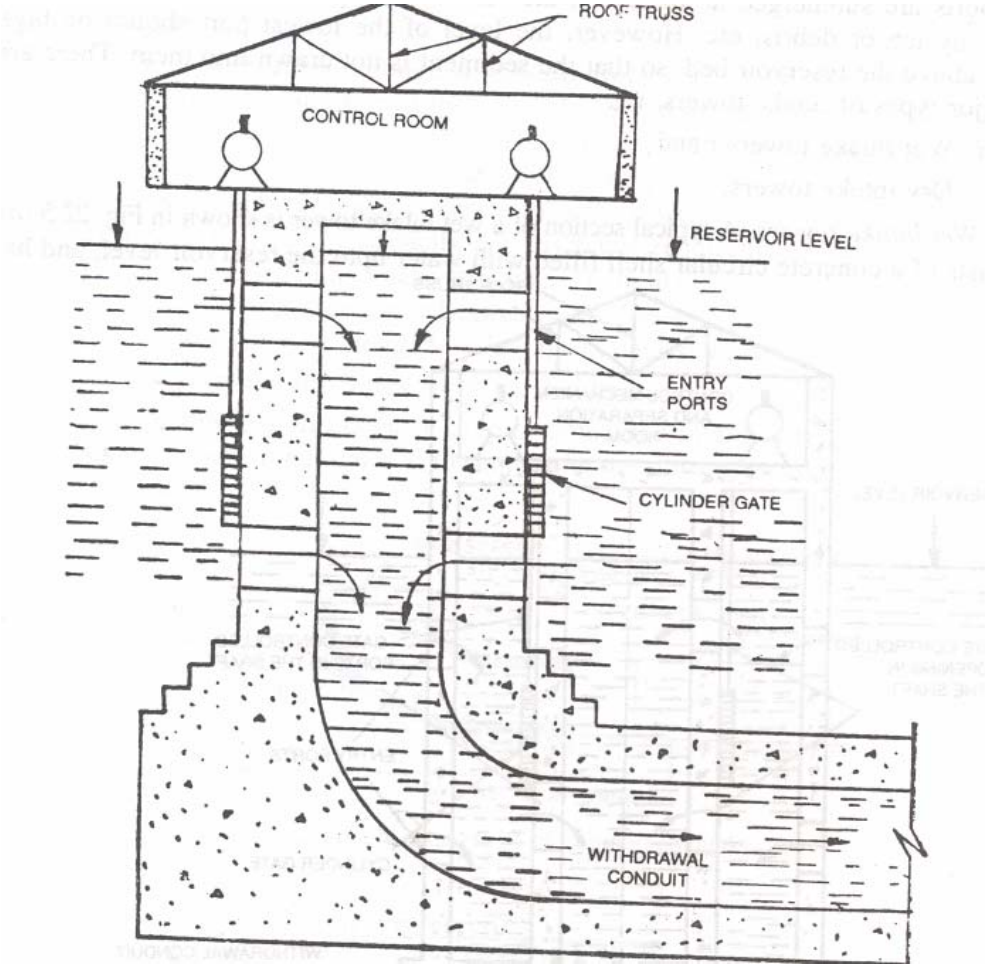
ii) Dry intake towers

Wet- intake tower:- A typical section of a wet intake tower shown below, consists of a concrete cylindrical well filled with water to the level of the reservoir. There is a vertical shaft inside the well which is connected to the withdrawal conduit. The water enters the well through entry ports. It then enters the vertical shaft through gate- controlled ports (openings). A control room is usually constructed over wet-intake tower.



Dry intake tower

The essential difference b/n a wet intake tower and a dry intake tower is that, in the case of a wet intake tower the water enters from the entry ports in to the tower and it enters in into the conduit pipe through separate gate controlled openings. Whereas in a dry intake tower the water is directly drawn in to the withdrawal conduit through the gated entry ports as shown.



A dry intake tower will therefore have water inside the tower if its gates are closed, whereas the wet intake tower will be full of water even if the gates are closed.

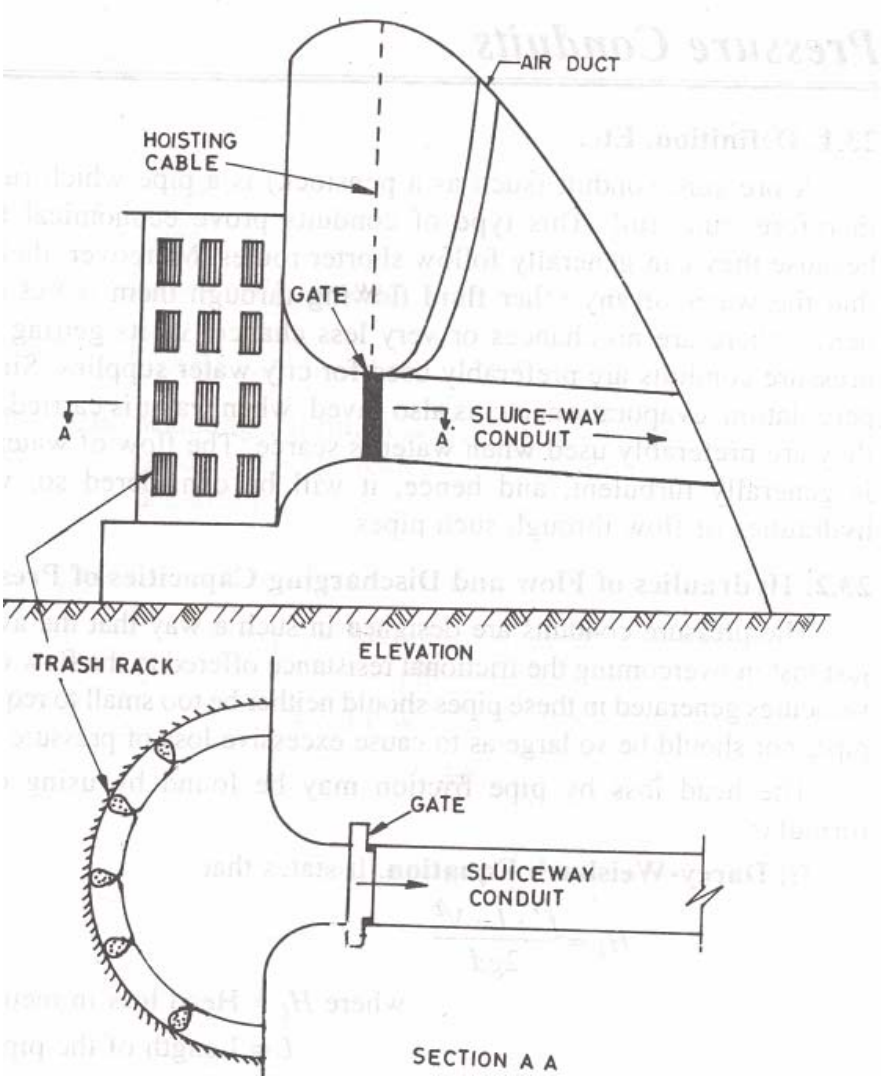
When the entry ports are closed a dry intake tower will subjected to additional buoyant force & hence must be of heavier construction than the wet intake towers. However, the dry intake towers are useful and beneficial in the sense that water can be withdrawn from any selected level of the reservoir by opening the port at that level

8.3 Trash racks

The entrance to intakes and dam outlets are generally covered with trash racks so as to prevent the entry of debris, ice etc in to the conduit.

These racks are generally bar screens, made from steel bars spaced at 5 to 15 cm center to center in both directions depending upon the maximum size of the debris required to be excluded from entering the conduit.

The velocity of flow through the trash rack is kept low (generally less than 0.62m/s) so as to minimize losses.



Half Cylinder-trash Rack and Tractor Gate installation