UNIT - 1

Storage Works - Reservoirs.

Dams are Constructed across the rivers and streams to Create an artificial lake or reservior behind it.

Dams and reservoirs are the most important and expensive elements in multi-purpose river basin development.

⇒ Depending upoon the purpose Served, reservoirs may be classified as under

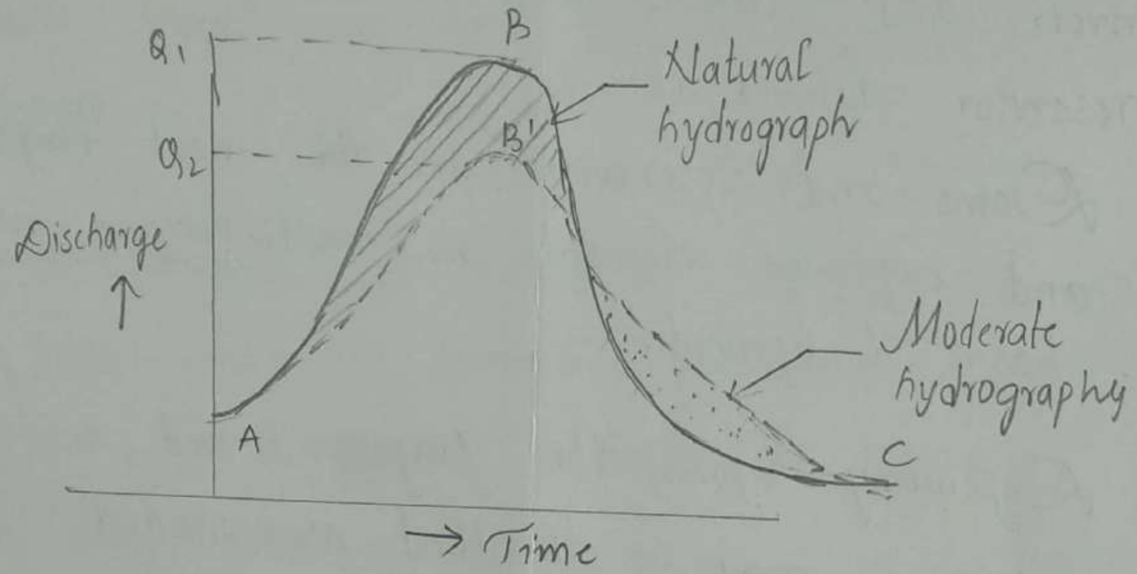
- Storage or Conservation reservoirs
- Flood protection reservoir
- Distribution reservoir
- Multipurpose reservoir

Storage or Conservation reservoirs :-

Storage reservoirs are primarly used for water Supplies for irrigation, hydroelectric development, domestic & industrial Supplies. A river doesn't carry the Same quantity of water throughout the year and may carry large quantities in the other part of the year.

Flood Control Reservoirs :-

Flood Control reservoirs are those which Store water during flood and release it Gradually at a Safe when the flood Reduces.



Distribution Reservoir :- A distribution reservoir is a Small storage reservoir used for water Supply in City.

Multipurpose Reservoir :- A Multipurpose reservoir is a Small Storages reservoir m Serves more than One purpose.

For ex, a Reservior designed to protect the down Stream area from floods, and to store water for irrigation & hydroelectric purposes is a Multipurpose Reservoir.

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Selection of Site for Reservoir.

⇒ The Geological Condition of the catchment area should be such that percolation looses are minimum & maximum run-off is obtained.

⇒ Suitable Dam Sigte must exist. The dam should be founded on sound watertight rock base, & percolation below the dam should be minimum.

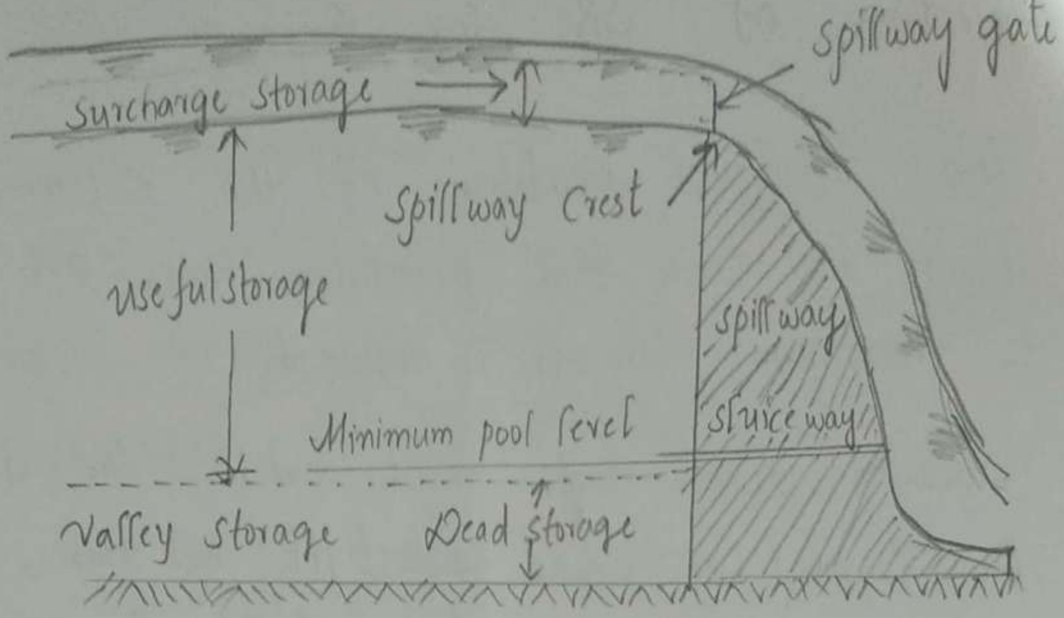
⇒ The reservoir basin should have narrow opening in the valley so that the length of the dam is less.

⇒ The cost of real estate for the reservior, including road, rail road, dwelling re-location etc must be as less as possible.

⇒ The Topography of the reservoir site should be such that it has adequate capacity without submerging excessive land & other properties.

Zones of Storage in a Reservoir :-

1) Useful Storage
2) Surcharge Storage
3) Dead Storage
4) Bank storage
5) Valley storage



Zones of Storage in a Reservoirs.

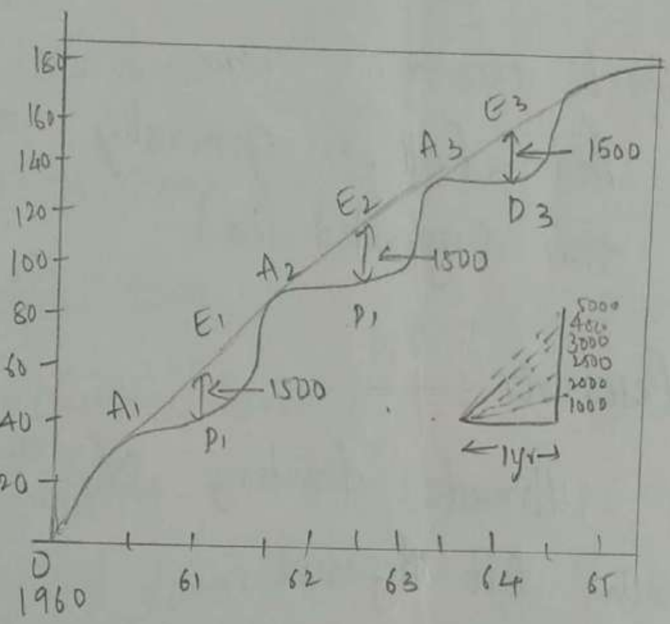
⇒ Storage Capacity And Yield :-

Yield :- Yield is the amount of water that can be supplied from the reservoir in specified interval of time. The interval of time chosen for the design varies from a day for small distibution reverviors to a year for large conservation reservoirs.

- Safe yield or firm
- Secondary yield
- Average yield.
- Mass inflow curve
- Demand Curve.

Determination of Yield From Reservoir of Specified Capacity.

The following is the procedure of determining the Safe yield from a reservoir of a storage capacity.



Reservoir Sedementation :-

⇒ All the rivers carry certain amount of slit eroded from the catchment area during heavy rains. The extension of erosion, & hence the slit load in the stream depends on following factors

→ Nature of soil of the catchment area

⇒ Topography of the catchment area

⇒ Vegetation Cover

⇒ Intensity of rainfall

The nature of the Soil of the Catchment area is an important factor. If the Soil is Soft, there is always a possibilety of Sheet erosion.

The bed load is generally much Smaller -10 to 15 % of the Suspended load.

Life of Reservoir :-

The ultimate denstiny of a reservoir is to be filled with silt deposits.

To allow for silting, a Certain percentage of the Total Storage is usually left unutilised and is called "Dead storage"

The trap efficiency is a function of the ratio of reservoir Capacity to total flow.

$$\boxed{\eta = f\left(\frac{\text{Capacity}}{\text{Inflow}}\right)}$$

Dams :-

A Dam is a hydraulic structure constructed across a river to store water on its upstream side.

Types of Dams.

⇒ Storage dam

⇒ Diversion dam

⇒ Detention dam

⇒ Overflow dam

⇒ Non-overflow dam

⇒ Rigid dams

⇒ Non-rigid dams.

Factors ~~Affecting~~ Governing Selection of type of dam :-

⇒ The Selection of a type of a dam at a given site depends up on many physical factors.

1) Topograph

2) Geology and Foundation Condition.

3) Materials of Construction.

4) Spillway Size and Location

5). Road way

6). Length and Height of Dam.

7) Life of Dam

⇒ Topography :- The first choice of dam is usually governed by the topography for the Site.

A low narrow V-shapped valley Suggests an arch dam, with a Separate Spillway.

⇒ Geology and Foundation Condition :- The next important Factor is the Geology and Foundation Condition.

If the foundation Consists of Sound rock, with no fault or fissures,

Silt or fine Sand foundation have the problems of Settlement.

Gravity Dams or rockfill dams are not Suitable on clay foundation.

⇒ Materials of Construction :-

The next important factor is the availability of materials of Construction for dams.

The Cost of Construction of particles of a particular type of dam will depend upon the availabity of the material is nearby area so that transportation charges are reduced.

⇒ Spill way Size and Location ÷

Good site for the location of a seperate spill way is essential especially in the case of earth or Rockfill dam.

The best site for a dam way be considered to be one where a deep gorge and a flank at its sides are seperated by a hillock while the spillway can be located in flank.

⇒ Materials ÷ Materials required for a particular type of Dam should be available nearby, This would very much reduce the cost of construction.

⇒ Reservoir and Catchment Area ÷ The side should ensure adequate area storage capacity of reservoir basin at a minimum cost.

- The cost of land and property submerged in the water spread area should be minimum.

- The Geological Conditions of the catchment area Should be Such that percolation lossess are minimum and maximum run-off is obtained.

  - less evaporation losses because of reduction in the water Spread area and less likelihood of weed growth.

⇒ Communication :- It would be preferable to Select a site which is connected by a road or rail link or can be conveniently connected to the site for transportation of cement, labour, machinary food and other equipment.

⇒ Locality :- The Surroundings near the site should perferably by healthy and free of mosquitoes etc, as labour and staff colonies have to be constructed near the site.

UNIT-II Dam-II Gravity Dams.

Introduction:

→ A Gravity dam is a structure so proportioned that its own weight resists the force Exerted upon it.

→ This type of dam is most permenent one, requires little maintanence & is most commonly used.

→ A gravity dam is mosty straight in plan & is known as "Straight Gravity dam". However, it may also be slightly curved in plan.

→ A curved gravity dam resists the External forces by its weight & not by arch action.

→ Most of the gravity dam are solid, so that no bending stress is introduced at any point & hence, they are sometimes known as 'Solid Gravity dams'.

→ A Gravity dam, however, can be hallow & is known as "Hollow Gravity dam"

→ When good foundations are available, gravity dam can be built upto any height. the heighest dam in the world are gravity type

Forces acting on a Gravity dam:-

1. water pressure
2. Weigth of dam
3. Uplift pressure
4. Pressure due to earthequake
5. Ice pressure
6. wave pressure
7. Silt pressure
8. wind pressure.

Water pressure:-

→ This is the major External force acting on dam.

→ when the upstream face of the dam is vertical, the water pressure acts horizontally.

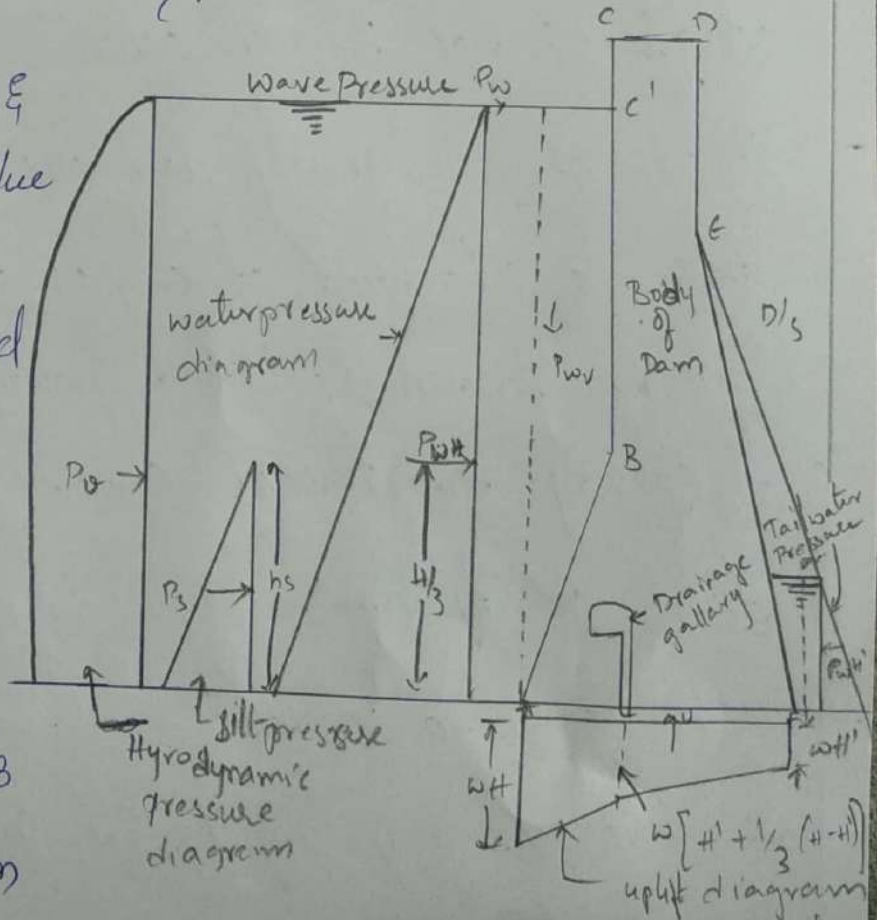
i) Horizontal component P_{WH} &

ii) Vertical Component P_{WV} due to weight of water "supply" supported by the inclined face

Thus, horizontal force

$$P_{WH} = \frac{wH^2}{2}$$

→ This acts at a heigth $H/3$ from the base of the dam



Vertical Compondnt, P_{wV} = weight of water contained by Coloum AA'C'B & acting at the Center of Gravity of the area.

Ily, if there is tail water of Height H'. on the downsteam Side, it exerts both horizontal pressure ($P_{wH'}$) as well as Vertical pressure ($P_{wV'}$)

Thus, horizontal pressure $P_{wH'} = \frac{wH'^2}{2}$

& Vertical pressure $P_{wV'}$ = Weight of water Contained by coloumn FF'E'

Weigth of the dam:

- The weigth of the dam is major resisting force.
- The Cross section of the dam may be divided into Several triangles & rectangles, & the weigths w_1, w_2, w_3 etc of each of these may be computed conveniently.

Uplift Pressure:

- The uplift pressure is defined as the upward of water
- as it flows or Seep through the body of the dam or its foundation. A proportion of the weight of the dam will be Supported on upward pressure of water; hence net foundation reaction due to vertical force will reduce

Area over which uplift acts:-

→ there are two schools of thought existing in the old literature for this.

→ one school of thought recommands that a value one-third to two-third of the area should be considered as effective over which the uplift acts.

→ The second school of thought, recommands that the effective area may be taken appro. equal to the total area.

The present practice in the design is to follow U.S.B.R recommendation that the total area should be considered as effective to account for uplift.

U.S.B.R Recommendation:-

→ U.S.B.R suggests the adaption of uplift pressure intensities equal to the hydrostatic pressure of water at the toe & heel joined by by a straight line in b/w. Sometimes drainage galleries are also provided in the body of the dam which releases the uplift pressure built up under it.

The uplift pressure at heel $A = \omega H$.

uplift pressure at toe $F = \omega H'$

uplift pressure at gallery $= \omega [H' + \frac{1}{3}(H - H')]$.

Indian standard Recommandations:-

The area factor or the percentage of area on which uplift acts & the intensity factor or the ratio which the actual intensity of uplift pressure bears to the intensity gradient extending from heading water to tail water at various points.

Criteria ~~A~~ for design:

(a) uplift pressure distribution in the body of the dam shall be assumed, in case of both preliminary & finial designs, to have an intensity which at the line at the formed drains exceeds the tailwater pressure by 1/3 the differential b/w reservoir level & tail water level

(b) uplift pressure distribution at contact plane b/w the dam & its foundations & within the foundation shall be assumed for preliminary designs to have an intensity which at the line of drain exceeds the tail water pressure by 1/3rd the differential b/w the reservoir & ~~w~~ tail water heads

(c) it is assumed that uplift pressure are not affected by earthquackes.

Pressure due to Earthquake:

The wave impact accelerations to the foundations under the dam and causes its momente. In order to avoid rupture, the dam must also more along with it. this accelaration introdues an inertia force in the body of dam & Sets up stresses initially in Lower Layers & gradually in the whole body of the dam. Earthquake wave may travel in any direction.

Intensity of Earthquake:

The intensity of an Earthquake is measure of strength of Sheaking during the earthquake. it is rated in numbers from 1 to 12.

Spectra of Earthquarke: Spectrum of an earthquacke is representation of maximum dynamic response of idealised Structure during an earth quacke. The maximum response is plotted against the natural period of vibration (T) & Can be Expressed in terms of the following:

(i) Maximum absolute acceleration
(ii) Maximum relative velocity
(iii) Maximum relative displacement.

Design Seismic Coefficient for diffrent zones.

(a) Seismic coefficient method.

As per IS:1893-1984, the design value of horizontal Seismic Coefficient (α_h) shall be computed as given by the following Expression.

$$\alpha_h = BI\alpha_0$$

Where B = Soil-foundation System factor, the value of which may be taken as 1.0 for dams.

I = Importance factor, the value of which is taken as 2.0 for dams

α_0 = Basic Seismic Coeffecient, the value of which for each of the four zones is given in table 8.1 below.

Seismic zone	II	III	IV	V
Basic Seismic Coefficient α_0	0.02	0.04	0.05	0.08

Thus, Substituting the values of B & I for dams ~~redu~~

$$\alpha_h = 2\alpha_0$$

As α_0 varis from 0.02 to 0.08 the value of α_h will vary from 0.04 to 0.16 for zone II to zone V.

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(b) Response Spectrum method:

- As per IS :- 1893 - 1984, the design value of horizontal Seismic Coefficient is Computed from the following Expression.

$$\alpha_h = \beta I F_0 \frac{S_a}{g}$$

Where β = ~~Soil~~ foundati

F_0 = Seismic zone factor for average accelation spectra, the value of which for each of four zones in table.

$\frac{S_a}{g}$ = average acceleration ~~pee~~ spectra as read from the graph for appropriate natural period (T) & damping of the Structure.

Seismic Zone	II	III	IV	V
Seismic Zone factor F_0.	0.10	0.20	0.25	0.40

Sub the value of $\beta = 1.0$ & $I = 2.0$. dams ~~for~~ &.

$$\alpha_h = 2 F_0 \frac{S_a}{g}$$

$$T = 5.55 \frac{H^2}{B} \sqrt{\frac{w_m}{g E_s}}$$

where

H = height of the dam (m)

B = base width of the dam (m)

w_m = unit weigh of the material of dam (KN/m3)

E_s = Modulus of elasticity of the dam (KN/m3)

Taking $w_m = 24 KN/m^3$, $E_s = 2.1 \times 10^7 KN/m^2$, $g = 9.81 m/sec^2$ &

$B = 0.7H$,

$$T = 0.55 \left(\frac{H^2}{0.7H}\right) \sqrt{\frac{24}{9.81 \times 2.1 \times 10^7}} = 2.41 \times 10^{-3} H$$

thus when $H = 100m$, $T = 0.241$ Seconds for which S_a/g for 5% damping comes out to be 0.196

Vertical Seismic Coeffient:- the vertical semic coefficent may be taken as half of the horizontal semic coeffient. Hence

$$\alpha_v = 0.5 \alpha_h$$

$$A_h = \frac{Z}{2} \cdot \frac{I}{R} \cdot \frac{S_a}{g}$$

where Z = zone factor

I = Importance factor

R = Response reduction factor

$\frac{S_a}{g}$ = average response acceleration coeffient for rock

$$\frac{S_a}{g} = \begin{cases} 1 + 15T & (for\ 0.00 \le T \le 0.10) \\ 2.50 & (for\ 0.10 \le T \le 0.40) \\ 1.00/T & (for\ 0.40 \le T \le 4.00) \end{cases}$$

Seismic zone	II	III	IV	V
Seismic intensity	Low	moderate	Severe	very Severe
zone factor Z	0.10	0.16	0.24	0.36

(a) Effect of a Horizontal Earthquake acceleration

1. Inertia force in the body of the dam
2. Hydrodynamic pressure of water.

Horizontal Inertia force:-

The inertia force dF on any elemental lamina at any height of the dam

dw = weight of the element limina at any point heigth of the dam

g = accelaration due to gravity

α_h = horizontal accelaration Coefficient = $\frac{\text{Earth quake acceleration}}{\text{Acceleration due to gravity}}$

Horizontal earthquake acceleration = $\alpha_h g$

Inertia force = mass x acceleration

$$= \frac{dw}{g} \cdot \alpha_h g = dw \cdot \alpha_h$$

Seismic Coefficient method:-

Consider an elementary strip of thickness dy at a depth y from the top, the width of the strip is

$$B_y = \frac{B}{H} \cdot y$$

$$\alpha_y = 1.5\alpha_h\left(1-\frac{y}{H}\right)$$

if w_m is the unit weight of the material,

$$dw = B_y \cdot dy \cdot w_m$$

∴ Elementary horizontal inertia force on the strip $= dF_y = dw \cdot \alpha_y$

∴ Base Shear

$$F_H = \int_0^H dF_y = \int_0^H B_y \cdot dy \cdot w_m \cdot \alpha_y$$

$$= \int_0^H \left(\frac{B}{H}y\right)\left(1-\frac{y}{H}\right)1.5\alpha_h \cdot w_m\, dy$$

$$= 1.5\,\alpha_h\,\frac{B}{H}\,w_m\left(\frac{y^2}{2}-\frac{y^3}{3H}\right)_0^H$$

$$F_H = 0.25\,\alpha_h B H\, w_m = 0.5\alpha_h W.$$

or

$w = \frac{1}{2}BH\,w_m$ = total weight of the dam.

$$F_H \simeq 0.6\alpha_h W$$

$$F_B = 0.6\,W\alpha_h$$

Computation of total horizontal force on the dam its moment.

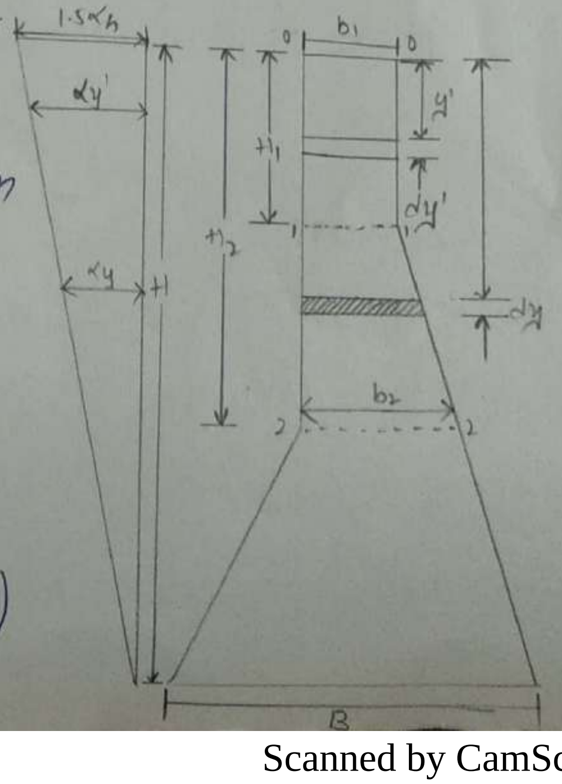
At any depth y' below the top, consider a strip of thickness dy'. the width of the strip $b'_y = b_1$ & the acceleration Coefficient $\alpha'_y = 1.5\alpha_h(1-y'/H)$

Hence elementary horizontal force

$$dF_1 = (1\times b_1 dy')\,w_m \cdot \alpha'_y$$

$$F_1 = \int_0^{H_1}(b_1 dy' \cdot w_m)\,1.5\,\alpha_h(1-y'/H)$$

$$F_1 = 1.5\,\alpha_h\,w_m\,b_1 H_1\left(1-\frac{H_1}{2H}\right)$$



$$M_1 = \int_0^{H_1} (b_1 dy' w_m)\, 1.5 \alpha_h \,(1 - y'/H)(H_1 - y')$$

$$M_1 = 1.5 \alpha_h w_m b_1 \frac{H_1^2}{2}\left(1 - \frac{H_1}{3H}\right)$$

Consider an elementary strip of thickness dy at any depth y below the top of the dam. The width $b_y = b_1 + n(y + H_1)$.

$$F_2 = F_1 + \int_{H_1}^{H_2} \{b_1 + n(y - H_1)\} dy\, w_m \,(1 - y/H)\, 1.5 \alpha_h$$

$$M_2 = M_1 + F_1 (H_2 - H_1) + \int_{H_1}^{H_2} (b_y \cdot dy \cdot w_m)(1 - y/H)\, 1.5\, \alpha_h (H_2 - y)$$

(b) Effect of vertical Earthquake acceleration.

(a) for Seismic Coefficient method:

At the top of the dam, it would be equal to 0.75 times α_h ~~value given by~~ & reducing linerly to zero value at the base

(b) for responce Spectrum method.

At the top of the dam, it would bequal to 0.75 times α_h ~~value given to~~ and reducing Linearly to zero value at the base

ICE pressure:

→ The ice pressure is more important for dams Constructed in Cold Countries, or at higher elevations.

→ The ice formed on the water Surface of the reservoir is Subjected to Expansion & Contraction due to temp. variations

Wave pressure.

. wave are genrated on the reservior Surface because of the wind blowing over it. wave pressure depends on the weight of the waves developed

$$h_w = 0.0322\sqrt{V \cdot F} + 0.763 - 0.271(F)^{1/4} \text{ for } F < 32\,km$$

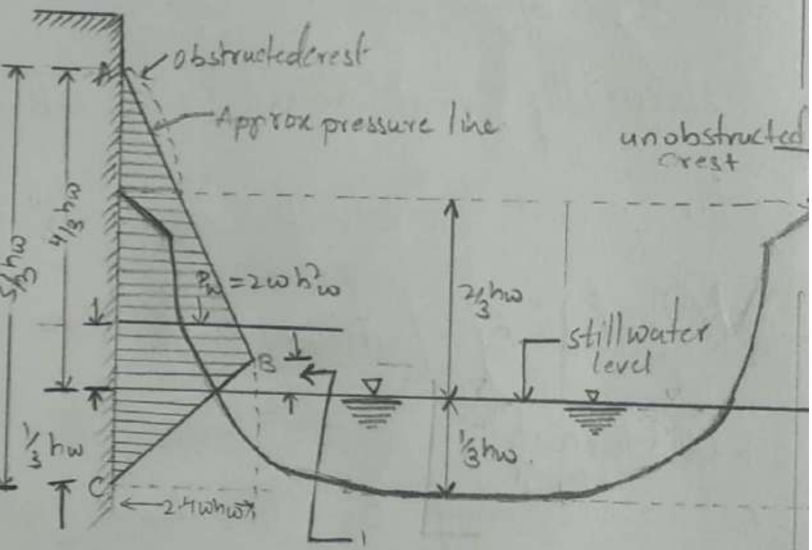
$$h_w = 0.0322\sqrt{V \cdot F.} \text{ for } F > 32\,km$$

where

h = height of waves in meter, b/w trough & Crust.

V = wind Velocity in km per hour

F = fetch or straight length of water Expansion in km



The pressure intensity due to waves is given by

$$P_w = 2.4wh_w$$

$$P_w = (2.4wh_w) \times \frac{1}{2}\left(\frac{5}{3}h_w\right) = 2wh_w^2$$

Silt pressure.

→ The river bring silt and be debris along it.

→ The Silt Load get deposited to an appreciable Extent when dam is Constructed.

$$P_s = \frac{1}{2}\gamma_s' h_s^2 \frac{1-\sin\phi}{1+\sin\phi}$$

→ If the upstream is inclined, the vertical weight of silt supported on the slope also acts as vertical force.

Wind pressure:-

→ it is a minor force & need hardly be taken into account for the design of dams.

→ wind pressure is required to be considered only on that portion of the super structure which is Exposed to the action of wind. &

→ Normally wind pressure is taken as 1 to 1.8 kN/m^2 for the area Exposed to the wind pressure

Modes of failures:

1. overturning:-

→ the overturing of the dam sections takes place when the resultant force at any section cuts the base of the dam downstream of the toe.

→ $F.S = \dfrac{\Sigma \text{Righting moments}}{\Sigma \text{overturing moments}} = \dfrac{\Sigma M_s}{\Sigma M_o}$.

→ ~~Bn~~ ~~&~~ In that case as metioned above the resultant moment at the toe becomes clockwise.

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Sliding:-

→ A dam will fail in sliding at its base, or at any other Level, if the horizontal forces cause sliding are more than the resistance available to it at at that Level

→ the resistance against sliding maybe due to friction alone, or due to friction & Shear strength of the joint.

→ Shear strength also comes into play because of the interlocking of stone in masonary dams.

$$SF = \tan\theta = \frac{\Sigma H}{\Sigma V}$$

and the ~~Sa~~factor of safty against sliding is

$$F.SS = \frac{\mu}{\tan\theta} = \frac{\mu \Sigma V}{\Sigma H}.$$

$$S.F.F = \frac{\mu \Sigma(V) + Ac}{\Sigma H} = \frac{\mu \Sigma(V) + b.c}{\Sigma H}.$$

SNo	Loading condition	F.S against sliding (F.SS)	S.F.F
1	A, B, C	2.0	4.0
2	D, E	1.5	3.0
3	F, G	1.2.	1.5

Compression;

The normal stress at any point on the base will be the sum of the direct Stress & the bending stress.

Thus direct stress $= \frac{V}{b \times 1}$

Bending stress $= \pm \frac{M.y}{I} = \pm \frac{V.e}{\frac{1}{6}b^2} = \pm \frac{6.V.e}{b^2}$

$$P_n = \frac{V}{b}\left(1 \pm \frac{6e}{b}\right)$$

$$(P_n)_{toe} = \frac{V}{b}\left(1 + \frac{6e}{b}\right)$$

$$(P_n)_{heel} = \frac{V}{b}\left(1 - \frac{6e}{b}\right)$$

$$\frac{V}{b}\left(1 + \frac{6e}{b}\right) \leq f.$$

When the eccentricity e is equal to $b/6$, we get

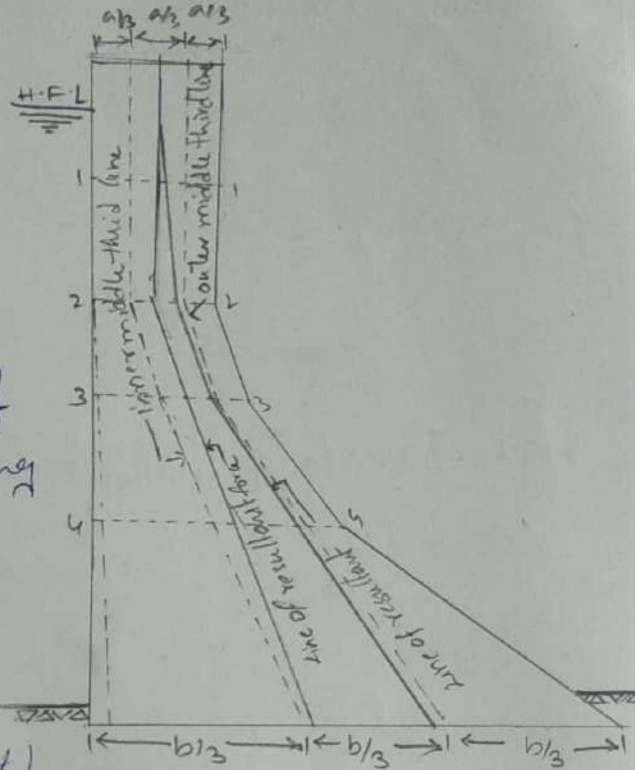
$$(P_n)_{toe} = \frac{V}{b}\left(1 + \frac{6}{b} \times \frac{b}{6}\right) = \frac{2V}{b}$$

Tension: from $(P_n)_{heel} = \frac{V}{b}\left(1 - \frac{6e}{b}\right)$ is normal stress

it is evident that if $e > \frac{b}{6}$, the normal stress at heel will be -ve or tensile

8.15 Stability analysis.

1. Gravity method:- This is an approximate method in which is dam is considered to be composed of parallel sided vertical cantilevers, each of which is free to act without supporting of interfering with the adjoining cantilevers.



Graphical method:-

The sum of all horizontal forces (ΣH) and sum of all vertical forces (ΣV), acting above that section are calculated and their line of action are graphically located. the resultant force R. is then found, and its line of action is located graphically. this done for each section, and finally a line is drawn joining the point at which the resultants cut the various sections.

Trial Load twist method:- The assumption is very nearly true if the dam is constructed in a wide U-shaped valley, and transvers joints are not keyed or grouted.

(i) Cantilever structural system to resist the External load by gravity action through bending and shear along horizontal planes.

~~ii) Beam~~

(ii) Beam structural system to resist bending.

(iii) Twist structural system to resist twisting moments caused by shearing of horizontal elements and angular rotation of vertical elements.

Slab analogy method:-

According to this method, dam is considerd as a slab and the analysis carried out is simialar to the calculation of stresses in bridge slabs. It is a time consuming and laborious method.

Lattice Analogy method. the dam is considerd as to be composed of diagonally braced squre frames. The analysis, though simple than the slab analysis method, is quite cumbersome.

Experimental methods:-

Direct method:- In this method geometrically similar 3-d models, ~~are~~ which are exact replica of the prototype, are made of elastic materials.

Indirect method:- photo elastic method and methods of magnetic & electrial analogy are usally considerd under the indirect method of determining the stress in the model and hence in the prototype dam.

Elementary profile of a Gravity dam.

we shall consider the following forces acting on the elementary profile of a gravity dam.

(1) weight of the dam (W)

$$W = \frac{1}{2} bH\rho w$$

where ρ = Specific gravity of dam material

w = unit weight of water

(2) water pressure (P)

$P = \frac{1}{2} wH^2$. acting at $\frac{1}{3}H$ from the base

(3) uplift pressure (U)

$$U = \frac{1}{2} C \cdot w \cdot b \cdot H$$

where C = uplift pressure intensity coefficient

Base width of elementary profile : The base width of the elementary profile is to be found under two Criteria.

(1) stress Criterion : we have

Taking the moment of all forces about M_2 and equating it to zero, ~~mom~~ we get

$$\frac{1}{2} wH^2 \cdot \frac{H}{3} + \frac{1}{2} CwbH \cdot \frac{b}{3} - \frac{1}{2} bH\rho w \cdot \frac{b}{3} = 0$$

multiplying all the term by $\frac{6}{wH}$, we get

$$H^2 + Cb^2 - b^2\rho = 0$$

$$b^2(\rho - C) = H^2$$

$$b = \frac{H}{\sqrt{\rho - C}}$$

$$\frac{W-U}{P} = \frac{H/3}{b/3} = H/b \quad \text{or} \quad \frac{\frac{1}{2} bH\rho w - \frac{1}{2} wcb H}{\frac{1}{2} wH^2} = H/b$$

$$b^2(\rho - c) = H^2 \quad \text{or} \quad b = \frac{H}{\sqrt{\rho - c}}$$

if uplift is not considering, $c = 0$

$$b = H/\sqrt{\rho}$$

(2) Stability or Sliding criterion: for no sliding to occur, horizontal force causing sliding should be balanced by the friction forces opposing the same. Hence.

$$P = \mu (W - U)$$

$$\frac{1}{2} wH^2 = \mu \left(\frac{1}{2} bH\rho w - \frac{1}{2} c b w H\right)$$

from which

$$b = \frac{H}{\mu(\rho - c)}$$

if uplift is neglected, $b = H/\mu.\rho$

Stress developed in elementary profile.

$$P_n = \frac{V}{b}\left(1 \pm \frac{6e}{b}\right)$$

where $V = (W - U)$ in this case and $e = b/6$

Hence for full reservoir case the normal stress at the toe is

$$P_n = \frac{(W - V)}{b}(1+1) = \frac{2(W-U)}{b}$$

$$P_n = \frac{2}{b}\left(\frac{1}{2} b H\rho w - \frac{1}{2} (cbwH)\right)$$

$$P_n = wH(\rho - c)$$

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The Corresponding stress at the heel will be

$$p_n = \frac{(W-U)}{b}(1-1) = 0$$

principal stress at the toe

$$\sigma_1 = p_n \sec^2\phi = wH(\rho - c)\left[\left(\frac{b}{H}\right)^2 + 1\right]$$

But

$$\left(\frac{b}{H}\right)^2 = \frac{1}{\rho - c}, ~~from~~$$

Hence

$$\sigma_1 = wH(\rho - c)\left[\frac{1}{\rho - c} + 1\right]$$

$$\sigma_1 = wH(\rho - c + 1)$$

Shear stress at the toe

~~F b~~

$$\tau = p_n \tan\phi$$

$$\tau = wH(\rho - c) \times \frac{b}{H}$$

$$\tau = wH(\rho - c) \times \frac{1}{\sqrt{\rho - c}}$$

$$\tau = wH\sqrt{\rho - c}.$$

Practical Profile of Gravity dam.

→ The elementary profile of the gravity dam is only a theorectial profile. Howere such a profile is not possible in pracitia because of the provision of

i) road way at the top

ii) additional load due to the roadway

iii) free board.

free board

-> free board is the margin provied between the top of dam and H.F.L in the reservoir to prevent the splashing of the waves over the non-overflow section

-> the free board usually provided is $3/2\, h_w$, where h_w can be found. ~~from E~~. the top width equal to 14% of h has been found to be economical.

Limiting height of gravity Dam: High & Low Gravity Dams.

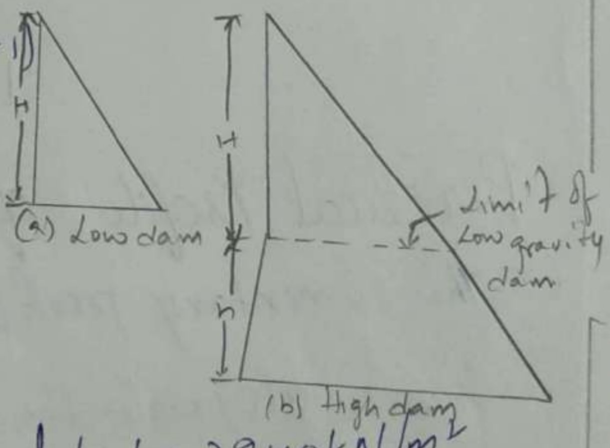
The principal stress at the toe is given by

$$\sigma_1 = wH(\rho - C + 1)$$

the maximum value of this principal stress should not exceed the allowable stress (f) for the material In the limiting case

$$f = \sigma_1 = wH(\rho - C + 1)$$

$$H = \frac{f}{w(\rho - C + 1)}$$



when $C = 0$

$$H = \frac{f}{w(\rho + 1)}$$

-> if common value of f is taken equal to be 2940 kN/m² for concrete dam, the limiting height of low gravity dam comes out to be

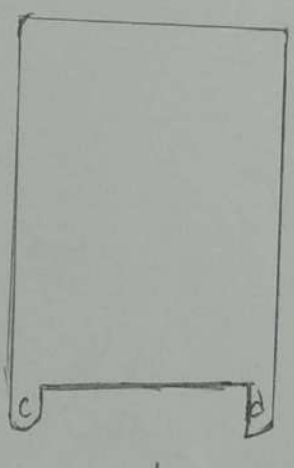
$$H = \frac{2940}{9.81(2.4+1)} \simeq 88\,m$$

Galleries:-

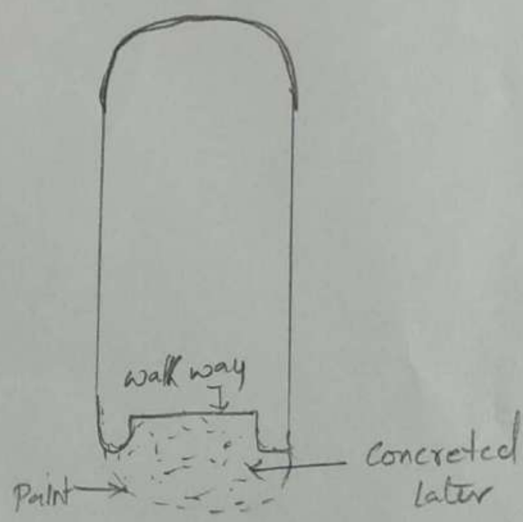
→ To provide drainage of the dam section. Some amount of water constantly seeps through the upstream face of the dam which is drained ~~to~~ off through galleries.

→ To provide space for header & return pipes for post cooling of concrete & grouting the longitudinal joints after completion of dam

→ To provide access to observe & measure the behaviour of the structure after its completion by fixing thermo couples & examining development of cracks etc.



(a) Rectangular



(b) oval shaped

UNIT-III

EARTH DAMS

TYPES OF EARTH DAMS :-

Earth dams can be divided into two categories

(i) Rolled fill dam (ii) Hydraulic dam.

1. Rolled fill dam

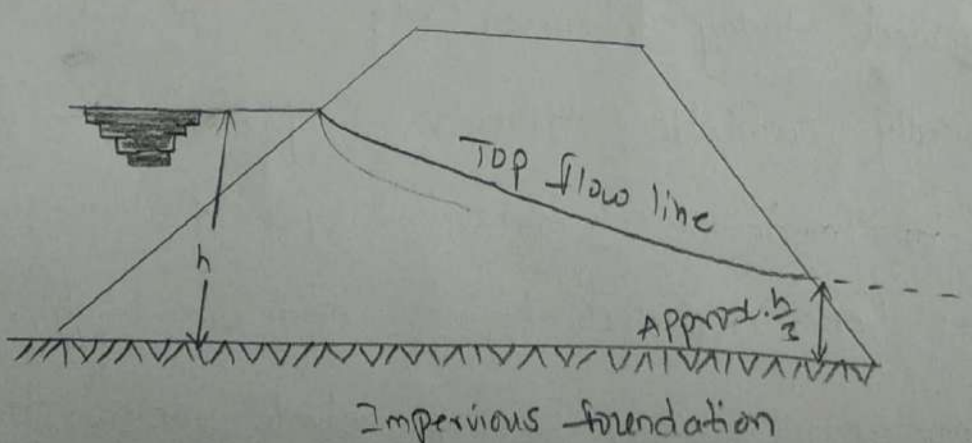
Rolled fill dams can further be sub-divided into following types.

1. Homogeneous embankment type,
2. Zoned embankment type
3. Diaphragm embankment type

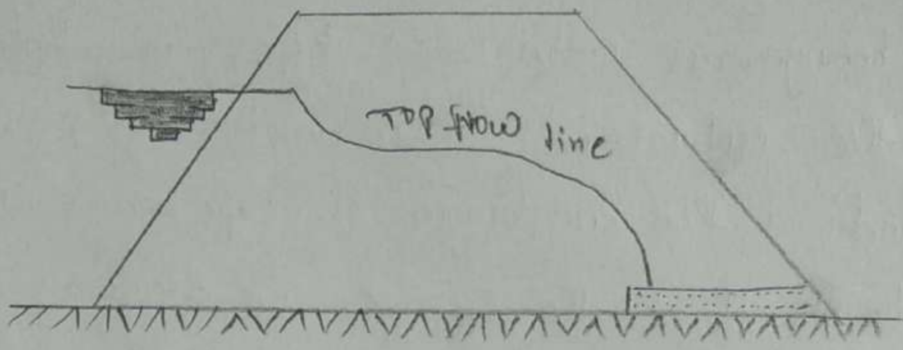
1. Homogeneous embankment type :-

A purely homogeneous type earth dam is composed of single kind of material (exclusive of the slope protection). Homogeneous dams have been built since the earliest times and are used today whenever only one type of material is economically available. However, they are used only for low to moderate heights. A purely homogeneous section has been replaced by a moderate heights. A purely homogeneous section has in which internal drainage system in the form of horizontal filter drain or rock toe, is provided, This controls the action of steeper slopes, The drainage system also keeps the phreatic line well within the body of the dam. for the purpose of further discussions, however, we shall designate the "homogeneous section" as the one composed of one type of material, with or without internal drain.

Homogeneous dams are usually composed of impervious or semi-impervious soils to provides an adequate water barrier. However, the upstream slope has to be flatter to make it safe during the "sudden drawdown condition". Many successfull embankments have been built to relatively previous sands and sand-gravel mixtures.



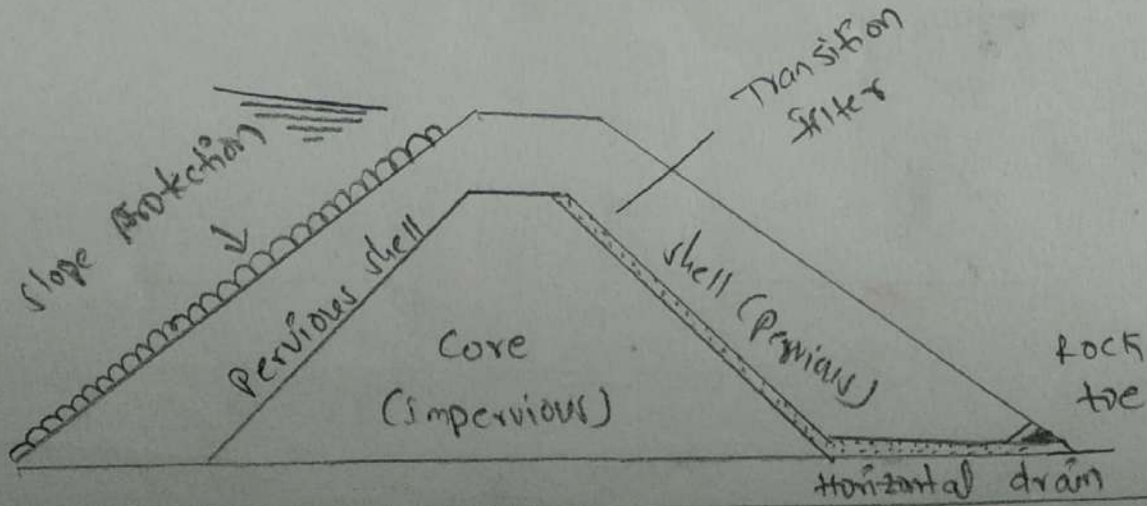
(a) Purely homogeneous dam



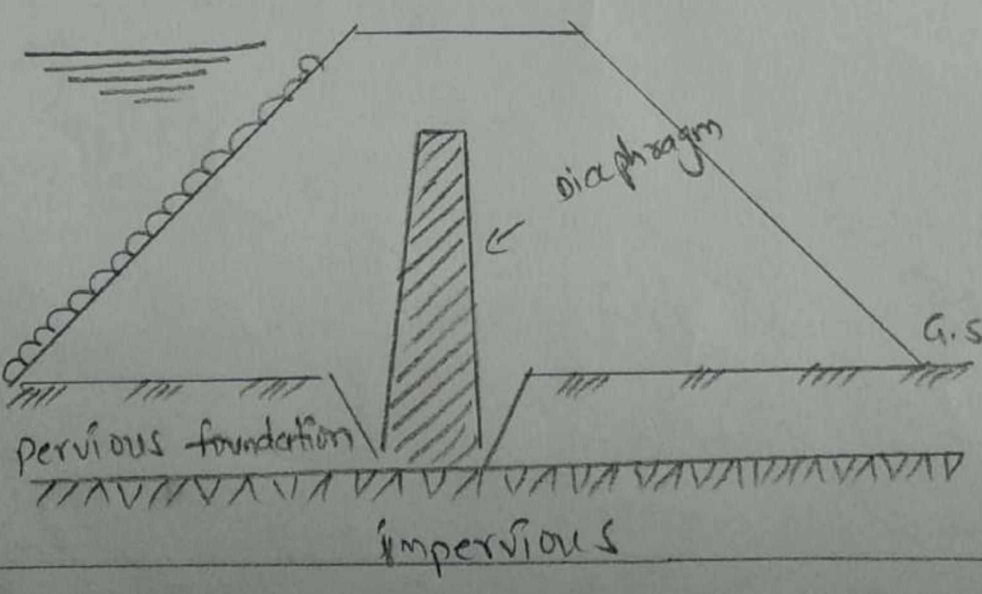
(b) modified homogeneous dam

Homogeneous dam.

(2) Zoned Embankment Type :- Zoned embankment type earth dam is the one in which the dam is made up of more than one material. The most common type of a rolled earth dam section is that in which a central impervious core is flanked by zones of material considerably more pervious. A suitable drainage system, in the form of a horizontal drain or a rock toe is also provided at the d/s side will lead to economy in the cost of construction.



3. Diaphragm Embankment Type :- This is a modification over the homogeneous embankment type, in which the bulk of the embankment is constructed of previous material and a thin diaphragm of impermeable material is provided to check the seepage. The diaphragm may be of impermeable material is provided to check the seepage. The diaphragm any other material and may be placed either at the central vertical core, or at the up stream face as a blanket However, the distinction between a diaphragm type and zoned type must be clearly known. If the horizontal thickness of the diaphragm at any elevation is less than 10m. or less than the height of embankment above any dam is considered to be 'zoned' embankment type.



CAUSES of FAILURES of EARTH DAMS :-

On the basis of investigation reports on most of the past failures it has now been possible to categories the types of failures into three main classes.

1. Hydraulic failures : 40%.
2. seepage failures ; 30%.
3. structural failures ; 30%.

1. Hydraulic failures :- Hydraulic failures include the following.

(i) over-topping (ii) wave erosion

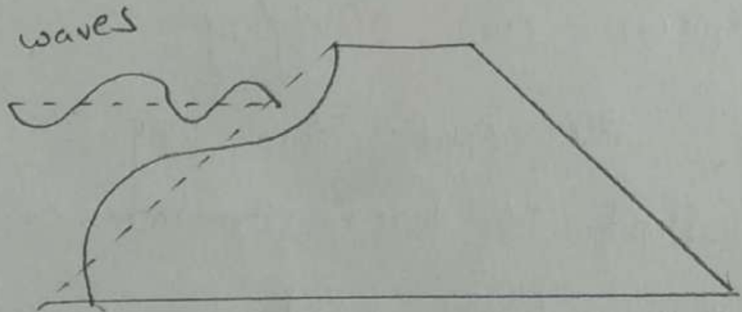
(iii) Toe erosion (iii) Gullying

(i) overtopping :- The earth dam may get overtopping if the design flood is under-estimated, or if the spilway is of foundation and embankment may also lead to overtopping.

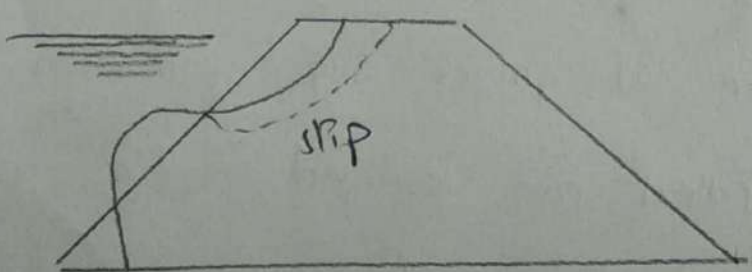
(ii) wave erosion :- failures due to wave erosion the effects of wave is to notch out earth. from the upstream slope in absence of proper slope protection in the from of riprap. Rollers are developed in the waves which try scoop out the earth. waves can also cause upstream slips.

(iii) Toe Erosion :- Toe Erosion may occur due to two reasons ; (a) erosion due to tail water, and (b) erosion due to cross currents that may come from spillway buckets or from spillway buckets or from exist areas of outlets. The toe erosion can be avoided by providing flow towards the earth dam.

iv) Gullying :- Down stream slope may fail due to the formation of gullies by heavy downpour. To eliminate failure due to gullying vided to the down stream side.



(a) Roller action



(b) upstream slip

WAVE EROSION

CRITERIA FOR SAFE DESIGN OF EARTH DAM :-

An earth dam must be safe and stable during phases of construction and operation of the reservoir. The practical criteria for the design of earth dams may be stated briefly as follows.

1. The embankment must be safe against overtipping during occurrence of the inflow design flood by the provision of sufficient spilway and outlet work capacity.
2. The dam must have sufficient free board so that it is not overtopped by wave action.
3. The seepage line should be well within the d/s face so that no sloughing of the slope takes place.
4. The upstream slope should be stable during rapid draw down condition.
5. The dam as a whole should be earthquake resistance.

The above criteria of design have been covered at length in the subsequent articles.

SEEPAGE CONTROL MEASURES :-

The seepage control measures are necessitated to prevent adverse effects of water percolating through embankment and its foundation. The following devices are used for seepage control through earth dam.

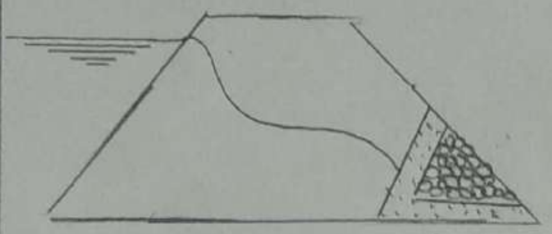
(A) Embankment seepage control

1. Toe filter
2. Horizontal drainage filter
3. protective filter d/s of the toe
4. D/S coarse section (embankment zoning).
5. Chimney drain extending upward into embankments.

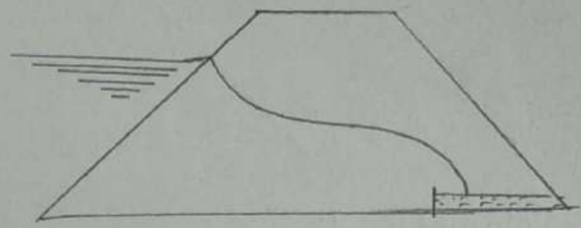
(B) foundation seepage control

6. Impervious cut off
7. upstream impervious blanket
8. D/S seepage berms
9. Drainage trenches
10. Relief wells.

1. **Toe filter** :- Rock toe keeps the phreatic line well within the section and also facilitates drainage. its height is generally kept equal to 30 to 40% of reservoir head. The gradation of should satisfy filter criteria.



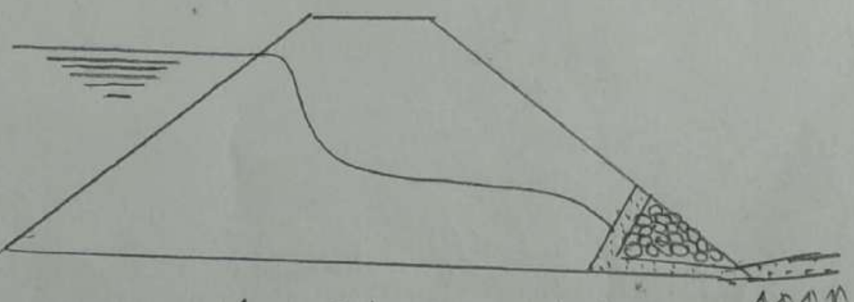
(a) Rock toe filter



(b) Horizontal drainage

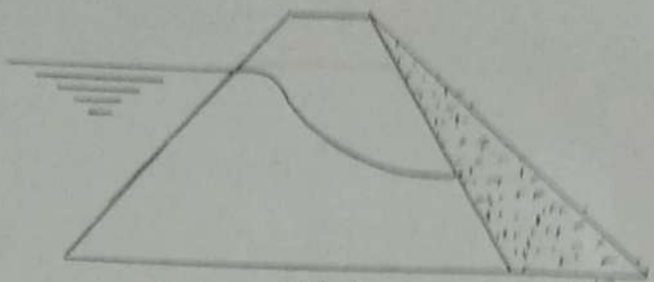
2. Horizontal drainage filter:- The horizontal drainage filter may extend from 25 to 100% of the distance from down stream toe to the centre line of the dam. The horizontal filter serves the following purposes.

3. filter downstream of the toe:-
The provides of such filter through also intercept Provides additional weight and thus makes the upward flow more safe,



(a) filter D/s of toe.

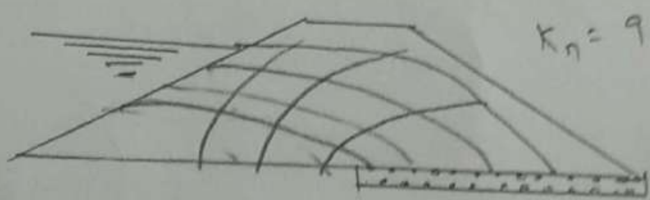
4. Downstream coarse section:-
This is also intercept the flow through the embankment and makes the d/s slope safe against piping. It is also an earth-quake resistant measure.



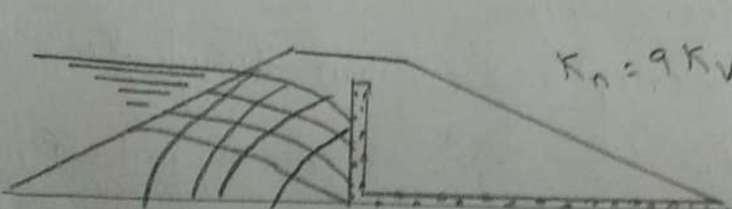
(b) Coarse section

5. chimney drain :-
when there is high degree of embankment stratification, the horizontal permeability is greater than the vertical, resulting in greater horizontal spread of seepage of the embankment. it is also earthquake resistant.

6. Impervious cutoffs :- whenever economically possible, the seepage should be cutoff. However, no cutoff is absolutely impervious and the reduction of seepage is a relative matter of cutoff reduces the discharge only by about 64% and 50% depth by 25%.

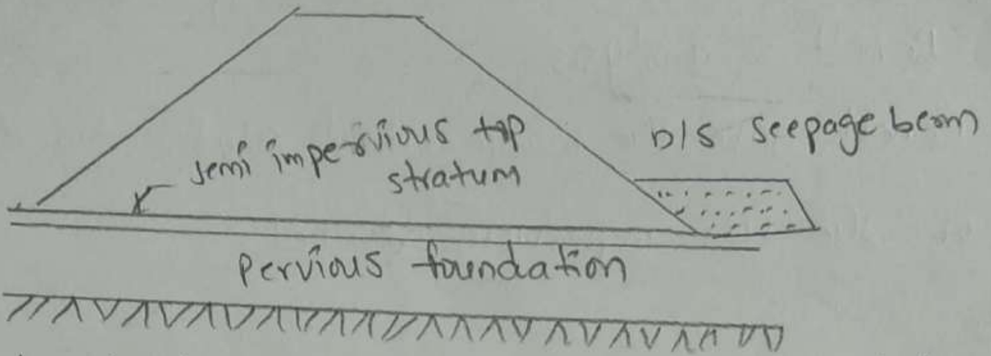


(a) horizontal filter drain relatively ineffective

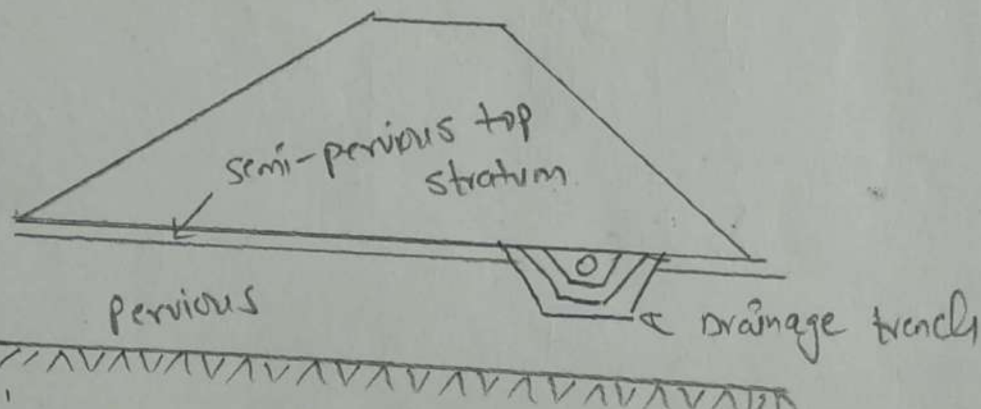


(b) Chimney filter drain intercepts embankment seepage.

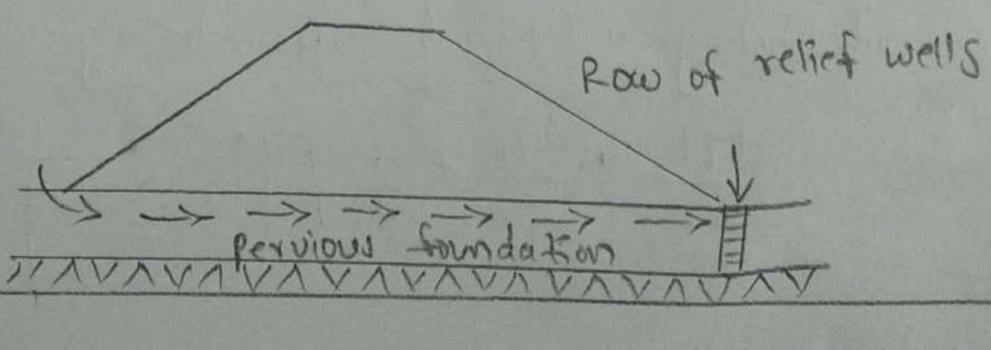
7. Downstream seepage berms:-
Berms can be used to control seepage efficiently where the d/s top stratum is relatively thin and uniform, or where no d/s stratum is present.



8. Drainage trenches:- They are provides when top stratum is thin and pervious stratum is also shallow.



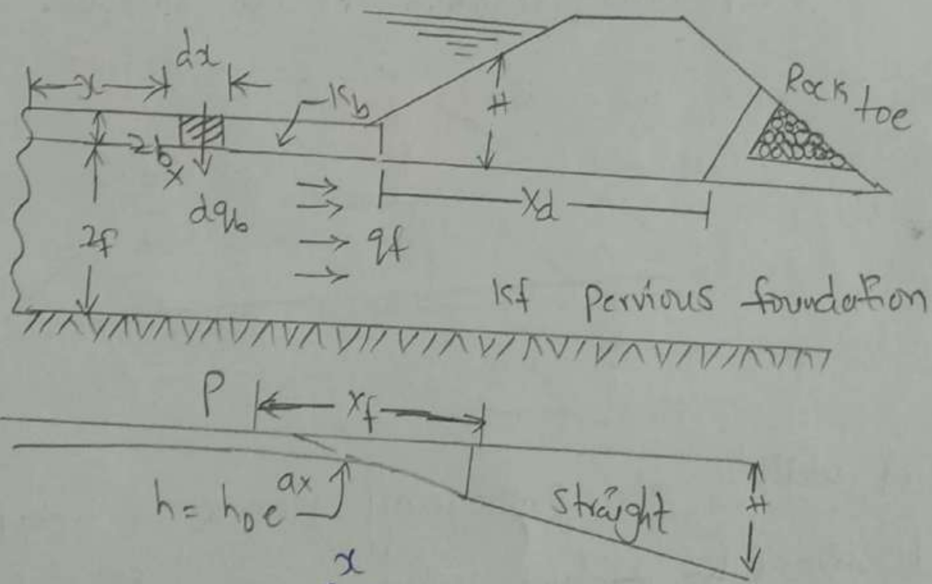
9. Relief wells:- The preliminary purpose of relief well is to reduced the sub-stratum uplift pressure d/s of the dam with otherwise would cause formation were first used by U.S Corps of Engineers.



10. upstream impervious blanket :- Impervious upstream or riverside blankets overlaying a pervious foundation are effective in reducing the quantity of seepage. They also blanket can be determined by the Benett 's analysis.

Benett 's analysis

Benett gave the mathematical solution for the performance of the u/s impervious blanket,



$$q_f = q_0 + \int_0^x dq_b \rightarrow ①$$

$$dq_b = K \cdot i \cdot A = K_b \frac{h}{z_b} dx \rightarrow ②$$

substituting in (1), and differentiating with respect to x, we get

$$\frac{dq_f}{dx} = \frac{dq_0}{dx} + \frac{d}{dx}\left[\int_0^x k_0 \frac{h}{z_b} dx\right]$$

$$\frac{dq_0}{dx} = 0, \text{ since } q_0 \text{ is independend of } x.$$

$$\frac{dq_f}{dx} = \frac{k_b h}{z_b} \longrightarrow (3)$$

$q_f = k_f \frac{dh}{dx} z_f$ for the foundation

$$\frac{dq_f}{dx} = k_f z_f \frac{d^2h}{dx^2} \longrightarrow (4)$$

equating (3) & (4), we get

$$\frac{d^2h}{dx^2} = \frac{k_b h}{k_f z_b z_f} = a^2 h$$

$$a = \sqrt{\frac{k_b}{k_f z_b z_f}} = \text{Constant}$$

Eq. 10.28 is the differential equation for the pressure dissipaton. Its solution is to be obtained in two cases: (i) Infinite length of the blanket (ii) finite length of the blanket.

Also, we will consider two cases of blanket thickness;

(a) Blanket of uniform thickness, and (b) blankent on variable thickness.

(a) Blanket of uniform thickness :-

For blanket of uniform thickness, z_b is constant and hence $a = \sqrt{k_b (k_f z_b z_f)}$ is Constant.

(i) Solution for Infinite length of blanket :-

The solution of Eq. 10.28 for infinite length is;

$$h = h_0 e^{ax}$$

where h_0 = total head lost through blanket at $x=0$

Equivalent resistance x_r of the foundation :-

The resistance x_r is a measure of the efficiency of the blanket of any length x. it may be defined as the length of a prism of the foundation material, thickness z_f and permeability k_f, under head loss h_f. the blanket system.

$$k_f z_f \frac{dh}{dx} = k_f z_f \frac{h}{x_r}$$

$$\frac{dh}{dx} = \frac{h}{x_r} \rightarrow (5)$$

$$\frac{dh}{dx} = h_0 \cdot a e^{ax}$$

$$\frac{dh}{dx} = a h_0 e^{ax} = ah$$

substituting in (5), we get $ah = h/x_r$

$$x_r = \frac{h}{ah} = \frac{1}{a} = \sqrt{\frac{k_f z_b z_f}{k_b}}$$

Discharge reduction:-

If there were no blanket

$$q_f = k_f \frac{H}{x_d} \cdot z_f$$

Due to the provision of blanket

$$q_f = k_f \frac{H}{x_r + x_d} z_f$$

$$\% \text{ reduction} = \frac{q_f - q_f}{q_f} \times 100 = \frac{x_f}{x_r + x_d} \times 100$$

(ii) solution for finite length of blanket:-

The solution of Eq. 10.28 for finite length is

$$h = h_n (e^{ax} - e^{-ax})$$

where h_n = constant = total head loss through the blanket at the d/s edge of the blanket to the remainder of the system. Differentiating Eq.

$$\frac{dh}{dx} = a h_n (e^{ax} + e^{-ax})$$

if x_r is the equivalent resistance of the foundation we have,

$$\frac{dh}{dx} = h / x_r$$

$$x_r = \frac{h}{dh/dx} = \frac{h_n (e^{ax} - e^{-ax})}{a h_n (e^{ax} + e^{-ax})}$$

$$x_r = \frac{(e^{2ax} - 1)}{a(e^{2ax} + 1)}$$

If there were no blanket $q_f = k_f \dfrac{H}{x_d} \cdot z_f$

Due to the provision of blanket

$$q_f' = k_f \frac{H}{x_r + x_d} \cdot z_f$$

$$\% \text{ reduction is discharge} = \frac{q_f - q_f'}{q_f} \times 100$$

$$= \frac{x_r}{x_r + x_d} \times 100$$

$$x_o = \sqrt{2}/a$$

(ii) steps for the design of blanket of uniform thickness:–

(i) find a by the expression $a = \sqrt{\dfrac{K_b}{k_f \cdot z_b \cdot z_f}}$

(ii) find the optimum length x_o by $x_o = \sqrt{2}/a$

(iii) find x_r by the relation $x_r = 1/a$ for infinite length

$$x_r = \frac{(e^{2ax} - 1)}{a(e^{2ax} + 1)} \text{ for finite length}$$

iv) find the hydrulic gradient $i = \dfrac{H}{x_r + x_d}$

v) find the loss of head (or head dissipated) through the blanket by the expression $h_o = \dfrac{x_r}{x_r + x_d} \cdot H$

vi) find percentage reduction in discharge

$$x_o = \sqrt{2}/a$$

(b) BLANKET of VARIABLE THICKNESS :-

It is seen that the blanket is more effective towards the downstream end. Hence for more efficient use of the blanket, we should provide thus making z_b variable. Hence Eq. 10.28 can be expressed as follows.

$$\frac{d^2h}{dx^2} = \frac{ah}{z_b}$$

where

$$a = \frac{k_b}{k_f z_f} = \text{Constant}$$

while z_b varies with x.

Considering blanket thickness to vary triangularly we have $z_b = s \cdot x$ where 's' is the slope of blanket thickness. However the solution of Eq. 10.36 for this case is obtained in terms of a series which can not be computed conveniently. On the contrary, if we assume the blanket to be of parabolic profile with is represented by the following equation:

$$z_b = \frac{ax^2}{n(n-1)}$$

Then the solution of Eq is given by

$$h = h_n x^n$$

for such a case, $x_r = \frac{h}{(dh/dx)} = x/n$

The head dissipated through the blanket is given by $h_o = \frac{x_r}{x_r + x_d} H$

The percentage reduction in discharge is given by Eq.

Design steps:-

1. Compute the value of x from Eq. Corresponding to the known values of k_b, k_f and z_f.
2. Assume a suitable value of length of the blanket, and compute z_b and x_r by taking different values of $n = 1.25, 1.5, 1.75, 2.0$ etc.
3. Compute head dissipated through the blanket & reduction in discharge.
4. Repeat the above procedure for several other lengths of the blanket.

Energy Dissipaters and stilling Basins :-

when flood discharge passes over the spillway crest it has high potential energy which gets converted into kinetic energy as it the down stream toe. There is a varied practies regarding the extent can be rendered by two ways.

(i) By dissipating the jet energy by means of hydraulic jump

(ii) By directing the jet of water so as to fall away from the structure by a deflector bucket the jet or lip, and dissipating the energy by impact.

Hydraulic Jump Computations :-

The hydraulic jump that occurs at the stilling basin has some distinctive charecteristics and assumes a definite form, depending upon the energy and flow charecteristics depends upon the following factors.

(i) Discharge entering the basin,

(ii) Critical depth of flow dc

iii) ~~Froude~~ froude number parameter $V/\sqrt{gd}$

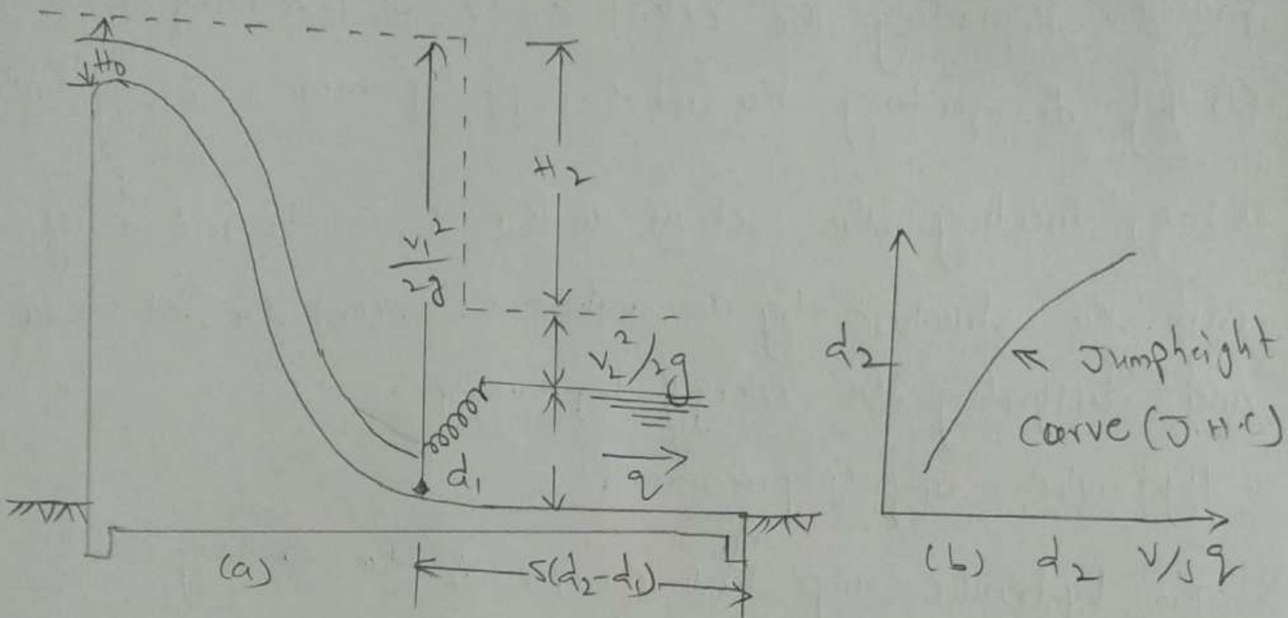
1. for the given discharge q per metre length of the spilway, calculate the head Hc over the crest to

the total energy level;

$$H_e = \left(\frac{q}{C}\right)^{2/3}$$

2. find total energy level at U/s;

$$U/s\ T \cdot E \cdot L = \text{Crest level} + H_e$$



3. Assuming no losses, the specific energy E_1 at the toe of the spillway will be equal to the T.E.L u/s

$$E_1 = T \cdot E \cdot L\ u/s$$

4. Knowing E_1 and q, find the prejump depth d_1 by trial and error from the relation.

$$E_1 = d_1 + \frac{v_1^2}{2g} = d_1 + \frac{q^2}{2gd_1^2}$$

5. Calculate froude Number F_1:

$$F_1 = \frac{v_1}{\sqrt{gd_1}} = \frac{q}{\sqrt{gd_1^3}}$$

6. Compute the post Jump depth d_2 from the relations.

$$d_2 = \frac{d_1}{2}\left[\sqrt{1+8F_1^2}+1\right]$$

$$d_2 = \frac{d_1}{2}\left[\sqrt{1+\frac{8q^2}{gd_1^3}}-1\right]$$

plotting Jump height curve:-

Equation can be used for calculating post Jump depths d_2 for various values of flow discharge q. shows that a curve d_2 with the discharge q.

plotting tails water curve (T.W.C):-

The efficiency of the hydraulic Jump in dissipating the energy, and the corresponding protection works in the stilling basin will channel retrogression must be taken into account.

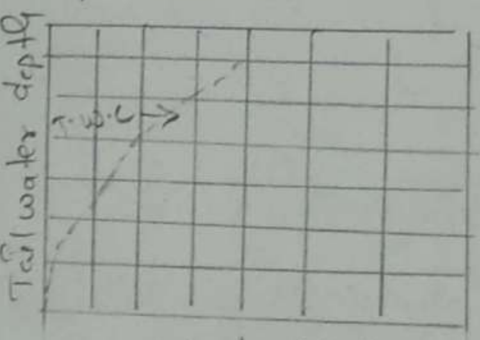
Relative position of Jump height curve and tail water curve.

There may be five conditions that govern the relationship b/w the Jump height curve (J.H.C) & tail water curve (T.W.C)

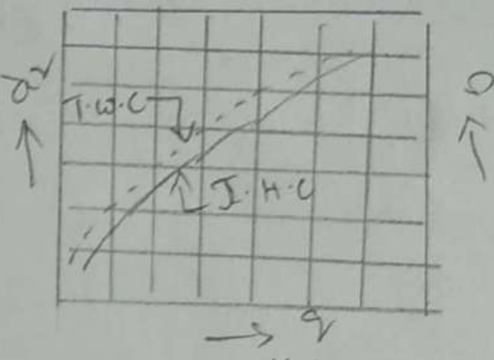
1. Both the curve coincide.
2. at large discharge.

1. protection works for condition I :-

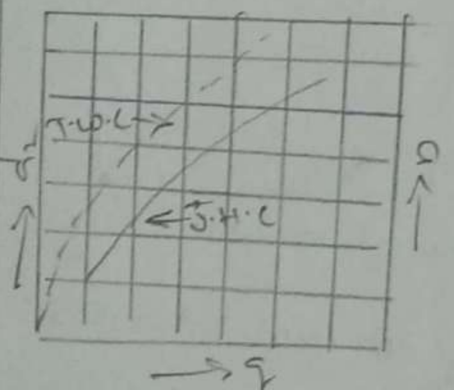
In this condition, both the curves coincide, as shown in fig 11.25(b). This is an ideal condition, since the post jump depth required for the channel. The jump formation will thus be perfect at all discharges and hydraulic jump will be formed at the toe of the spillway.



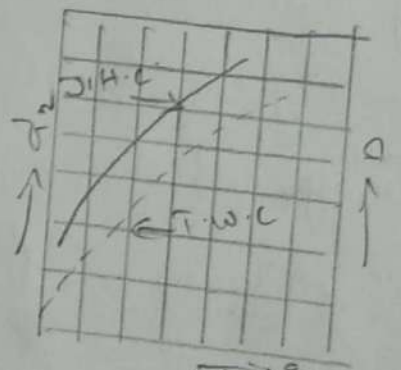
Discharge q
(a)



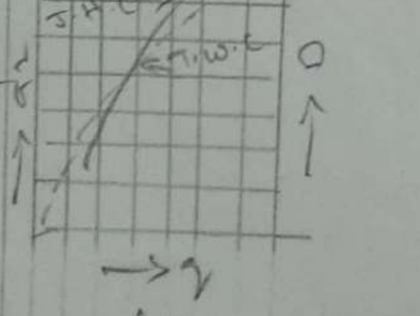
(b)



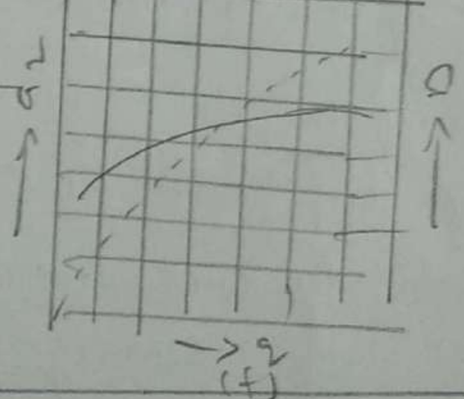
(c)



(d) q



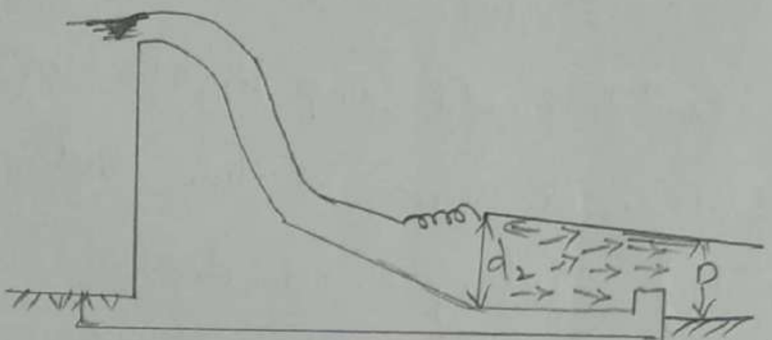
(e)



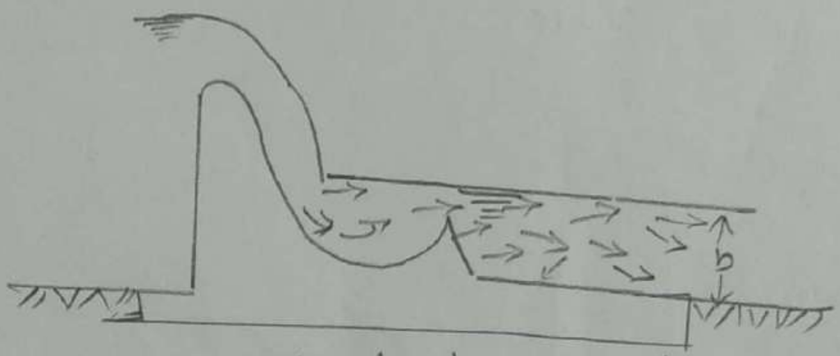
(f)

2. protection works for condition - II

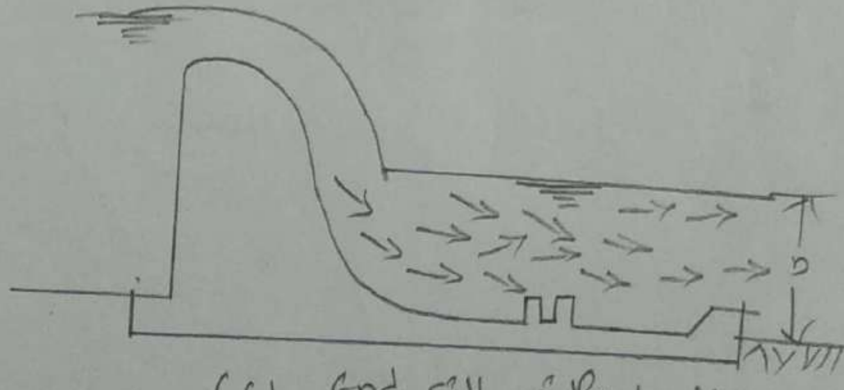
In this condition, shows in fig 11.25(c), the jump height curve lies lower than the tail water curve at all discharges. In other words the available tail water depth is greater the tail water depth at the point of formation of hydraulic jump.



(a) sloping glacis above bed



(b) sharply upturned bucket



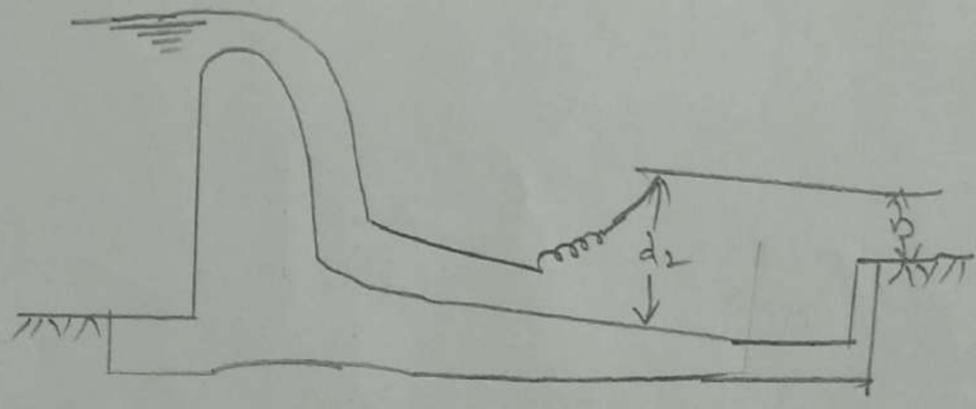
(c) End sill with baffles

3. protection works for condition :- III

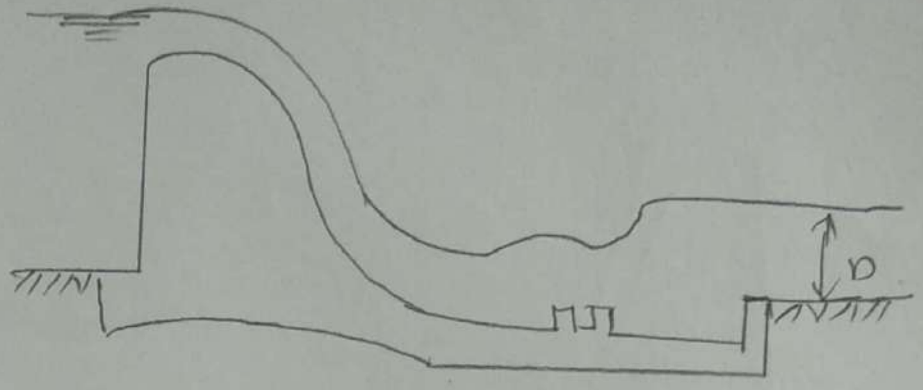
In this condition the jump height curve is heighter then the tail water curve at all discharge, as shown in fig 11.25(d). Thus the avalible tail water depth is lesser than the depth required for the formation of hydraulic jump.

4. protection works for condition :- IV

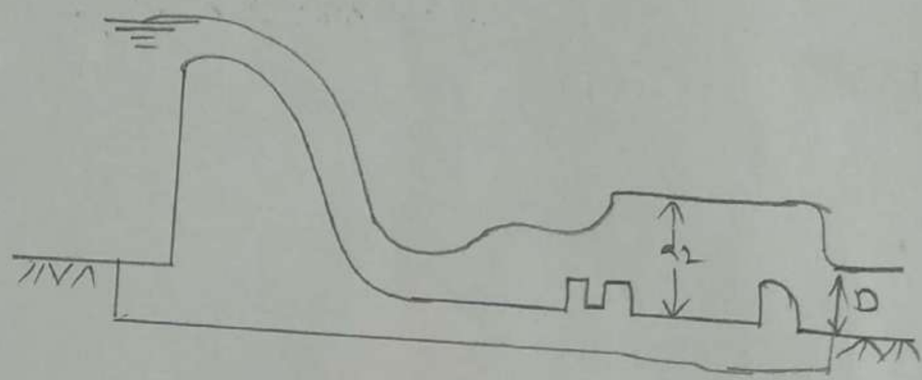
In this condition the jump height curve lies lower than the tail water curve at low discharge, and higher at high discharges, as shown in fig 11.25(e). thus, at the low discharges, insufficient tail water depth will be there.



(a) sloping glacis below G.L



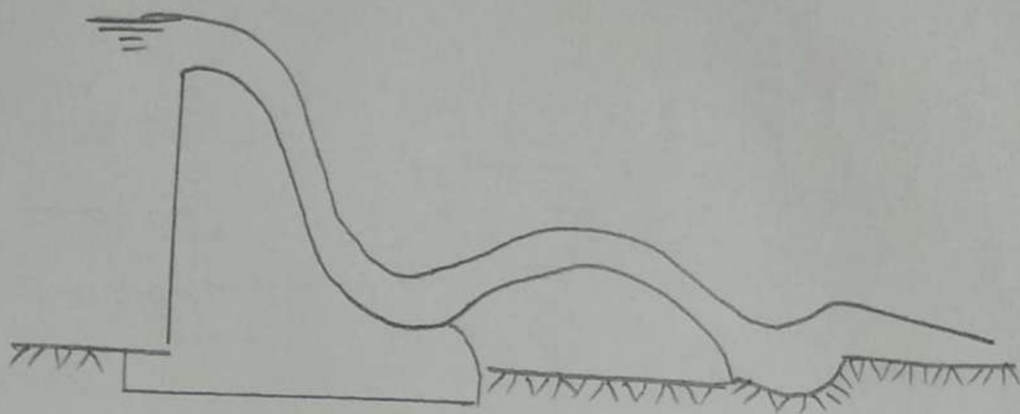
(b) cistern excavated in rock



(c) low secondary dam

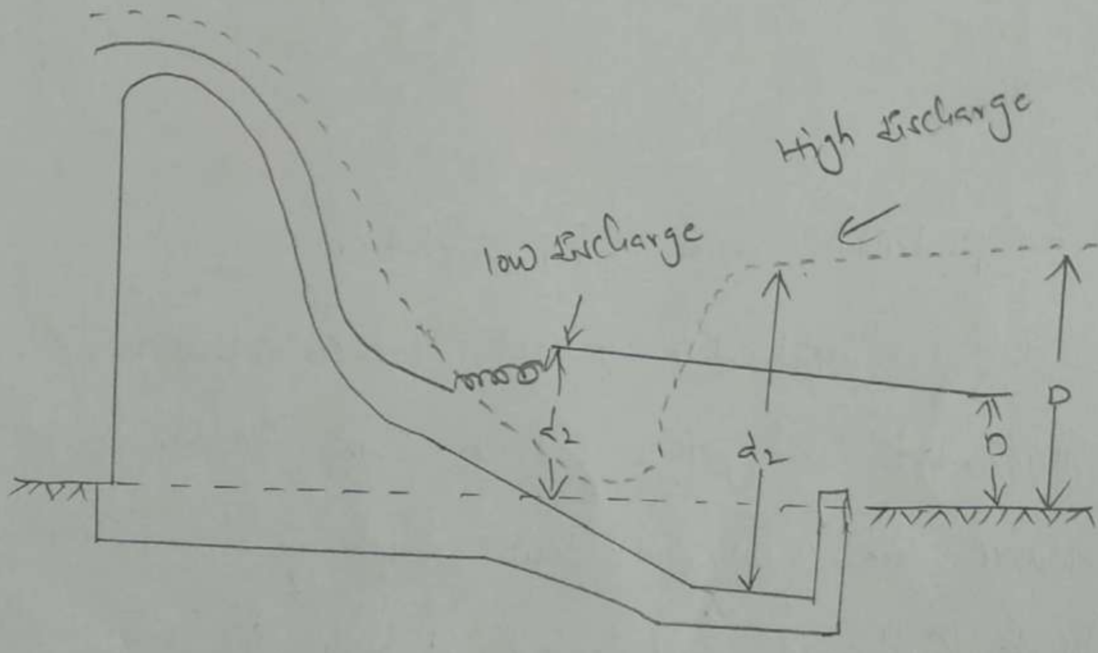
5. protection works for condition v :-

This is just the reverse of condition IV, As illustrated in fig 11.25(f), the jump height curve lies above the T.W.C at low discharge, and lower at high discharge; and lower at high discharges Thus at low discharge in sufficient tail water will be there while at higher discharges the jumps will be at high discharges, the jump will be formed higher up.



(d) sharply upturned bucket

Protection works for Low Tail water Depth



sloping apron partly below and partly above the bed.

DESIGN PRINCIPLE OF OGEE SPILLWAYS :-

This is the most common type of spillway provides on gravity dams. The profile of the spillways is ogee or 's' shaped. The overflowing water is guided smoothly over the creast and profile of the spillway so that the over flow water does not break contact with the spillway surface. This reverse curve turns the flow on the apron of a stilling basin or into the spillway discharge channel.

High overflow spillway :-
overflow ogee spillway are classified as high and low depending on whether the ratio of the height (P) of spillway creast measured from the river bed to the design head (H_d) is greater than, and equal to or less than 1.33 respectively. In case of high of 0.04 H or less, may be considered negligible.

Head :- The head (H) is the distance measured vertically from the water surface (upstream) of the commencement of drawdown) to the crest axis.

Design head (H_d) :-

The design head is that value of head for which the ogee profile is designed.

Head due to velocity of approach (H_a) :-

It is the velocity head given by

$H_a = v_a^2/2g$, where v_a is the velocity of approach.

Total energy head (H_e) :-

It is equal to the actual head plus the head due velocity of apporach Thus.

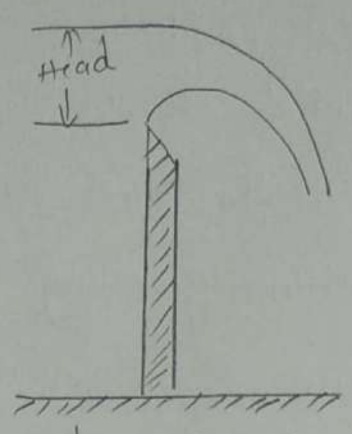
$$H_e = H + H_a$$

$$H = H_d, \quad H_{ed} = H_d + H_a$$

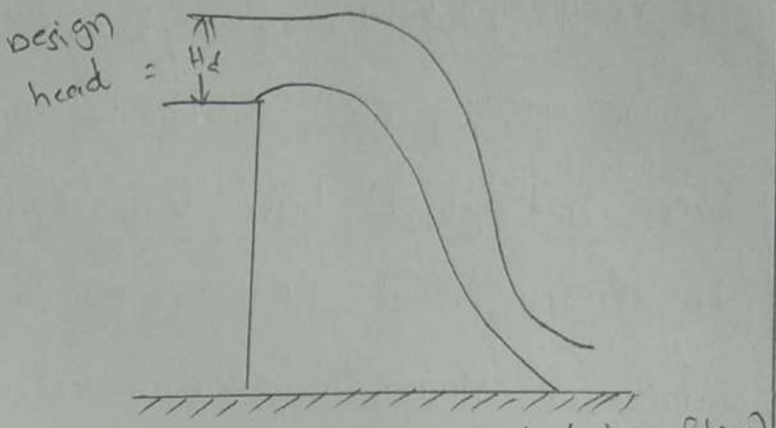
Ogee profile :-

The ogee profile to be acceptable should provide maximum possible hydraulic efficiency, structural stability, and economy and also avoid the formation of objection sub-atmospheric pressure at the surface.

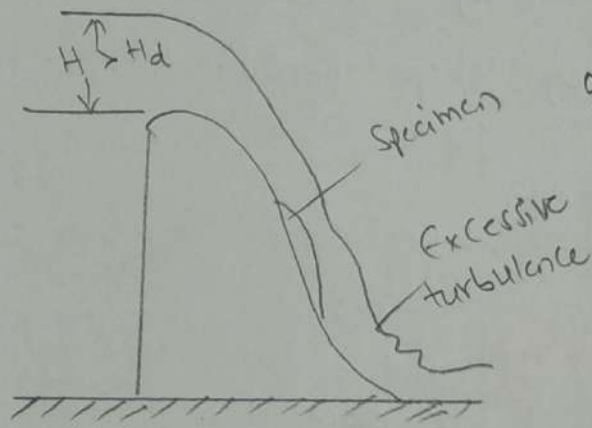
The falling jet would adhere to the profile of ogee, causing a positive hydrostatic pressure and reducing the discharge capacity.



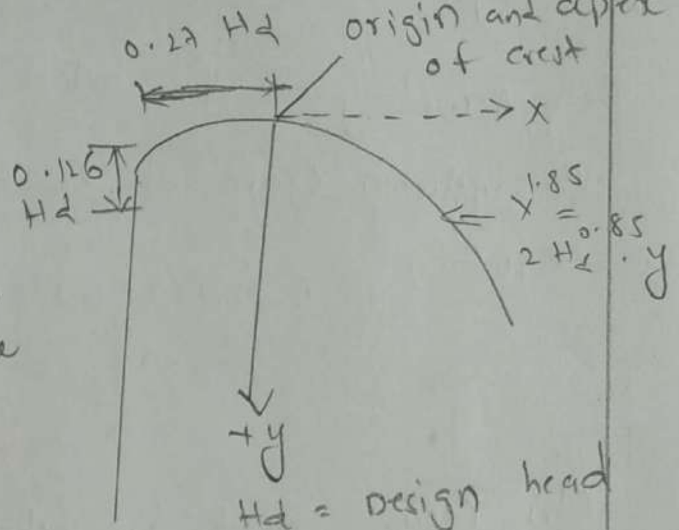
(a) shap creste weir



(b) ogee spilway at design flow



(c) ogee spillway with greater flow



(d) Elements of napps-shaped crest profile.

OGEE SPILLWAY

(a) Crest profile for vertical u/s face

(i) Design criteria of downstram profile

The details of the dowstream crest profile for vertical u/s face are shown in fig. The downstream curve of the ogee has the following equations.

$$x^{1.85} = 2 H_d^{0.85} \cdot y$$

(ii) Design criteria of upstream Crest profile :-

The u/s profile should be tangential to the vertical face and should have zero slope at the crest axis to ensure that there is no discontinuity along the surface of flow. The recommended curve, given below, incorporates data over the remaining portion.

According to the latest analytical studies of U.S Army the upstream Curve of the ogee shape has the following equation.

$$y = \frac{0.724(x + 0.270 H_d)^{1.85}}{H_d^{0.85}} + 0.126 H_d - 0.4315 H_d^{0.375} (x + 0.27 H_d)^{0.625}$$

(b) Creast profile for inclined u/s face

In general, the equation for the nappe-shaped profile depends upon (i) head over the creas, (ii) inclined of the upstream face of the spillway and (iii) height of the spillway crest above the stream bed or above the bed of approrach/entrance channel on which the approach velocity depends. Based on extensive. These shapes can be represented by the following general equation.

$$x^n = k H_d^{n-1} \cdot y$$

$$y = \frac{0.7228(x + 0.27 \times 17.58)^{1.85}}{(17.58)^{0.85}} + 0.126 * 17.58 - 0.34715$$

$$(17.58)^{0.375}(x + 0.27 \times 17.58)^{0.825}$$

$$y = 0.0673\,(x + 4.7466)^{1.85} + 2.2151 - 1.2643$$

$$(x + 4.7466)^{0.625}$$

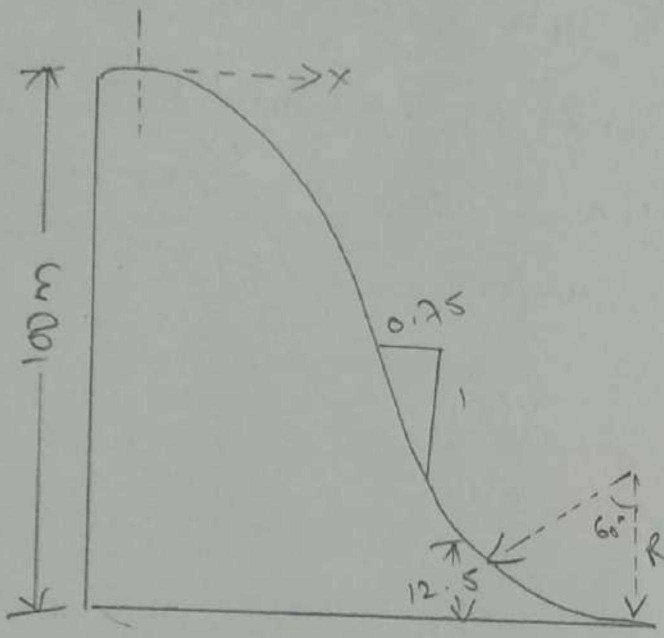
The curve will extend upto $x = 0.27 \times 17.58 \Rightarrow -4.75m$

Hence the value of y coordinates are calculated for the values of $x = -0.5m$, $-1.0\,m$, $-1.5m$, $-2m$, $-3m$, $-4m$ and $-4.75\,m$ and tabulated as under.

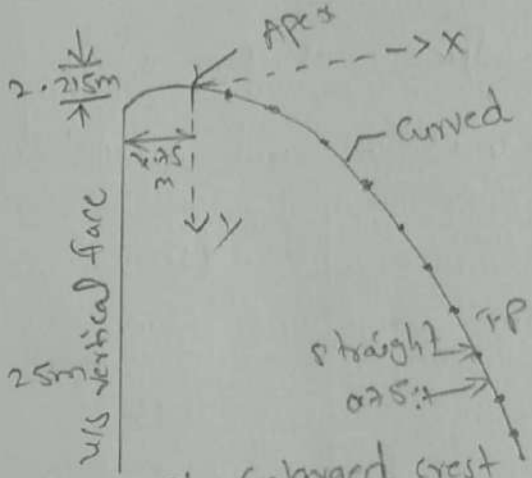
x (m)	y (m)
−0.5	0.012
−1.0	0.057
−1.5	0.135
−2.0	0.248
−3.0	0.601
−4.0	1.199
−4.75	2.215

4. Step 4; Design of d/s bucket :-

The profile of the spillway is shown in fig 11.27 (a). A reverse curve at the toe is provided to form a bucket. The radius of the bucket is generally kept at $R = P/4 = 100/4 = 25m$. The bucket will subtend an angle of 60° at the centre c as shown.



(a) spillway section



(b) Enlarged crest

UNIT-IV

DIVERSION HEADWORKS

Diversion Headworks:- Any hydraulic structure which supplies water to the off-taking canal is called a headwork. Headworks may be divided in to two classes:

i, storage headwork.

ii, Diversion headwork.

⇒ i, storage headwork:- A storage Headwork comprises the construction of a dam across the river. it stores water during the period of excess supplies in the river and releases it when demand overtakes available supplies.

⇒ ii, Diversion headwork:- A Diversion Headwork serves to divert the required supply into the canal from the river. and it serves the following purposes.

* It regulates the intake of water into the canal.
* It controls the silt entry into the canal.
* It raises the water level in the river so that the commanded area can be increased.

Types of Diversion Headworks:-

A Diversion Headworks can further be sub-divided in two principal classes

1. Temporary spurs (or) bunds.
2. permanent weirs and barrages.

⇒ 1. Temporary spurs (or) bunds are those which are temporary and are constructed every year after the floods.

⇒ 2. permanent weirs and barrages:-

weir:- The weir is a solid obstruction put across the river to raise its water level and divert the water into the canal. if a weir also stores water for tiding over small periods of short supplies, it is called a storage weir.

* The main difference between a storage weir and dam is only in height and the duration for which the supply is stored.

* A dam stores the supply for a comparatively longer duration.

~~Barrage:-~~

weirs are classified into two heads, depending upon the criterion of the designs of their floors.

1. Gravity weirs
2. Non-gravity weirs

⇒ 1. Gravity weir:- A Gravity weir is the one in which the uplift pressure due to the seepage of water below the floor is resisted entirely by the weight of floor.

2) Non-gravity weirs :-

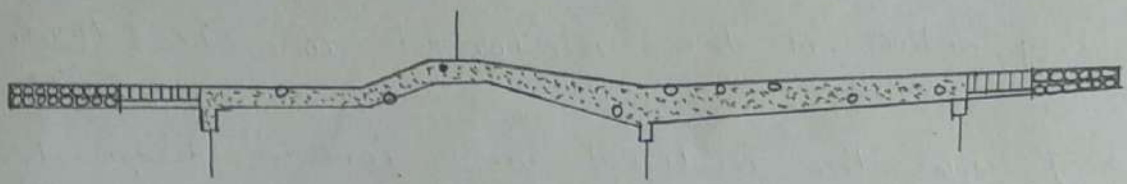
The non-gravity type, the floor thickness is kept relatively less, and the uplift pressure is largely resisted by the bending action of the reinforced concrete floor.

Depending upon the material and certain design features, gravity weir (or simply) can further be sub-divided in to the following types:

1. vertical drop weir
2. sloping weir
   a, Mansory or concrete slope weir
   b, Dry stone slope weir.
3. parabolic weir.

1. vertical drop weirs:-

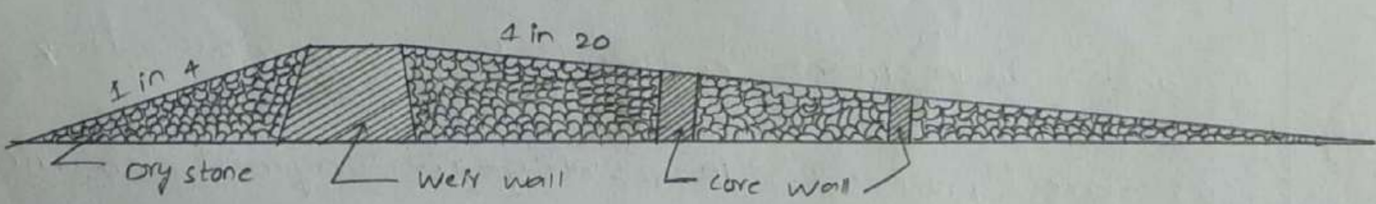
A vertical drop weir consists of a vertical drop wall or) crest wall, with (or) without crest gates. At the upstream and downstream ends of the impervious floor, cutoff piles are provided. To safegaurd aganist socuring action, laughing aprons are provided both at upstream and down stream end of the floor. vertical drop weirs are suitable for any type of foundation.



(a) sloping weir of concrete.

2. Mansory (or) concrete sloping weir :-

weirs of this type are of recent origin. They are suitable for soft sandy are suitable for foundations, and are generally used where the difference in weir crest and down stream river bed is limited to 3 metres, when water passes over such a weir, hydraulic jump is formed on the slopping glacis.



(b) sloping weir of mansory and dry stone

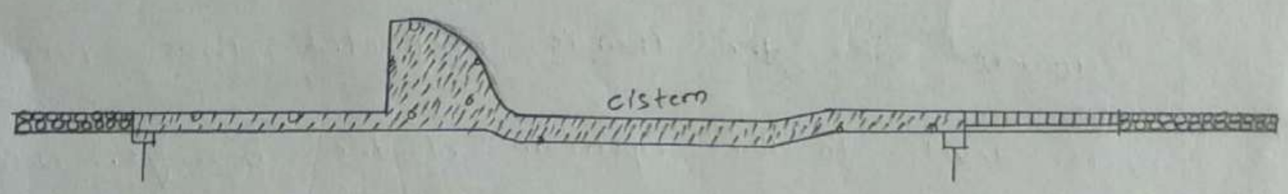
sloping weirs.

3. Dry stone slope weir:-

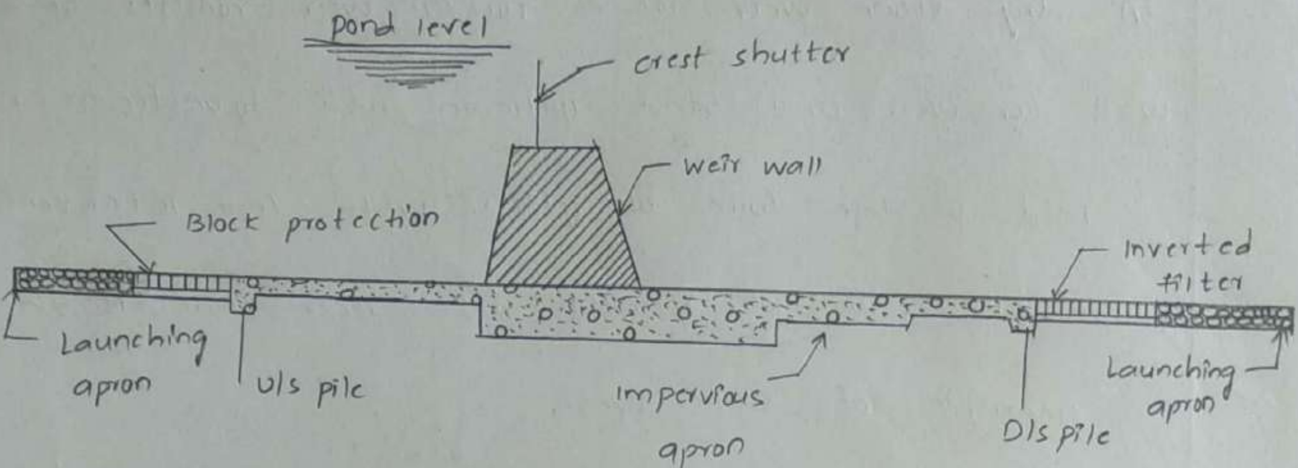
A dry stone weir (or) a rockfill weir consists of a body wall (or weir wall) and upstream and downstream rockfills laid in the form of glacis, with few intervening core walls. okhla wier on Yamuna river, near Delhi, is the example of such weir.

4. parabolic weir:-

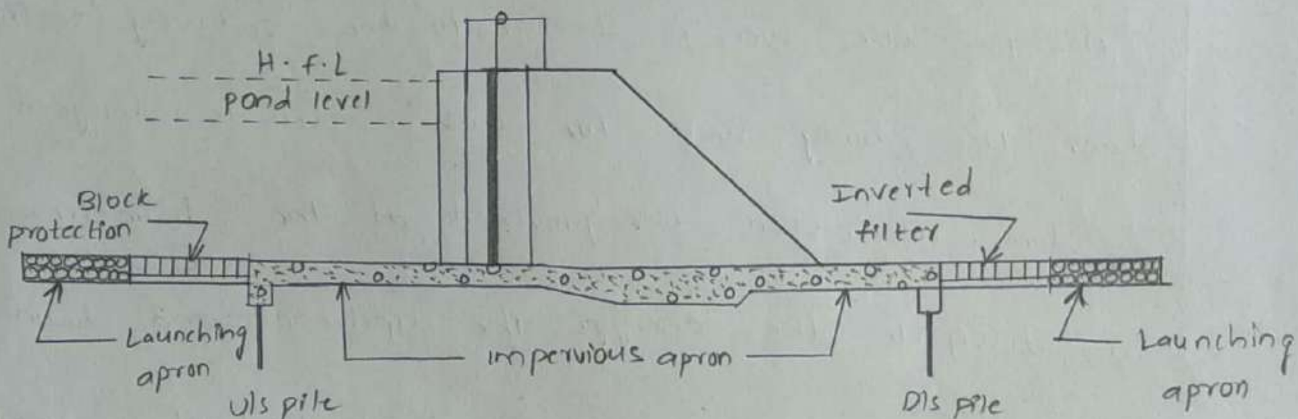
A parabolic weir is similar to the spillway section of a dam. The body wall for such a weir is designed as low dam. A cistern is provided at the downstream side to dissipate the energy. The upstream and down-stream protection works are similar to that of a vertical drop (or) sloping glacis weir.



weir & barrage :-



Vertical drop weir

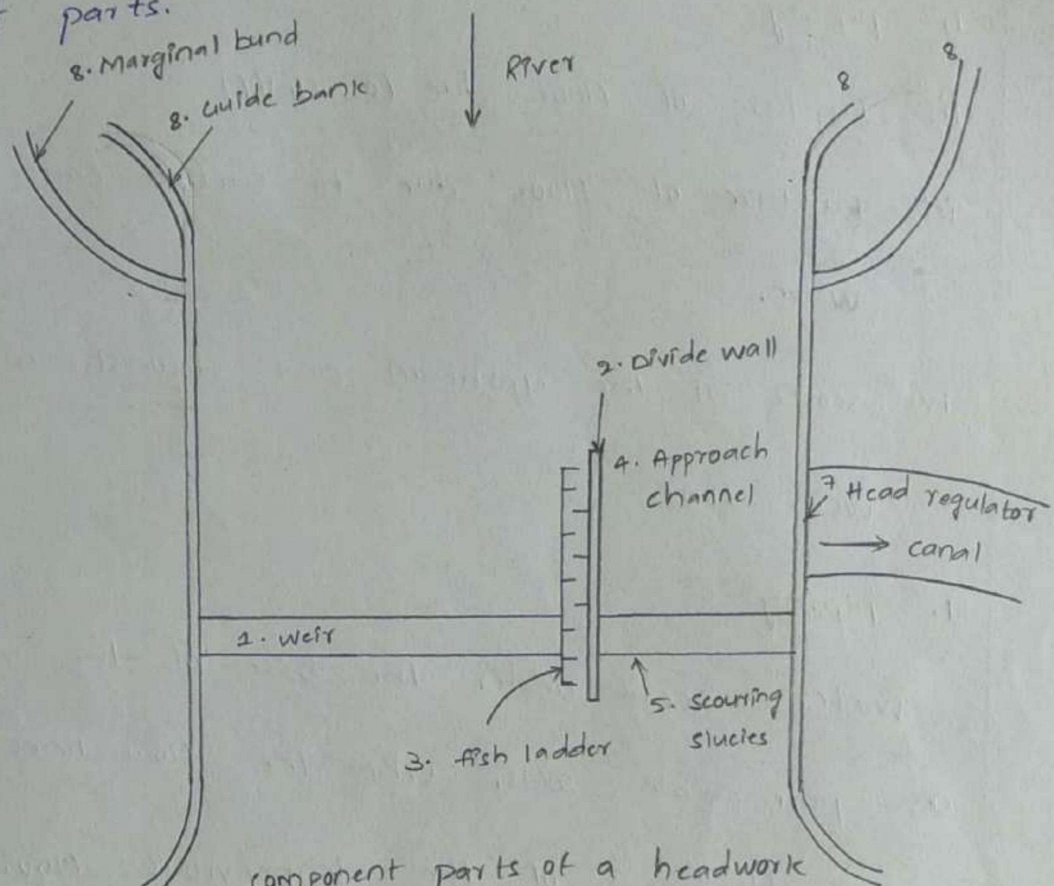


Barrage

the gates are raised to clear off the high flood level, enabling the high flood to pass downstream with minimum afflux. when the flood recedes, the gates are lowered and the flow is obstructed, thus raising the water level to the upstream of the barrage. Due to this there is less silting and better control over the levels. However, barrages are much more costlier than the weirs.

component parts of a diversion headwork

A diversion headwork consists of the following component parts.



component parts of a headwork

1. weir or barrage
2. Divide wall or divide groyne
3. fish ladder
4. pocket (or) approach channel
5. scouring sluices
6. silt prevention devices
7. canal head regulator
8. River training works (Marginal bunds and guide banks)

The description and design of these parts are discussed in the following articles.

Causes of failure of weirs and their ~~m~~ remedies

A weir may fail due to the following reasons:

i) piping

ii) Rupture of floor due to uplift

iii) Rupture of floor due to suction caused by standing wave.

iv) scour at the upstream and downstream side of the weir floor.

1. piping

water seeps under the base of the weirs founded on permeable soils, when the flow lines emerge out at the d/s end of the impervious floor of the weir, the hydraulic ~~Jump~~ gradient or the exit gradient may may exceed a certain critical value for the soil. In that case, the surface soil starts boiling and is washed away by percolating water. with the removal of the surface soil, there is further soil, there is further concentration of flow lines in to the resulting depression and still more soil is removed.

Remedies :-

piping failures can be prevented by

i) providing sufficient length of the impervious floor so that path of percolation is increased and the exit gradient is decreased.

ii) providing pile at downstream end.

2. Rupture of floor due to uplift.

If the weight of floor is insufficient to resist the uplit pressure, the floor may burst and effective length of impervious floor is thereby reduced. The final failure, however, is due to the reduction of the effective length with the consequent increase in the exit gradient.

Remedies :- failures due to rupture of floor may be prevented by.

i) providing impervious floor of sucfficent length.

ii) providing impervious Floor of appropriate thickness at various points

3. Rupture of floor due to suction caused by standing wave,

The standing wave (or) hydraulic jump forced at the d/s of the weir causes suction which also acts in the direction of uplift pressure. if the floor thickness is insufficient, it may fail by rupture. Examples of such failures are Marala weir on the chenab and Rasul weir.

Remedies:- failures can be prevented by

i, providing additional thickness of floor to counter-balance the extra pressure due to the standing wave.

ii, constructing the floor thickness in one concrete mass instead of in masonry layers.

4. Scour on upstream and downstream of the weir.

when the natural waterway of a river is contracted, the water may scour the bed both at upstream and downstream of the structure. The scour holes so formed may progress towards the structure, causing its faiure, Examples of such failures are Islam weir and Deoha weir.

Remedies:- such failures can be prevented by

i, Taking the piles at upstream and downstream ends of the impervious floor, much below the calculated socur level.

ii, providing suitable length and thickness of launching aprons at u/s and d/s sides, so that stones of the aprons may settle in the socur holes.

Weirs on permeable foundations

Bligh's creep theory:- The design of the impervious floor, or the apron is directly dependent on the possiblities of percolation in the porous soil on which the apron is built. Bligh assumes as an approximation that the hydraulic slope or gradient is constant throughout the impervious that the hydraulic length of the apron.

He designed the length of the travel as the 'creep length' which is the sum of horizontal as well as vertical length of creep. Bligh asserted that no amount of sheet priing or another cut-off could ever stop the percolation unless the cut-off extends upto the imperable sub-soil strata. Thus according to Bligh's theory, the total creep length L for the case

$L = l$

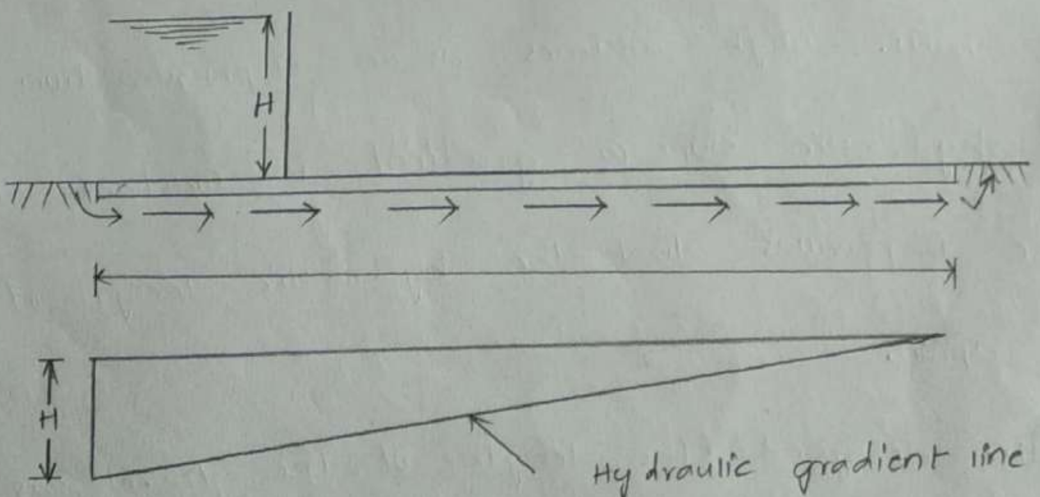
the total creep length is

$L = 2d_1 + l + 2d_2$

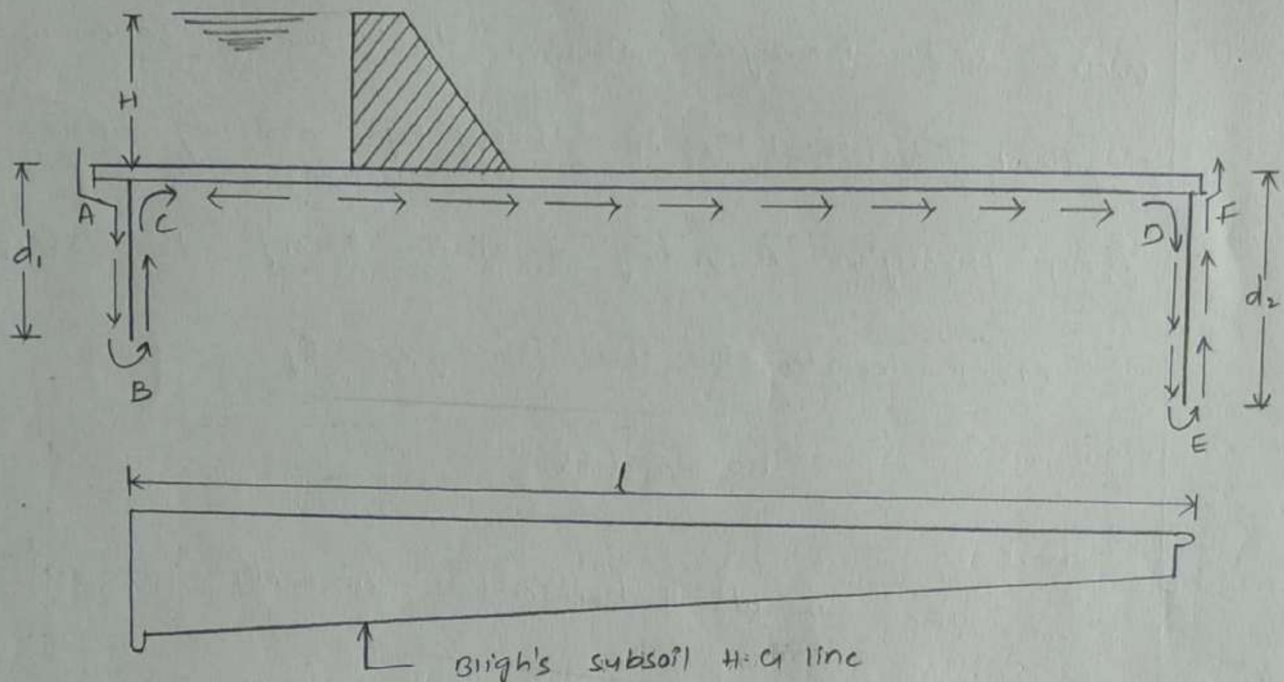
This means that in calculating the length of creep, the depth of every cutoff vertical creep is mutiplied by the coefficient 2.

if H is the total loss of head, the 'loss of head per unit length of the creep' 'c' would be.

$$c = \frac{H}{2d_1 + l + 2d_2} = \frac{H}{L}$$



(a) floor without vertical cutoff



(b) floor with vertical cutoffs

Bligh's creep theory

Limitations of Bligh's theory:-

1. Bligh made no distinction between horizontal and vertical creep.

2. Bligh's method holds good so long as the horizontal distance between the pilines is greater than twice their depth.

3. Loss of head does not take place in the same proportion as creep length. Also, the uplift pressure distribution

is not linear but follows a sine curve.

ii) Lane's weighted creep theory:-

Based on statistical investigations of as many as 278 dams, weirs and barrages all over the world, Lane observed that vertical creep theory by evolving 'lanes weighted creep theory'. According to this theory, the weighted creep length the (L_w) is given by

$$L_w = \frac{1}{3} l + V$$

where

l = the sum of all horizontal contacts and all the sloping contacts having slope less than 45°

V = sum of all the vertical contacts and all the sloping contacts steeper than 45°.

$$L_w = \frac{1}{3} l + 2d_1 + 2d_2$$

To ensure safety against piping, lane suggested that the weighted creep length must not be less than the following.

$$L_w = C_w H.$$

where

L_w = weighted creep length given

C_w = Lane's creep coefficient, the value of which depends on the type of the soil.

S.NO	Type of soil	Lane's creep coefficient C_w	Lane's safe Hydraulic gradient $(1/C_w)$
1	Very fine sand (or) silt	8.5	1/8.5
2	fine sand	7.0	1/7.0
3	Medium sand	6.0	1/6.0
4	coarse sand	5.0	1/5.0
5	fine gravel	4.0	1/4.0
6	Medium gravel	3.5	1/3.50
7	Coarse gravel including cobbles	3.0	1/3.0
8	Boulders with same cobbles & gravel	2.5	1/2.5
9	soft clay.	3.0	1/3.0

Example 12.1

calculate the average hydraulic gradient. Also, find the uplift pressures at points 6, 12 and 18m from the u/s end of the floor and find the thickness of the floor at these points taking $\rho = 2.24$.

Sol:- Total length of creep $= (2\times6) + 22 + (2\times8) = 50$

Hydraulic gradient $= \frac{4}{50} = \frac{1}{12.5}$

i) uplift pressure at a point A, 6m from u/s

length of creep upto $A = (6\times2) + 6 = 18m$

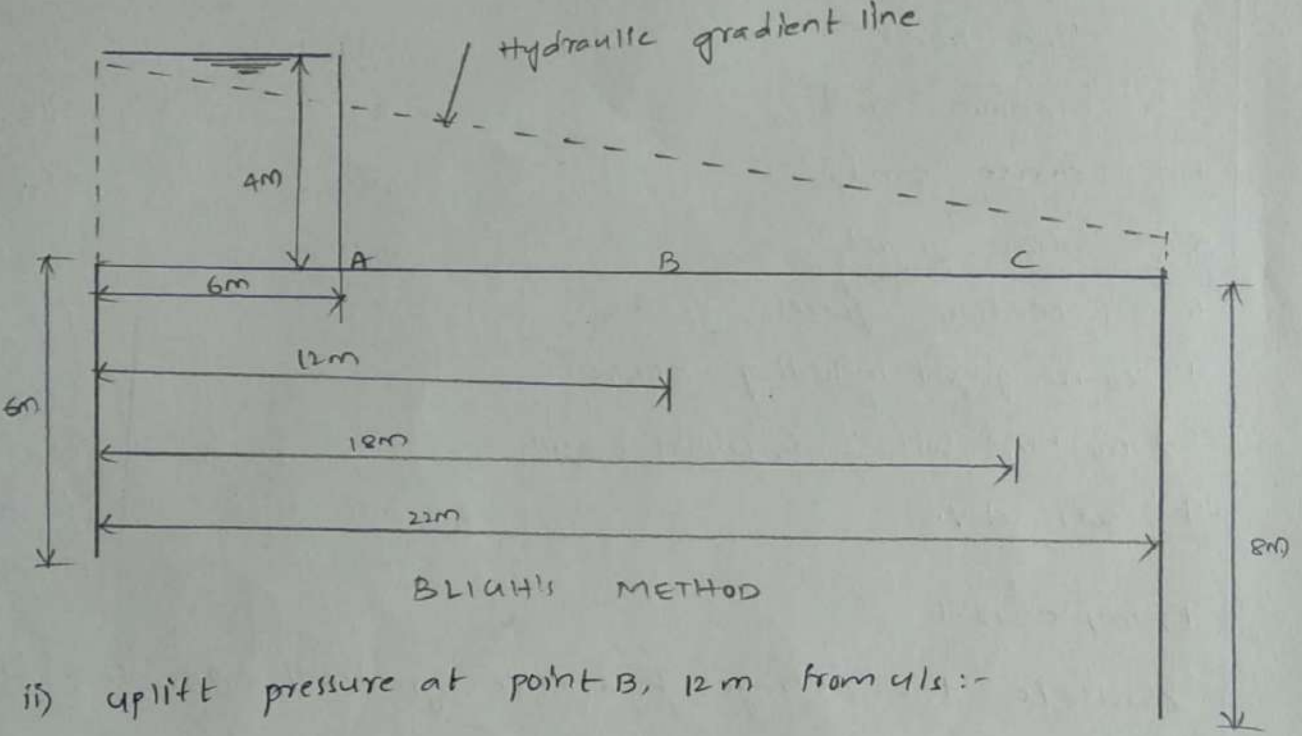
unbalanced head $= h_1 = 4\left(1 - \frac{18}{50}\right) = 2.56m$

uplift pressure $= wh_1 = 9.81 \times 2.56$

$= 25.11 \ kN/m^2$

thickness $t = \frac{4}{3} \frac{h_1}{\rho - 1}$

$= \frac{4}{3} \times \frac{2.56}{3.24 - 1} = 2.76m$



BLIGH's METHOD

ii) uplift pressure at point B, 12 m from u/s :-

length of creep upto $B = (6 \times 2) + 12 = 24m$

unbalanced head $h_2 = 4 \left[1 - \frac{24}{50}\right] = 2.08m$

uplift pressure $= wh_2 = 9.81 \times 2.08 = 20.4$ kN/m³

Thickness $t = \frac{4}{3} \frac{h_2}{\rho - 1}$

$= \frac{4}{3} \frac{2.08}{2.24 - 1} = 2.23m$

iii) uplift pressure at point C, 18m from u/s:-

length of creep upto $C = (6 \times 2) + 18 = 30m$

unbalanced head $h_3 = 4 \left[1 - \frac{30}{50}\right] = 1.6m$

uplift pressure $= wh_3 = 9.81 \times 1.6 = 15.7$ kN/m³

Thickness $t = \frac{4}{3} \frac{h_3}{2.24-1}$

$= \frac{4}{3} \times \frac{1.6}{2.24-1} = 1.72m$

3, Khosla's theory:-

In 1926-27, some siphons on upper chenab canal, designed on Bligh's theory gave trouble. Actual pressure measurements made with the help of pipes inserted in the floors of two of these siphons did not show any relationship with the pressure calculated on the basis of Bligh's theory.

This led to the following provisions conclusions by khosla.

1. The outer faces of the end sheet piles were much more effective than the inner ones and the horizontal length of the floor.

2. the intermediate piles if smaller in length than the outer ones were ineffective except for the local redistribution of pressure.

3. It was absolutely essential to have a reasonably deep vertical cutoff at the downstream end to prevent undermining.

UNIT-V Canal Regulation Works

CANAL FALLS

A fall is an irrigation structured constructed across a canal to lower down its water level and destory the surplus Energy liberated from the failing water which may otherwise scour the bed and banks of the canal.

The location of a fall is decided from the following considerations

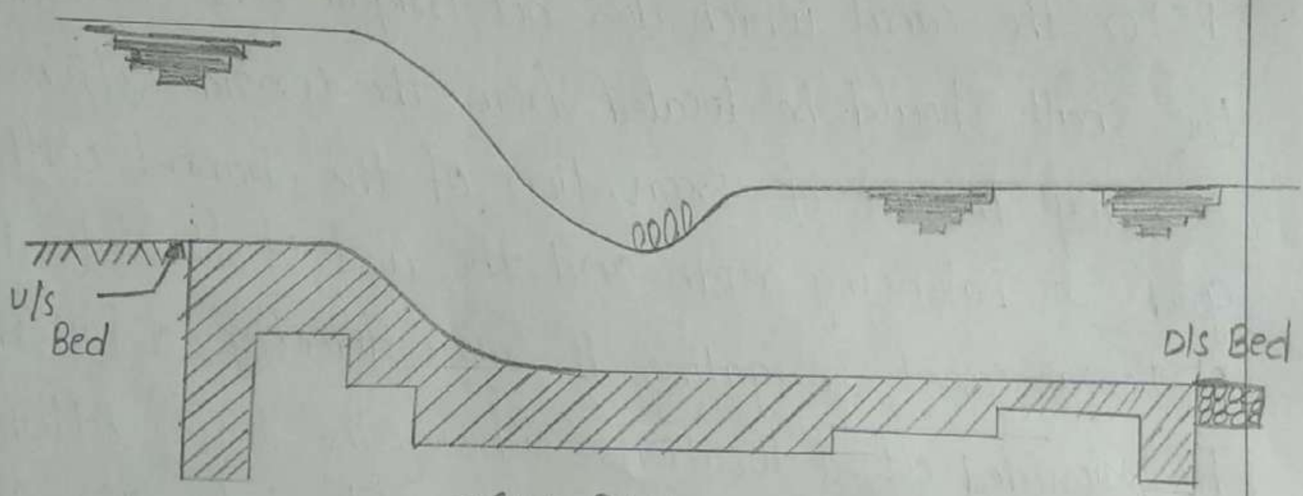
1. For the canal which does not irrigate the area directly, the shall should be located from the considerations of the channel with re- Economy in cost of Excavation of the channel with re- gard to balancing depth and the cost of the falls itself
2. For a canal irrigating the area directly, a fall may be provided at a locations where the F.S.L. After the Drop, The F.S.L of the canal may be below the ground level for $\frac{1}{2}$ to $\frac{1}{4}$ kilometer.
3. The location of the fall may also be decided from the consideration of the possibility of combination it with a regulator or a bridge or any other masonry work.
4. A relative Economy of providing large number of small falls v/s small Number of large falls should be worked out. The provision of small number of big falls resists in unbalanced Earthwork.

DEVELOPMENT OF Falls

1. orge fall: The orge fall was first constructed by Sir proby cautley on the Ganga canal. This type of fall has gradual convex and concave curves.

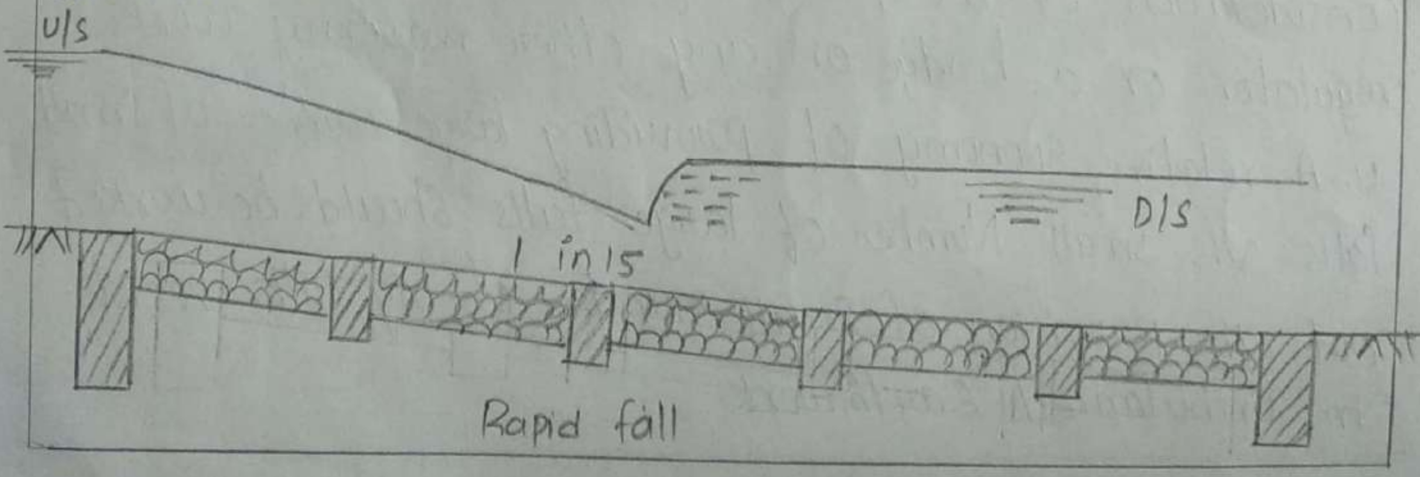
i) There was considerable draw down Effect on the U/s resulting in bed Erosion

ii) Due to Smooth transition, the kinetic Energy was preserved till sufficient depth.



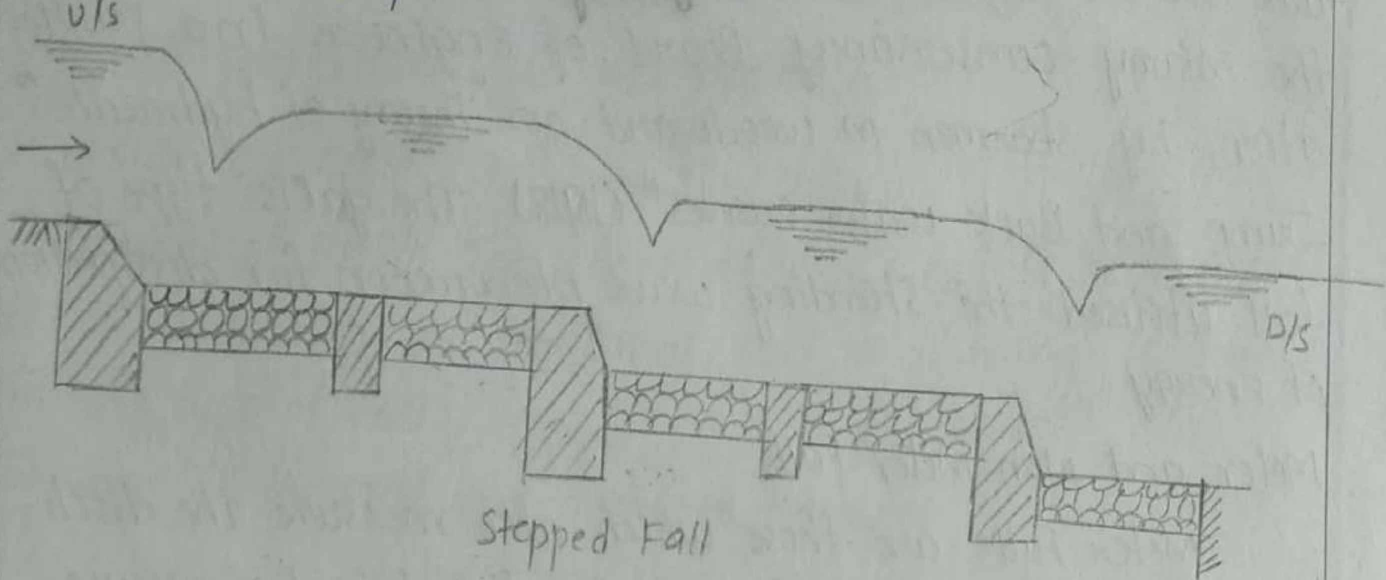
OGEE FALL

Rapid Fall: Such falls are provided on western yamuna canal and were designed by Lieut R.F crofton such a fall consist of a glacis sloping at 1 vertical to 10 to 20 Horizontal



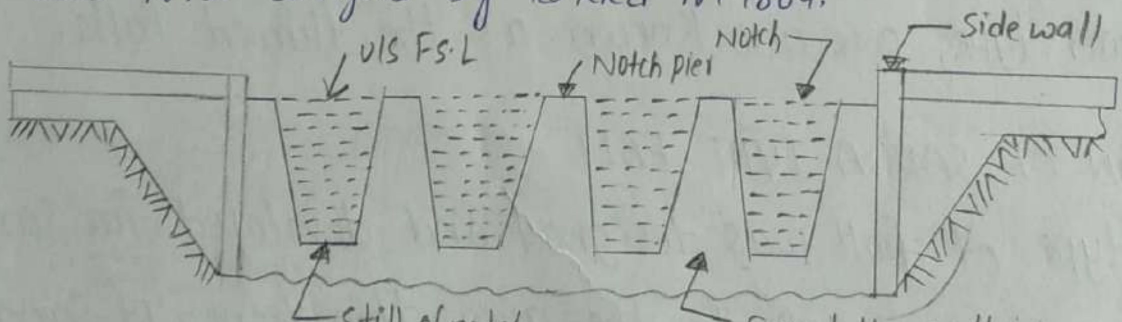
Rapid fall

Stepped Fall: Stepped Fall was a next development of the rapid fall. one such type was provided at the tail of main canal Escape of Sarda canal.



Stepped Fall

Notch Fall: Soon after the development of stepped fall, the Efficiency of vertical impact on the floor for Energy dissipation came to be recognished. The vertical fall came in the field along with cistern. The trapezoidal notch fall was first designed by ~~18~~ Ried in 1864.



Vertical drop fall: In the vertical drop fall, the nappe imagires clear into the water cushion below. In the Eariler type of vertical falls, the dimension of cistern were put in arbitrarly the light of Experience of the designer.

Glacis type fall: The Efficiency of the hydraulic Jump as a very potent means of destorying the Energy of canal falls was brought out clearly by the research work of the Miamy conservancy Board of Engineers in a publication by Sharman M. woodward on "Theory of hydraulic Jump and Back water curves" (1918). The glacis type of fall utilised the standing wave phenomenon for dissipation of Energy.

Meter and Non-Meter Falls:

Meter Falls are those which also measure the discharge of the canal. The non-meter fall do not measure the discharge.

Contracted and Full width falls.

A fall may either be contracted of the fall channel ~~and~~ width or it may be contracted. The contracted Falls, the later type, are also known as the flumed falls.

DESIGN OF SARDA TYPE FALL

This type of ~~f~~fall was designed and developed for Sarda Canal Systems of U.P. In the area, thin veener of Sandy-clay overlies a stratum of pure sand. This fall has, therefore been constructed ~~for~~ drops varying from 0.9 to 1.8 meters. In the Earlier designs, the cistern was not depressed below the d/s floor and the d/s wings were not flared.

The completed design consists of the design of the following component parts:

1) Crest 2) Cistern 3) Impervious ~~factor~~ floor.

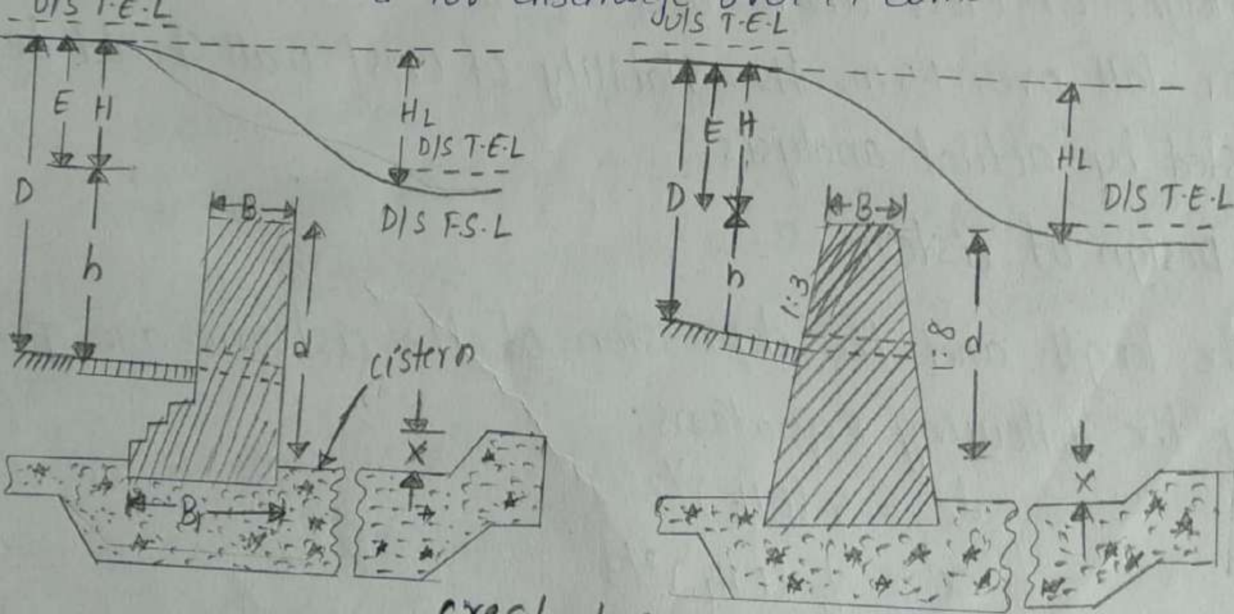
4) D/S protection (5) U/S approach

1: Design of crest

i) length of crest: The length of the crest is kept equal to the bed with of the canal, and no fluming is done in this type of fall.

ii) Shape of the crest and discharge formula:

Two types of crest are used, The rectangular crest is used for discharges up to 14 cumecs (500 cusecs) and trapezoidal crest is used for discharge over 14 cumecs.



crest design

For the rectangular crest:

Top width of crest is given by $B = 0.55\sqrt{d}$ meter

Base width is given by $B_1 = \frac{H+d}{e}$

for masonry crest, e may be taken equal to 2

Discharge is given by $Q = 1.835\, L H^{3/2} \left(\frac{H}{B}\right)^{1/6}$

Where, Q = discharge in cumecs

L = length of th crest in metres.

For a trapezoidal crest:

The Top width of crest is given by $B = 0.55\sqrt{H+d}$

U/S Batter = 1:3

D/S Batter = 1:8

Thus the base width is determined by the better

Discharge is given by, $Q = 1.99\, L H^{3/2} \left(\frac{H}{8}\right)^{1/6}$

iii) Crest level:

The value of H is known R.L of crest = U/S F.S.L − H

Height of crest above bed = $h = D - H$

For fall over 1.5 m, the stability of crest wall should be tested by actual analysis.

2. Design of cistern

The length and the depression of the cisterns are given by the following Equations:

$$l_c = 5 (E H_L)^{1/2}$$

$$x = \frac{1}{4} (E H_L)^{2/3}$$

3. Design of Impervious floor:

The total length of impervious floor is determined Either by Bligh's theory or by Khosla's theory. The maximum Seepage head is equal to d. out of the total imprevious floor length, a minimum length (l_d), to be provided to the d/s of the crest.

$$l_d = 2(D+1.2) + H_L \text{ metres}$$

The balance of the imprevious floor length may be provided under and u/s of the crest.

4. D/s protection: The d/s protection consists of (i) bed protection, ii) side protection, and iii) d/s wings.

i) Bed protection: The bed protection consists of dry brick pitching about 20cm thick resting on 10cm ballast. The length of the pitching and the number of curtain walls to be provided.

ii) Side protection: Side pitching, consisting one brick on edge, is provided after the warped wings. Generally, Warping masnory wings is done from vertical for slope of 1:1. Hence, the side pitching is warped from a slope of 1:1 to $1\frac{1}{2}:1$. The pitching supported on a toe wall $1\frac{1}{2}$ brick thick and of depth equal to half the d/s water depth.

iii) D/s wings: The d/s wings are kept vertical for a length of 5 to 8 times $\sqrt{EH_L}$ from the crest, and are then warped or flared to a slope of 1:1 or $1\frac{1}{2}:1$. An average ~~slay~~ splay of 1 to 2.5 to 1 in 4 for attaining the required slope is given to the top of the wings.

5. Design of U/s approach: for discharge up to 14 cumecs,

the u/s wings may be splayed, straight at angle of 45 for greater discharges, the wings are kept segmental with radius equal to 5 to 6 times H, subtending an angle of 60° at the centre, and then are carried straight into the berm

Head Regulators and cross-Regulators.

Head Regulator and cross-Regulator regulate the supplies of the off-taking channel and the parent channel respectively. A distributary head is a regulator, a metre of supply and a silt selective structure. A cross regulator is provided on the main canal at the d/s of the off-take to head up to the water level.

Functions of distributary head regulator:

1. They regulate or control the supplies to the off-taking channel
2. They serve as a meter for measuring the discharge entering in to the off-taking canal
3. They control the slit entry in the off-taking canal.
4. They help in shutting off the supplies when not needed in the off-taking canal, or when the off taking channel is required to be closed for repairs.

h = difference in water level u/s and d/s of the channel, in metres

h_a = head due to velocity approach

C_1 = constant = 0.557

C_2 = constant = 0.80

Knowing the discharge Q, the length of water-way L can be calculated

2. Design of d/s floor:

The level and length of the d/s floor is determined under two flow conditions: (i) full supply discharge passing through both the head regulator and cross regulator and (ii) the discharge in the parent channel being insufficient; the cross regulator gate is partially opened and the off taking channel is running full.

The discharge intensity q and the head loss $H_L (= h)$ are known. Hence, the value of E_{f2} can be found from the Blench curves

$$D/s \text{ floor level} = d/s\ T.E.L - E_{f2} \simeq d/s\ F.S.L - E_{f2}$$

The d/s floor level, calculated from the above relation should never be provided higher than the d/s bed level

$$\text{Now,} \quad E_{f1} = E_{f2} + H_L$$

Hence, the depth D_1 and D_2 ~~cros~~ corresponding of E_{f1} and E_{f2} respectively are found from Specific Energy curves

Then length of d/s floor = 5 $(D_2 - D_1)$

Functions of ~~distributor~~ cross-regulator:

1. The Effective regulation of the whole canal system can be done with help of cross-regulator

2. During the periods of low discharges in the parent channel, the cross-regulator raises water level of the u/s and feeds the off-take channel in rotation.

3. It helps in closing the supply to the d/s of the parent channels, for the purpose of repairs Etc

4. They help in absorbing fluctuation in various sections of the canal system, and in preventing the possibilites of breaches in the tail reachers

5. Incidently, bridges and other communication work can be combined with it

DESIGN OF CROSS-REGULATOR AND DISTRIBUTARY HEAD REGULATOR.

1. Design of crest:

The discharge is determined by the drowned weir formula:

$$Q = \frac{2}{3} C_1 L \sqrt{2g}\left[(h+h_a)^{3/2} - h_a^{3/2}\right] + C_2 L d \sqrt{2g(h+h_a)}$$

where, Q = discharge in cumecs

L = length of water-way, in metres

d = ~~difference~~ depth of d/s ~~in~~ water level in the channel, measured above the crest

However, the d/s floor should be at least 2/3 rd of the total impreuious length of the floor.

3. Design of impervious floor:

Total length of the imprevious floor should be found from the consideration of the permissble exit gradient.

The depth of u/s cutoff $d_1 = \frac{1}{3}$ u/s water depth + 0.6 m

The depth of u/s cutoff $d_2 = \frac{1}{3}$ d/s water depth + 0.6 m

Maximum static head H_S = u/s F.S.L − d/s floor level

$$G_E = \frac{1}{\pi\sqrt{\lambda}} \frac{H_S}{d_2}$$, from which $\frac{1}{\pi\sqrt{\lambda}}$ is known.

From the exit gradient curves, $\alpha = bd_2$ is known. Hence the total length b of the impervious floor is known.

The minimum thickness of 0.3 to 0.5 m is provided from the pratical considerations.

4) Design of u/s and d/s protection:

u/s scour depth d_1 is taken equal ($\frac{1}{3}$ u/s water depth + 0.6m). The d/s ~~ccur~~ scour depth d_2 is taken equal to ($\frac{1}{2}$ d/s water depth + 0.6 m).

a) u/s protection: The u/s protection consists of a block protection having cubic contents = d_1 cubic metres/m. The cubic contents of u/s launching apron is kept equal to 2.25 d_2 cubic metre/metre width of regulator

b) d/s protection/ The cubic contents ~~d~~ of d/s inverted filter is kept equal to d_2 cubic metre/metre. The cubic contents of d/s launching apron is kept equal to 1.25 d_2 cubic metre/metre width of regulator.

Design a sarda type of fall for the following data

i) Full supply discharge : $\frac{u/s}{d/s}$ = 40 cumecs

ii) Full supply level : $\frac{u/s}{d/s} = \frac{218.30m}{216.80m}$

iii) Full supply depth : $\frac{u/s}{d/s} = \frac{1.8m}{1.8m}$

iv) Bed width : u/s/ds = $\frac{26m}{26m}$

v) Bed level : u/s/ds = $\frac{216.50m}{215.00m}$

vi) Drop : 1.5m.

Design the floor on Bligh's theory taking coefficient of creep = 8. Check the design by Khosla's theory and make changes if necessary. Safe exit gradient may be taken equal as $\frac{1}{5}$

Sol: Step 1 Calculation of H and d

The discharge is more than 14 cumecs, trapezoidal crest will be. The discharge is given by

$$Q = 1.99\, L\, H^{3/2} \left[\frac{H}{B}\right]^{1/6}$$

Here L = width of canal = 26m

Q = 40 cumecs

$B = 0.55\sqrt{H+d}$

$H + d = D +$ drop in the level

$H+d = 1.8 + 1.5 = 3.3\ m$

$B = 0.55\sqrt{H+d} = 0.55\sqrt{3.3} = 1.0\ m$

Substiting the value of B in (1), we get

$$40 = \frac{1.99 \times 26\ H^{(3/2 + 1/6)}}{(1.0)^{1/6}}$$

$$H^{5/3} = \frac{40 \times 1}{1.99 \times 26} = 0.774$$

$$H = (0.774)^{3/5} = 0.86\ m$$

Substituting this in (2) we get

$$d = 3.3 - H = 3.3 - 0.86 = 2.44\ m$$

Height of crest above bed = $D - H = 1.8 - 0.86 = 0.94\ m$

Step 2. Design of crest!

A trapezoidal crest will be adopted

Top width $B = 1m$; d/s batter = 1:3 ; u/s batter = 1:8

Assuming canal side slope 1:1, velocity of approach is given by

$$V_a = \frac{40}{(27+1.8)1.8} = 0.8\ m/sec$$

$$\text{velocity head} = \frac{V_a^2}{2g} = \frac{(0.8)^2}{2 \times 9.81} = 0.032\ m$$

$\therefore$ U/s T.E.L = U/s F.E.L + velocity head = $218.30 + 0.032 = 218.332\ m$

RL of crest = U/s F.S.L $- H = 218.30 - 0.86 = 217.44\ m$

value of

$E =$ U/s T.E.L $-$ R.L of crest $= 218.332 - 217.44$

$= 0.892\ m$

step 3

Design of cistern:

Depth of cistern $= x = \frac{1}{4}(EH_L)^{2/3} = \frac{1}{4}(0.892 \times 1.5)^{2/3} = 0.304m$

length of cistern $= l_c = 5(EH_L)^{1/2} = 5(0.892 \times 1.5)^{1/2} = 5.8m$

length of ~~distance~~ cistern = 6m

R.L of Bed of cistern = R.L of d/s bed − x

= 215.00 − 0.304

= 214.696m

Keep the cistern at R.L 214.69m

final value of $x = 215.00 - 214.69 = 0.31m$

step 4 Design of impervious of floor:

Seepage head $= H_S = d = 2.44m$

Bligh's coefficient = 8

length of impervious floor or creep length $= C \times H_S = 8 \times 2.44$

$= 19.5m$

provide u/s cut off $d_1 = 1.0m$ and d/s cut off $d_2 = 1.6m$

The vertical length of creep $= 2(1+1.6) = 5.2m$

length of horizontal imprevious floor $= 19.5 - 5.2 = 14.3m$

provide 15m length of imprevious floor

Minimum length of imprevious floor to the d/s of toe of the crest wall $= l_d = 2(D+1.2) + H_L = 2(1.8+1.2) + 1.5 = 7.5m$

Provide $l_d = 8m$. The balance of length $= 15 - 8 = 7m$ is to be provided under and u/s of the crest.

CROSS DRAINAGE WORKS

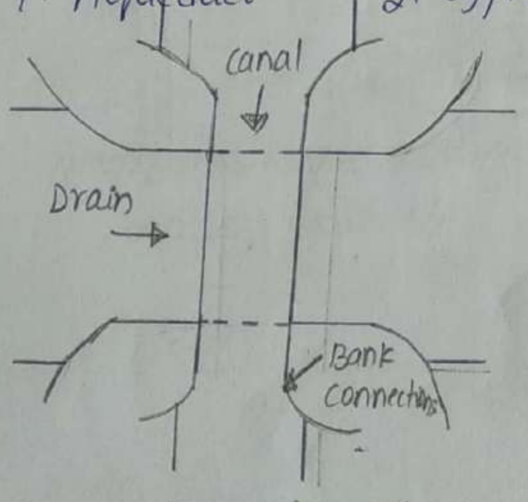
Types of Cross-Drainge Works:

Depending upon the relative levels and discharges, cross-drainage work may be of the following types:

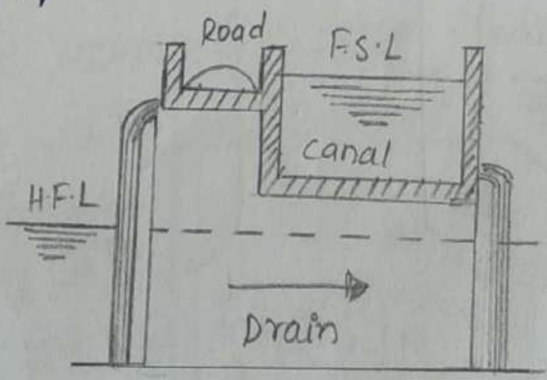
i) C.D Works carrying canal over the drainage:

In this type of C.D Work, the canal is carried ~~outer~~ over the natural drain. The advantage of such arrangment is that the canal, running perennially, is above the ground and is open to inspection. This is the usual type of work constructed when the drain is very big in comparsion to the section of the canal. The structures that fall under this type are:

1. Aqueduct
2. Syphon Aqueduct.



a) plan of crossing



b) Aqueduct

c) syphon aqueduct

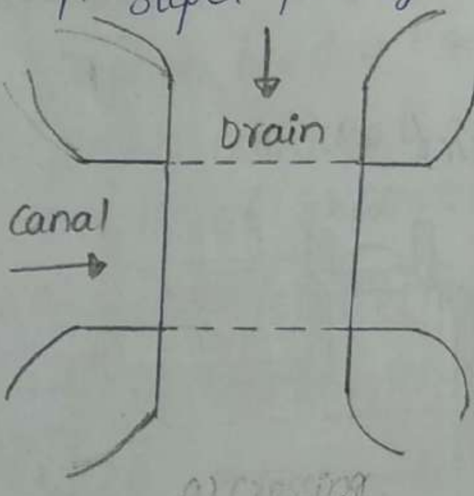
2) C.D Works carrying driange over the canal

In this type of C.D Work, driange is carried ~~out~~ over the canal. The advantage of this type is that the C.D works themselves are less liable to damage than the Earth work of the canal.

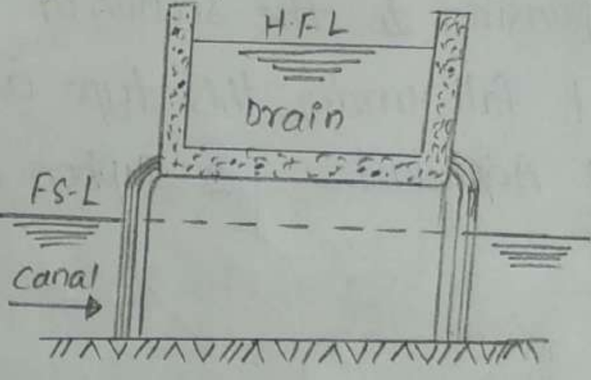
The major disadvantage of this work is that the Perennial canal is not open to inspection, Also if the slit is deposited in the barrels of the work, it is difficult to clear it out

The structure that fall under this type are

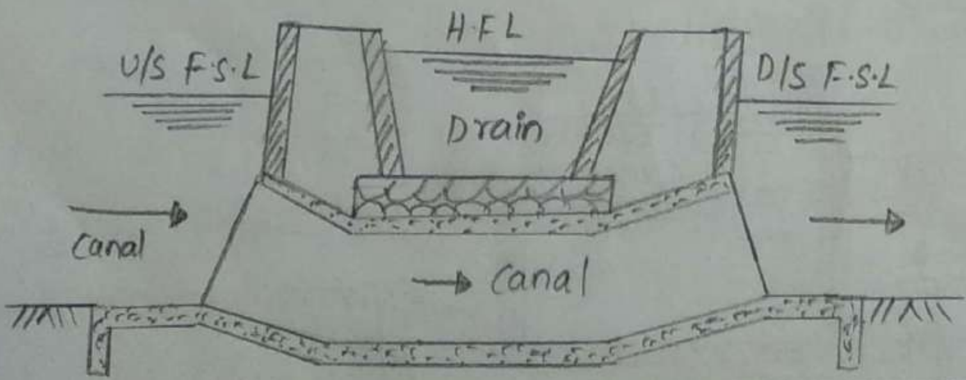
1. super-passage
2. Canal syphon



a) crossing



b) super passage



c) syphon

iii) CD Works admitting the drainage water in to the canal

In this type of work, the canal water and the drainage water are permitted to inter mingle with each other. The only advantage of this type of work is its low initial cost. Such type of works have the following disadvantages:

i) Regulation of such a work is difficult and requires additional staff

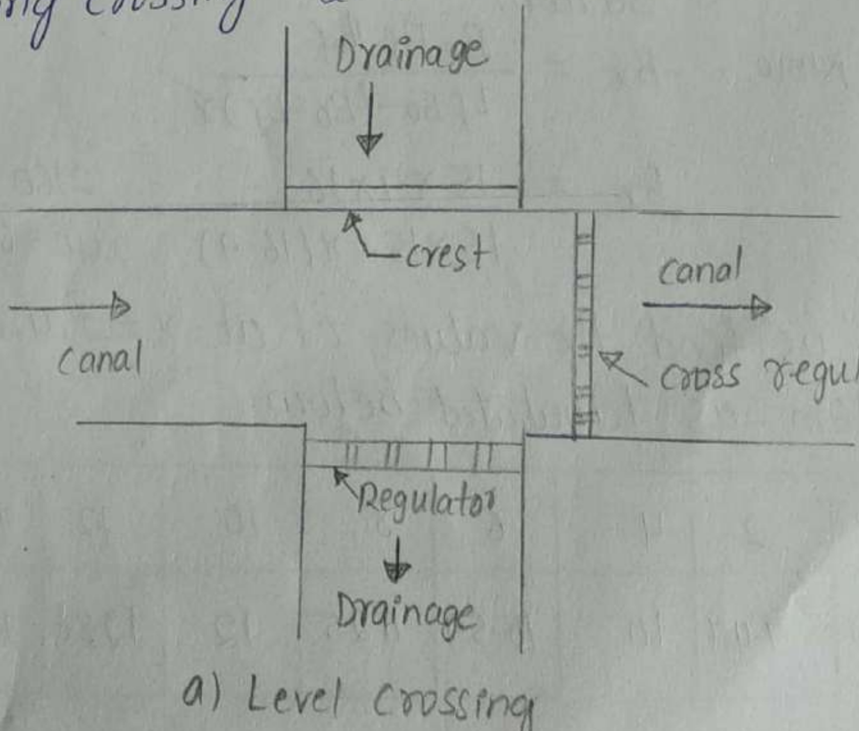
ii) the canal has to be designed to carry the increased flood discharge of the drain.

iii) the faulty regulation of gates may damage the canal

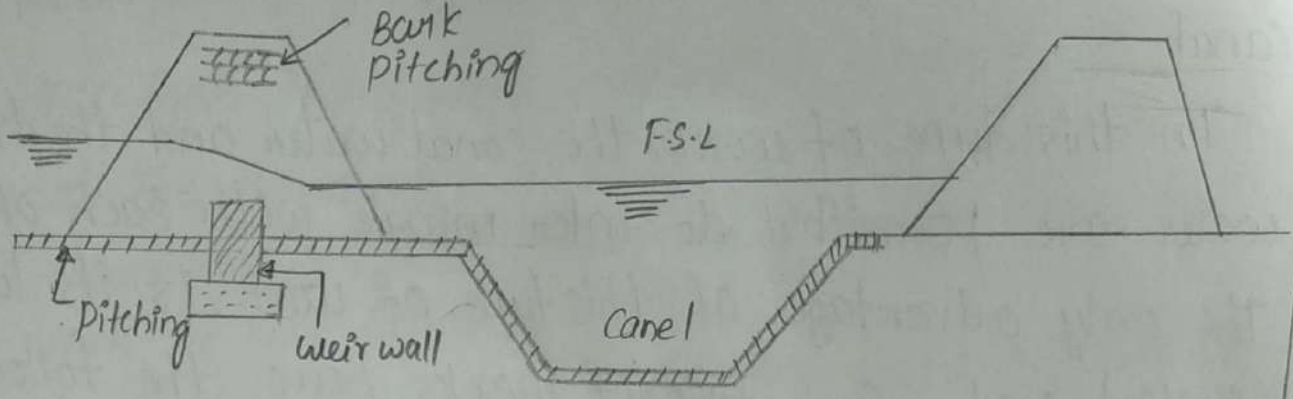
iv) there is additional expenditure of slit clearance

Following are the structure under this type CD Works:

1. Levelling crossing 2. Inlet and outlets



a) Level crossing



b) inlet

1. Design an Expansion transition for a canal for the following data, using mitra's method

length of flame = 16m

width of throat = 9m

width of Canal = 15m

Soll Given $B_f = 9m$, $B_0 = 15m$, $L_f = 16m$

let B_x = width at any distance x from the flumed Section,

Now, $$B_x = \frac{B_0 B_f \cancel{B_f}}{L_f B_0 - (B_0 - B_f)x}$$

$$B_x = \frac{15 \times 9 \times 16}{16 \times 15 - x(16-9)} = \frac{2160}{240 - 6x}$$

let us find the values of at $x = 2, 4, 6, 8, 10, 12, 14$ and 16m as tabulated below.

S(m)	2	4	6	8	10	12	14	16
Bs(m)	9.47	10	10.59	11.25	12	12.86	13.85	15

Solve. Using Chaturvedi's method.

Sol:

$$x = \frac{L_f B_0^{3/2}}{B_0^{3/2} - B_f^{1/2}} \left\{1 - (B_f / B_x)^{3/2}\right\}$$

$$x = \frac{16 (15)^{3/2}}{15^{3/2} - 9^{3/2}} \left\{1 - \left(\frac{B_f}{B_x}\right)^{3/2}\right\}$$

$$= 29.893 \left[1 - (B_f / B_x)^{3/2}\right]$$

$$(B_f / B_x)^{3/2} = 1 - (x - 29.893)$$

The computations are tabulated below

S(m)	2	4	6	8	10	12	14	16
$(B_f/B_x)^{3/2}$	0.933	0.8662	0.7993	0.7324	0.6655	0.5986	0.5317	0.4648
B_f/B_x	0.9549	0.9087	0.8613	0.8125	0.7622	0.7102	0.6563	0.6000
B_x	9.43	10.90	10.45	11.08	11.81	12.67	13.71	15.000