Irrigation → Necessity and importance, principal crops and crop seasons, types, Methods of application, Soil-Water-plant relation ship, soil moisture constants Consumptive use, crop Water requirement, duty and delta, factors effecting duty, depth and frequency of Irrigation, Irrigation efficiencies, Water logging and drainage, standards of quality of irrigation Water, crop rotation.

Definition →

Irrigation may be defined as the process of artificially supplying Water to soil for raising crops. It is a science of planning and designing an efficient, low cost, economic irrigation system tailored to fit natural conditions. It is the engineering of controlling and harnessing the various natural sources of Water, by the construction of dams and reservoirs, canals and head Works and finally distributing the Water to the agricultural fields. Irrigation engineering includes the study and design of Works in connection With river control, drainage of Water logged areas, and generation of hydroelectric power.

Necessity →

India is basically an agricultural country and all its resources depend on the agricultural output. Water is ~~ee~~ evidently the most vital element in the plant life. Water is normally supplied to the plants by nature through rains. However the total rainfall in a particular area may be eaither in sufficient, or ill timed. In order to get the max yield it is essential to supply the optimum quantity of Water, and to maintain correct timing of Water This is possible only through a systematic Irrigation System. [by collecting Water during the periods of excess rainfall and releasing it to the crop as and When it is needed.]

Thus the necessity of irrigation can be summarised in the following points :

→ When the total rainfall is less than needed for crop (i.e less Rainfall), artificial supply is necessary. In such a case, Irrigation Works may be constructed at a place where more Water is available and then to convey the Water to the area Where there is deficiency of Water. for Example Rajastan canal conveys Water to the arid zones of Rajastan, where the annual rainfall hardly exceeds 100 to 200 mm.

→ In case of Non uniform rainfall

The rainfall in a particular area may not be uniform over the crop period. due to these results either the yield may be less (or) the crop may die altogether. [Mainly in the case of Rabi season]

→ Growing a number of crops during a year.
The rainfall in an area may be sufficieant to raise only one type of crops during the rainy season (i.e kharif crops), for which no irrigation may be required. However with the ~~position~~ provision of irrigation facilities in that area crops can be raised in other season also (i.e Rabi crops).

→ Growing perennial crops :
perennial crops such as sugar cane etc which need water through out the year can be raised only through the provision of irrigation facilities in the area.

→ Commercial crops with additional water :
The rainfall in a particular area may be sufficient to raise the usual crops, but more water for raising commercial crops may be necessary

Scope →
The scope of irrigation can be devided into two heads.
(a) Engineering aspect.
(b) Agricultural aspect.

Engineering aspect →
(1). Storage, Diversion or lifting of water
This is the first phase of irrigation engineering. By the construction of a dam across the river, a suitable reservoir can be created and water can be stored.

Alternatively, if river is perennial and carries sufficient discharge, suitable diversion works, such as a weir, barrage and bhandara can be constructed across the river and water can be diverted to the canal. places where ground water table is high, suitable wells can be dug and water can be lifted and fed to small channels (or) pipes.

(2) Convey of water to the agricultural fields.
(3). Application of water to Agricultural fields.
(4). Drainage and Relieving water logging.
(5). Development of water power.

Agricultural Aspect →

The agricultural aspect deals with the through study of the following:

1. proper depth of water necessary in single application of water for various crops.
2. Distribution of water uniformly and periodically.
3. Capacities of different soils for irrigation water, and the flow of water in soils.
4. Reclamation of waste and alkaline lands, where this can be carried out through the agency of water.

Principal crops and crop seasons →

More than 70% of the indian population is directly (or) indirectly connected with agriculture.

The chief crops of india are →

Rice, wheat, sugarcane, tea, cotton, groundnut, jute, coffee, rubber, garden crops (like coconuts, orange, etc) etc.

Different types of soils are needed for raising for different types of crops. For example ~~heo~~

→ Heavy retentive soil (40% clay) is favourable for raising crops like sugarcane, rice, etc.. requiring more water.

→ light sandy soil (2 to 8% clay) is suitable for crops like gram, fodder, etc. requiring less water

→ Medium or normal soil having (10 - 20% of clay) is suitable for crops like wheat, cotton, maize, vegetables, oil seeds, etc requiring normal amount of water.

From the agricultural point of view, The year can be devided into two principal cropping seasons i.e Rabi and kharif

Rabi starts from ⇒ 1st october - 31st March

kharif starts from ⇒ 1st april - 30th september.

The kharif crops are Rice, bajra, jowar, maize, cotton, tobbacco, ground nut, etc. The Rabi crops are wheat, barley, gram, linseed, mustard, potatoes, etc. kharif crops are also called summer crops and Rabi crops as winter crops. karif crops require about 2-3 times the quantity of water required by the Rabi crops.

Types of Irrigation →

Irrigation may broadly be classified into
1. Surface irrigation, 2. sub-surface irrigation.

Surface irrigation can be futher classified into
a. Flow irrigation.
b. lift Irrigation.

When the water is available at a higher level and it is supplied to lower level by the more action of Gravity, then it is called Flow irrigation. But, if the water is lifted up by some mechanical (or) manual means, such as by pumps etc. and then supplied for irrigation, then it is called lift Irrigation. use of wells and tubewells for supplying irrigation water fall under this category of irrigation.

Flow Irrigation can be futher sub-devided into
(i) perennial irrigation; and
(ii) Flood irrigation

Perennial Irrigation →

In this system of irrigation. constant and continuous water supply is assured to the crops in accordance with the requirements of the crop throughout the 'crop period'.

Flood Irrigation →

this type of irrigation is called inundation irrigation In this method of irrigation soil is kept submerge & and thoroughly flooded with water, so as to cause thorough saturation of the land

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Subsurface Irrigation →

In this type of irrigation, water doesn't wet the soil surface. The underground water nourishes the plant roots by capillarity. It may be devided into the following two types.

(a) Natural sub-irrigation
(b) Artificial " "

Natural sub Irrigation →

Leakage water from channels etc., goes underground and during passage through the sub-soil it may irrigate crops, sown on lower lands by capillarity. Some times, leakage causes the water table to rise up, which helps in irrigation of crops by capillarity. When underground irrigation is achieved simply by natural process, with out any extra efforts, it is called natural sub-irrigation.

Artificial Sub-Irrigation →

When a system of open jointed drains is artificially laid below the soil, so as to supply water to the crops by capillarity, then it is known as Artificial sub irrigation.

Techniques of Water Distribution in the farms →

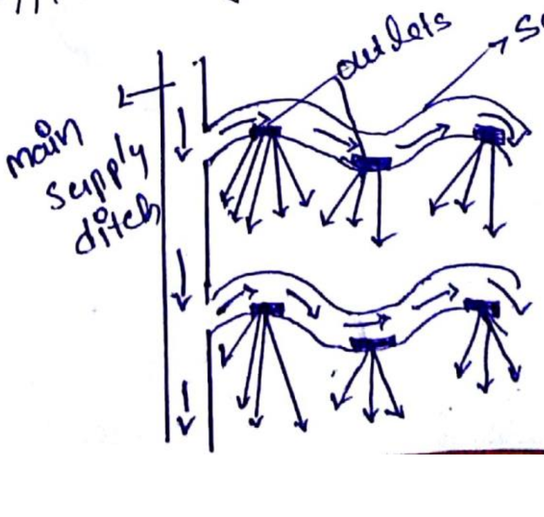
There are various ways in which the irrigation water can be applied to the fields. their main classification is as follows.

1. Free Flooding.
2. Border flooding.
3. Check flooding.
4. Basin flooding
5. Furrow irrigation method.
6. sprinkler irrigation method.
7. Drip irrigation method.

Free flooding (or) ordinary flooding →

In this method, ditches are excavated in the field and, they may be eaither on the contour (or) up and down the slope. Water from these ditches, flows across the field. After the water leaves the ditches, no attempt is made to control the flow by means of levees, etc. Since the movement of water is not restricted it is sometimes called wild flooding.

Although the initial cost of land preparation is low, labour requirements are usually high and water application efficieancy is also low.



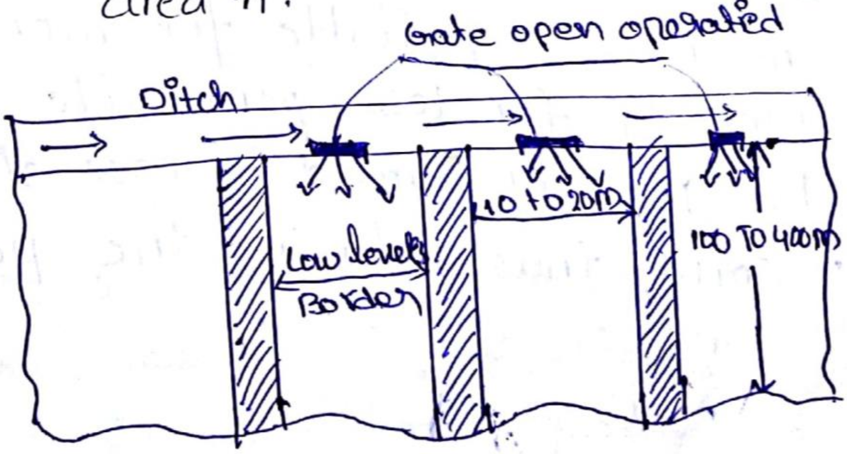
Border flooding →

In this method, the land is divided into a number of strips, seperated by low levees called borders. The land areas confined in each strip is of the order of 10 to 20 metres in width, and 100 to 400 metres in length,

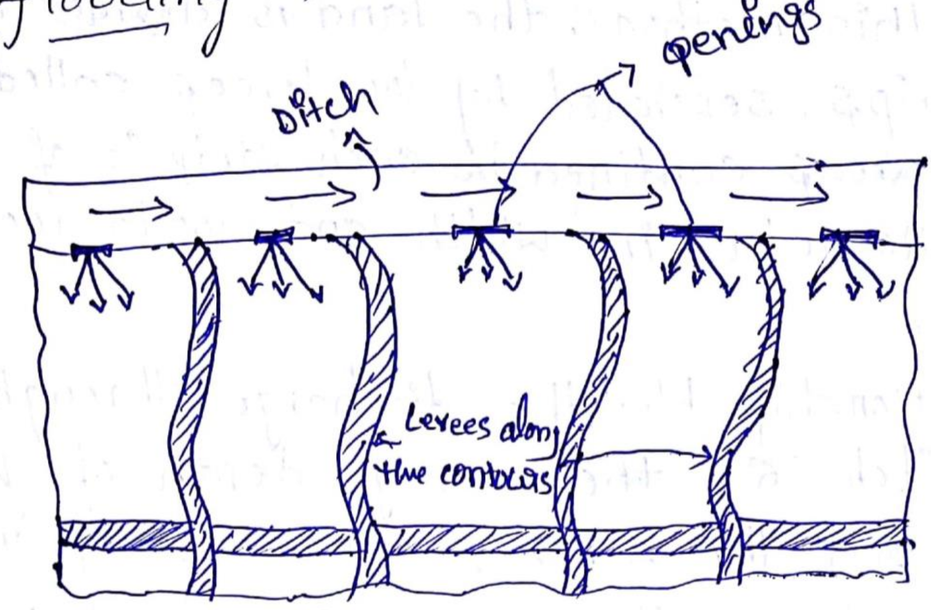
A relationship b/w the discharge through the supply ditch (Q), the average depth of water flowing over the strip (y), the rate of infiltration of the soil (f), the area of the land irrigated (A), and the approximate time required to cover the given area with water (t), is given by the eq,

$$t = 2.3 \, \frac{y}{f} \log_{10}\left[\frac{Q}{Q - fA}\right]$$

where
- Q = Discharge through the supply ditch
- y = Depth of water flowing over the border strip
- f = Rate of infiltration of soil
- A = Area of land strip to be irrigated
- t = Time required to cover the given area A.



Check flooding →

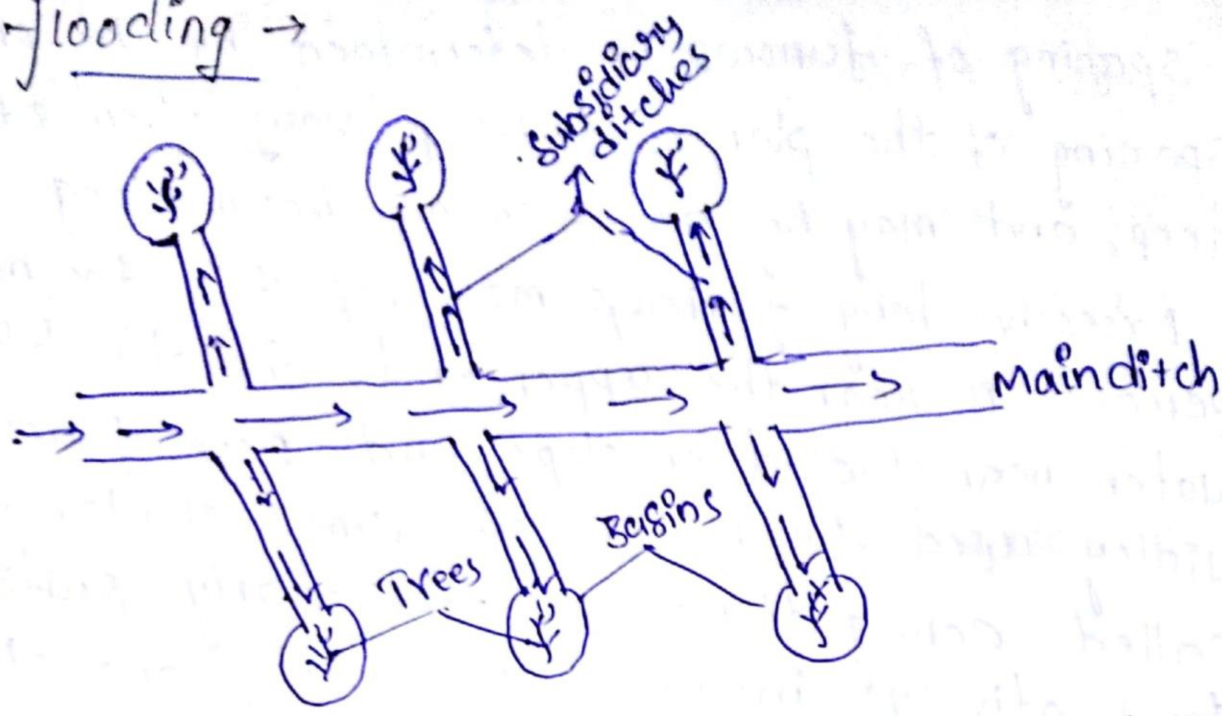


check flooding is similar to ordinary flooding except that the water is controlled by surrounding the check area with low and flat levees. levees are generally constructed along the contours, having vertical interval of about 5 to 10cm these levees are connected with cross-levees at convenient places as shown in above figure. The confined plot area varies from 0.2 to 0.8 hectare.

In check flooding, the check is filled with water at a fairly high rate and allowed to stand until the water infiltrates.

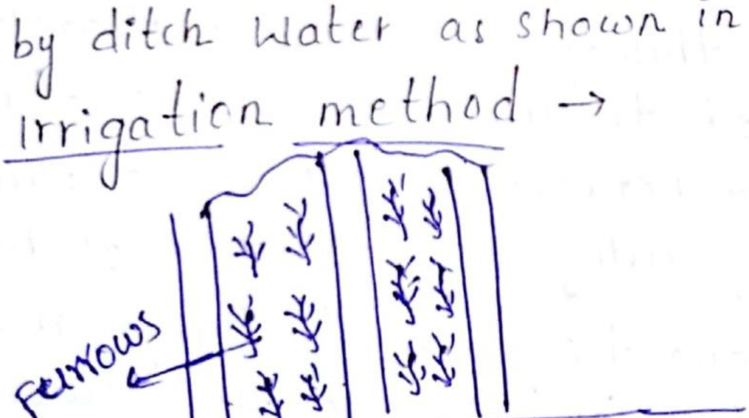
this method is suitable for more permeable soils as well as for less permeable soils. the water can be quickly spread in case of high permeable soils, thus reducing the percolation losses.

Basin flooding →



this method is a special type of check flooding and is adopted specially for orchard trees. One or more trees are generally places in the basin, and the surface is flooded as in check method. by ditch water as shown in above figure.

Furrow irrigation method →



In flooding methods, described above, water covers the entire surface; while in furrow irrigation method, only one fifth to one half of the land surface is wetted by water. It results less evaporation, less pudding of soil and permits cultivation sooner after irrigation.

Furrows are narrow field ditches, excavated b/w rows of plants and carry irrigation water through them.

spacing of furrows is determined by the proper spacing of the plants. Furrows vary from 8 to 30cm deep, and may be as much as 400m long.

Excessive long furrows may result in too much percolation near the upper end, and too little water near the down slope end. Deep furrows are widley used for row crops. Small shallow furrows called corrugations, are particularly suitable for relatively irregular topography and close growing crops, such as meadows and small grains.

Sprinkler irrigation →

In this farm-water application method water is applied to the soil in the form of a spray through a network of pipes and pumps.

Conditions favouring the adoption of this method

→ When the land topography is irregular, and hence unsuitable for surface irrigation.

→ When the land gradient is steeper, and soil is easily erodible.

→ When the land soil is excessively permeable, so as not to permit good water distribution by surface irrigation; or when the soil is highly impermeable.

→ When the watertable is high.
→ When the area is such that the seasonal water
requirement is low, such as near the coasts.
→ When the crops to be grown are such
(a) as to require humidity control, as in tobacco;
(b) Crops having shallow roots; or
(c) crops requiring high and frequent irrigation.
→ When the water is available with difficulty
and is scarce.

Types of sprinkler systems. A sprinkler system
can be classified under three heads, as

1. permanent system
2. semi-permanent system; and
3. portable system.

Advantages →

(i) Seepage losses, which occur in earthen channels
of surface irrigation methods, are completley
eliminated. Moreover, only optimum quantity of
water is used in this method.

(ii) land levelling is not required, and thus
avoiding removal of top fertile soil, as happens
in other surface irrigation methods.

(iii) No cultivation area is lost for making ditches
happens in surface irrigation methods. It, thus
results in increasing about 16% of the cropped area.

(iv) In sprinkler system, the water is to be applied at
a rate lesser than the infiltration capacity of the
soil, and thus avoiding surface run off, and its
bad effects such as loss of water, washing of top soils etc

(v) Fertilisers can be uniformly applied, because they are mixed with irrigation water itself.

(vi) This method leaches down salts and prevents Water-logging (or) Salinity.

(vii) It is less labour oriented, and hence useful where labour is costly and scarce.

(viii) upto 80% efficiency can be achieved i.e upto 80% of applied water can be stored in the root zone of plants.

The limitations of sprinkler irrigation are also enumerated below →

(i) High winds may distort sprinkler pattern, causing non-uniform spreading of water on the crops.

(ii) In areas of high temparature and high wind velocity, considerable evaporation losses of water may take place.

(iii) They are not suited to crops requiring frequent and larger depths of irrigation, such as paddy.

(iv) Initial cost of the system is very high, and the System requires a high technical skill.

(v) Only sand and silt free water can be used, as otherwise pump impellers lifting such waters will get damaged.

(vi) It requires larger electrical power.

(vii) Heavy soil with poor intake cann't be irrigated efficiently.

(viii) A Constant water supply is needed for commercial use of equipment.

Drip irrigation method →

Drip irrigation also called tricle irrigation, is the latest field Irrigation technique, and is meant for adoption at places where there exists acute sacricity of irrigation water and other salt problems. In this method water is slowly and directly applied to the root zone of the plants, there by minimising the losses by evaporation and percolation.

Soil - water - plant relationship →

soil will have eaither single phase system, 2-phase system [i.e in case of super saturated system (S+L), In case of Dry moist soil (S+A) or 3-phase system (i.e S+L+A).

Soil → 1-P → solids
2-P → S+L (super saturated state)
less plant growth.
→ S+A (Dry moist soil)
No plant growth.

Any two phase system is not advisible becaue the first one will be lack of Oxygen, it hinders the growth of the plant. The second one will be lack of water in which there is no plant growth.

The voids in a soil sample are 2 types
1. Capillary pores
2. Non capillary pores.

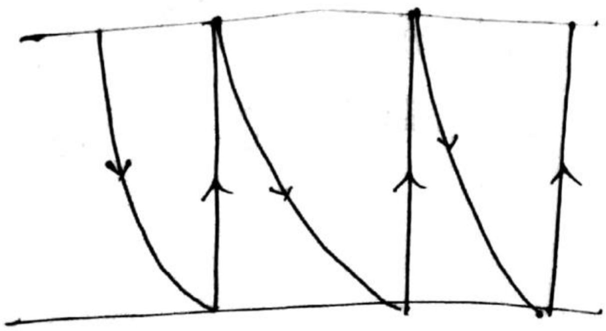
Saturation capacity →
it is the moisture content of the soil where both capillary & Non capillary pores are filled with water. This state is not advisible

Field capacity →
it is the moisture content of the soil, any capillary force are filled with water & Non capillary force are filled with air. it is the best state definable capacity

Optimum moisture content →
it is the moisture content of the soil at which irrigation water must be provided in the safest way as an irrigation engineering you must ensure that moisture capacity is equal to optimum moisture capacity to release the moisture content.

The relation b/w FC & OMC will be shown in below figure

Field capacity



Temporary Wilting point →

it is the first dangeorous signal state of MC of the soil, indicate that plants not able to extract soil water easily. They can extract the water at cooler parts of the day [evenings & Nights].

Permenant Wilting point →

it is the final state of moisture content where plant needs essentially irrigation water for its survival. it is the ultimate dangeoulouy signal.

ultimate wilting point →

in this state the plant is no mole.

Based on the moisture content the irrigation water 3 types.

1. ~~Ground~~ Gravity water → stored b/w saturation capacity and field capacity.

{ This water should not be available in the soil for the plants saftey.

2. Capillary water → Most useful water for plant growth.

3. Hygroscopic water → it is the water stored in the soil b/w PWP - UWP. this water is not at all usefull for the plant growth, it is hidden and entrapped in the soil & can't be taken out by applying a pressure

force of any magnitude.

Frequency of Irrigation → $f = \frac{dw}{cu}$ $\frac{cm}{cm/day}$

f = days.

Consumptive use factor (c.u) →

⇒ Evaporation + Transpiration + ΔQ metabolic + ΔQ photosynthetic.

∴ $Q_{\text{Released to canals}}$ − Canal field losses = $Q_{\text{released to fields}}$.

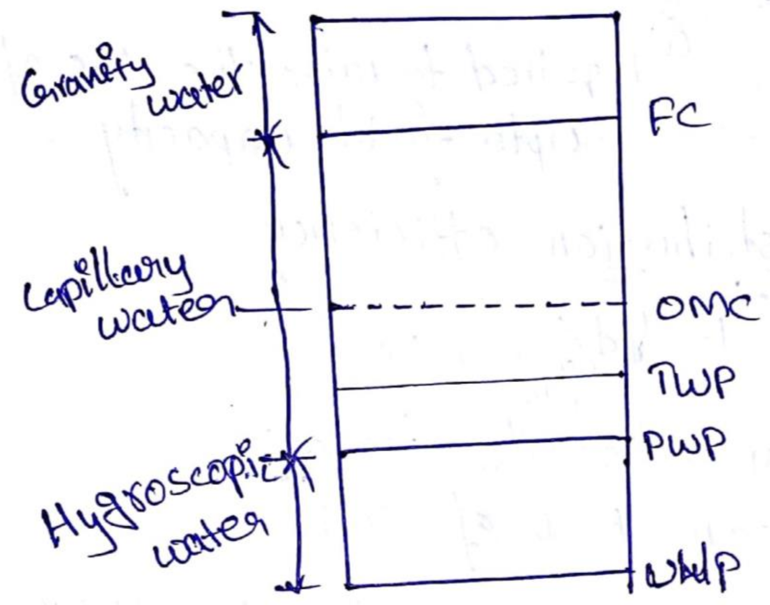
∴ $Q_{\text{Released to fields}}$ − field losses =

$Q_{\text{plant root zone depth of soil}}$ + Q_{Leaching}.

∴ $Q_{\text{Plant root zone depth of soil}}$ ⇒ $\underbrace{E + T + \Delta Q_{\text{metabolism}} + \Delta Q_{\text{Photosynthetic}}}$

{ i.e nothing but consumptive use factor (c.u) }.

Figure → Soil-moisture-plant relationship



Types of irrigation efficiency →

① Water conveyance efficiency

$$\eta_c = \frac{Q \text{ released to the fields}}{Q \text{ released to the canal}}$$

② Water application efficiency

$$\eta_a = \frac{Q \text{ stored in the root zone depth of soil}}{Q \text{ released to the fields}}$$

③ Water use efficieancy

$$\eta_u = \frac{Q_{\text{stored in RZD of soil}} + Q_{\text{leaching}}}{Q \text{ released to the field}}$$

(4) **Water storage efficiency**

$$\eta_s = \frac{Q_{RZD}}{Q_{\text{required to raise the M.C of the soil upto field capacity}}}$$

(5) **Water distribution efficiency**

$$\eta_d = \left(1 - \frac{y_d}{y_m}\right) 100$$

y_d = mean absolute deviation.
y_m - mean RZD of soil.

Note → For homogeneous soils $\eta_d = 100\%$. With increase in hetroginity η_d goes on decreasing.

[Or]

Irrigation efficiencies →

Efficiency is the ratio of water output to the water input and is usually expressed as percentage. Input minus output is nothing but losses and hence if losses are more output is less and, therefore, efficiency is less. Hence efficiency is inversly proportional to the losses. Water is lost in irrigation during various processes & therefore, there are different kinds of irrigation efficiencies as given below

(i) **Efficiency of water-conveyance** →

It is the ratio of the water delivered into the fields from the outlet point of the channel, to the water entering into the channel at its starting point. It may be represented by η_c. It takes the conveyance (or) transit losses into consideration.

(ii) Efficiency of Water-application →

It is the ratio of the quantity of water stored into the root zone of the crops to the quantity of water actually delivered into the field. It may be represented by η_a. It may also be called on farm efficiency as it takes into consideration the water lost in the farm.

(iii) Efficiency of Water-storage →

It is the ratio of the water stored in the root zone during irrigation to the water needed in the root zone prior to irrigation [i.e field capacity - existing moisture content]. It may be represented by η_s.

(iv) Efficiency of Water use →

It is the ratio of the water beneficially used, including leaching water, to the quantity of water delivered. It may be represented by η_u.

Water Requirements of Crops

Every crop requires a certain quantity of water after a certain fixed interval, throughout its period of growth. The term water requirement of a crop means the total quantity and the way in which a crop requires water from the time it is sown to the time it is harvested.

Duty and Delta of a Crop →

Delta → Each crop requires a certain amount of water after a certain fixed interval of time, throughout its period of growth. The depth of water required every time, generally varies from 5-10 cm depending upon the type of the crop, climate & soil.

The time interval b/w two such consecutive waterings is called the frequency of irrigation (or) rotation period.

The rotation period may vary b/w 6–15 days for different crops. The total quantity of water required by the crop for its full growth [maturity] may be expressed in hectare-metre (or) in million cubic metres (million-cubic-ft) or simply as depth to which water would stand on the irrigated area, if the total quantity supplied were to stand above the surface without percolation (or) evaporation. This total depth of water (in cm) required by a crop to come to maturity is called its delta (Δ).

problem → If rice requires about 10 cm depth of water at an average interval of about 10 days, and the crop period for rice is 120 days, find out the delta for rice.

$$\Delta = 12 \times 10\,cm = 120\,cm.$$

10d = 10 cm
120 = ?

Δ for rice = 120 cm.

Delta for certain crops →

Delta on field which includes the evaporation and percolation losses.

1.	Sugarcane	120 cm
2.	Rice	120 cm
3.	Tobacco	75 cm

4. Garden fruits — 60cm
5. cotton — 50cm
6. vegetables — 45cm
7. Wheat — 40cm
8. Barley — 30cm
9. Maize — 25cm
10. Fodder — 22.5 cm
11. peas — 15cm.

Duty of Water →

The duty of water is the relationship b/w the volume of water and the area of the crop it mature. it may be defined as the number of hectares of land irrigated for full growth of a given crop by supply of 1 m^3/sec of water continuously during the entire base period (B) of that crop.

Relation between duty and delta → let there be a crop of base period 'B' days. let one cumec of water be applied to this crop during B days.

Now the volume of water applied to this crop during B days.

$$V = (1 \times \underset{sec}{60} \times \underset{min}{60} \times \underset{hours}{24} \times \underset{days}{B})\ m^3$$

$$= 86400 \text{ (cubic metre)}$$

By definition of duty(D), one cubic metre supplied for B days matures D hectares of land.

∴ This quantity of water (V) matures D hectares of land for $10^4 D$ sq.m of area. [∵ 1 hectare = 10,000 m^2]

Total depth of water applied on this land

$$= \frac{\text{Volume}}{\text{Area}} = \frac{86{,}400B}{10^4 D} = \frac{8.64B}{D} \text{ metres}$$

By definition, this total depth of water is called delta (Δ).

$$\Delta = \frac{8.64B}{D} \text{ metres}$$

$$\Delta = \frac{864B}{D} \text{ cm.}$$

where Δ is in cm,
B is in days; and
D is duty in hectares/cumec.

Problem →
Find the delta for a crop when its duty is 864 hectares/cumec on the field, the base period of this crop is 120days.

$$\Delta \text{ (cm)} = \frac{864B}{D}$$ where B is in days and D is in hectares/cumec.

B = 120 days and D = 864 hectares/cumec

$$\Delta = \frac{864 \times 120}{864} = 120 \text{cm.}$$

Note →
* In direct irrigation, duty is always expressed in hectares/cumec. it is then called as <u>flow duty (or) duty</u>.
* When duty is expressed in hectares/million cubic metre of water available in the reservoir is called <u>Quantity duty (or) storage duty</u>.

Duty for certain crops

Sugarcane	730 hectares/cumec.
Rice	775 "
other kharif	1500 "
Rabi	1800 "
perennials	1100
Hot fodder	2000 "

Factors on which duty depends →

Duty of Irrigation water depends upon the following factor.

(i) Type of crop → Different crops require different amount of water, and hence the duties for them are different. A crop requiring more water will have less flourishing acreage for the same supply of water as compared to that requiring less water. hence duty will be less for a crop requiring more water and viceversa.

(ii) Climate and season → As stated earlier, duty includes the water lost in evaporation and percolation. these losses will vary with the season. hence duty varies from season to season, and also from time to time in the same season.

(iii) Usefull rainfall → If some of the rain falling directly over the irrigated land, is useful for the growth of the crop, then so much less Irrigation water will be required to mature. More the usefull rainfall, less will be the requirement of Irrigation water, and hence more will be the duty of Irrigation water.

(iv) Type of soil → If the permeability of the soil under the irrigated crop is high, the water lost due to percolation will be more and hence, the duty will be less. therefore, for sandy soils, where the permeability is more, the duty of water is less.

(v) Efficiency of cultivation method →

If the cultivation method (including tillage & irrigation) is faulty and ~~less~~ efficient, resulting in the wastage of water, the duty of water will naturally be less. If the irrigation water is used economically then the duty of water will improve, ~~so~~ the same quantity of water would be able to irrigate more area.

Definitions

Kharif - Rabi ratio (or) Crop ratio →

The area to be irrigated for Rabi crop is generally more than that for the kharif crop. This ratio of proposed areas, to be irrigated in kharif season to that in the Rabi season is called. kharif - Rabi ratio.

This ratio is generally 1:2, i.e kharif area is one-half of the Rabi area.

Paleo - Irrigation →

Some times, in the initial stages before the crop is sown, the land is very dry. This particularly happens at the time of sowing of Rabi crops because of hot september, when the soil may be too dry to be sown easily. In such a case, the soil is moistened with water, so as to help in sowing of the crops. This is known as paleo irrigation.

Kor-Watering →

The first watering which is given by to a crop when the crop is a few centimetres high is called Kor-watering. It is usually the max single watering followed by other waterings at usual intervals, as required by drying of leaves.

for Eg → The optimum depth of Kor-Watering for rice is 19cm,
→ for wheat is about 13.5cm
→ for sugar cane is 16.5cm.

The kore watering must be applied within a fixed limited period called Kor period. The Kor period for rice varies from 2-4 weeks, and that for wheat varies from 3 to 8 weeks

Cash crops →

A cash crop may be defined as a crop which has to be encashed in the market for processing etc. it can't be consumed directly by the cultivators. All non food crops, are thus, included in cash crops. The food crops like wheat, rice, barley, maize etc are excluded from the list of cash crops.

Crop rotation →

When the same crop is grown again and again in the same field, the fertility of land gets reduced as the soil become deficient in plant foods favourable to that particular crop.

In order to enhance the fertility of the land and to make the soil regain its original structure it is often found necessary and helpful to give Some rest to the land.

This can be achieved either by allowing the land to lie fallow without any cultivation for Some time, or to grow crops which do not mainly require those salts or foods which were mainly required by the earlier grown crop.

This method of growing different crops in rotation one after the other in the same field is called Rotation of crops.

A cash crop may be followed by a fodder crop, Which in turn may be followed by soil renovating crop like gram which being a liguminous crop helps in giving nitrogen to the fields thereby renovating the soil.

Ex rotations of crops

(i) Wheat - Juar - Gram

(ii) Rice - Gram

(iii) Cotton - Wheat - Gram - Fallow

(iv) Cotton - Juar - Gram

(v) Sugarcane (12 months) - Thadua - wheat (or) gram.

Water distribution (or) uniformity coefficient efficiency →

The effectivness of irrigation may also be measured by its water distribution efficiency (η_d) which is defined below

$$\eta_d = \left[1 - \frac{d}{D}\right]$$

where η_d = Water distribution efficiency
D = Mean depth of water stored during Irrigation.
d = Average of the absolute value of deviations from the mean.

Consumptive Use (or) Evapotranspiration (C_u) → (E_t) →

Consumptive use for a particular crop may be defined as the total amount of water used by the plants in transpiration and evaporation from adjacent soils, in any specified time.

The value of consumptive use may be different for different crops, and may be different for the same crop at different times and places.

The consumptive use for a given crop at a given place may vary throughout the day, throughout the month, and throughout the crop period.

The transpiration is defined as the process by which the water leaves a living plant during photosynthesis through its leaves, to enter the atmosphere as water vapour. Transpiration occurs when the plant manufactures carbohydrates for its growth by the process of photosynthesis.

On the other hand, the evaporation continues throughout the day & night although its rate may be different.

$$\text{Transpiration ratio (TR)} = \frac{\text{Total mass of water transpired by the plant during its full growth}}{\text{Mass of dry matter produced}}$$

TR has no units

* When sufficient moisture is freely available to Completely meet the needs of the vegetation fully Covering in area the resulting evapotranspiration is called the potential evapo-transpiration.

* The real evapo-transpiration occuring in a specific situation in the field however, is called the actual evapo transpiration. AET can be measured by lysimeter.

Effective rainfall → (Re)

precipitation falling during the growing period of a crop that is available to meet the evapo-transpiration needs of the crop, is called effective rainfall.

Consumptive Irrigation Requirement (CIR) →

It is the amount of irrigation water required in order to meet the evapotranspiration needs of the crop during its full growth. It is therefore nothing but the consumptive use itself, but ~~excessive~~ exclusive of effective precipitation, stored Soil moisture, (or) ground water. When the last two are ignored, then we can write

$$C.I.R = C_u - R_e.$$

Net Irrigation requirement → (NIR)

It is the amount of irrigation water required In order to meet the evapotranspiration need of the crop as well as other needs such as leaching.

$$NIR = C_u - R_e + \text{water lost as percolation}$$

Factors effecting consumptive use →

Consumptive use (or) evapotranspiration depends upon all those factors on which evaporation and transpiration depend; such as temp, sunlight, humidity, wind movement, etc.

Estimation of consumptive use →

1. Blaney-criddle Equation, and
2. Hargreaves class A pan evaporation method.
3. penman's equation

Blaney-Criddle eq →

$$C_u = \frac{k \cdot p}{40}\left[1.8t + 32\right]$$

If $\frac{p}{40}\left[1.8t+32\right]$ is reprented by f,

$$C_u = k \cdot f$$

Hargreaves class A pan evaporation method →

$$K = \frac{\text{Evapotranspiration } (E_t \text{ or } C_u)}{\text{Pan evaporation } (E_p)} = K$$

$$E_t \text{ (or) } C_u = k \cdot E_p.$$

Penman's equation → While the blaney criddle eq and the Hargreaves class A pan equation using christiansen formula had been in use for the last many yeay for computing the consumptive use values, and net irrigation requirements for different crops.

The penman equation has however more recently been introduced for determining the consumptive use of different areay (or) different segments of a basin ... each

The advantage with this eq, lies in the fact that the different specified value of coefficient of reflection (albedo), a factor used in this eq, are available for different types of areas, which can be used in penman's eq.

$$A = \frac{e_s' - e_s}{T_a' - T_a}$$

E_t (Daily potential evapo-transpiration)

$$= \frac{A \cdot H_n + E_a \cdot \gamma}{A \cdot \gamma}$$

where γ = psychromatic constant
= 0.49 mm of Hg/°C.

The parameter E_a of penman's eq is estimated as

$$E_a = 0.35\left[1 + \frac{V_2}{160}\right]$$

where V_2 = mean wind speed at 2m above the ground in km/day.

e_s = saturation vapour pressure at mean air temp in mm of Hg.

e_a = actual mean vapour pressure of air in mm of Hg.

Standards of Quality of irrigation water →

A good irrigation water is the one which performs the above mentioned functions without any side effects which retards the plant growth. irrigation water may be said to be unsatisfactory for its intended use if it contains

1. Chemicals toxic to plants (or) the persons using plant as food.
2. Chemicals which react with the soil to produce unsatisfactory moisture characteristics and
3. bacteria injurious to persons (or) animals eating plants irrigated with the water.

Impurities in irrigation water →

The Quality of irrigation water depends upon various types of impurities present in water, the following being the prominent ones

1. Concentration of sediments in water
2. Total concentration of soluble salts
3. proportion of sodium ions to other cations.
4. Concentration of toxic elements such as boron Concentration.
5. concentration of bicarbonate in relation to the Concentration of calcium and magnesium.
6. Bacterial concentration.

Water logging and drainage →

An agricultural land is said to be waterlogged When its productivity (or) fertility is affected by high water-table. The depth of water-table at which it tends to make the soil water-logged and harmful to the growth and subsistence of plant life depends upon the height of capillary fringe, which is the height to which water will ~~be~~ rise due to capillary action.

The normal height of the capillary fringe met with in agricultural soil varies from 0.50 to 1.60m. The depth of water table which adversely affects the growth of different crops is given below

Crops	Depth of Water table
1. Wheat	0.9 – 1.2 m
2. Cotton	1.5 – 1.8 m
3. Rice	0.6 m
4. Sugarcane	0.3 m
5. Fooder crop	1.2 m
6. Luceine	2.1 to 2.4 m.

Note → Effects, causes & Remedial measures of water logging ⇒ please Refer textbook B.C. punmia pageno – 747-750.

UNIT - II
CANALS

Syllabus ⇒ classification, design of non-erodible canals, methods of economic section and maximum permissible velocity, economics of canal lining, design of erodible canals - kennedy's silt theory & lacey's regime theory, balancing depth of cutting.

Classification →

A canal is an artificial channel, generally trapezoidal in shape constructed on the ground to carry water to the fields either from the river (or) from a tank (or) Reservoir.

Canals can be classified in following ways

(a) classification based on the nature of source of supply

(1) permanent canal (2) Inundation canal.

* A canal is said to be permanent when it is fed by a permenent source of supply.

* Inundation canals usually draw their supplies from rivers whenever there is a highstage in the river.

(b) classification based on the function of the canal

(1) Irrigation canal (2) carrier canal (3) feeder canal

(4) Navigation canal (5) power canal

→ Irrigation canal carries water to the agricultural fields.

→ A carrier canal besides doing irrigation, carries water for another canal.

→ A feeder canal is constructed with the idea of feeding two (or) more canals.

(C) classification based on financial output →

(1) productive canal. (2) protective canal

→ productive canals are those which yield a net revenue to the nation after full development of irrigation in the area.

→ protective canal is a sort of relief work constructed with the idea of protecting a particular area from famine.

f) Classification based on boundary surface of the canal →

Based on the type of boundary surface, canals may be of the following type

(1) Alluvial canal (2) Non-Alluvial Canal

(3) Rigid boundary canal. [lined canals].

e) Classification based on the discharge and its relative importance in a given network of canals →

1. Main canal 2. Branch canal

3. Major distributory 4. Minor distributory

5. Water course.

→ Main canal generally carries water directly from the river (or) Reservoir. such a canal carries heavy supplies and is not used for direct irrigation. these are act as water carriers to feed supplies to branch canals and Major distributaries.

→ Branch canals are the branches of the main canal in either direction taking off at regular intervals. these canals are usually feeder channels for major & minor distributaries. They usually carry a discharge of 5 cumecs

→ Major distributaries usually called Rajbha. takeoff from branch canal. They are real irrigation channels their dischage varies from 1/4 to 5 cumecs.

→ Minor distributaries (or) Minor take off from branch canals (or) from distributaries. Their dischage usually less than 1/4 cumecs.

→ A water couse (or) field channel is a small channe which ultimately feeds the water to irrigation fields.

(f) Classification based on canal alignment →

According to the alignement, a canal may be classified as (1) contour canal (2) watershed canal (3) side slope canal.

→ Ridge canal (or) watershed canal is aligned along a watershed and runs for most of its length on a watershed.

→ Contour canal aligned nearly parallel the contours of the area

→ Side slope canal is a channel aligned roughly at right angles to the contours of the country and is neither on the watershed nor in the valley.

Design of Non-erodible (or) Non-Alluvial channe

Non-alluvial channels are those which flow through non-alluvial soils such as loam, clay, moora and other hard soils, including boulders and rocks.

In the non alluvial soils, water is relatively clear due to which there is no problem of silting these channels are therefore considered as stable channels. Non-alluvial channels are designed on the basis of maximum permissible velocity that could be permitted in the channel without causing any scour either in the bed (or) in the sides.

Values of permissible velocities.

Type of Bed Material	Permissible velocity (m/s).
1. ordinary soil	0.6 - 0.9
2. loam; lean clay	0.4 - 1.0
3. Ordinary clay	0.5 - 1.2
4. Heavy clay	0.6 - 1.6
5. light loose Sand	0.3 - 0.6
6. Hardsoil & Moorum	1.0 - 1.2
7. Gravel	1.2 - 1.5
8 Boulders	1.5 - 1.8
9 Soft Rock	1.8 - 2.4
10 Hard rock	2.5 - 5.

Side slopes → The side slopes (H:V) of channels in ordinary soils, including clay, are generally kept as 1:1 in cutting and 1.5:1 in filling. However in grit (or gravel), soft rock, and hardrock the side slopes are 0.5:1, 0.25:1 & 0.125:1 respectively though the sides may be kept even vertical in very hard rocks.

Flow equations →

The design of non-alluvial channels is done on the basis of the following two flow eq

(i) Chezy's formula (ii) Manning's formula.

Chezy's formula →

$$V = C\sqrt{RS}.$$

Where C is chezy's coefficient usually determined from the following eq by bazin

$$C = \frac{87}{1+K/\sqrt{R}}.$$

Where

K - Bazin's coefficient for earth channels = 1·2 - 1·4 {good condition}

for earth channels = 1·7 - 1·8 {poor condition}

Manning's formula →

$$V = \frac{1}{N} R^{2/3} S^{1/2}.$$

Where

N - Manning's coefficient.

Values of N

Type of surface	Value of N
1. Earth channel; clean straight & uniform	0·016 - 0·020.
2. Earth channel clean but weathered	0·018 - 0·025

Design procedure

for a channel design, the dischauge Q and bed slope S are generally known. The value of max permissible velocity and side slopes (r:1) are choosen on the basis of type of soil through which the channel has to flow.

Step 1 : Determine the Area of C/s from the continuity eq,

$$A = Q/V \Rightarrow \{ \because \text{from } Q = AV \}.$$

Step 2 : Determine hydraulic mean radius from Mannings formula

$$R = \left[\frac{VN}{S^{1/2}} \right]^{3/2}.$$

Step 3 : Determine wetted perimeter (P) from the Relation

$$P = A/R.$$

Step 4 : for a trapezoidal channel with sideslopey (r:1), the area of cross-section (A) & wetted perimeter (P) are given by

$$A = (B + rD) D$$

$$P = B + 2\left(D\sqrt{1+r^2}\right).$$

Note → for the most efficient channel trapezoidal section having side slopes of $1/\sqrt{3} : 1$ & $R = D/2$.
$H : V$

Lining of Irrigation channels →

Necessity → lining of canal is necessary for
(i) to minimise the seepage losses in canal
(ii) to increase the discharge in canal section by increasing the velocity.
(iii) to prevent erosion of bed and side due to high velocities.
(iv) to retard the growth of weeds, and
(v) to reduce Maintanance of canal.

Advantages of lining →

→ The lining of canals prevents seepage loss and thus more area can be irrigated by the water so saved. The cost of irrigation is therefore reduced.
→ The lining of canal is an important anti-water logging measure as it reduces seepage to the adjoining land.
→ The lining provides smooth surfaces. The rugosity coefficient, therefore decreases.
→ The increased velocity minimises the losses due to evaporation
→ The increased velocity helps to provide a narrow c/s for lined channels.
→ Higher velocity prevents silting of channel.
→ lining makes the banks more stable in light textured soil
→ lining prevents (or) reduces weed growth.
→ canal lining assures economical water distribution.

Types of lining →
1. cement concrete lining
2. shotcrete lining
3. precast concrete lining
4. Cement mortar lining
5. Brick lining
6. stone blocks, (or) undressed ston lining
7. Asphaltic lining.
8. soil cement lining.
9. clay puddle lining
10. Road oil lining.

Economics of canal lining →

At times, the choice of canal lining is to be done on financial considerations besides the technical feasibility. For determining its economic viability, an analysis of benefits from canal lining has to be worked out in terms of money. The benefits from lining should be greater than extra cost to be incurred on lining.

Calculation of benefit →

The major benefit then can be readily assessed in terms of money is from the saving of seepage water which would have been lost from unlined channel. this water when supplied to farmers will yield revenue.

let q cumecs be the water saved in the lined reach and R_1 rupees be the cost of water per cumec. then the total saving $= q \cdot R_1$ rupees.

let p be the percentage saving in annual maintanence cost which is rupees R_2 for unlined channel. then the saving in annual maintanance cost $= pR_2$ rupees.

total benefit (B) per year $= (qR_1 + pR_2)$ rupees.

Annual cost of extra expenditure on lining →

Annual cost of extra expenditure on lining ⇒

$$C \times i(i+1)^N \left[(1+i)^N - 1\right]^{-1}.$$

∴ Benefit cost ratio (B.C.R) is

$$BCR = \frac{(qR_1 + pR_2)\left[(1+i)^N - 1\right]}{C \times i\,(1+i)^N}$$

$$= \frac{B\left[(1+i)^N - 1\right]}{C\,i\,(1+i)^N}$$

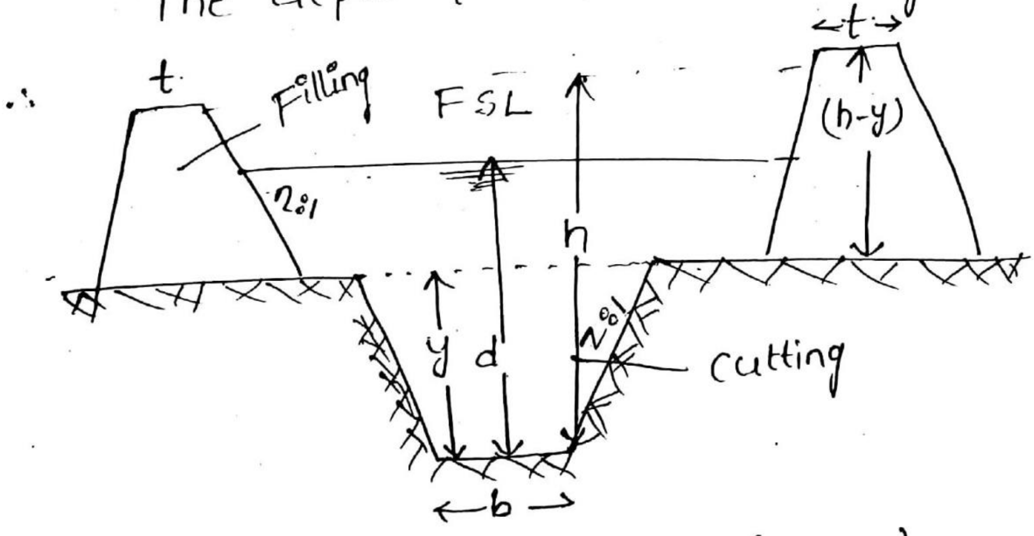
If rate of interest, is 8% per annum $i = 0.08$.

Balancing depth of cutting & filling →

A canal section will be economical when the earth work involved at a particular section has an equal amount of cut & fill. Usually a canal section has a part in cutting and part in filling as shown in fig. If the amount of cut is equal to the amount of fill, it has to be paid for once only.

The necessity of a borrow pit (or) soil bank is entirely avoided. For a given cross-section there is always only one depth of cutting for which the cutting & filling will be equal.

The depth is known as balancing depth.



∴ Area of the cut $= y(b+zy)$

$\simeq by + zy^2$

Area of filling $= 2\left[(h-y)t + n(h-y)^2\right]$

*Note → A canal is usually constructed with a side slope of 1:1 in cutting & a slope 1.5:1 in filling.

Design of Erodible (or) Alluvial canals by using Kennedy's & lacey's design principle →

The channel (or) canal which takes off from a river has to draw a fair share of silt flowing in the river. this silt is carried either in suspension or along the bed of the channel.

The silt load carried by the channel imposes a difficult problem in a channel design. in alluvial soils. If the sides and bed of a channel are eroded away, the c/s increases and besides other damages because of scour, its full supply depth decreases; it can therefore command much less area. A velocity which will just keep the silt in suspension without scouring the channel is known as non-silting & non-scouring velocity.

Many investigations have worked on various existing channels towards the design of Non-silting, Non-scouring channel section.

Notable amongst them are Mr R.G. kennedy & Mr. Gerald lacey as Kennedy's theory and Lacey's theory respectively.

For the design of an irrigation channel, the design discharge Q, and the surface and soil properties such as rugosity coefficient N and silt factor f are known. The problem consists in determination of the four unknowns.

(i) Area of c/s (A)

(ii). Hydraulic mean depth (R).

(iii) Velocity of flow (V)

(iv) Bed slope (S)

Kennedy's theory →

* Kennedy selected a number of sites on upper Bari doab canal system, one of the oldest in punjab for carrying out investigations about velocity & depth of the channel.

* The following water has to counteract some amoun of friction against the bed of the canal. This gives rise to vertical eddies rising up gently to the surface.

* These eddies are responsible for keeping most of the silt in suspension.

* He also gave a relation b/w critical velocity to the depth of flowing water.

$$\boxed{V_c \text{ (or) } V_0 = 0.55\, m\, d^{0.64}} \approx \boxed{V_c = CD^n}$$

m = critical velocity ratio
= 1.1 to 1.2 for coarse sand,
= 0.8-0.9 for fine sand,

$$\therefore m = CVR = \frac{\text{critical velocity for the area}}{\text{Critical velocity for upper bari doab canal system}}$$

$$C = \frac{\left[\frac{1}{n} + \left(23 + \frac{0.00155}{S}\right)\right]}{\left[1 + \left(23 + \frac{0.00155}{S}\right) \frac{n}{\sqrt{R}}\right]}$$

$$V = C\sqrt{RS}.$$

Value of C.

Types of material	'C'
light sandy silt	0.53
coalser light silt	0.59
Sandy loam	0.65
Coalse silt	0.70.

Value of m

Type of silt	Value of m
light sandy silt in the rivers of Northen India	1.00
Somewhat coalser light sandy silt	1.10
Sandy, loamy silt	1.20
Rather coalser silt (or) debris of hard soil	1.30
silt of river Indus in sind	0.70

Fixation of longitudinal slope (S) of channel →

Slope	B(m)	D(m)	B/D.
1 in 5000	7	0.68	10.8
1 in 4000	3.2	0.85	3.8
1 in 2000	1.5	1.4	1.07.

Fixation of B/D ratio ⇒

(a) Woods table

(b) Emprical formula & table for B/D ratio

for channels having $Q < 15$ cumecs

$$D = 0.5\sqrt{B}.$$

for $Q \geq 15$ cumecs follow the table

Value of Water depth (D)

Discharge	Depth D (m)
15	1.7
30	1.8
75	2.3
150	2.6
300	3.0

(c) Recommendation of CWPC for B/D ratio →

$$B/D = r = [15 + 6.44 (Q)]^{0.382}$$

for Eg Q = 14 cumecs

$$B/D = [15 + 6.44 \times 14]^{0.382} \simeq 5.9 .$$

Drawbacks in Kennedy's theory →

1. Kennedy did not notice the importance of B/D ratio.
2. He aimed to find out only the average regime conditions for the design of a channel.
3. No account was taken of silt concentration and bed load, and the complex silt carrying phenomenon was incorporated in a single factor m.
4. silt grade and silt charge were not defined.
5. Kennedy did not give any slope equation.
6. Kennedy used Kutter's equation for the determination of the mean velocity and therefore, the limitations of Kutter's eq, got incorporated in Kennedy's theory of channel design.

Lacey's Regime theory →

<u>Regime channel</u> → lacey defined regime channel as a stable channel transporting a regime silt charge. A channel will be in regime if it flows in unlimited incoherent alluvium of the same character as that transported and the silt grade & silt charge are constant.

Incoherent alluvium → it is a soil composed of loose granular graded material which can be scoured with the same rate with which it is deposited.

Regime silt charge → it is the min transported load consistant with fully active bed.

Regime silt grade → this indicates the gradation b/w the small and the big particles. It should not be taken to mean the average mean diameter of a particle.

Regime conditions → A channel is said to be in regime when the following conditions are satisfied.

1. The channel is flowing in unlimited incoherent alluvium of the same character as that transported.
2. silt grade and silt charge are constant.
3. Discharge is constant.

nitial regime → Initial regime is the state of channel that has formed its section only and yet not secured the longitudinal slope.

inal regime → To attain the final regime the channel forms its section first before the final slope. The channel after attaining its section and longitudinal slope will be said to be in final regime.

ermanent regime → When a channel is protected on the bed and side with some kind of protecting material, the channel section cann't be scoured up & so there is no possibility of change of section (or) longitudinal slope.

∴ silt factor $(f) = 1.76\sqrt{m}$

m = mean particle size, (mm).

$\rightarrow \quad V = \left[\frac{QF^2}{140}\right]^{1/6}$ (velocity).

$\rightarrow \quad P = 4.75\sqrt{Q}$. (Wetted perimeter)

$\rightarrow \quad R = 5/2\left[V^2/f\right]$ (hydraulic radius)

$\rightarrow \quad S = \frac{f^{5/3}}{3340 \cdot Q^{1/6}}$ (slope)

Values of f.

Type of soil	value of f
1. fine silt	0.50 – 0.70
2. Medium silt	0.85
3. standard silt	1.00
4. medium sand	1.25
5. Coalse sand	1.50.

Defects in lacey's theory →

1. The theory doesn't give a clear description of physical aspects of the problem.
2. It does not define what actually governs the characteristics of an alluvial channel
3. lacey's eq do not include a concentration of silt as Variad
4. lacey did not properly define the silt grade and silt charge.
5. lacey introduced semi ellipse as ideal shape of a regime channel which is not correct.

Comparison of Kennedy's and Lacey's theory –

Kennedy's theory	Lacey's theory.
→ It states that the silt carried by the following water is kept in suspension by the vertical component of eddies which are generated from the bed of the channel	→ It states that the silt carried by the following water is kept in suspension by the vertical component of eddies which are generated from the entire wetted perimeter of the channel.
→ Relation b/w V & D	→ Relation b/w V & R.
→ critical velocity ratio 'm' is introduced to make the eq applicable to diff channels with diff. silt grades.	→ Silt factor 'f' is introduced to make the eq applicable to diff channels with diff silt grades.
→ Kutters eq is used for finding the mean velocity.	→ this theory given an eq for finding the mean velocity.
→ this theory gives no eq for bed slope.	→ this theory gives an eq for bed slope
→ In this theory, the design is based on trial & error method.	→ this theory does not involve trial and error method.

Unit-2 (Part 2)

Canal structures

1. Falls :-

Necessity and location of falls :-

A fall is an irrigation structure constructed across a canal to lower down its water level and destroys the ~~supply~~ surplus energy liberated from the falling water which may otherwise scour bed & and banks of the canal.

① For the canal which does not irrigate the area directly, the fall should be located from the considerations of economy in cost of excavation of the channel with regard to balancing depth and cost of the falls itself.

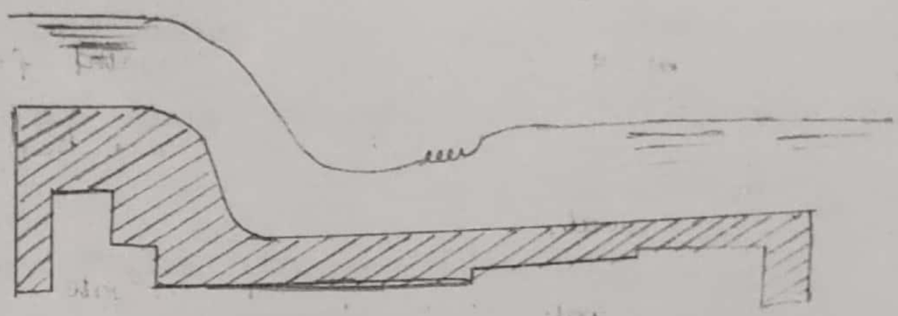
② For a canal irrigating the area directly, a fall may be provided at a location where the F.S.L outstrips the ground level, but ~~at~~ before the bed of the canal comes into falling.

③ The location of the fall may ~~be~~ also be decided from the consideration of the possibility of combining it with a regulator or a bridge or any other masonry works.

④ A relative economy of providing large number of small falls v/s small number of big falls should be worked out. The provision of small number of big falls results in unbalanced earthwork, but there is always some saving in the cost of the fall structure

Types of falls —

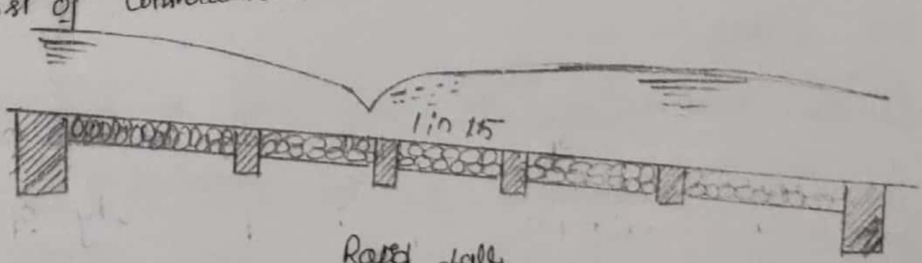
1. **Ogee fall** : — The ogee fall was first constructed by Sir Proby Cautley on the Ganga Canal. This type of fall has gradual convex and concave curves with an aim to provide a smooth transition and to reduce disturbance and impact.



OGEE FALL

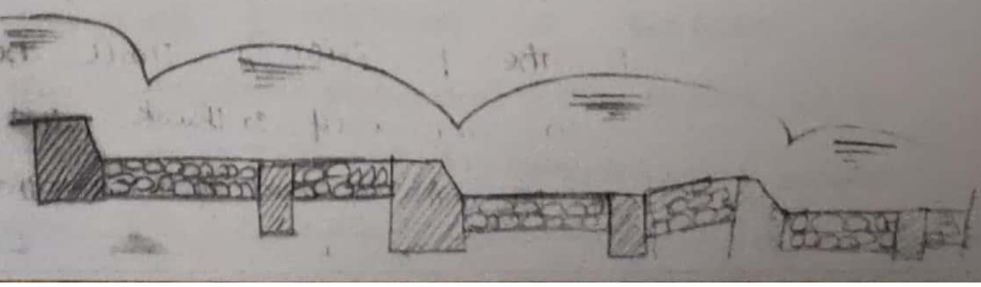
2. **Rapid fall** :—

Such falls were provided on western Yamuna Canal and were designed by Lieut R.L Crofton. Such a fall consists of glacial sloping at 1 vertical to 10 to 20 horizontal. Hence a fall worked admirably, However there was very high cost of construction.



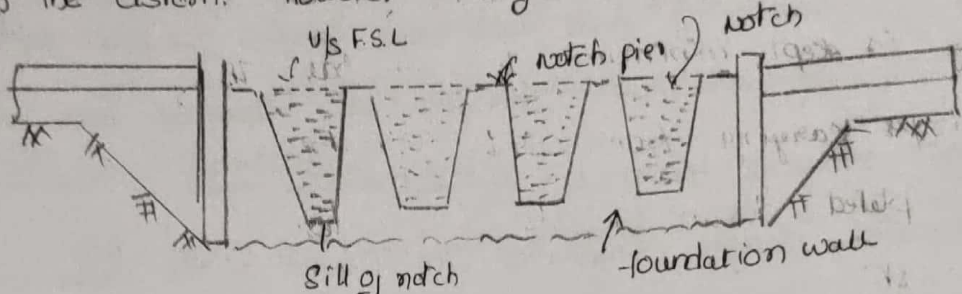
Rapid fall

3. **Stepped fall** :- Stepped falls was a next development of the rapid fall one such type was provided at the tail of main canal escape of Sarda Canal.



4. Notch fall :—

Soon after the development of stepped fall, the efficiency of vertical impact on the floor for energy dissipation came to be recognised. The vertical fall came in the field along with the cistern. However with greater discharges.



5. Vertical Drop fall :—

In the vertical drop fall, the nappe impinges clear into the water cushion below. The dimensions of cistern were put in arbitarily in light of experiance of the disigners. Another device in the form of grid was usually used in the cistern intercepting the dropping Jet of water

6. Glacis Type fall :—

The efficiency of the hydraulic Jump as a potent means of distroying the energy of canal falls was brought out clearly by the research work of the many Conservency. The glacis fall may be in straight glacis Type or parabolic glasic Type Commonly known as Montague Type. The straight glacis fall may be with baffle platform and baffle wall. formation of Jump Takes place from baffle platform.

Meter and Nonmeter falls :—

Meter falls are those which also measure the discharge of the Canal. The non metre falls do not measure discharge. For a fall to act as a metre, it must have a broad weir Type crest so that discharge Coefficient is Constant under Variable head.

Design of SARDA Type fall :-

This Type of fall was designed and developed for Sarda Canal System of U.P. In that area, thin veneer of sandy clay overlies a stratum of pure sand. Hence, the main requirement was to provide a number of falls, with small drops. So that depth of cutting is kept minimum. This fall has, therefore been Constructed for drops varying from 0.9 to 1.8 metres.

The Completed design Consists of the design of the following Parts.

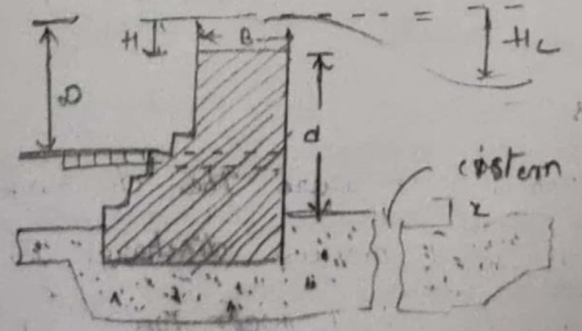
1. Crest
2. cistern
3. Impervious floor
4. d/s protection
5. u/s approach.

1. Design of Crest

(i) Length of crest : The length of the crest is kept equal to bed width of the Canal. and no flumming is done in this Type of fall. Hence the length of the crest is kept equal to bedwidth of the Canal. plus the water depth to take into account the anticipated increase in discharge at a future date

(ii) Shape of the crest and discharge formula

Two Types of crests are used. the rectangular crest is used for discharge upto 14 cumecs (500 cusecs) and Trapezoidal crest is used for discharge over 14 cumes



Rectangular crest

For the rectangular crest :-

The width of the crest is given by $B = 0.55\sqrt{d}$ metre

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

for masonry crest P may be Taken equal to 2

Discharge is given by $Q = 1.835 LH^{3/2} (\frac{H}{B})^{1/6}$ → (1)

Q = discharge in cumes   L = length of crest in metres

For a Trapezoidal crest :-

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 width is determined by the batter

$Q = 1.99 LH^{3/2} (H/B)^{1/6}$ → (2)

(ii) Crest level :-

From eq (1) & (2) the value of H is known

RL of crest = U/s F.S.L − H

Height of crest above bed = h = D − H

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

(2) Design of Cistern :-

The length and depression of the cistern are given by the following equation

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

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

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 occurs when there is water on the u/s side upto the top of the crest and there is no flow to the d/s side. out of the total impervious floor length, a maximum length (l_d) to be provided to the d/s of the crest is given by the following expression,

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

The thickness of the impervious floor is determined a minimum thickness of 0.3 to 0.4m is provided for the floor to u/s crest.

(4) D/S protection :-

The d/s protection consists (i) bed protection (ii) side protection (iii) d/s wings.

(i) Bed protection :- The bed protection consists of dry bricks pitching about 20cm thick resting on 10cm ballast.

(ii) Side protection :- The side protection consists of one dry brick pitching on edge, is provided after the wrapped wings. the side pitching is wrapped from a slope of 1:1 to 1½:1.

(iii) D/s wings :- The d/s wings are kept vertical for a lengths of 5 to 8 metres times $\sqrt{EH_L}$ from the crest and are then warped or flared to a slope of 1:1 or 1½:1

The wing walls are designed as earth retaining structure. In the absence of elaborate stability calculation the width of wings at any level may be kept equal to ⅓rd the height above bed level

(5) Design of u/s approach :-

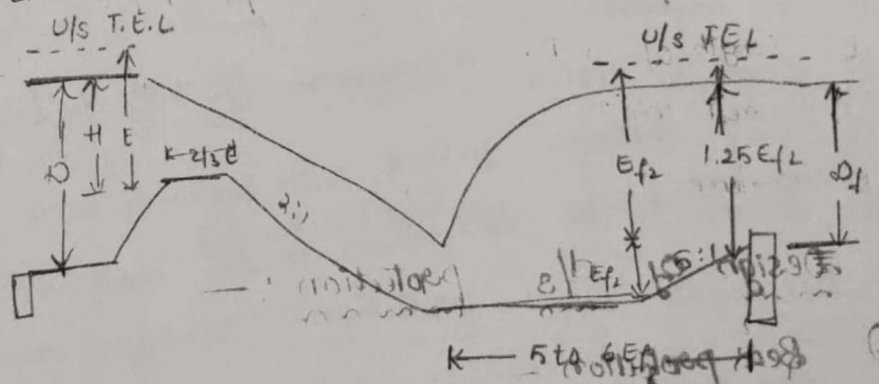
For discharge upto 14 cumes the u/s wings may be splayed, straight at an angle of 45°. For greater discharges the wings are kept segmental with radius equal to 5 to 6 times H, subtending at angle of 60°. at the time of centre, and then are carried straight into the berm.

* Design of Straight Glacis fall

1. Crest Design :-

(a) For a non-meter falls, the u/s glacis is given a slope of 1/2 : 1 and is Joined Tangentially to the crest with a radius of E/2 where E = height of u/s TEL above crest

(b) The d/s glacis is given a slope of 2:1 and in joined Tangentially to the cistern with a radius = E.



Straight glacis fall

(c) The width of the crest is kept equal to 2/3 E.

(d) The glacis fall may sometimes be flumed if it is to be combined with a bridge etc. However, the minimum clear length of the crest.

The value of E calculated from the discharge equation

$$Q = 1.84 \, L_t \, E^{3/2}.$$

L_t = length of the crest
Q = discharge in cumecs

If there are n number of piers, the effective length to be equal to $(L_t - 0.2n)$

$$\text{crest level} = U/S\ T.E.L - E$$
$$U/S\ T.E.L = U/S\ F.S.L + \text{velocity head.}$$

② **Design of cistern:—**

Knowing the discharge intensity (q) and the drop H_L the energy of flow (E_{f_2}) below the hydraulic Jump is known.

Then $R.L\ \text{Cistern} = d/s\ T.E.L - 1.25\ E_{f_2}$

3. **Design of Impervious floor:—**

The total length of the impervious floor is found from the consideration of permissible exit gradient. The total length of the impervious floor may be provided in the following

① length of cistern ② Horizontal length of d/s glacis
③ crest width ④ " " of u/s glacis
⑤ Balance to be provided to the u/s

④ **Design of d/s protection:—**

ⓐ **Bed protection.—**

Since a deflector wall is provided at u/s end of the floor no bed pitching is provided.

ⓑ **side protection**

brick pitching of 1½ brick thick is done upto a length of $3D_1$. The pitching is supported on a Toe will 0.4m wide and $D_1/2$ deep.

ⓒ curtain wall: Depth of curtain wall $= D/2$ width 0.4m

(5) Design of Energy Dissipators :—

(a) friction blocks :—

Four rows of friction blocks are provided in case of flummed glacial falls. The height of each block is kept equal to $D_1/8$ and length equal to 3 times the height of the block. The distance between the rows is equal to height of the blocks

(b) Glacis blocks :—

The falls more than 2m one row of glacis blocks is of the same dimensions as that of friction blocks. is provided at the d/s end of glacis

(c) Deflector wall :—

Deflector wall of height $D_1/10$ and width 0.4m is provided at the d/s cistern. The same wall may ~~works~~ as curtain wall

6. Design of d/s Expansion :—

In a flummed wall the d/s Expansion starts from the toe of the glacis. The A rectangular hyperbolic expansion is generally preferred

The Bed width B_x at a distance x from the d/s toe of the glacis is given by

$$b_x = \frac{B_1 \times B_2 \times L_e}{(L_e \times B_2) - (B_2 - B_1)x}$$

(7) upstream Approach :—

For a non meter falls the side walls are splayed at an angle of 45° from the u/s edge of the crest.

Head Regulators and Cross Regulators

Head regulator and cross-regulator regulate the supplies of the off-taking channel and the parent channel respectively. The distributory head regulator is provided at the head of the distributory and control the supply of entering the distributory.

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 into the off-taking.
3) They control silt entry in the off taking canal.
4) They help in shutting of the supplies and not needed in the off taking canal or when the offtaking canal is required to be closed for repairs.

Functions of Cross regulator :-

1) The effective regulation of the whole canal's 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 up stream and feed's the off take channel in rotation.
3) It help's in closing the supply to the D/S of the parent channel for the purpose of repair's.
4) They help in absorbing fluctuation in various section's of the canal system and in preventing the possibilities of breaches in the tail reaches.
5) Incidently bridges and other communication work's can be combined with it.

Design of Cross Regulator :-

1) Design of Crest :-

The discharge is determined by the drowned weir formula :-

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

⑥

Q = discharge, in cumecs

L = length of water-way, in metres

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

h_a = Head due to velocity of approach.

d = Depth of d/s water level in the channel measured above the crest

C_1 = Constant = 0.557

C_2 = Constant = 0.80

Generally the velocity of approach is small, and may be neglected while using Eq. 18.24 Knowing the discharge Q, the length of waterway L can be calculated.

For the cross regulator, the crest level is kept equal to the upstream bed level of the parent channel.

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.

(ii) The discharge in the parent 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 floor level = d/s T.E.L − E_{f2} = 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. $E_{f1} = E_{f2} + H_L$

The depth D_1 and D_2 corresponding to E_{f1} and E_{f2} respectively are found from specific energy curves.

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

3) <u>Design</u> of <u>Impervious</u> floor:-

Total length of the impervious floor st. to be found from the Consideration of permissible Exit gradient.

The depth of u/s. Cutoff $d_1 = \frac{1}{3}$ u/s waterdepth + 0.6m

The depth of d/s Cut off $d_2 = \frac{1}{2}$ d/s water depth + 0.6m

Maximum static head H_s = u/s F.S.L. - d/s floor level

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

The Exit gradient curves, $\alpha (= b d_2)$ is known. The floor thickness is found from Considerations of uplift pressure. A minimum thickness 0.3 to 0.5m is provided from the pratical Consideration.

4) <u>Design</u> of <u>u/s</u> and <u>d/s</u> <u>protection</u>:-

u/s Scour depth d_1 is taken Equal ($\frac{1}{3}$ u/s water depth + 0.6m). The d/s Scour depth d_2 is taken Equal to ($\frac{1}{2}$ d/s water depth + 0.6m). There ~~scour depth~~ ~~is~~ below the Corresponding bed levels, and protection works are to be designed Corresponding

(a) <u>U/s Protection</u>:-

The U/s Protection Consist of a block Protection having Cubic Contents = d_1 Cubic Meters/m. Cubic Contents of u/s launching apron is kept equal to 2.25 d_1 Cubic Meter/Meter width of Regulator.

(b) <u>D/s Protection</u>:-

The Cubic Contents d of d/s inverted filter is kept Equal to d_2 Cubic Meter/Meter. The Cubic Contents of d/s launching apron is kept equal to 2.25 d_2 Cubic Meter/Meter width regulator

Cross Drainage Works :-

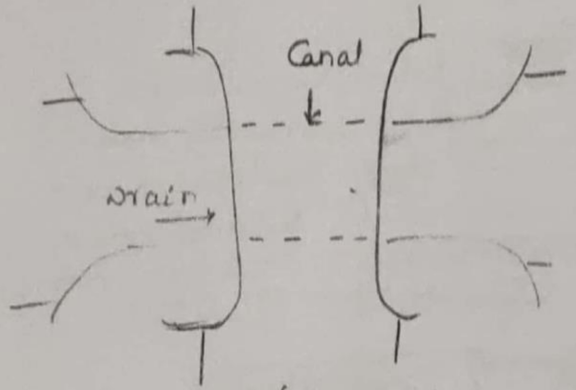
A cross drainage work is a structure carrying the discharge of a natural stream across a canal intercepting the stream.

Types of cross Drainage Works :-

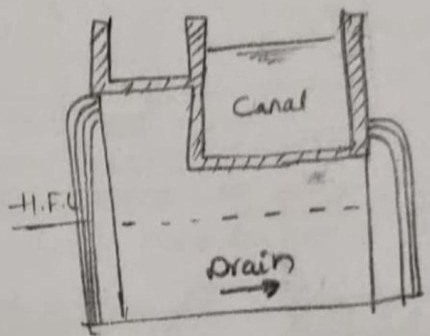
1. C. D works carrying canal over the Drainage

In this type of C.D work, the canal is carried over the natural drain the advantage of such arrangement, is that the canal running permenantly is above the ground.

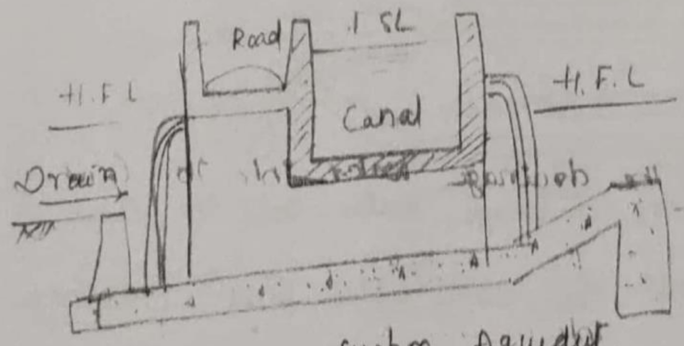
① Aqueduct ② syphon Aqueduct.



plan of crossing



Aqueduct.



syphon Aqueduct.

Shows the Aqueduct and syphon Aqueduct respectively. the H.F.L of the drain much below the bottom of the canal Trough in the case of aqueduct so that discharge water flows freely under gravity.

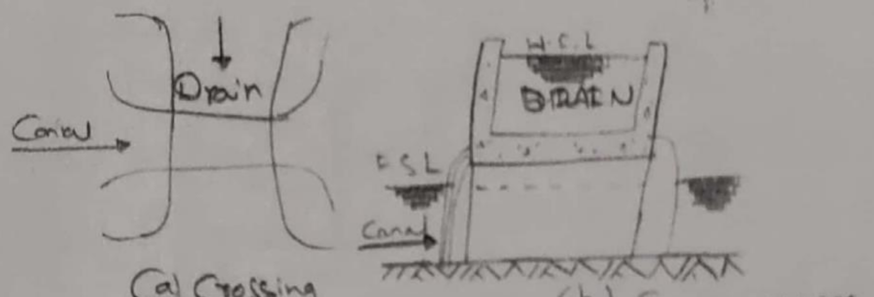
② C.D works carrying Drainage over the canal.

In this type of C. D works drainage is carried over the canal. The advantage of this Type is that the C.D work themselves are less liable to damage than the earth work of the canal.

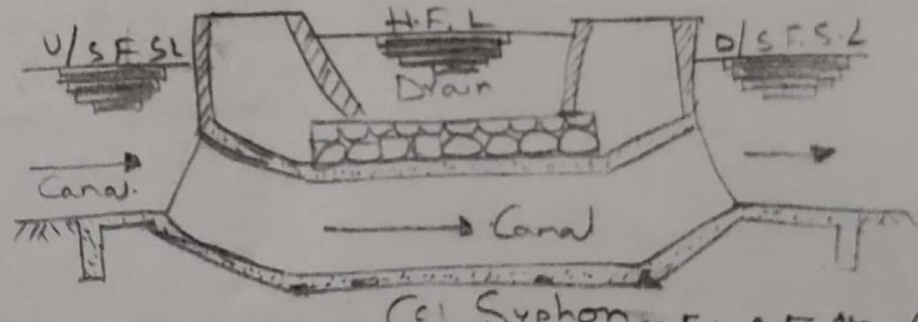
The major disadvantage of this work is that the perennial canal is not open to inspection.

The structures that fall under this Type are 1 Super passage
2 Canal syphon

Shows a supper passage. A super passage is similar to an adequet except in this case the drain is over the canal



(a) Crossing (b) Super passage



(c) Syphon

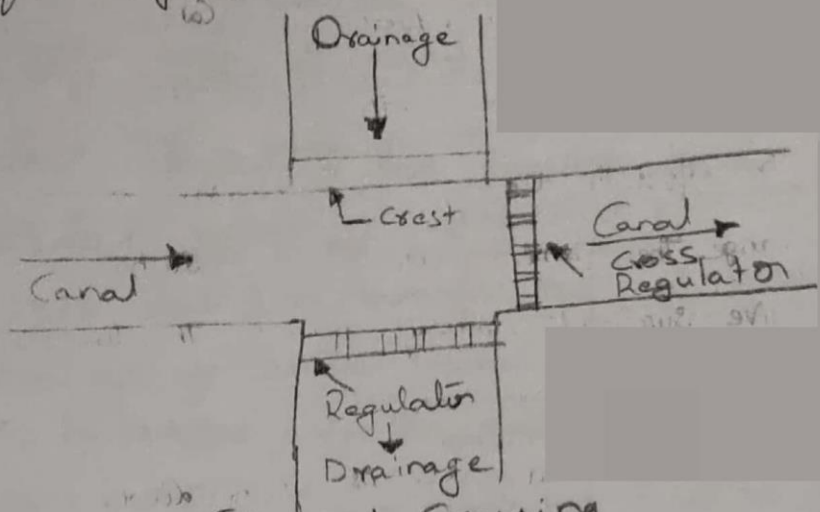
③ C.D Works admitting the drainage water into the Canal

In this Type of work the canal water and drainage water are permitted to intermingle with each other

① Level crossing ② Inlet and outlets

(i) Construction of a Crest with its top at the F.S.L of the Canal and drainage at the U/s junction with Canal.
(ii) Provision of the head regulator across the drainage at its d/s junction with the drainage.
During the floods, however the drainage regulator is opened so that the flood discharge, after spilling over the Crest and mixing with the Canal water.

The accurate supplies in the Canal are maintained by a Cross Regulator. level Crossing are suitable for Canals of all sizes.



(a) level Crossing

A Canal inlet [Fig. 19.3b] is Constructed when Cross drainage flow is small, and its water may be absorbed into the Canal without Causing appreciable rise. However, if the Canal is smaller an outlet may be Constructed to pass out the additional discharge which has Entered the Canal.

(b) Inlet

Selection of Suitable Type of cross Drainage works

The factors which affects the Selection of the Suitable Type of cross Drainage works are (i) relative bed levels and water levels of the Canal and Drainage.

(ii) Size of the Canal and the Drainage.

1. When the bed level of the Canal is much above the H.F.L of the drainage, so that sufficient head way is available for floating rubbish etc. and also for the

structural elements of the work, an adequate is the obvious choice.

2. The necessary headway between the canal bedlevel and the Drainage H.F.L can be increased by shifting the crossing to the d/s of the Drainage. If, however it is not possible to change the canal alignment or if such shifting doesn't not give sufficient headway between the two levels, a syphon aqueduct may be provided.

3. When the canal bed level is much lower, but the F.S.L of the canal is higher than the bedlevel of the drainage a canal syphon is preferred.

4. when the drainage and the canal cross each other practically at the same level, a cross drainge may preferred

Outlets

An outlet is small structure which admits water from the distributing channel to a water course or field channel. Thus, an outlet is a sort of head regulator for the field channel delivering water to the irrigation fields.

Types of outlets: —

Outlets may be classified under the following heads

1. Non modulur outlet
2. Semi - module or flexible module
3. Rigid module.

Non modulur outlet: — A non-modulur outlet is the one in which the discharge depends upon the difference in level between the water level in the distributing channel and the water course. The discharge through such an outlet varies in wide limits with the flutations of the water levels in the distributing and the field channels.

Semi module or Flexible outlet: — A flexible outlet or semi-module is one which the discharge is affected by the flutuation in the water level of the distributing channel while the flutation in water levels of the field channel do not have any effect on its discharge. The various outlets in common use that fall under this category are pipe outlet, Kennedy's gauge outlet, Crump's open flume outlet

③ Rigid Module :— A Rigid module is the one in which maintains Constant discharge within limits irrespective of the fluctuations in water levels in the distributing channel and/or field channel. This is most Common outlet that falls under this Category is the Gibb's rigid module.

④ proportionality :—

A proportional outlet is the one in which the flexibility (F) is equal to unity. Thus in a proportional outlet, the rate of change of its discharge is equal to the rate of change of the discharge of the distributing channel. For proportionality, putting F=1 in Equation $F = \frac{m}{n}\frac{D}{H}$

The ratio of H/D is known as the setting. In a proportional unit Therefore, sitting is equal the ratio of outlet and canal indices.

From the view of proportionality, an outlet is classified into three Types.

ⓐ proportional outlet
ⓑ Hyper proportional outlet
ⓒ Sub proportional outlet.

⑤ Flexibility :— It is the ratio of rate of change of discharge of an outlet to the rate of change of the discharge of the distributing channel.

Thus $F = \frac{dq}{q} / \frac{dQ}{Q}$.

Thus where F = Flexibility

q = Discharge through the outlet

Q = Discharge of the distributing channel

Now for the field channel

$$q = KH^m$$

K = Constant m = outlet index

H = head acting on the outlet

$$dq = mKH^{m-1}\,dH$$

$$\frac{dq}{q} = \frac{mKH^{m-1}\,dH}{KH^m} = m\frac{dH}{H} \longrightarrow (1)$$

Similarly for the parent channel

$$Q = CD^n$$

C = Constant n = Canal index

D = Depth of water in the Canal

$$dQ = nCD^{n-1}\,dD$$

$$\frac{dQ}{Q} = \frac{nCD^{n-1}\,dD}{CD^n} = n\frac{dD}{D} \longrightarrow (2)$$

Dividing (1) & (2) we get

$$F = \frac{\frac{dq}{q}}{\frac{dQ}{Q}} = \frac{m\frac{dH}{H}}{n\frac{dD}{D}} = \frac{m}{n}\frac{D}{H}\frac{dH}{dD}$$

Since any change in the water depth results in an equal change in the head causing flow we have $dH = dD$. Thus the expression for flexibility becomes

$$\boxed{F = \frac{m}{n}\frac{D}{H}}$$

Sensitivity :—

It is defined as the ratio of rate of change of discharge of an outlet to the rate of change in the level of distributing surface, referred to normal depth of the channel. Thus,

$$S = \frac{dq/q}{dG/D}$$

where

S = Sensitivity of the outlet

q = Discharge through the outlet

dq = change in the discharge of the outlet

G = Guage reading, so set that $G=0$ when $q=0$

D = Depth of water in the Distributing channel

$dG = dD$

$$S = \frac{dq/q}{dD/D} \longrightarrow (1)$$

$$F = \frac{dq/q}{dQ/Q} \quad \text{where } \frac{dQ}{Q} = n\frac{dD}{D}$$

$$F = \frac{dq}{q} \Big/ n\frac{dD}{D} \longrightarrow (2)$$

Comparing (1) and (2) we get

$$\boxed{S = nF}$$

River Training

The expression for river Training implies various measures adopted on a river to direct and guide the river flow, to Train and regulate the river bed or to increase the low water depth. The purpose of river Training is to establish the channel along a certain alignment. There may be various objects for Training a river; these are discribed below.

(1). High flood discharge may pass safely and quickly through the reach.

(2) Sediment load including bed and suspended load may be Transported efficiently

(3) To make the river course stable and reduce bank erosion to minimum

(4) To provide a sufficient draft for navigation as well as good course for it.

(5) To fix direction of flow through certain defined reach

Classification of River Training Works

1. High water Training :- This is also called Training for discharge. The river is Trained to provide sufficient and efficient cross sectional area for the expedition passage of maximum flood.

2. Low water Training :- In case the river is Trained to provide sufficient depth for navigation during low stage of river. This is also called Training for depth. and is usually achieved by contraction of the width of the channel.

③ Mean Water Training

In the case of river is Trained to Correct the configuration of river bed for the efficient Transport of sediment load in order to keep the channel in good shape It can be called Training for Sediment.

Kennedy's theory.

Example 4.7. *Design an irrigation channel to carry 50 cumecs of discharge. The channel is to be laid at a slope of 1 in 4000. The critical velocity ratio for the soil is 1.1. Use Kutter's rugosity coefficient as 0.023.*

Solution. $Q = 50$ cumecs, $S = \dfrac{1}{4000}$, $m = 1.1$, $n = 0.023$

Use equation (4.19), as, $V_0 = 0.55m \cdot y^{0.64}$

Assume a depth equal to 2 m

$$V_0 = 0.55 \times 1.1 \times (2)^{0.64} = 0.605 \times 1.558 = \mathbf{0.942} \text{ m/sec}$$

$$A = \frac{Q}{V_0} = \frac{50}{0.942} = 53.1 \text{ m}^2.$$

Assume side slopes as $\dfrac{1}{2} : 1 \left(\dfrac{1}{2} H : 1V\right)$

Now, $A = y\left(b + y \cdot \dfrac{1}{2}\right)$

$\therefore$ $53.1 = 2\,(b + 1)$

or $26.55 = b + 1$

or $b = 25.55$ m

and $P = b + 2\sqrt{\left(1 + \dfrac{1}{4}\right)} \times y$

or $P = b + 2\dfrac{\sqrt{5}}{2}y = 25.55 + \sqrt{5} \times 2 = 30.03$

$$R = \frac{A}{P} = \frac{53.1}{30.03} = 1.77 \text{ m}.$$

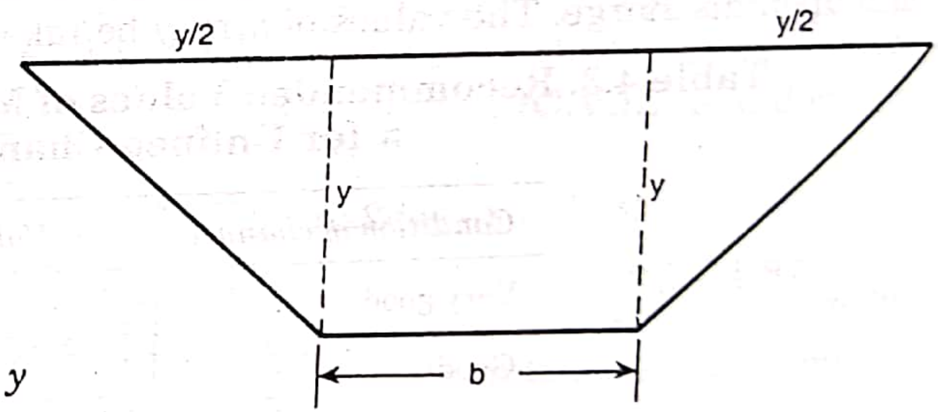


Fig. 4.9.

But, from eqn. (4.20),

$$V = \left[\frac{\frac{1}{n} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{n}{\sqrt{R}}}\right]\sqrt{RS}$$

$$\therefore \quad V = \left[\frac{\frac{1}{0.023} + 23 + \frac{0.00155}{1/4000}}{1 + \left(23 + \frac{0.00155}{1/4000}\right)\frac{0.023}{\sqrt{1.77}}}\right]\sqrt{1.77 \times \frac{1}{4000}}$$

$$= \left[\frac{43.5 + (23 + 6.2)}{1 + \frac{29.2 \times 0.023}{1.33}}\right]\left[1.33 \times \frac{1}{63.3}\right]$$

$$= \frac{72.7}{1 + 0.505} \times 1.33 \times \frac{1}{63.3} = \frac{72.7}{1.505} \times 1.33 \times \frac{1}{63.3}$$

$$= 1.016 \text{ m/sec} > 0.942 \text{ ; or } V > V_0.$$

In order to *increase* the critical velocity (V_o),, we have to *increase* the depth. So increase the depth.

Use 3 m depth :

$$V_0 = 0.605 \times (3)^{0.64} = 0.605 \times 2.02 = 1.22 \text{ m/sec.}$$

$$A = \frac{50}{1.22} = 40.8 \text{ m}^2.$$

$$40.8 = 3\left(b + \tfrac{1}{2}\cdot 3\right) \quad \text{or} \quad 13.6 - 1.5 = b = 12.1 \text{ m.}$$

$$P = 12.1 + 2 \times \frac{\sqrt{5}}{2}3 = 12.1 + 6.72 = 18.82$$

$$R = \frac{A}{P} = \frac{40.8}{18.82} = 2.17 \text{ ; therefore } \sqrt{R} = 1.47.$$

$$V = \frac{43.5 + 29.2}{1 + \frac{29.2 \times 0.023}{1.47}} + \left[1.47 \times \frac{1}{63.3}\right] = \frac{72.7}{1.45} \times 1.47 \times \frac{1}{63.3}$$

$$= 1.16 \text{ m/sec.} < 1.22 \text{ ; or } V < V_0, \text{ So reduce the depth.}$$

Use 2.5 m depth

$$V_0 = 0.605 \times (2.5)^{0.64} = 0.605 \times 1.797 = 1.087 \text{ m/sec.}$$

$$A = \frac{50}{1.087} = 46$$

$$46 = 2.5\left(b + \tfrac{1}{2}2.5\right)$$

$$18.4 - 1.25 = b = 17.15 \text{ m}$$

$$P = 17.15 + \sqrt{5} \times 2.5 = 17.15 + 5.58 = 22.73$$

$$R = \frac{A}{P} = \frac{4}{22.73} = 2.02 \text{ ; therefore } \sqrt{R} = 1.42$$

$$V = \frac{72.7}{1 + \frac{29.2 \times 0.023}{1.42}}(1.42)\left(\frac{1}{63.3}\right) = \frac{72.7}{1.472} \times \frac{1.42}{63.3}$$

$$= 1.1 \text{ m/sec} > 1.087 \text{ ; } V > V_0$$

So increase the depth.

Use 2.7 m depth

$$V_0 = 0.605 \times 1.189 = 1.147$$

$$A = \frac{50}{1.147} = 43.5$$

$$43.5 = 2.8\,(b + \tfrac{1}{2} 2.8)$$

$$15.54 - 1.4 = b = 14.14 \text{ m}$$

$$P = 14.14 + \sqrt{5} \times 2.8 = 14.14 + 6.26 = 20.40$$

$$R = \frac{43.5}{20.4} = 2.13, \text{ therefore, } \sqrt{R} = 1.46$$

$$\therefore \qquad V = \left[\frac{72.7}{1 + \dfrac{29.2 \times 0.023}{1.46}}\right]\left[\frac{1.46}{63.3}\right] = \left[\frac{72.6}{1.46}\right]\left[\frac{1.46}{68.3}\right]$$

$$= 1.148 \text{ m/sec} \approx 1.147 \text{ or } V \approx V_0.$$

Actual velocity V tallies with V_0.

Hence, use the depth equal to 2.7 m and base width 14.14 m. (say 14.2 m) with slopes $\frac{1}{2}$: 1 of trapezoidal section. **Ans.**

Example 4.8. *Design an irrigation channel to carry 40 cumecs of discharge, with B/D, i.e. base width to depth ratio as 2.5. The critical velocity ratio is 1.0. Assume a suitable value of Kutter's rugosity coefficient and use Kennedy's method.*

Solution. $V_0 = 0.55\,(y)^{0.64}$ $(\because\ m = 1)$

Here $\quad y = D$

$\therefore \quad V_0 = 0.55 \cdot D^{0.64}$

$\quad Q = AV$

Using $\frac{1}{2}$: 1 slopes, area (A) of trapezoidal section is given as :

$$A = BD + 2 \cdot \frac{1}{2} \cdot D\frac{D}{2} = D\left[B + \frac{D}{2}\right]$$

$$\therefore \qquad 40 = D\left[B + \frac{D}{2}\right]V_0 \qquad \left(\because\ \frac{B}{D} = 2.5\right)$$

But $\quad B/D = 2.5$; or $B = 2.5D$

$\therefore \quad 40 = D\,[2.5\,D + 0.5D]\,V_0 = D\,[3D]\,V_0 = 3D^2 \cdot V_0$

But $\quad V_0 = 0.55 \cdot D^{0.64} \qquad \therefore\ 40 = 3D^2\,(0.55 \cdot D^{0.64})$

or $$D^{2.64} = \frac{40}{3 \times 0.55} = 24.2$$

or $$D = (24.2)^{\frac{1}{2.64}} = (24.2)^{0.379} = \mathbf{3.34\ m}$$

Now $\quad B = 2.5\,D = 2.5 \times 3.34 = \mathbf{8.35\ m}$

Now determine the slope S

$$A = 3D^2 = 3 \times (3.34)^2 = 33.5 \text{ m}^2.$$

$$P = \left[B + 2 \cdot \frac{\sqrt{5}}{2}D\right] = (8.35 + \sqrt{5} \times 3.34) = (8.35 + 7.46) = 15.81 \text{ m}$$

$$R = \frac{33.5}{15.81} = 2.12, \text{ or } \sqrt{R} = 1.456$$

$$V_0 = 0.55\,(3.34)^{0.64} = 0.55 \times 2.163 = \mathbf{1.19}$$

Assume $n = 0.023.$

Using Eq. (4.20), we get

$$V = \left[\frac{\frac{1}{0.023} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{0.023}{1.456}}\right] 1.456\sqrt{S} \qquad ...(i)$$

Assume $S = 1/4000.$

Putting this value of S and computing the value of V, we get

$$V = \frac{43.5 + (23 + 6.2)}{1 + 29.2 \times \frac{0.023}{1.456}} \times \frac{1.456}{63.3} = 1.114$$

$$1.114 < 1.19 \quad \text{or} \quad V < V_0$$

Therefore, to increase the value of V, we must increase/steepen the slope ; hence, use a slope = 1 in 3700 (say)

Putting $S = \frac{1}{3700}$ in (i) above, we get

$$V = 1.189 \approx 1.19$$

or $V = V_0$ for value of $S = \frac{1}{3700}$ So use $S = \frac{1}{3700}$

Hence, use a trapezoidal channel section as follows :

$$\left.\begin{aligned} \text{Depth} &= 3.34 \text{ m} \\ \text{Base width} &= 8.35 \text{ m} \\ \text{Side slopes} &= \tfrac{1}{2} H : 1V \\ \text{Bed slope} &= 1 \text{ in } 3700 \end{aligned}\right] \quad \textbf{Ans.}$$

Kennedy's Theory. A

Example 4.10. *Design a regime channel for a discharge of 50 cumecs and silt factor 1.1, using Lacey's Theory.*

Solution. $Q = 50$ cumecs, $f = 1.1$

$$V = \left[\frac{Qf^2}{140}\right]^{1/6} = \left[\frac{50 \times (1.1)^2}{140}\right]^{1/6}$$

$$A = \frac{Q}{V} = \frac{50}{0.869} = 56.3 \text{ m}^2$$

$$R = \frac{5}{2} \cdot \frac{V^2}{f} = \frac{5}{2} \cdot \frac{(0.869)^2}{1.1} = 1.675 \text{ m.}$$

$$P = 4.75\sqrt{Q} = 4.75 \cdot \sqrt{50} = 33.56 \text{ m}$$

For a trapezoidal channel with $\frac{1}{2} H : 1V$ slopes

$$P = b + \sqrt{5}\cdot y \qquad \text{and} \qquad A = \left(b + \frac{y}{2}\right)y$$

$\therefore$ $$33.56 = b + \sqrt{5} \cdot y \qquad ...(i)$$

and $$56.3 = by + \frac{y^2}{2} \qquad ...(ii)$$

From Eq. (*i*), we get, $b = 33.56 - 2.24y$

Putting this value of b in Eq. (*ii*)

$$56.3 = [33.56 - 2.24y]\, y + \frac{y^2}{2}$$

$$= 33.56y - 2.24y^2 + 0.5y^2 = 33.56y - 1.74y^2$$

or $$1.74y^2 - 33.56y + 56.3 = 0$$

or $$y^2 - 19.3y + 32.4 = 0$$

$\therefore$ $$y = \frac{19.3 \pm \sqrt{372 - 129.6}}{2} = \frac{19.3 \pm \sqrt{242.4}}{2} = \frac{19.3 \pm 15.6}{2}$$

...id confusion

Neglecting unfeasible + ve sign, we get

$$y = \frac{19.3 - 15.6}{2} = 1.65 \text{ m}$$

$\therefore$ $y = 1.65$ m. **Ans.**

$$b = 33.56 - 2.24 \times 1.65 = \mathbf{29.77\ m}$$

or $b = 29.77$ m. **Ans.**

$$S = \frac{f^{5/3}}{3340\, Q^{1/6}} = \frac{(1.1)^{5/3}}{3340 \cdot (50)^{1/6}} = \frac{1}{5420}$$

Use a bed slope of 1 in 5420. **Ans.**

UNIT-3

Diversion Head-Works

Diversion Head works: —

Any hydraulic structure which supplies water to the off Taking canal is called a headwork. Headwork may be divided into Two classes:

① Storage Headwork

② Diversion Headwork

A storage headwork Comprises the Construction of dam across the river. It stores water during the period, of excess supplies in the river and releases it when demand overtakes available supplies.

A Diversion Headwork serves to divert the required supply into the canal from the river. A diversion Head work serves the following purposes.

1. It rises the water level in the river so that the commanded area can be increased.
2. It regulate the ~~tad~~ Intake of water into the canal
3. It control the silt entry into the canal.
4. It reduces the fluctuation in the level of supply in the river
5. It stores water for tiding over small periods of short supplies.

Weir:— The weir is a solid obstruction put across the river to raise its water level and diverts the water in to the canal. If a weir also stores water for tiding over small period 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 in which the supply is stored. A dam stores the supply for a comparitively longer duration.

Weirs are classified into two heads depending upon the criterion of the design of their floors.

① Gravity dam ② Non Gravity dam.

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

In 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 feature, can be further divided into.

1. Vertical drop weir
2. Masonry or Concrete slope weir
3. Dry stone slope weir
4. parabolic weir.

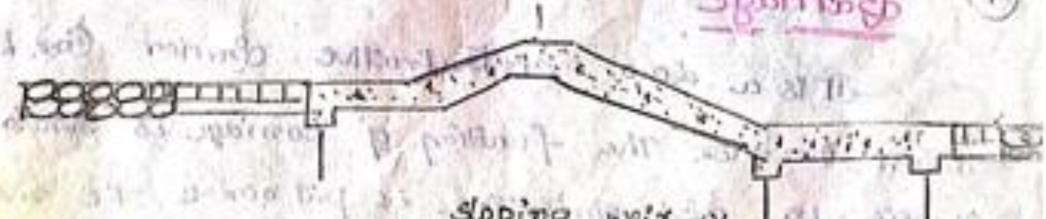
① Vertical drop weir :—

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 safeguard against scouring action, launching aprons are provided both at upstream & downstream ends of the floor.

Vertical drop weir

② Masonry or concrete sloping weir :—

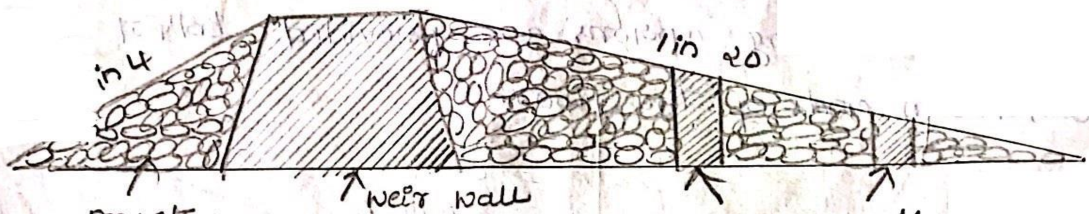
Weirs of this type are of recent origin. They are suitable for soft sandy foundations and are generally used where the difference in weir crest and downstream river bed is limited to 3 metres. When water passes over such a weir hydraulic jump is formed on the sloping glacis



Sloping weir of concrete

(3) Dry stone slope weir: —

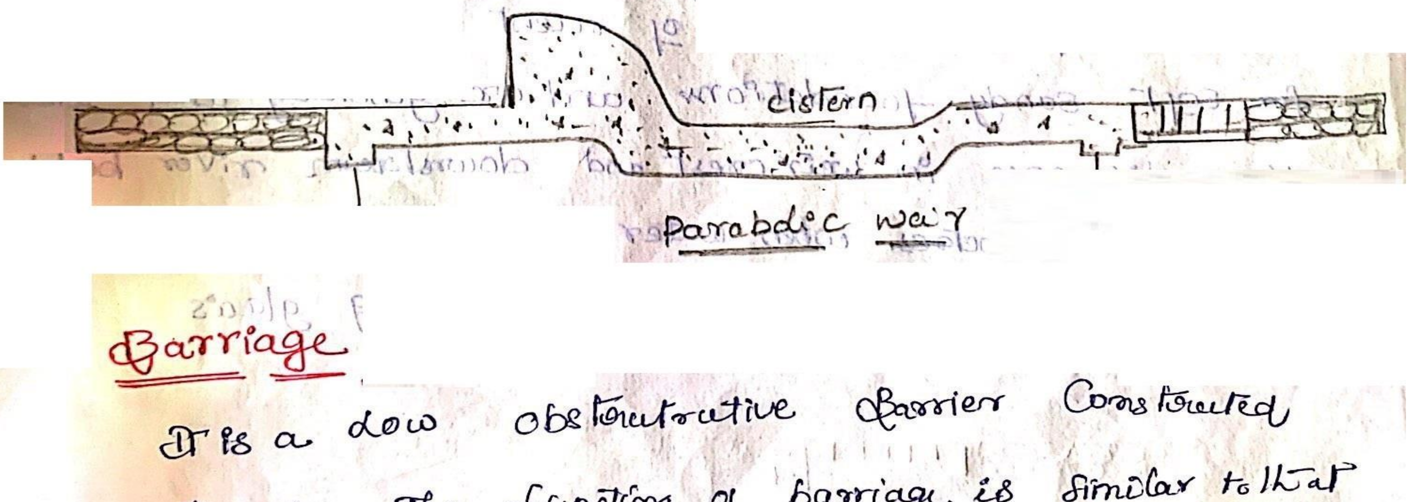
A Dry stone weir or a rockfill weir Consists of a body wall (weir wall) and u/s and d/s rockfills laid in the form of glacis with a few intervening core walls



Masonry dry stone weir

(4) parabolic weir: —

A parabolic weir is similar to the spillway section of a dam. The body wall for such weir is designed as a low dam. A cistern provided at the d/s side to dissipate energy

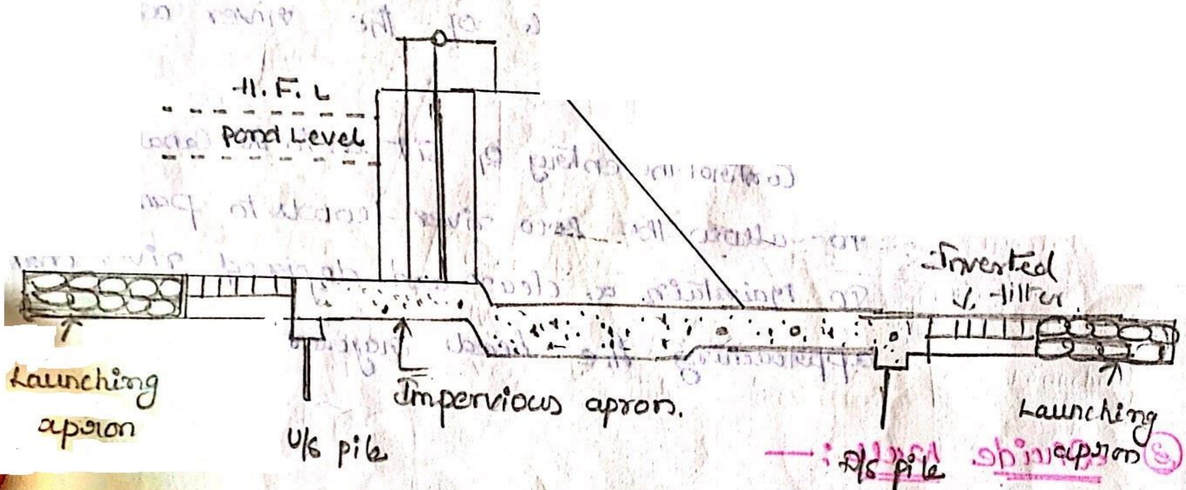


Parabolic weir

Barriage

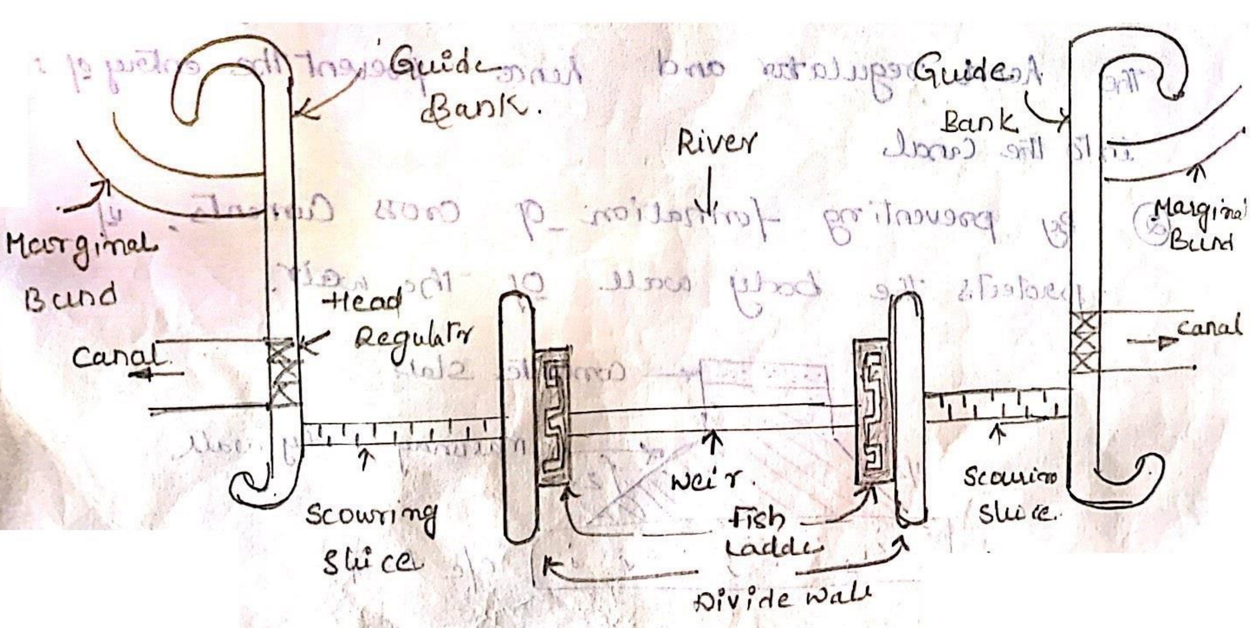
It is a low obstructive barrier constructed across the river. The function of barriage is similar to that of weir. No solid obstruction is put across the river but heading up of water is effected by gates alone. Gates are provided on the crest of the barrier and they are housed in the grooves made in the the piers and abutment. The piers are also constructed on the crest and

supporting a platform used for lifting and lowering ③
by the gates. Thus the flow is perfectly controlled by
gates. Due to this, there is less silting and better control
over the levels. However barriages are much more
Costlier then the weirs.



Barrage

Layout of Diversion Head works :-



Diversion Head work

① **Weir** :– A masonry structure Constructed across the river with or without shutters is Called weir.

Function :– It Can raise the water to the Designed Level.

② **Scouring Sluice** :–

The openings provided in the body wall of the weir almost at the bed level of the river are Called scouring sluices.

Functions : To Control the entry of silt in to the Canal

To allow the low river floods to pass Safely

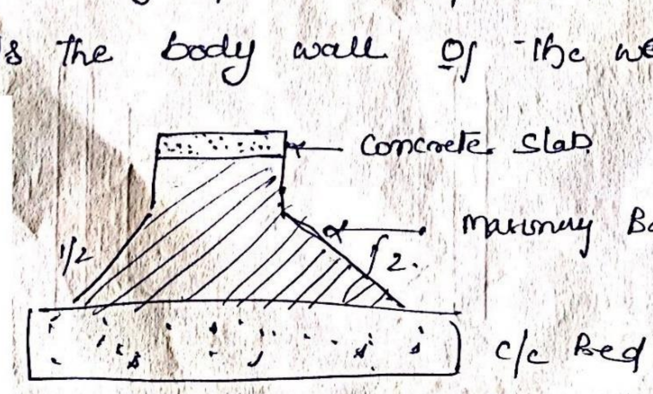
To Maintain a clear and defined river channel approaching the head regulator.

③ **Divide wall** :–

A long solid wall Constructed perpendicular to the axis of weir between the Scouring slice and the fish ladder is Called Divid wall. It Divide the challel into two Components.

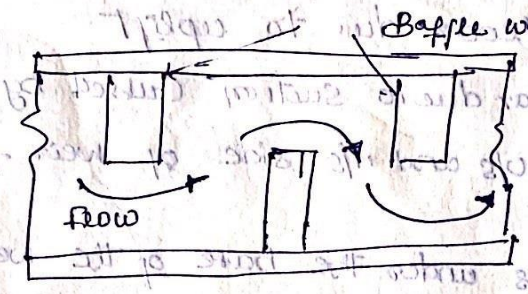
Function :– It Can create a still pond very near to the head regulator and hence prevent the entry of silt into the Canal

② By preventing formation of cross Currents, it protects the body wall of the weir.



④ **Fish Ladder :—**

A passage provided just by the side of the divide wall for the movement of fish from U/S & D/S or vice versa is called fish ladder.



⑤ **Head Regulator :—**

A structure constructed at the head of the canal to regulate the supply of water in to the canal is called "Head Regulator"

Functions

1. It is used as a measuring device.
2. It does not allow flood water to enter the canals.

⑥ **Guid Banks :—**

Guid Banks are provided on either side of the Diversion Head works in alluvial soils for a smooth non tortous approach to the diversion Head works to prevent the river from outflanking the work.

⑦. **Marginal Bunds :—**

Marginal Embankment are provided on either Bank of the river U/S of Diversion head works in alluvial soil to protect the land and property which is likely to be submerged during ponding of water during floods.

* Causes of failure of weir :—

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 u/s and d/s side of weir floor.

1) Piping :—

water seeps under the base of the weirs founded on permeable soils. when the flow line emerge out at the d/s end of the impervious floor of the weir. The hydraulic gradient or the exit gradient 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 surface soil, there is further concentration of flow lines into the resulting depression and still more soil is removed.

Remedies :— piping providing sufficient length of the impervious floor so that path of percolating is increased and the exit gradient is decreased.

② providing pile at d/s end.

② Rupture of floor due to uplift :

If the weight of floor is insufficient to resist the uplift pressure, the floor may burst and effective length of impervious floor is thereby reduced.

Remedies :— 1. providing impervious floor of sufficient length

2. providing impervious floor of appreciat thickness of various point.

3. providing pile at the d/s end. so that the uplift pressure to the d/s is reduced.

③ Rupture of floor Due to Suction Caused by standing wave

⑤

The standing wave or Hydralic jump formed at 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.

Remedies: → 1. providing additional thickness of floor to counterbalence the extra pressure due to the standing wave.

2. Constructing the floor thickness in one concrete mass instead of in masonry layers.

④ Scour on the U/s & D/s of the weir :—

when the natural waterway of a river is contracted, the water may scour the bed both at upstream & Downstream of the structure. The scour holes so formed may progress towards the structure.

Remedies ① Taking the piles at U/s & D/s ends of the impervior floor much below the calculated scour level.

✱ Design of Impervious floor for Subsurface flow

we have alredy seen that the subsurface flow or the foundation seepage may cause harm in 2 ways.

1. piping (or) uplift.

1. Bligh's creep theory :—

The design of impervious floor, or the apron is directly dependent on the possibilities of percolation in the porous soil on which the apron is built.

Bligh's assumes as an approximation that the hydraulic slope or gradient is constant througout the impervious length of the apron. He further assumed the percolating water to creep along the contact of the base profile of the apron with the subsoil.

He designated the length of the Travel as the creep length, which is sum of the horizontal as well as vertical length of creep.

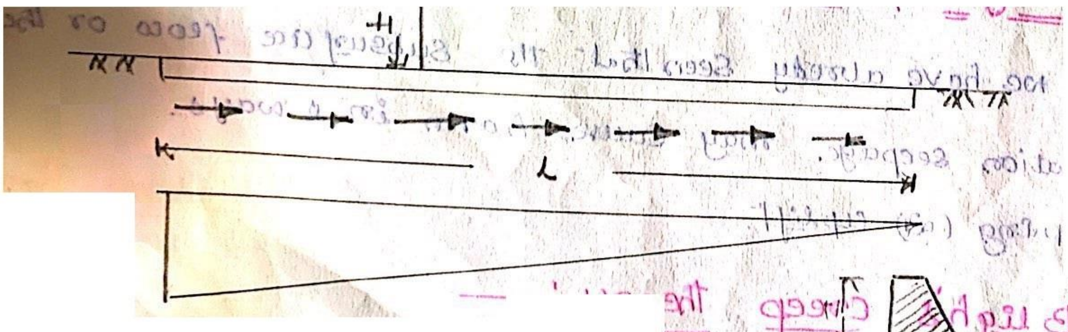
Bligh's asserted that no amount of sheet piling or another cut off could ever stop the percolation unless the cut off extends up to the impermeable soil strata.

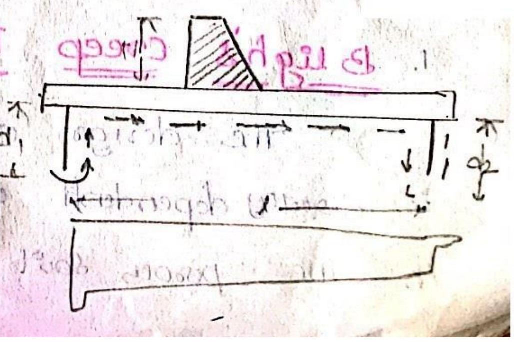
Thus according to the Bligh's theory, the Total creep length L for the case

$$L = l.$$

In the fig the Total creep length

$$L = 2d_1 + l + 2d_2$$





Bligh's creep Theory

This means that in calculating length of creep the (6)
depth of every cutoff is multiplied by the coefficient 2.

If H is the total loss of head. the loss of head per unit length of creep. (C) would be

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

Coefficient of creep

Type of soil	value of $c = 1/C$
1. light sand and mud	18
2. Fine micaous sand	15
3 Coarse grained sand	12
4 Boulders or shingle gravel mixed	5 to 9.

Design criteria :-

① Safety against piping :- The length of creep should be (1) sufficient to provide a safe hydraulic gradient according to the type of the soil.

Thus, the safe creep length is given by

$$L = CH$$

C = coefficient of creep. $= 1/C$.

② Safety against uplift pressure

Let h' be uplift pressure head at any point of the apron

The uplift Pressure $= wh'$ → ①

If t = thickness of the floor at the point

ρ = specific gravity of the floor material.

Then downward force (resisting force) per unit area

$= t w \rho \rightarrow$ ②

Equating ① & ② we get

$$w h' = t w \rho$$

$$h' = t\rho$$

$$h' - t = t\rho - t = t(\rho - 1)$$

$$t = \frac{h' - t}{\rho - 1} = \frac{h}{\rho - 1}$$

h = ordinate of the hydraulic-gradient line measured above the top of the floor providing a factor of safety of 4/3 we have

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

① problem

Fig shows the section of a hydralic structure founded on sand. Calculate the average hydralic gradient. Also find the uplift pressure at points 6 and 12, 18 m 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 $= (2\times6) + 22 + 2\times8 = 50$ $\quad 2l + d_1 + 2l_2$

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

(i) uplift pressure at a point A, 6 m from U/S.

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

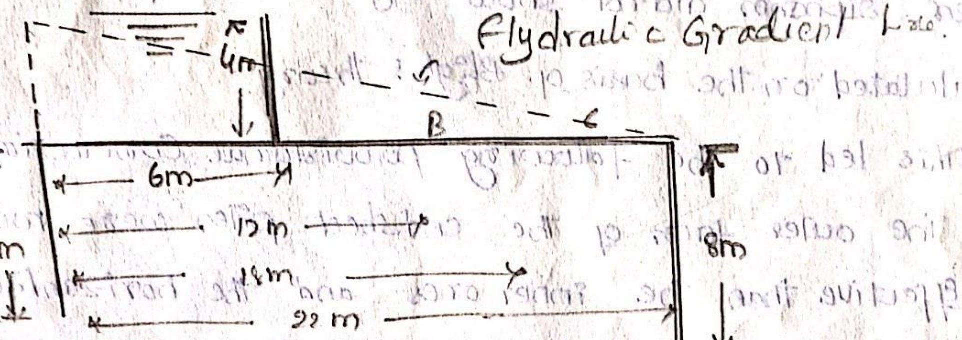
unbalanced head $h_1 = 4\left(1 - \frac{18}{50}\right)$

$4\left(1 - \frac{A}{L}\right)$

uplift pressure $wh' = 9.81 \times 2.56 = 25.11\ kN/m^2$

Thickness $t = \frac{4}{3} \frac{h_1}{\rho - 1} = \frac{4}{3} \times \frac{2.56}{2.24-1} = 2.76m$

(2) 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 $= wh_2 = 9.81 \times 2.08 = 20.4\ kN/m^2$

Thickness $t = 4/3 \frac{h_2}{\rho - 1} = \frac{4}{3} \frac{2.08}{2.24-1} = 2.23\ m.$

(3) 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 pressures $wh_3 = 9.81 \times 1.6 = 15.7\ kN/m^2$

Thickness $t = 4/3 \frac{h_3}{\rho - 1} = 4/3 \times \frac{1.6}{2.24-1}$

$1.72m$.

(D) Khosla's Theory :—

Some siphon's on upper Chinab Canal designed on Bligh's theory gave trouble. Actual pressure measurements made with the help of pipes inserted in the floors of these siphones didnot show any relationship with the pressure calculated on the basis of Bligh's theory.

This led to the following provisional conclusions by Khosla.

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

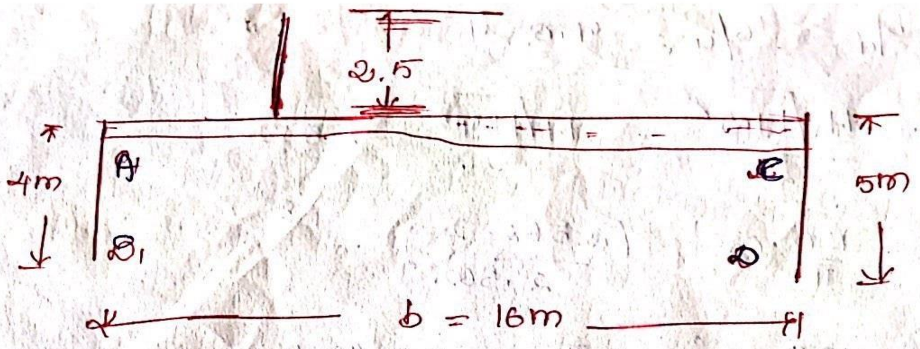
⇒ The intermediate piles of smaller in length than the outer one were ineffective except for the local redistrib of pressure.

⇒ undermining of the floor started from Tail end. If the hydraulic Gradient at exit was more than the critical gradient for the particular soil.

⇒ It was absolutely essential to have a ~~relationship~~ de reasonabley deep vertical cutoff at d/s end to prevent undermining.

problem :—

(1) An impervious floor of weir on permeable soil is 16m long and has sheet piles at both the ends. The upstream pile is 4m ~~long.~~ deep and d/s pile is 5m deep. The weir creates a net head of 2.5m. Negleting the thickness of the weir floor. Calculate the uplift pressure at the junctions of inner faces of the pile with the weir floor. by using Khosla theory.



1) pressure at point E

$$\phi_E = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right). \qquad b = 16. \quad d = 5.$$

$$\alpha = b/d = \frac{16}{5}$$

$$= 3.2$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+(3.2)^2}}{2} = 2.176.$$

$$\phi_E = \frac{100}{\pi}\cos^{-1}\left(\frac{2.176-2}{2.176}\right)$$

$$= \frac{100}{\pi}\, 85°35' \times \frac{\pi}{180} = 47.42\ \%$$

let us now apply Corrections for interference of c/s pile

$$c = -19\sqrt{\frac{D}{d}}\left(\frac{d+D}{b}\right)$$

$d = 5m \quad D = 4m \quad b' = 16m$

$$c = -19\sqrt{\frac{4}{16}}\left(\frac{5+4}{16}\right) = -5.34\%.$$

$$\phi_E = 47.42 - 5.34 = 42.08\ \%.$$

$$P_E = 0.4208 \times 2.5 = \underline{1.052m}$$

② pressure at point A :-

$$\phi_A = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right).$$

$\alpha = b/d = 16/4 = \cancel{4} \; 4$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+4^2}}{2} = 2.562$$

$$\phi_E = \frac{100}{\pi} \cos^{-1}\left(\frac{2.562-2}{2.562}\right)$$

$$= \frac{100}{\pi} \, 77.34^\circ \times \frac{\pi}{180} = 42.96\,\%$$

Let us know apply corrections of d/s pile

$$C = +19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

$d = 4\,m$ $D = 5m$ $b' = b = 16m$

$$C = 19\sqrt{\frac{5}{16}}\left(\frac{4+5}{16}\right) = 5.97\,\%$$

$$\phi_A = 42.96 + 5.97 = 48.\,-\,\%$$

$$P_c = 0.\cancel{48}.9 \times 2.5 = \underline{1.575m}$$

UNIT-4 ①

~~Resource Engineering~~

Reservior planning

Investigation :—

The following investigation are required for reserviour planning

① Engineering surveys. 2. Geological Investigation 3. Hydrological Investigation

1. Engineering Surveys

The area at the dam site is surveyed in detail and contour plan is prepared. From the plan, the following characteristics are prepared

ⓐ Area Elevation curve ⓑ Storage Elevation curve

ⓒ Map of the area to indicate the land property to be surveyed.

ⓓ Suitable site selection for the dam.

1. Area Elevation & Storage Elevation Curves

Figure shows the typical contour plan at the reservior site. The hatched area shows the water spreaded area. The area A_1, A_2, A_3 enclosed by the successive contours can be determined with a planimeter.

The reservior capacity, or the volume of the storage corresponinding to a given water level in the reservior may be calculated either by a Trapizoidal formula or by prismoidal formula. Thus, if V is the storage volume and h is the contour intervel the formula are.

1. $V = \Sigma \frac{h}{2} (A_1 + A_2) \ldots$ Trapizoidal formula

2. $= \left\{ \frac{A_1 + A_n}{2} + A_2 + A_3 \ldots \ldots A_{n-1} \right\}$

2. $V = \Sigma \frac{h}{3} (A_1 + A_2 + \sqrt{A_1 A_2} \ldots$ (cone formula)

3. $V = \frac{h}{3} [A_1 + A_n + 4 (A_2 + A_4 + \ldots) + 2 (A_3 + A_5 + \ldots)]$

..... prismoidal formula.

where A_n is the area of the Contour Corresponding to the water surface elevation in the proposed reservior. The volume corresponding to various water surface elevation may be calculated and a curve in below figure.

(2) Geological Investigation

Geological investigation are required to give detailed information about the following items.

1. water Tightness of reservior basin.
2. Suitability of foundation for the dam
3. Geologically and structural features such as folds faults, fissure etc. of the rocks basin.
4. Type and depth of over burden
5. Location of permeable and Suitable rocks if any
6. Ground water Conditions in the region.
7. Location of quarry sites for material required for the dam Construction and quantities available from them.

The geology of the Catchment area should also be studied since it affects the proportions of runoff percolations

The special requirement for the geology of the reservior site is that there should be no danger of serious leakage when the ground is under pressure from the full head of water in the reservior.

③ Hydrological Investigations :-

The hydrological investigation is a very important aspect of reservior planning. The capacity of the irrigation canals. The capacity of the irrigation canals and or the installed capacity of the power houses will depend upon the available supplies from the reservior.

1. Study of runoff pattern at the proposed dam site, to determine the storage capacity corresponding to a given demand.
2. Determination of the hydrograph of the worst flood, to determine the spillway capacity and design.

④ Site Selection of Sites For a Reservior

The final selection of site for a reservior depends upon the following factors.

① The geological condition of the catchment area should be such that percolation losses are minimum and maximum runoff is obtained.

② The reservior site should be such that quantity of leakage through it is a minimum. Reservior site having the presence of highly permeable rocks reduce the water tightness of the reservior. Rocks which are not likely to allow passage of water includes shales and slates, schists, gneisses and crystalline igneous rocks such as granite.

③ Suitable dam site must exist the dam should be founded on sound waterotight rock base, and percolation below the dam should be minimum. The cost of the dam is often a Controlling factor in selection of a site.

④ The reservior basin should have narrow openining in the valley so that the length of the dam is less.

⑤ The cost of the real estate for the reservior, including road railroad dwelling relocation etc. must be as less as possible

⑥ The Topography of the reservior site should be such that it has adequate capacity without submerging excessive land and other properties.

⑦ The site should be such that a deep reservior is formed a deep reservior is preferable to a shallow one because of
(i) lowest cost of land submerged per unit of Capacity
(ii) less evaporation losses because of reduction in the water spread area and (iii) less likelyhood of weed growth

⑧ The reservior site should be such that it avoids or excludes water from those Tributeries which carries a high percentage of silt in water.

Zones of Storage :-

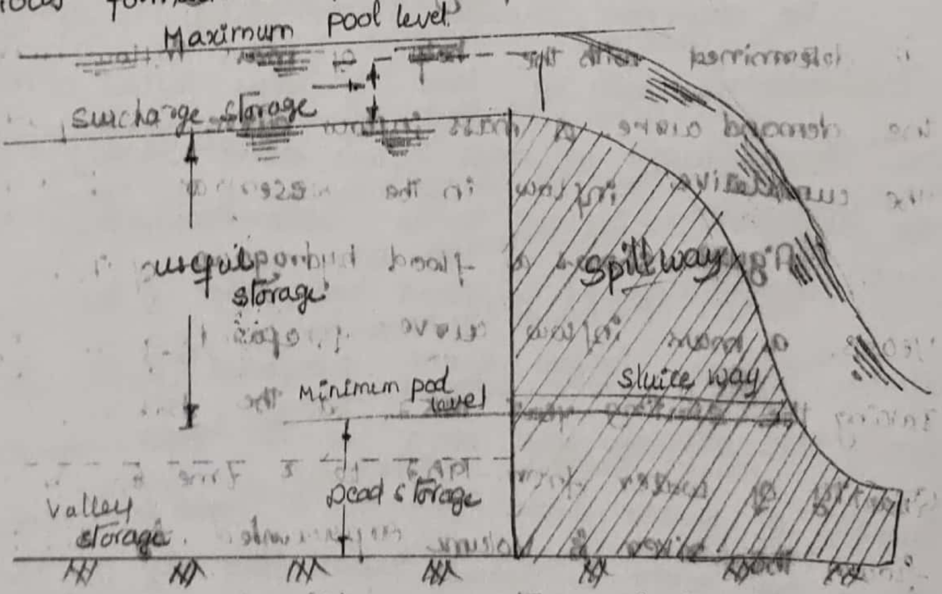
The following are the various zones in reservior

1. useful storage
2. Dead storage
3 valley storage
④ surcharge storage
5. Bank storage

The maximum level to which the water will rise in reservior during ordinary operation Condition is called normal pool level. The normal pool level is corresponding to either

r. spillway crest, or to the top level of spillway gates. The level to which water rises during the design flood is known as the maximum pool level. The lowest elevation to which the water in the reservoir to be drawn under ordinary operating conditions is known as maximum pool level. (3)

The volume of water stored between the normal pool level and maximum pool level is known as the useful storage. The volume of water below the minimum pool level is known as the dead storage. and is not useful under ordinary condition. The volume of water stored between the normal pool level and maximum pool level corresponding to a flood is called surcharge storage. The terms bank storage and valley storage are referred to the volume of water stored in the pervious formations of the river banks and the soil above it.



Zones of storage

* Storage capacity and yeild

yeild:- yeild is the amount of water that can be supplied from the reservoir in a specified interval of time. The interval of time chosen for the design varies from the day for small distribution reservior to a year for 6 ge Conservation reservior. for example, if 25000 cubic metres of water is supplied from a reservior in one year, its yeild is 25000 cubic metres/year or 2.5 hectare - metres per year.

* Safe yeild or firm yeild :— The maximum Quantity of water that can be guaranteed during a critical dry period is known as the safe yeild or firm yeild.

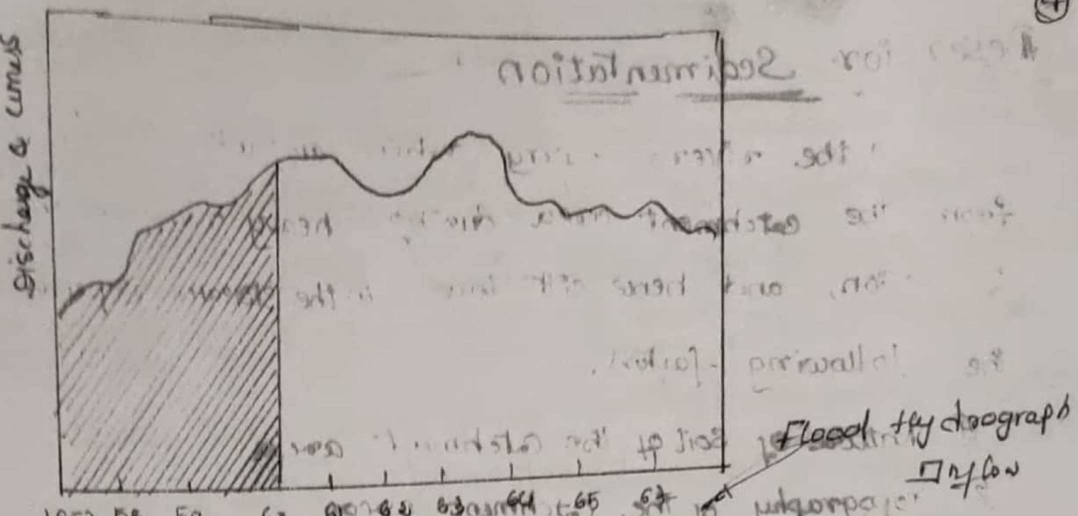
* Secondary yeild :— It is the Quantity of water available in excess of safe yeild during periods of high floods.

* Average yeild :— The arithmetic Average of the firm and secondary yeild over a long period of Time is called Average yeild.

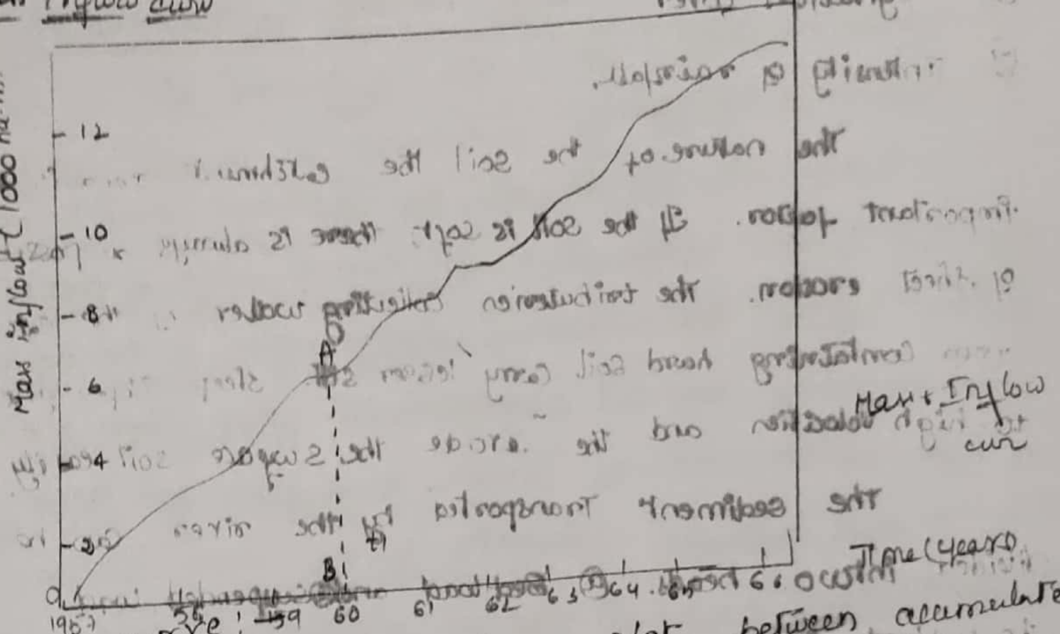
Mass Inflow curve :—

The reservior capacity corresponding to specified yeild is determined with the help of mass inflow curve and the demand curve. A mass inflow curve is a plot between the cumulative inflow in the reservior with time.

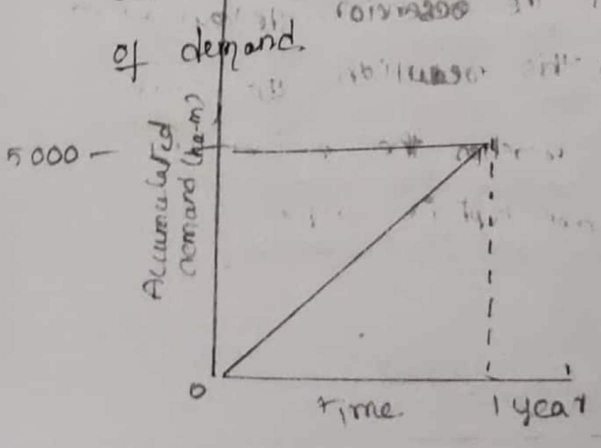
figure shows a flood hydrograp of inflow for several years. a mass inflow curve prepared from flood hydrograph Taking the starting year 1957 at the base. The Total Quantity of water from 1957 to a time t_1 that has flown the river is volume represented by the hatched area. In the mass inflow curve the corresponding ordinate at time t_1 will therefore, be Equal to the volume of water indicated by the hatched area. similarly the ordinates of the mass inflow curve corresponding to other can be computed from A fig. and plotted

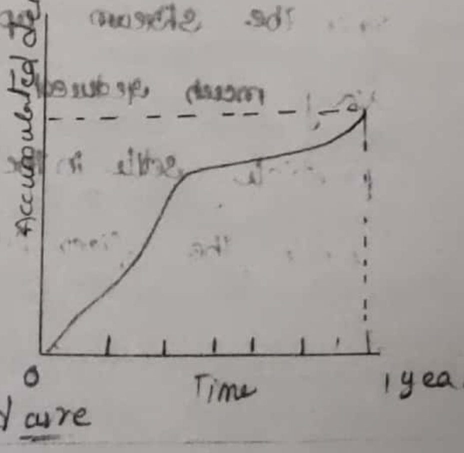


Mass inflow curve



Demand curve: A demand curve is a plot between accumulated demand with time. The demand curve representing a uniform rate of demand is a straight line having the slope equal to the demand rate. A demand curve may be curved also, indicating variable rate of demand.





Demand curve

Reservior Sedimentation :—

All the rivers carry certain amount of silt eroded from the catchment area during heavy rains. The extent of erosion, and hence silt load in the stream depends upon the following factors.

1. Nature of soil of the catchment area
2. Topography of the catchment area
3. vegitation cover
4. Intensity of rainfall.

The nature of the soil the catchment area is an important factor. If the soil is soft, there is always a possibility of sheet erosion. The tributaries collecting water of the catchment area containing hard soil carry lesser silt. steep slopes give rise to high velocities and the erode the surface soil easily.

The sediment Transported by the river can be divided into two heads. (a) Bed load and (b) suspended load

The bed load is dragged along the bed of the stream. The suspended load is kept in suspension load is kept in suspension because of the vertical component of the eddies formed due to friction of flowing water against the bed. The bed load is generally much smaller - 10 to 15% of suspended load.

when the stream approaches the reservior the velocity is very much reduced. Due to this reduction the coarse particle settle in the head reaches the reservior. while the finer particles are kept in suspension

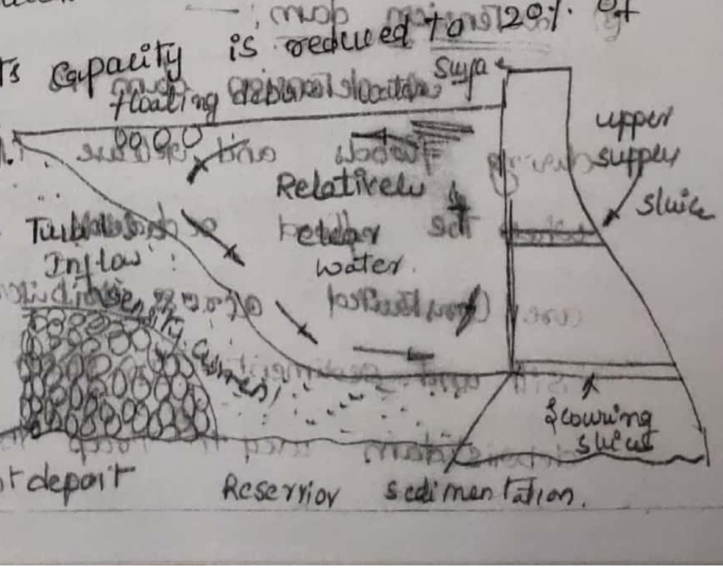
* **Density currents:-** Density current may be defined as gravity flow of fluid under another fluid of approximately equal density. In case of reservoir the water stored is usually clean and the inflow during the flood is generally muddy. The two fluids have therefore different densities. The heavy turbid water flows bottom along the channel of gravity as shown. This is known as density current.

* **Measurement of sediment load:-**

The amount of silt or the sediment load carried by the stream is determined by taking the sample of water carrying silt at various depth. The sample are then filtered and the sediment is removed and dried. The sediment load measured in the units of parts per millions parts of water (ppm).

* **Life of Reservior:-**

The ultimate density of a reservior is to be filled with silt deposits. To allow for silting, a certain percentage of total storage is usually left unutilised and is called dead storage. However the time passes on more and more silting taken place and the live or effective storage is gradually reduced. The useful life of reservior is terminated when its capacity is reduced to 20% of the designed capacity.



Reservior sedimentation.

Types of Dams

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

Classification according to use :-

① Storage Dam ② Diversion Dam ③ Detension Dam

① **Storage Dam**:— This is most common type of dam. Normally constructed to impound water to its upstream side during the period of excess supply in the river and is used in periods of deficient supply. Behind such a dam a reservior or lake is formed. The storage dam may be constructed for various purposes such as irrigation, water power, irrigation or for water supply for public health purpose or it may be for a multipurpose project.

② **Diversion Dam**:—

The purpose of a Diversion dam is essentially different. While storage dam stores water at its upstream for future use, a diversion dam simply rises water slightly in the river and thus provide head for carrying and diverting water into ditches, canals or other conveyence systems to the place of use.

③ **Detension dam**:—

A detension dam is constructed to store water during floods and release it gradually at safe rate when the flood recedes. Sometimes detension dams are constructed across tributories carrying large silt and sediment. In such cases it is known as a debris dam used to trap the sediment and thus to exclude the sediment to flow to the main reservior formed on the main river to which the Tributary meets.

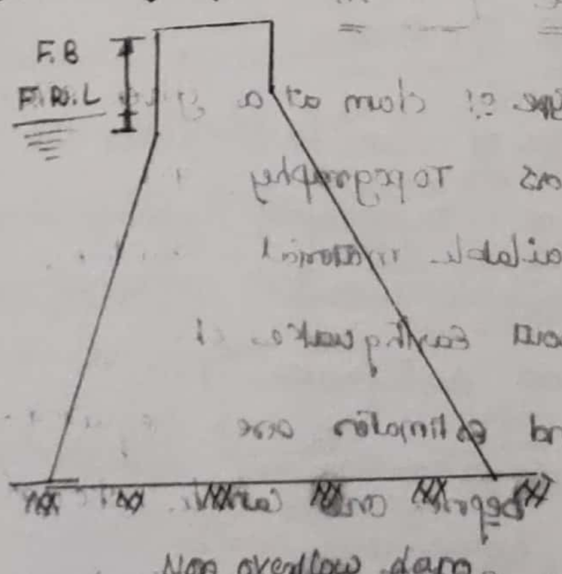
II Classification According to Design

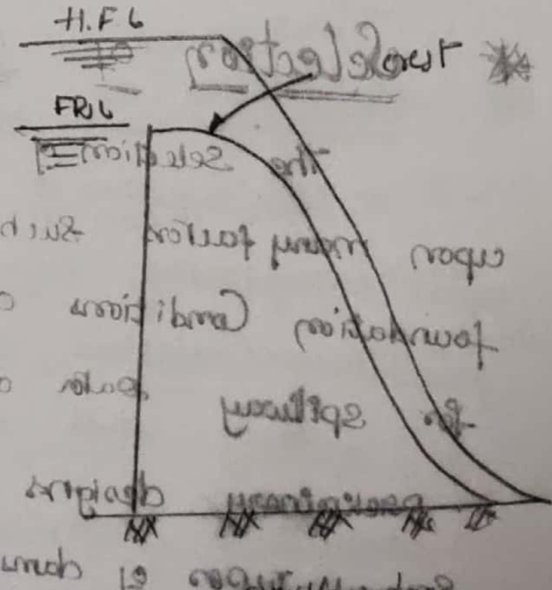
① Nonoverflow ② overflow:

1. Nonoverflow:— A non overflow dam is one in which the top of the dam is kept at a higher elevation than the maximum expected high flood level. water is not permitted to overtop the dam. Hence the non over flow dam may be constructed of wide variety of material such as earth rockfill masonry concrete.

② overflow dam:—

A overflow dam is the one in which is designed to carry surplus discharge over its crest. Its crest level is kept lower than the top of the other portion of dam. since water glides over its downstream face it should be made of such material which is not easily eroded by flowing water.





III classification According To material :

According to the most Common classification the dams may be classified as

① Rigid dams ② Non rigid dams

① **Rigid dam:** Rigid dams are those which are Constructed of rigid material such as masonry Concrete, steel or Timber

Rigid dams may further classified as

① Solid masonry or Concrete dams

② Arched masonry or "

3. concrete Buttress dam

4 Steel Dam ⑤ Timber dam.

② **Non rigid dam:**

Non rigid dams are those which are Constructed of non rigid material such as earth and/or rockfill. The most Common Type of rigid dams are

① Earthfill dam ② Rockfill dam ③ Combined Earth & rockfill dam.

* Selection of Type of Dam :

The Selection of Type of dam at a given site depends upon many factors such as Topography, geological and foundation Conditions available material suitable site for spillway Data about Earthquake etc.

preliminary designs and estimates are required for Several Types of dams before one can be shown to be the most economical. The choice of a dam may also guided by many local problems such as availability and labour and Equipment.

* Topography :— The first choice of dam is usually governed by the Topography for the site. A low rolling plains Country suggests an earth dam with a separate spillway A low narrow V-shaped valley suggests an arch dam, provide the Top width of valley is less than one-fourth of its height and separate site for spillway is available

& 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, any Type of dam Can be Constructed on it. The removal of disintegrated rock Together with the sealing of seams and fractures by grouting will frequently be necessary. Poor rock or gravel foundation for earth dam, rockfill dam or low Concrete gravity dam. silt or fine sand foundation have the problems of settlement dam or low Concrete gravity dam but not rockfill dam. Hence earth dams are suitable with foundation Treatment

* Materials of Construction :—

The next important factor is the availability of materials of construction for dam. The cost of construction of a particular Type of dam will depend upon the availability of the materials in the nearby area so that Transportation dam may be suitable. If however coarse and fine grained soils are available an earth dam may suitable.

* Spillway size and location :—

The safe discharge of flood water through dam is very essential. and for that suitable site for spillway should be available If The area is such that a large spillway capacity is required an overflow Concrete gravity dam should be preferred. where large discharge are to derived during the construction of dam. a Concrete gravity dam is preferred To an Earth dam.

* **Roadway:** If a roadway is to be passed over Top of the dam, an earth dam or gravity dam would be preferred.

* **Length and height of dam:** If the length of the dam is very long and its height is very low an earth dam would be a better choice.

* **Life of a Dam:** Concrete or masonry gravity dam have very long life. Earth and rockfill dams have intermediate life.

Selection of site for a dam:

1. **Foundations:** suitable foundation site should be available at the site selected for particular Type of dam. For gravity dam, sound rock is essential. For earth dams any types of foundation is suitable with proper Treatment.

② **Topography** :— The river cross section of at the dam site should have preferably have a narrow gorge to reduce the length of the dam. However the gorge should open out upstream to provide large basin for a reservoir

③ **Site for spillway** :— Good site for the location for a separate spillway is essential especially in the case of earthfill or rockfill dam. However, in the case of Gravity dam, spillway may be located at its middle. The best site for a dam may be considered to be one where a deep gorge and a flank at its sides are separated by hillock higher then the height of the dam

④ **Material** :— Materials required for a particular type of dam should be available nearby, without requiring much of transportation. This would very much reduce the cost of construction.

⑤ **Reservior and Catchment Areas** :—
(i) The side should ensure adequate storage capacity of reservior basin at a minimum cost.
(ii) The cost of land and property submerged in the water spread area should be minimum.
(iii) The reservior site should be such that quantity leakage through its side and the bed is minimum.
(iv) The geological condition of the catchment area should be such that percolation losses are minimum and maximum runoff is obtained.
(v) The reservior site should be such that it avoids or excludes water from those tributorities which carry a high percentage of silt in water

6. **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 machinery, food and other equipment.

7. **Locality**:- The surroundings near the site should preferably be healthy and free of mosquitoes etc. as labour and staff colonies have to be constructed near the site.

Gravity Dams

A gravity dam is a structure so proportioned that its own weight resists the forces exerted upon it. This type of dam is the most permanent one, requires little maintenance and is most commonly used.

Forces acting on a gravity dam :—

Following are the forces acting on a gravity dam

1. water pressure
2. uplift pressure
3. weight of the dam
4. pressure due to earthquake
5. Ice pressure
6. wave pressure
7. silt pressure
8. wind pressure.

1. **Water pressure :—**

This is the major external force acting on a dam. when the upstream face of the dam is vertical, the water pressure acts horizontally. The intensity of pressure varies triangularly with a zero intensity at the water surface to a value wh at any depth h below the surface. when the upstream face is partly vertical and partly inclined.

Intensity of the earthquake

The intensity of an earthquake at a place is a measure of the strength of shaking during earthquake and is indicated by a number according to the modified mer calle scale or M.S.K. scale of seismic intensities.

(5) ICE pressure:—

The ice pressure is more important for dams constructed in cold countries. The ice formed on the water surface of the reservoir is subjected to expansion and contraction due to temparature variations. The ~~coefficient~~ of thermal expansion of ice being five times ~~more than that~~ of concrete, the dam face has to resist the force due to expansion of ice. This force acts linearly along the length of the dam. at the reservior level.

(6) Wave pressure:—

Waves are generated on the reservior surface because the wind blowing over it. Wave pressure depends on the height of the waves developed. wave height may be calculated from the following formula

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

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

where h = height of waves in metres between Trough and crest.

V = wind velocity in km per hour

F = Fetch or straight length of water expanse in km.

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⑦ **Silt pressure :—**

The river brings debris and silts along with it. The silt load gets deposited to an appreciable extent when dam is constructed. If γ_s' is the submerged unit weight of silt and ϕ is the angle of internal friction. and h_s is the height to which the silt is deposited.

The silt pressure is given by $P_s = \frac{1}{2} \gamma_s' h_s^2 \frac{1-\sin\phi}{1+\sin\phi}$

If the upstream face is inclined the vertical weight of silt supported on the slope also act as vertical force.

⑧ **Wind pressure :—**

It is a minor force and 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 on the action of wind. Normally wind pressure is taken as 1 to 1.5 KN/m^2 for the area exposed to the wind pressure.

* **Modes of failures : Stability Requirements**

Following are the modes of failures of Gravity Dam

① overturning ② sliding ③ Compression or crushing ④ Tension

1. **overturning :—** The overturning of the dam section takes place when the resultant force at any section cuts the base of the dam downstream of the toe. In that

case the resultant moment at the Toe becomes clockwise or.-ve). on the other hand if the resultant cuts the base within the body of the dam. there will be no overturning.

For stability requirements the dam must safe against overturning. The factor of safty against overturning is defined as the ratio of the resultant movement (+ve) of the overturning moments i.e

$$f.s = \frac{\Sigma \text{ Righting moments}}{\Sigma \text{ overturning moments}} = \frac{\Sigma M_R}{\Sigma M_o}$$

The factor of safty against overturning should not be less than 1.5.

② Sliding :— a dam will fail in sliding at its base or at any other level if horizontal forces causing sliding are more than the resistant available at that level. The horizontal resistance against sliding may due to friction alone, or due to friction and shear strength of the joint. Shear strength develops at the base if benced foundation are provided and at other joints are carefully bid so that a good bond develops.

The factor of safty against sliding is defined as the ratio of actual coefficient of static friction (μ) on the horizontal joint to the sliding friction.

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

and factor of safty against sliding (F.S.S) is

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

The coefficient of friction μ varies from 0.65 to 0.75.

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③ **Compression or crushing**

In order to calculate the normal stress distribution at the base or at any section, let H be the total horizontal force & V be the total vertical force, & R be the resultant force cutting the base at an eccentricity e from the centre of the base width b.

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

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

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

Hence the Total normal stress P_n is given by

$$P_n = \frac{V}{b}\left[1 \pm \frac{6e}{b}\right]$$

The positive sign will be used for calculating normal stress at the Toe, since the bending stress will be compressive force and negative sign will be used for calculating normal stress at heel.

Thus, the normal stress at the Toe is

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

The normal stress at the heel is

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

④ Tension: —

The normal stress at heel is

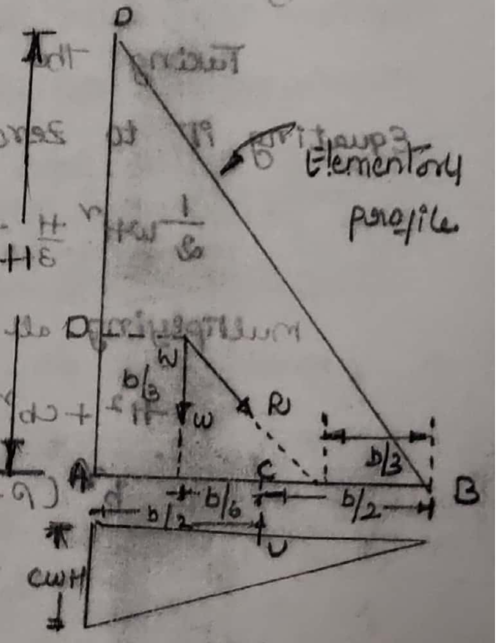
$$(P_n)_{heel} = \frac{V}{b}\left[1 - \frac{6e}{b}\right]$$

It is evident that if $e > b/6$ the normal stress at heel will be −ve or Tensile.

No Tension should be permitted at any point of the dam under any circumstances for moderately high dams. For no Tension develop, The eccentricity should be less than $b/6$. In other words the resultant should always lie within the middle third.

✳ Elementary profile of Gravity Dam: —

In the absence of any force other than the force due to water an elementary profile will be triangular in section. having zero width at the water level at the top, where water pressure is zero and maximum base width b, where maximum water pressure acts. Thus the section of the elementary profile is of the same shape as the hydrostatic pressure distribution diagram. For Reservoir empty condition a right angled triangular profile as shown in fig will provide the maximum profile.



Elementory profile

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

① weight of the dam (W)

$$W = \tfrac{1}{2} bH . \rho . w$$

ρ = specific gravity of dam material; w = unit weight of water

(2) water pressure (P)

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

(3) uplift pressure (U)

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

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 :- when the reservoir is empty for no tension to develop the resultant should act at the inner third point (M_1). For the reservoir full condition for no tension to develop, the resultant R must pass through the outer third point (M_2).

Taking the moment of all forces about M_2 and Equating it to zero we get

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

multiplying all terms by $\frac{6}{wH}$ we get

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

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

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

Limiting Height of a Gravity dam :—

The principal stress at the Toe is given by

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

In this expression the only variable, changing the value of σ_1 is H. The maximum value of this principal stress should not exceed the allowable stress. (f). for the material.

In the limiting case

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

From which the height H is given by

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

For finding the limiting H. but it is usual not to Consider the uplift. Hence putting $C = 0$ we get.

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

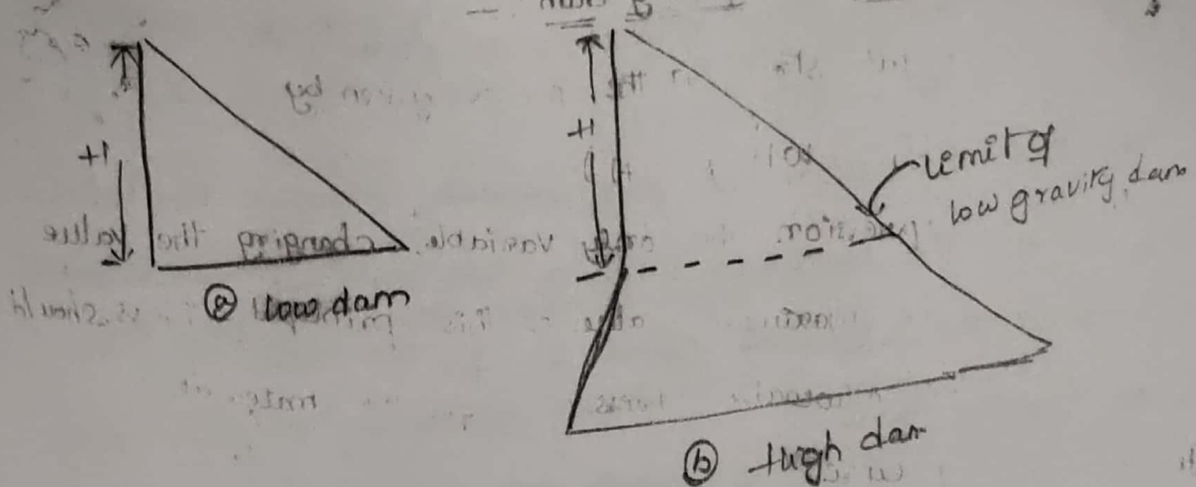
If the height of the dam is more than that given by Eq. The maximum compressive stress will exceeds the permissible stress

A low gravity dam is the one in which the height defines H is less then that given by so that maximum compressive stress is not greater then the allowable stress. For general case. Taking $w = 9.81 \ kN/m^3$ and $\rho = 2.40$. the limiting Height in metres is given by

$H = 0.03 f$ where f is the allowable stress in kN/m^2

$f = 2940 \ kN/m^2$

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

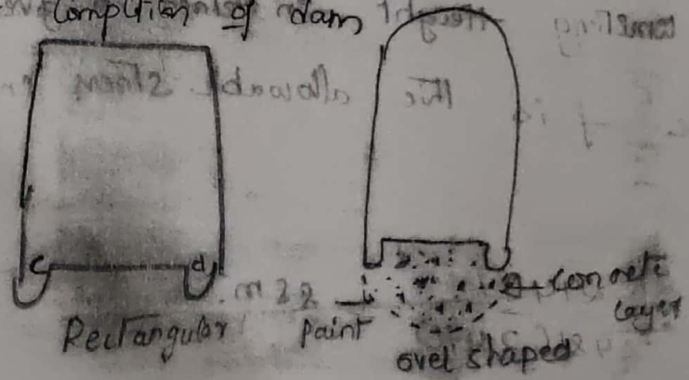


High & Low Gravity Dam.

(*) Drainage Galleries :—

A gallery is a formed opening left in a dam. This may run in Transverse or longitudinal direction and may run horizontally or on a slope. The shape and size varies from dam to dam. and is generally governed by the fluctuations it has to perform. Following are the purpose for which a gallery is formed on a Gravity dam

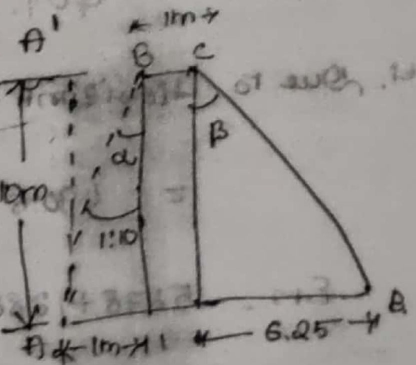
1. To provide drainage of the dam Section. Some amount of water Constantly seeps through the upstream face of the dam
2. To provide facilities for drilling and grouting operations for foundations etc. Drilling for drains is generally resorted to clean them if they are clogged.
3. To provide space for header and return pipes for post cooling of Concrete and grouting the longitudinal Joints after Completion of dam



Unit Gravity dams

1) A masonry dam 10m high is Trepizoidal in section with a top width of 1m and bottom width 8.25m. The face is exposed to water has a batter of 1:10. Test the stability of dam. Find the principal stress at the Toe and heel of the dam. Assume unit weight of masonry as 22.4 KN/m². w for water = 9.81 KN/m³ and permissible shear stress of Joint = 1400 KN/m².

Sol Consider, 1m length of the dam



1. vertical forces :—

a. Self weight of the dam

$\frac{a+b}{2} \times H \times$ unit length $\times 22.4$

$= \frac{8.25+1}{2} \times 10 \times 1 \times 22.4$

$= 1036$ KN

(b) weight of water in Column $AA'B$

$= \frac{10 \times 1}{2} \times 9.81 = 49.05$ KN.

(c) uplift force $= \frac{1}{2} B H C w$

$= \frac{1}{2} \times 8.25 \times 10 \times 9.81 = 404.66$ KN

$\Sigma V = 1036 + 49.05 - 404.66 = 680.39$ KN

(2) Horizontal water pressure :—

$\Sigma H = \frac{wh^2}{2} = \frac{9.81 \times 10^2}{2} = 490.05$ KN.

Moment due to various forces about Toe :—

(a) Due to self weight of Dam

Moment $= \left(\left(\frac{1}{2} \times 1 \times 10 \times 22.4\right) \times 7.2\right.$

$+ (1\times10\times22.4)(6.25+0.5) + \frac{1}{2}\times 6.25\times10\times22.4)(\frac{2}{3}\times6.25)$

$= 5278$ KN-m

2. Due to column of water in AA_1B

Moment $= \frac{1}{2}\times10\times1\times9.81 \times (8.25 - 1/3) = 388.31$ KN-m

3. Due to uplift force

Moment $= 404.66\times 2/3 \times 8.25 = 2225.63$ KN. m

4. Due to Horizontal water pressure.

$= 490.5 \times 10/3 = 1635$ KN/m.

$\Sigma M = -5278 + 388.31 - 2225.63 + 1635$

$= 1805.68$ KN-m

Calculation of factor of safety

Factor of safety against overturning

$= \frac{\Sigma M_R}{\Sigma M_O} = \frac{(+) M}{-(M)} = \frac{5668.31}{3860.63} = 1.47 < 1.5$ unsafe

Factor of safety against overturning

$= \frac{\mu \Sigma V}{\Sigma H} \quad \mu = 0.75. \quad = \frac{0.75\times680.39}{490.5} = 1.04 > 1$

Shear friction factor $= \frac{\mu \Sigma V + bS}{\Sigma H} = \frac{0.75\times680.39 + 8.25\times1400}{490.5}$

$= 24.6$

Calculation of Stress :—

The resultant acts at a distance x from toe,

$x = \frac{\Sigma M}{\Sigma V} = \frac{1805.68}{680.39} = 2.65$m

~~Calculation of~~

Its distance from Centre is $e = b/2 - \bar{x}$

$= \frac{8.25}{2} - 2.65 = 1.475\ m$

Compressive stress at Toe is

$P_n = \frac{\Sigma V}{b}\left[1 + \frac{6e}{b}\right] = \frac{680.39}{8.25}\left[1 + \frac{6(1.475)}{8.25}\right]$

$= 167.8\ kN/m^2 \cdot (+ve)$

Compressive stress at heel

$\frac{\Sigma V}{b}\left(1 - \frac{6e}{b}\right) = \frac{680.39}{8.25}\left[1 - \frac{6.(1.475)}{8.25}\right]$

$= -2.9\ kN/m^2$ ie Tension

From fig: -

$\tan\alpha = \frac{1}{10}$ $\tan\beta = \frac{6.25}{10}$

$\sec\alpha = \sqrt{1.01}$ $\sec\beta = \sqrt{1.391}$

Principle stress at Toe of dam $= P_n \sec^2\beta = 167.8 \times 1.391$

$= 233.41\ kN/m^2$

Principle stress at heel

$P_n \sec^2\alpha - p\tan\alpha = -2.91 \times 1.01$

(0.01)

$= -2.91 \times 1.01 - 9.81 \times 10 \times 0.01 = -3.91\ kN/m^2$

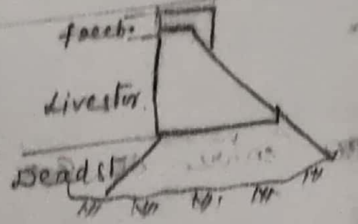
Shear stress at Toe $= \tau = P_n \tan\beta = \frac{167.8 \times 6.25}{10}$

$= 104.9\ kN/m^2$

Shear stress at heel $= -[P_n - p]\tan\alpha$

$= -(-2.91 - 9.81) \times 1/10 = 10.1\ kN/m^2$

General Terms



1. **High flood level:**— The maximum water Level that attains during flood is called high flood Level. The dam and spillway sections are designed to withstand the water pressure at this level is called as "Maximum water Level".

2. **Full Reservior level:**— It is also Called as full Tank Level (F.T.L). It is the Level upto which the water is stored. Obviousley the crest of the spillway is fixed at this Level.

3. **Free Board:**— To prevent the over topping of the dam during peak floods a sufficient margin is provided between the maximum water Level in the reservior and top of the dam.

① **Gross free Board:**— It is the difference of Level b/w F.R.L and Top of the dam.

② **Net free Board:**— It is the difference of Level b/w M.W.L and top of the dam.

④ **Dead storage:**— It is the part of the stored water in the reservior Basin which is not available for use and hence Termed as Dead storage. The Capacity of dead storage is so fixed that it Can allow the silting for about 100 years without reduction in the effective storage.

⑤ **Live storage** It is also Called the available or Effective storage. It is difference b/w. the Gross storage and Dead storage.

Live storage = Gross storage − Dead storage.

Irri 2

Gravity Dams.

1) Gravity Dam is a major hydraulic structure constructed across a river/reservior, with the purpose of storage of that water for about 5-6 months time, ~~if~~ with a storage head with a head of 100m - 200m, so that the stored water can be utilized for the purpose of irrigation & power production at a convenant time in conveniant quantities

2) profiles of gravity Dam :-

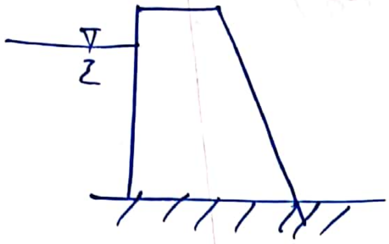
a) common profile

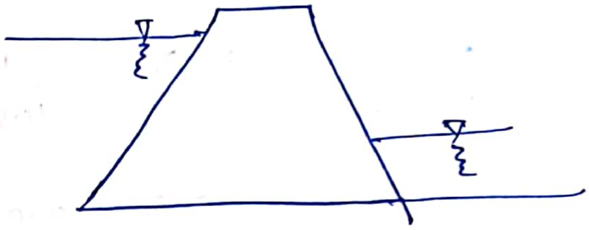
b) practical profile

c) Elementary profile

a) common profile:-

→ one face is essentially vertical & the other face is essentially inclined.





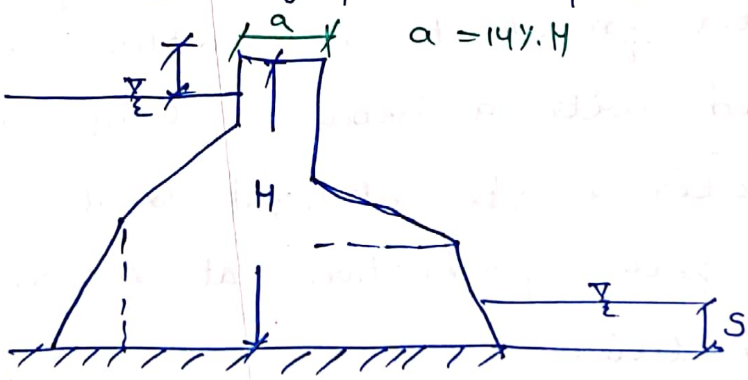
Trapezium c/s.

→ The shape of the c/s of the ~~p~~ common profile is a Trapezium

b) practical profile:-

→ practical profile for full ht. (or) part of the ht., if both upstream & downstream faces are inclined, it is a practical profile.

→ creager & mynotto has given 3 considerations to be implemented regarding practical profile.



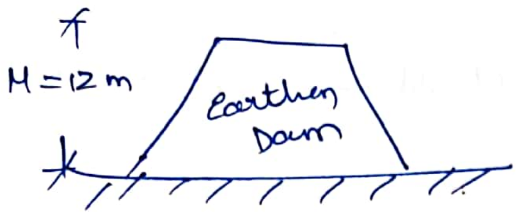
a) Top width (a) = 14% H.

b) Free board (FB) = 0.9 m (or) 1.5 h_w } which ever is greater

c) Any geometrical dimensions, Horizontal, vertical, inclined

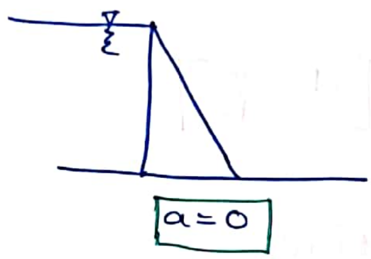
Function $\propto$ (H)

Note:- All the earthen dams will have both the faces inclined & hence earth dam is an example of practical profile



c) Elemantary profile :-

→ It is the common profile with a top width zero, therfore c/s of a elemantary profile is a right-angled Δ'e.

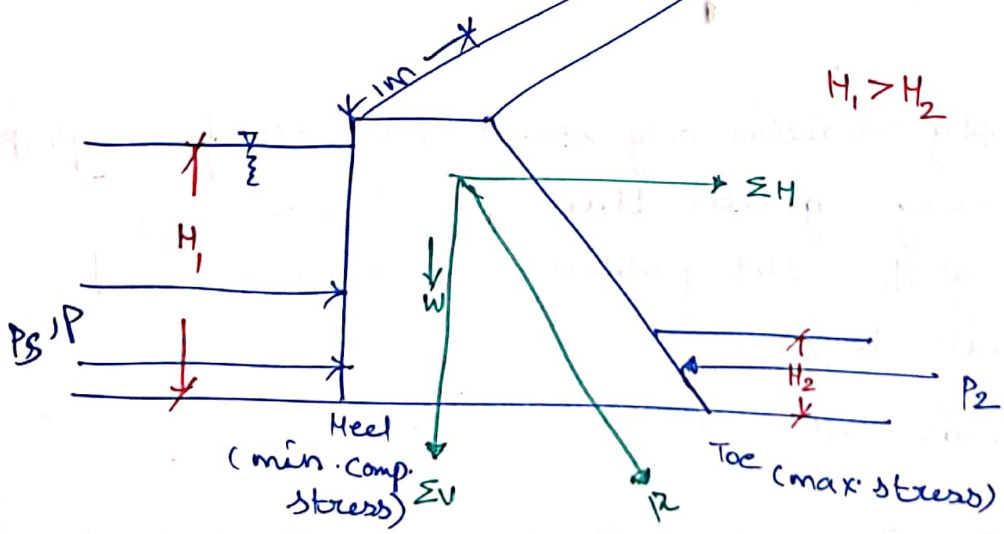


operating conditions of gravity dam :-

1. Reserviour full condition
2. Reserviour Empty condition

1. Reserviour full condition :-

→ upstream side water depth is greater than downstream side water depth. ($H_1 > H_2$)



→ In reserviour full cond. ΣH toward right, ΣV vertically down

→ The resultant force in the 4th co-ordinate

→ Its line of action cuts the base near the toe, thus toe will be the pt. of max. comp stress

$$(P_{max})_{toe} = \text{direct stress } (\sigma_d) + \text{bending stress } (\sigma_b)$$

$$= \frac{\sigma_v}{b}\left[1 + \frac{6e}{b}\right]$$

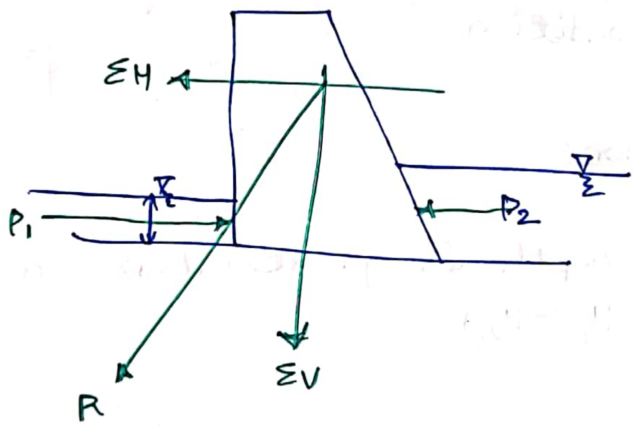
Heel will be pt. of min. comp. stress

$$(P_{min}) = \text{direct stress} - \text{bending stress}$$

$$= \frac{\sigma_v}{b}\left[1 - \frac{6e}{b}\right]$$

2. Reservioer empty condition:-

→ H_1 must be essentially less than H_2. $(H_1 < H_2)$



→ In reserviour empty condition only ~~zero~~ hydrostatic forces P_1, P_2 will act, P_2 being greater than P_1, because of negligible value of H_1, slit pressure force (P_s) & wind pressure (P_w) will be zero

→ ΣH acts towards Left, ΣV vertically down

→ Resultant force (R) = cuts the base near the heel, has the resultant lies in 3^{rd} quadrant.

→ Heel will be subjected to max. stress & toe will be subjected to min. stress.

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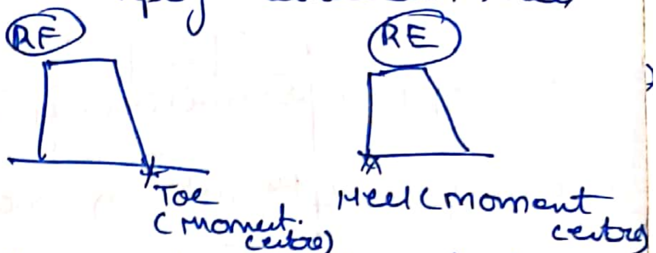
Note!-

→ In reservior full condition, Toe will be taken as the moment centre, and in reservior empty condition, heel will be the moment centre.



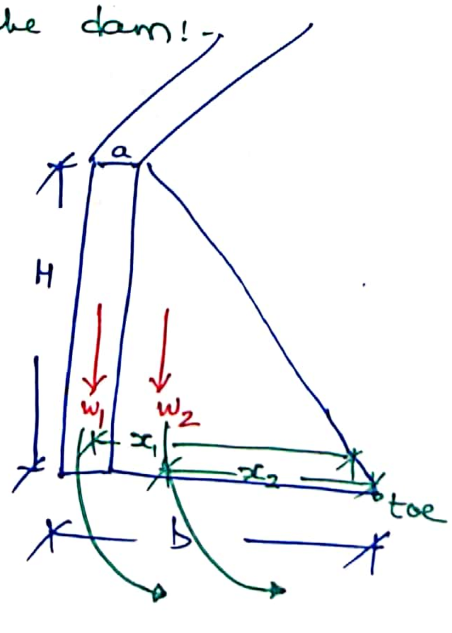
→ The Location of the pt. of max. comp. stress, should be choosen as moment centre

Forces acting on gravity dam:-

1. self wt. of the dam (W)
2. Hydro-static pressure on up-stream site (P_1)
3. Hydro-static force on down-stream side (P_2)
4. uplift Force (U)
5. Silt pressure force (Ps)
6. wave pressure Force (Pw)
7. Ice pressure force (Pi)
8. Earth quake force (Peq)

Zone 3, Zone 4, Zone 5 (Danger zone) (dangerous)

1. Self wt. of the dam:-



$W_1 = \gamma_c \cdot a \times H \times 1$

$= S w a H$

acting at $x_1 = b - \frac{a}{2}$ from toe

$W_2 = \gamma_c \cdot \frac{1}{2}(b-a) \times H \times 1$

$= \frac{1}{2} S w (b-a) H$

acting at $x_2 = \frac{2}{3}(b-a)$ from toe

$W = w_1 + w_2$

$M_W = w_1 x_1 + w_2 x_2.$

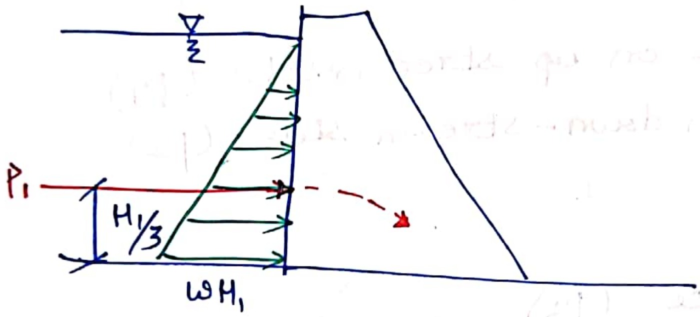
$\omega = 1000\ kgf/m^3$
$= 9810\ N/m^3$
$= 10\ kN/m^3$
$= 1\ t/m^3$

	ω
nature of force =	stabilizing force
nature of moment =	Righting moment.

ω = unit wt. of water.
γ_c = unit wt. of concrete

2. Hydrostatic pressure force on U/S :- (P_1)

(i) U/S face vertical :-



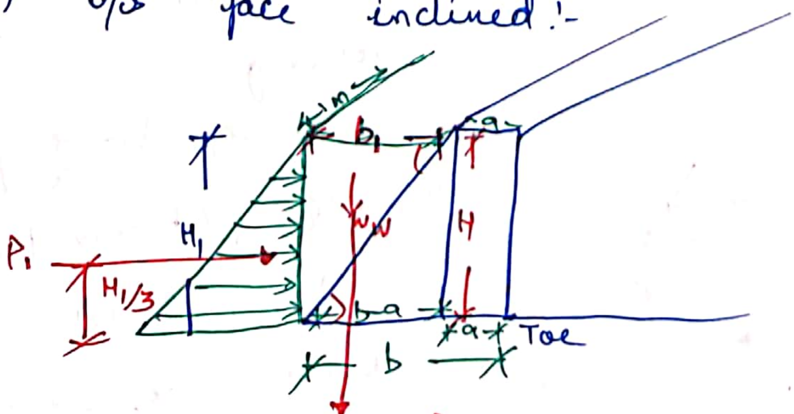
P_1 = Area of hydro static pressure distribution diagram.

$= \frac{1}{2}\omega H_1^2.$

$M_{P_1} = P_1 \frac{H_1}{3}$

$$\boxed{M_{P_1} = \frac{\omega H_1^3}{6}.}$$

(ii) U/S face inclined :-



$P_1 = \frac{\omega H_1^2}{2}$

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$$\boxed{P_1 = \frac{wH_1^2}{2}}$$

W_w = wt. of water entrapped by wedge

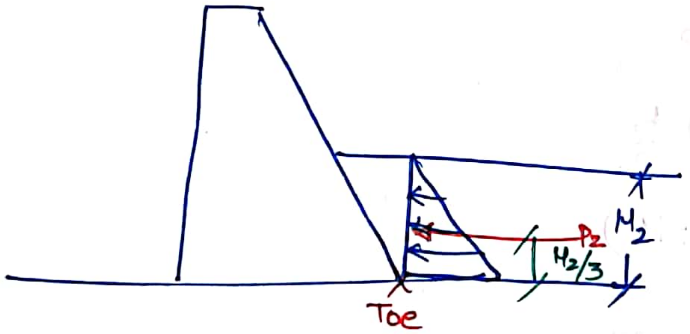
$$W_w = w \times \frac{1}{2} \times b_1 \times H_1 \; (1)$$

$$X_w = b - \frac{b_1}{3} \text{ (from toe)}$$

$$\frac{H}{b-a} = \frac{H_1}{b_1} \quad \text{(from similar } \Delta\text{le)}$$

Nature of force	destabilising force
Nature of moment	overturning moment.

3. Hydrostatic force on down-stream side (P_2)



P_2 = Area of D/S of Hydro-static pressure dist. diagram

$$P_2 = \frac{wH_2^2}{2}$$

$$M_{P_2} = P_2 \times \frac{H_2}{3} \rightarrow \boxed{\frac{wH_2^2}{6}}$$

P_2

Nature of Force	stabilizing force
Nature of momt	Righting force

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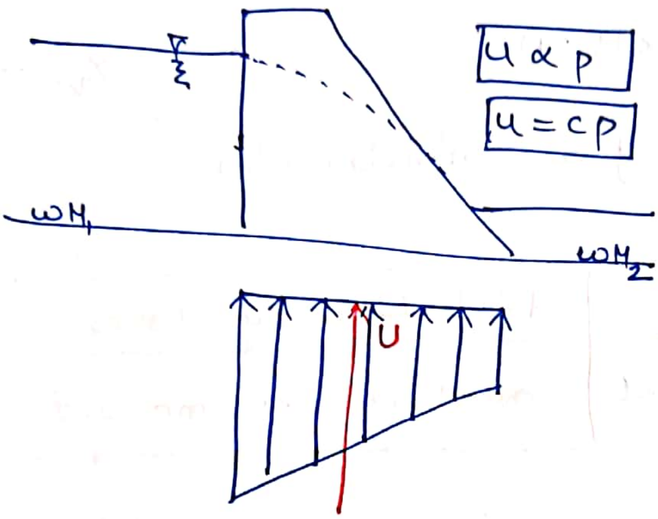
4. uplift force (U) :-

(i)

b = base width
c = uplift pressure coef
=
H_1, H_2 = head of water at u/s & d/s



$\boxed{u \propto p}$

$\boxed{u = cp}$

$\boxed{c = 0 \text{ to } 1}$

U = uplift force

= Area of UPDD

$= (cwH_1 + cwH_2)\frac{b}{2}$

$U = \frac{1}{2} bcw\,(H_1 + H_2)$

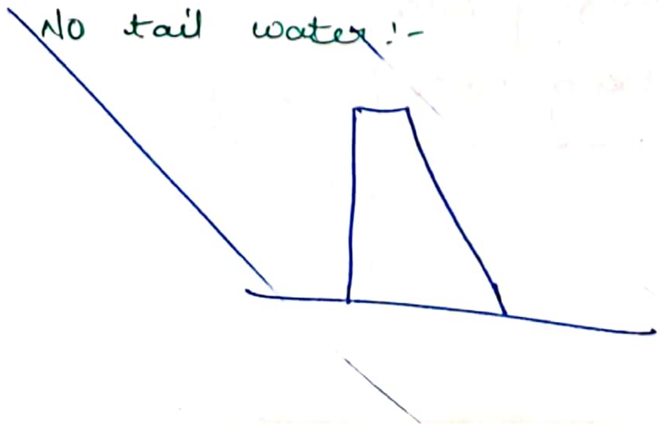
$x_U = \frac{b}{3}\left[\frac{2H_1 + H_2}{H_1 + H_2}\right]$

$M_U = U x_U$

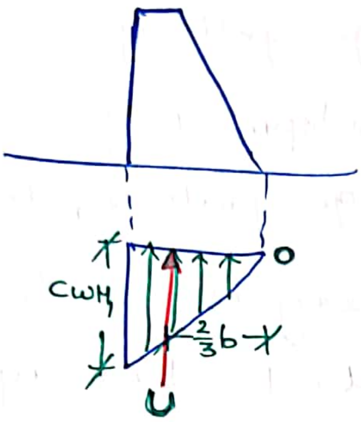
$= \frac{1}{2} bcw\,(H_1 + H_2)\,\frac{b}{3}\left(\frac{2H_1 + H_2}{H_1 + H_2}\right)$

$= \frac{1}{6} b^2 cw\,(2H_1 + H_2)$

(ii) when No tail water :-



(ii) No tail water:-



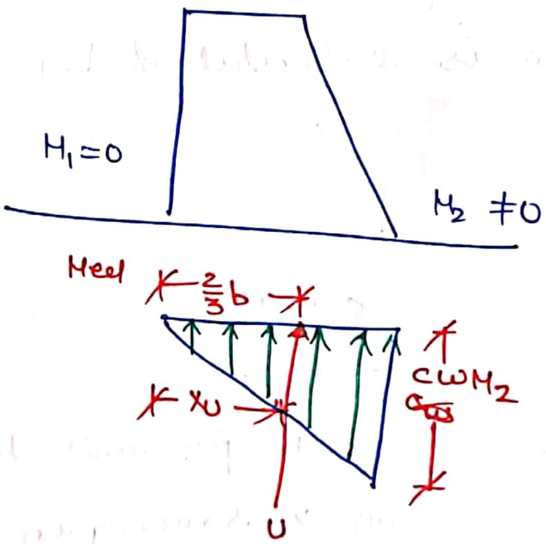
$H_2 = 0$

$U = \frac{1}{2} \times bcwH_1$

$$\boxed{\begin{aligned} M_U &= U x_U \\ &= \frac{1}{3} b^2 c\cdot w H_1 \end{aligned}}$$

about toe

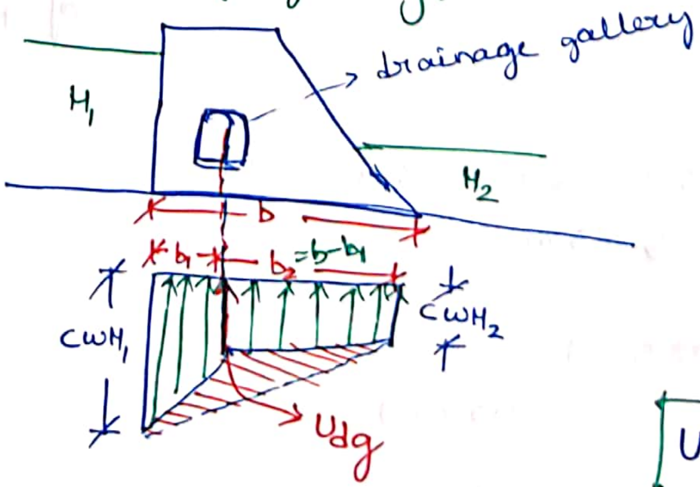
(iii) Reservoir empty condition:-



$U = \frac{1}{2} bcwH_2$

$\boxed{M_U = \frac{1}{3} b^2 cwH_2}$ → about Heel.

(iv) where there is a drainage gallery:-



$U_{dg} = cwH_2 + \frac{1}{3} cw(H_1 - H_2)$

$= cw\left[\frac{H_1 + 2H_2}{3}\right]$

$\boxed{U = U_1 + U_2}$

when there is.

→ Inspection gallery (or) drainage gallery is provided

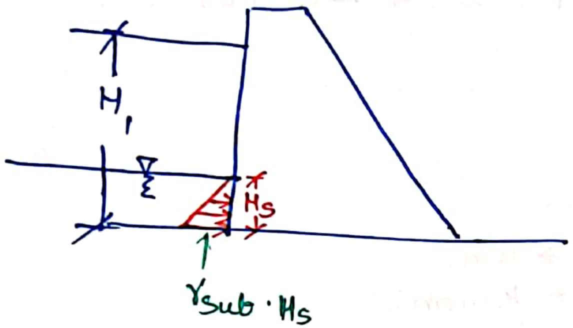
1) To check for the development of any cracks, development of seepage failure, leakages, etc.

2) when drainage gallery is provided self wt. will be reduced by 5 to 7%. but uplift force can be reduced by around 30%, Therefore provision of drainage gallery in a gravity dam is a highly desirable phenomena.

3) Intensity of uplift pressure at the location of the gravity dam is calculated by IS-code provision.

Eg:-

5. Silt pressure Force [PS]



$P_S = [\text{Area of SPDD}] K_a$

$= \frac{1}{2} \times \gamma_{sub} \cdot H_S \cdot H_S \cdot K_a$

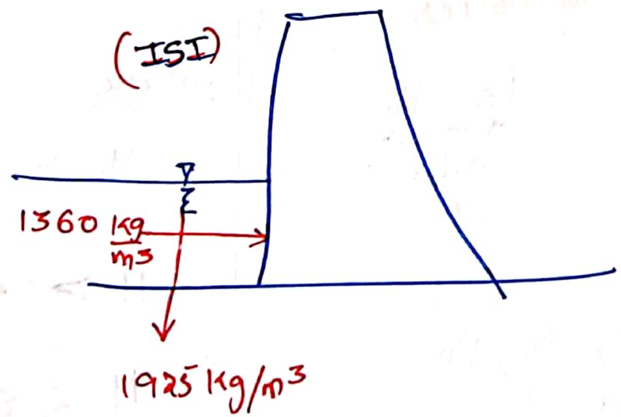
$$P_S = \left(\frac{\gamma_{sub} H_S^2}{2}\right) K_a.$$

It acts at $\frac{H_S}{3}$ about toe

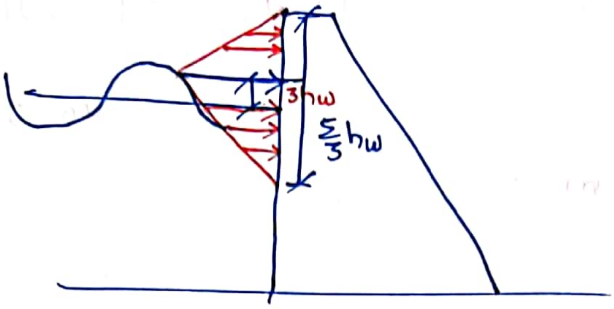
Nature of Force = Destabilising force.

Nature of moment = overturning moment.

b) If silt data is not given to you,



6. Wave pressure Force [Pw]:-



Fetch ⇒ $\boxed{F > 75\text{ km}}$ → generally for indian reservoir.
↳ given by molitor.

① $F < 32$ Km.

$$h_w = 0.032\sqrt{VF} + 0.263 - 0.761F^{1/4}.$$

② $F > 32$ Km

$$h_w = 0.032\sqrt{VF}$$

F → Km.
V → kmph.

$$P_w = 2.4\,w h_w$$

$$P_w = \frac{1}{2}(2.4\,w h_w)\frac{5}{3}h_w$$

$$\boxed{P_w = 2\,w h_w^2}$$

$$M_w = P_w\left[H_1 + \frac{3}{8}h_w\right]$$

Note:-

consideration of forces

Cp → profile	P_1, P_2, W, U (Major)	P_s P_w (minor)
PP	"	"
EP	only P, W, U	
High dam	only P_1, P_2, W, U, P_s, P_w	
Low dam	only P_1, P_2, W, U	

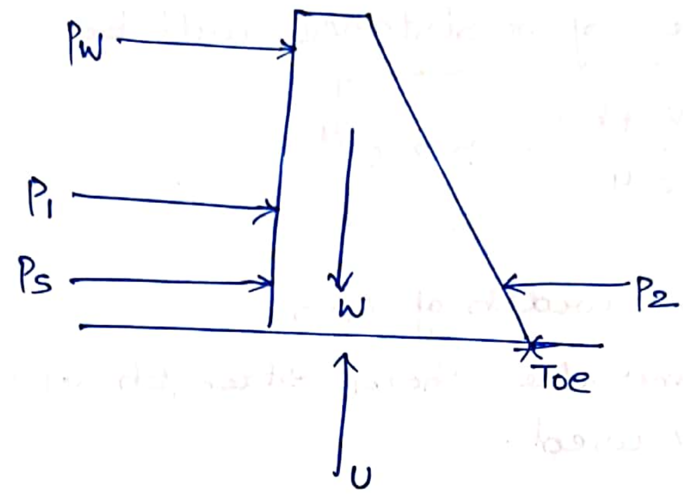
where

$$H_L = \frac{f}{w(S_c+1)}$$

$H > 88$ m high

$H < 88$ m Low gravity Dam.

Analysis of the forces on gravity Dam



$\Sigma H = P_1 - P_2 + P_s + P_w$

$\Sigma V = W - U$

$F_R = \mu \Sigma U$

$F_S = \Sigma H.$

$$\Sigma M_R = W_1 x_1 + W_2 x_2 + P_2 \cdot \frac{H_2}{3}$$

$$\Sigma M_o = P_1 \frac{H_1}{3} + P_s \cdot \frac{H_s}{3} + P_w \left(H_1 + \frac{3}{8} h_w \right) + U(x_U)$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{\Sigma M_R - \Sigma M_o}{W - U}$$

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

Modes of Failures:-

A gravity dam may fail due to any one (or) more of the following.

1) **Sliding Failure** :- Two factors of safety are defined

a) **FSS**:- (Factor of safety against sliding)

$$F_{SS} = \frac{F_R}{F_S}$$

F_R = force of resistance
F_S = " " sliding.

$$\boxed{F_{SS} = \frac{\mu \Sigma V}{\Sigma H} > 1.0}$$

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b) SFF :- (shear friction factors)

shear strength of the Joints is taken into consideration. The additional force of resistance will be

$$\boxed{SFF = \frac{\mu \Sigma V + bq}{\Sigma H} > 2.0}$$

where b = base width of G.D
q = permissible shear strength of concrete mix used.

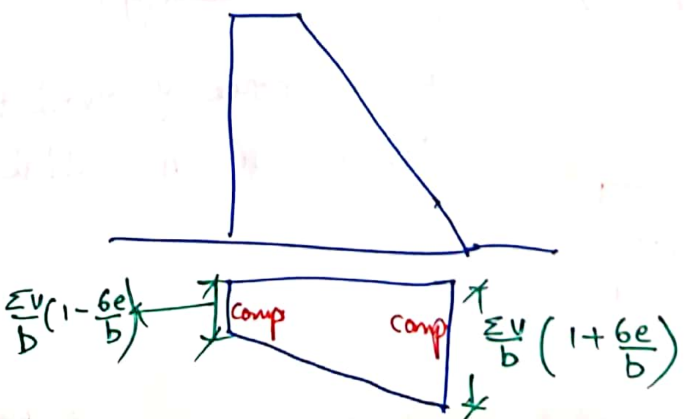
Note:- A gravity dam is said to be safe against sliding as Long as SFF exceeds 2, even though some times SFF < 1

S.FF < 2 = Dam in danger regarding sliding Failure.

2) overturning Failure:- A G.D is said to be safe against O.T, if the factor of safety against overturning exceeds 1.5.

$$\boxed{F.S.O.T = \frac{\Sigma M_R}{\Sigma M_O} > 1.5}$$

3) crushing failure:-



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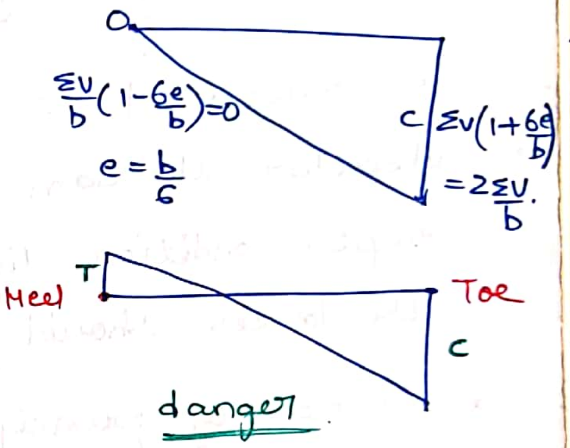
A dam is said to in the state of failure regarding crushing, when the max. comp. stress developed at the toe exceeds permissible comp. strength of the mix used.

→ For safety aganist compression

$$\boxed{P_{max} \le f}$$

$$\boxed{\frac{\Sigma V}{b}\left[1+\frac{6e}{b}\right] \le f}$$

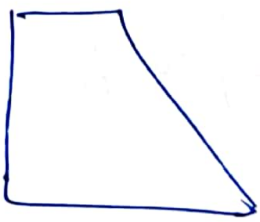


4) <u>Tension Failure</u>:-

→ Tensile stresses are not permitted to get developed at any pt. along the base of the dam. To be safe aganist tension, The min. stress at the heel of the dam should be +ve in nature.

→ compression = +ve.
Tension = −ve.

→ safest stress diagram at the base is compression at both toe & heel.



→ If the tensile ~~for~~ stresses are not to be developed

$$\boxed{P_{min} \ge 0}$$

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$$\frac{\Sigma V}{b}\left[1-\frac{6e}{b}\right] \geq 0$$

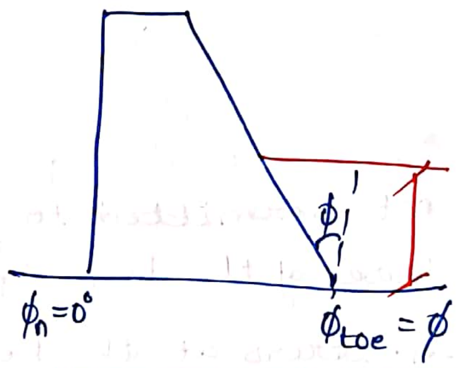
$$1 \geq \frac{6e}{b}$$

$$\Rightarrow \boxed{e \leq \frac{b}{6}} \longrightarrow \text{condition for "No Tension"}$$

→ This condition of no tension is called middle third rule.

→ wheather the dam is reserviour full condition (or) empty condition, light of action of the resultant of all the forces should fall in the middle $1/3^{rd}$ of the base.

calculation of principle stress & shear stress :-



principal stress $\boxed{(\sigma) = p_n \sec^2\phi - p \tan^2\phi}$

t = toe
h = heel.

$$\sigma_{toe} = (p_n)_t \sec^2\phi_t - p_t \tan^2\phi_t$$

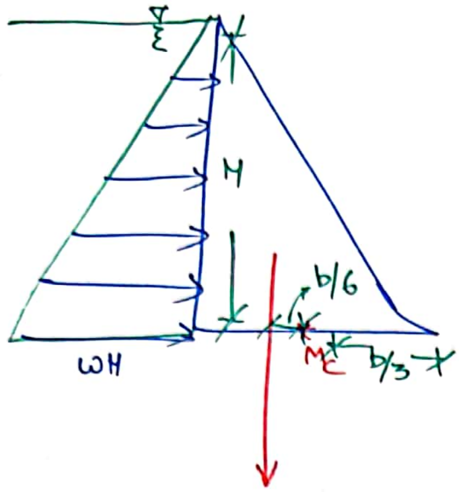
$$\sigma_{heel} = (p_n)_h \sec^2\phi_h - p_h \tan^2\phi_h$$

$$\tau = (p_n - p) \tan\phi$$

$$\tau_{toe} = \left[(p_{n})_t - p_t\right] \tan\phi_t$$

$$\tau_{heel} = 0$$

Elementary profile of a Gravity Dam:-



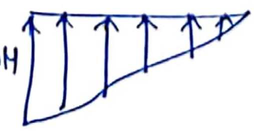
M_C = Moment about "C"

1) a=0 (a)
2) FB = 0 (Free board)
3) Tail water ($H_2 = 0$)
4) FSS = 1 (Factor of safety for sliding)
5) FSOT = 1 (" " " for overturning)
6) $e = \frac{b}{6}$
7) P, W, U only are taken.
8) Mc = b/3 inwards from toe.

$P = \frac{1}{2} \omega \cdot H \cdot H = \frac{\omega H^2}{2}$ (acting at $\frac{H}{3}$ about M_c).

$W = \frac{1}{2} b H\, SW$. (acting at b/3 from M_c).

uplift pressure force:-



$U = \frac{1}{2} \times b \times cwH$ (acting at b/3 from M_c).

$$\Sigma H = P = \frac{wH^2}{2}$$

$$\Sigma V = W - U$$

$$= \frac{1}{2} b w s H - \frac{1}{2} b c w H$$

$$= \frac{1}{2} b w H (s-c)$$

$$FSS = 1$$

$$F_R = F_S$$

$$\frac{1}{2} b w H (s-c) \mu = \frac{1}{2} \frac{wH^2}{2}$$

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

$$FSOT = 1$$

$$M_R = M_O$$

$$W \cdot \frac{b}{\cancel{3}} = P \frac{H}{\cancel{3}} + U \frac{b}{\cancel{3}}$$

$$\cancel{\frac{1}{2}} b^2 \cancel{H} s \cancel{w} = \frac{\cancel{w} H^2}{\cancel{2}} \cdot \cancel{H} + \cancel{\frac{1}{2}} b^2 c \cancel{w} \cancel{H}$$

$$b^2 (s-c) = H^2$$

$$\boxed{b = \frac{H}{\sqrt{s-c}}}$$

$$P_{max,toe} = \frac{\Sigma V}{b} \left[1 + \frac{6e}{b} \right]$$

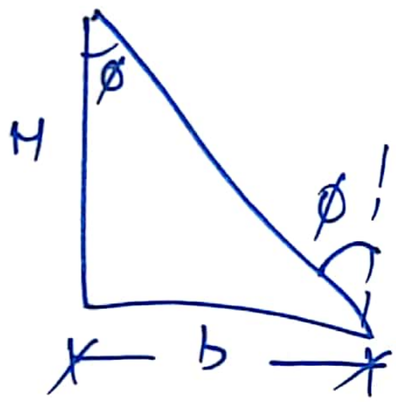
$$= \cancel{\frac{1}{2}} \times \frac{\cancel{b} w H (s-c)}{\cancel{b}} [\cancel{2}]$$

$$= wH(s-c)$$

$$P_{min,heel} = 0.$$

$$\sigma_{toe} = p_{nt}\sec^2\phi_t - p_t\tan^2\phi_t$$

$$= WH(s-c)$$



$$\tan\phi = \frac{b}{H} = \frac{1}{\sqrt{s-c}}$$

$$\sigma_{toe} = WH(s-c)\left[1+\frac{1}{s-c}\right] - 0$$

$$= WH(s-c+1)$$

$$\sigma_{heel} = 0\cdot\sec^2\phi - WH\tan^2\phi \rightarrow \text{zero.}$$

$$= 0.$$

$$\tau_{toe} = (p_{nt} - p_t)\cdot\tan\phi_t$$

$$= [WH(s-c) - 0]\frac{1}{\sqrt{s-c}}$$

$$\tau_{toe} = WH\sqrt{s-c}$$

$$\tau_{heel} = 0$$

Earthen Dams and Rock Fill Dams

20.1. Introduction

Earthen dams and earthen levees are the most ancient type of embankments, as they can be built with the natural materials with a minimum of processing and primitive equipment. But in ancient days, the cost of carriage and dumping of the dam materials was quite high. However, the modern developments in earth moving equipments have considerably reduced the cost of carriage and laying of the dam materials. The cost of gravity dams on the other hand, has gone up because of an increase in the cost of concrete, masonry, etc. Earthen dams are still cheaper as they can utilise the locally available materials, and less skilled labour is required for them.

Gravity dams and arch dams require sound rock foundations, but earthen dams can be easily constructed on earth foundations. However, earth dams are more susceptible to failure as compared to rigid gravity dams or arch dams. Before the development of the subject of Soil-Mechanics, these dams were being designed and constructed on the basis of experience, as no rational basis for their design was available. This led to the failure of various such earthen embankments. However, in these days, these dams can be designed with a fair degree of theoretical accuracy, provided the properties of the soil placed in the dam, are properly controlled. This condition makes the design and construction of such dams, thoroughly interdependent. Continuous field observations of deformations and pore water pressures have to be made during the construction of such dams. Suitable modifications in the design, are then made during construction, depending upon these field observations.

20.2. Types of Earthen Dams

The earthen dam can be of the following three types :

1. *Homogeneous Embankment type*
2. *Zoned Embankment type*
3. *Diaphragm type.*

These three types of dams are described below :

(1) **Homogeneous Embankment Type.** The simplest type of an earthen embankment consists of a single material and is homogeneous throughout. Sometimes, a blanket of relatively impervious material may be placed on the upstream face. A purely homogeneous section is used, when only one type of

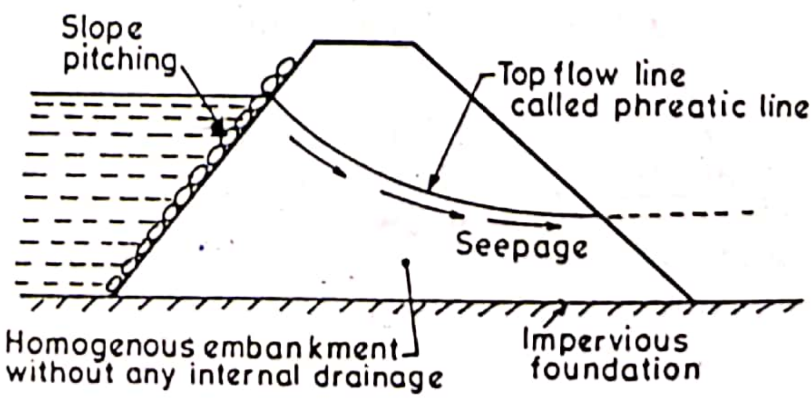


Fig. 20.1 (*a*). Homogeneous type embankment.

material is economically or locally available. Such a section is used for low to moderately high dams and for levees. Large dams are seldom designed as homogeneous embankments.

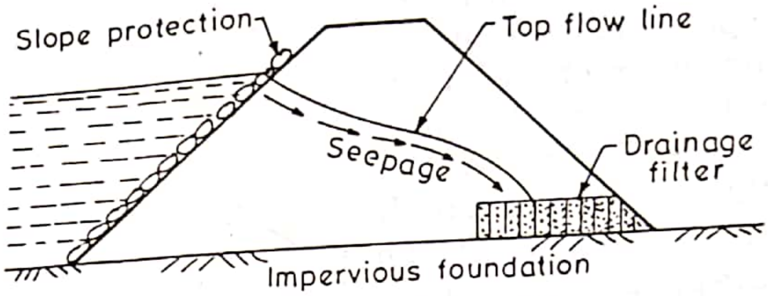


Fig. 20.1 (*b*). Homogeneous embankment provided with a drainage filter.

A purely homogeneous section poses the problems of seepage, and huge sections are required to make it safe against piping, stability, etc. Due to this, a homogeneous section is generally added with an internal drainage system ; such as a horizontal drainage filter [Fig. 20.1 (*b*)], or rock toe, etc. The internal drainage system keeps the phreatic line (*i.e.* top seepage line) well within the body of the dam, and steeper slopes and thus, smaller sections, can be used. The internal drainage is, therefore, always provided in almost all types of embankments.

(2) **Zoned Embankment Type.** Zoned embankments are usually provided with a central impervious core, covered by a comparatively pervious transition zone, which is finally surrounded by a much more pervious outer zone (Fig. 20.2).

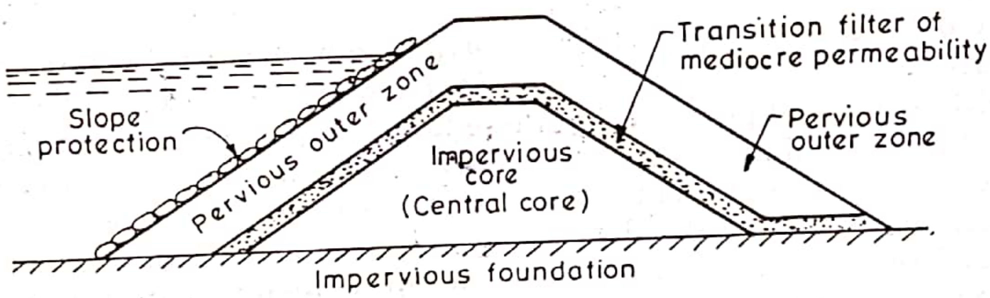


Fig. 20.2. Zoned type embankment.

The central core checks the seepage. The transition zone prevents piping through cracks which may develop in the core. The outer zone gives stability to the central impervious fill and also distribute the load over a large area of foundations.

This type of embankments are widely constructed and the materials of the zones are selected depending upon their availabilities. *Clay, inspite of it being highly impervious, may not make the best core, if it shrinks and swells too much. Due to this reason, clay is sometimes mixed with fine sand or fine gravel, so as to use it as the most suitable material for the central impervious core. Silts or silty clays may be used as the satisfactory central core materials. Freely draining materials, such as coarse sands and gravels, are used in the outer shell.* Transition filters are provided between the inner zone and the outer zone, as shown in Fig. 20.2. This type of transition filters are always provided, whenever there is an abrupt change of permeability from one zone to the other.

(3) **Diaphragm Type Embankments.** Diaphragm type embankments have a thin impervious core, which is surrounded by earth or rock fill. The impervious core, called diaphragm, is made of impervious soils, concrete, steel, timber or any other material. It acts as a water barrier to prevent seepage through the dam. The diaphragm may be placed either at the centre as a central vertical core or at the upstream face as a blanket. The diaphragm must also be tied to the bed rock or to a very impervious foundation material, if excessive under-seepage through the existing previous foundations has to be avoided (Fig. 20.3).

The diaphragm type of embankments are differentiated from zoned embankments, depending upon the thickness of the core. If the thickness of the diaphragm at any elevation is less than 10 metres or less than the height of the embankment above the corresponding elevation, the dam embankment is considered to be of 'Diaphragm Type'. But if the thickness equals or exceeds these limits, it is considered to be of zoned embankment type.

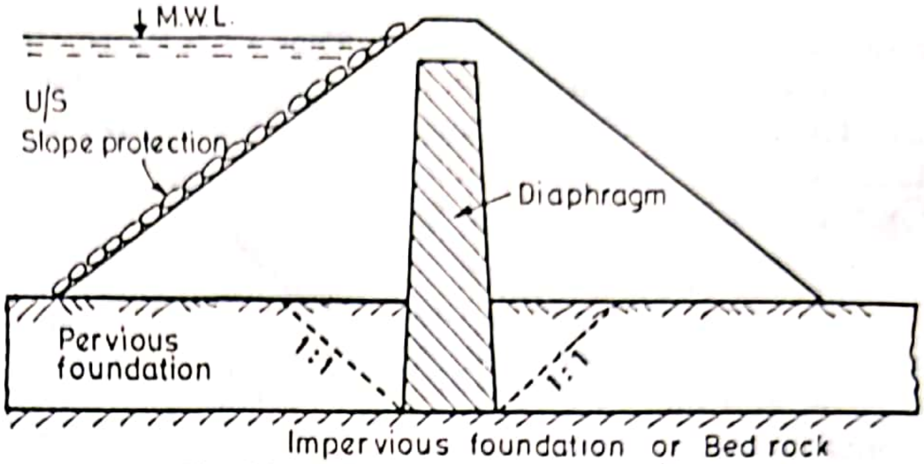


Fig. 20.3. Diaphragm type embankment.

20.3. Methods of Construction

There are two methods of constructing earthen dams ;

(1) *Hydraulic-fill Method* ; and

(2) *Rolled-fill Method.*

(1) **Hydraulic-fill Method.** In this method of construction, the dam body is constructed by excavating and transporting soils by using water. Pipes called flumes, are laid along the outer edge of the embankment. The soil materials are mixed with water and pumped into these flumes. The slush is discharged through the outlets in the flumes at suitable intervals along their lengths. The slush, flowing towards the centre of the bank, tends to settle down. The coarser particles get deposited soon after the discharge near the outer edge, while the fines get carried and settle at the centre, forming a zoned embankment having a relatively impervious central core.

Since the fill is saturated when placed, high pore pressures develop in the core material, and the stability of the dam must be checked for these pressures. This type of embankment is susceptible to settlement over long periods, because of slow drainage from the core.

Hydraulic-fill method is, therefore, seldom adopted these days. Rolled-fill method for constructing earthen dams is, therefore, generally and universally adopted in these modern days.

(2) **Rolled-fill Method.** The embankment is constructed by placing suitable soil materials in thin layers (15 to 30 cm) and compacting them with rollers. The soil is brought to the site from burrow pits and spread by bulldozers, etc. in layers. These layers are thoroughly compacted by rollers of designed weights. Ordinary road rollers can be used for low embankments (such as for levees or bunds) ; while power-operated rollers are to be used for dams. The moisture content of the soil fill must be properly controlled. The best compaction can be obtained at a moisture content somewhere near the optimum moisture content. (The optimum moisture content is the moisture required for obtaining optimum density in the fill). Compaction of coarse gravels cannot be properly done by rolling and is best done by vibrating equipment. Detail of rolling and compactiing different types of soils are available in "Soil Mechanics and Foundation Engineering" by the same author.

20.7. Causes of Failure of Earthen Dams 2

Earth dams are less rigid and hence more susceptible to failure. Every past failure of such a dam has contributed to an increase in the knowledge of the earth dam designers. Earthen dams may fail, like other engineering structures, due to improper designs, faulty constructions, lack of maintenance, etc. The various causes leading to the failure of earth dams can be grouped into the following three classes :

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

HSS

These causes are described below in details :

20.7.1. Hydraulic failures. About 40% of earth dam failures have been attributed to these causes. The failure under this category, may occur due to the following reasons:

(*a*) *By over topping.* The water may overtop the dam, if the design flood is under-estimated or if the spillway is of insufficient capacity or if the spillway gates are not properly operated. Sufficient freeboard should, therefore, be provided as an additional satety measure.

(*b*) *Erosion of upstream face.* The waves developed near the top water surface due to the winds, try to notch-out the soil from the upstream face and may even, sometimes, cause the slip of the upstream slope. Upstream stone pitching or riprap should, therefore, be provided to avoid such failures.

(*c*) *Cracking due to frost action.* Frost in the upper portion of the dam may cause heaving and cracking of the soil with dangerous seepage and consequent failure. An additional freeboard allowance upto a maximum of say 1.5 m should, therefore, be provided for dams in areas of low temperatures.

(*d*) *Erosion of downstream face by gully formation.* Heavy rains falling directly over the downstream face and the erosive action of the moving water, may lead to the formation of gullies on the downstream face, ultimately leading to the dam failure. This can be avoided by proper maintenance, filling the cuts from time to time especially during rainy season, by grassing the slopes and by providing proper berms at suitable heights (Fig. 20.5), so that the water has not to flow for considerable distances. The proper drainage arrangements are made for the removal of the rain water collected on the horizontal berms. Since the provision of berms ensures the collection and removal of water before it acquires high downward velocities, the consequent erosion caused by the moving water (run off) is considerably reduced.

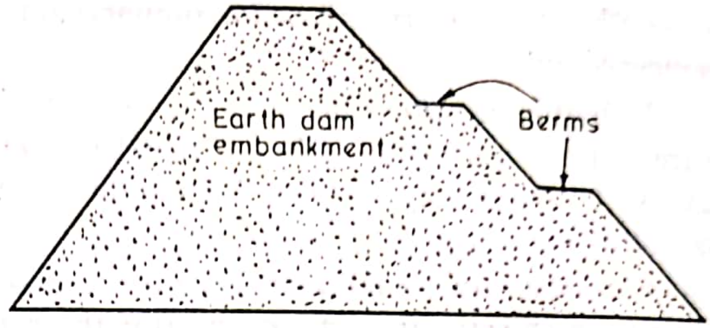


Fig. 20.5

(e) *Erosion of the d/s toe.* The d/s toe of the earth dam may get eroded due to two reasons, *i.e.* (*i*) the erosion due to cross currents that may come from the spillway buckets; and (*ii*) the erosion due to tail water. This erosion of the toe can be avoided by providing a downstream slope pitching or a riprap up to a height slightly above the normal tail water depth. Side walls of the spillway (called diaphragm walls) must be of sufficient height and length, as so to prevent the possibility of the cross flow towards the earthen embankment.

20.7.2. Seepage Failures. Controlled seepage or limited uniform seepage is inevitable in all earth dams, and ordinarily it does not produce any harm. However, uncontrolled or concentrated seepage through the dam body or through its foundation may lead to piping or sloughing and the subsequent failure of the dam. Piping is the progressive erosion and subsequent removal of the soil grains from within the body of the dam or the foundation of the dam. Sloughing is the progressive removal of soil from the wet downstream face. More than 1/3rd of the earth dams have failed because of these reasons.

(a) *Piping through foundations.* Sometimes, when highly permeable cavities or fissures or strata of coarse sand or gravel are present in the foundation of the dam, water may start seeping at a huge rate through them (Fig. 20.6). This concentrated flow at a high gradient, may erode the soil. This leads to increased flow of water and soil, ultimately resulting in a rush of water and soil, thereby creating hollows below the foundation. The dam may sink down into the hollow so formed, causing its failure.

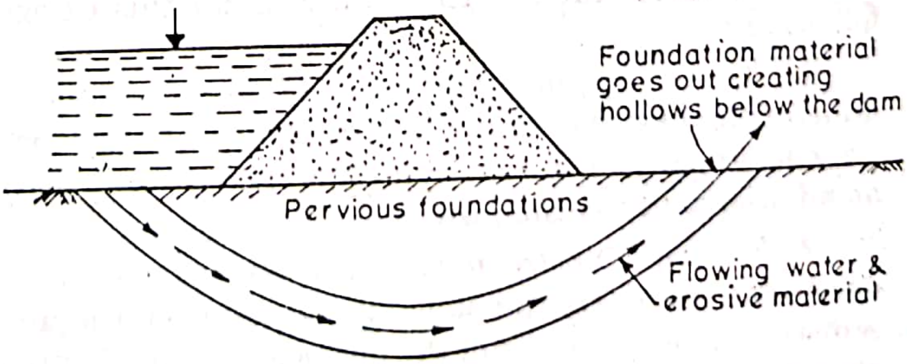


Fig. 20.6. Piping through the dam foundation.

(b) *Piping through the dam body.* When the concentrated flow channels get developed in the body of the dam, (Fig. 20.7) soil may be removed in the same manner as was explained in foundation piping, leading to the formation of hollows in the dam body, and subsequent subsidence of the dam. These flow channels may develop due to faulty construction, insufficient compaction, cracks developed in embankment due to foundation settlement, shrinkage cracks, animal burrows, etc. All these causes can be removed by better construction and better maintenance of the dam embankments.

Piping through the dam body, generally get developed near the pipe conduits passing through the dam body. Contact seepage along the outer side of conduits may either develop into piping, or seepage through leaks in the conduits may develop into piping. This can be avoided by thoroughly and properly compacting the soils near the outlet conduits and by preventing the possibilities of leakage through conduits, but preventing the formation of cracks in the conduits. These cracks in the conduits are caused by differential settlement and by overloading from the embankment. When these factors are controlled, automatically, the possibility of piping due to leakage through the conduits is reduced.

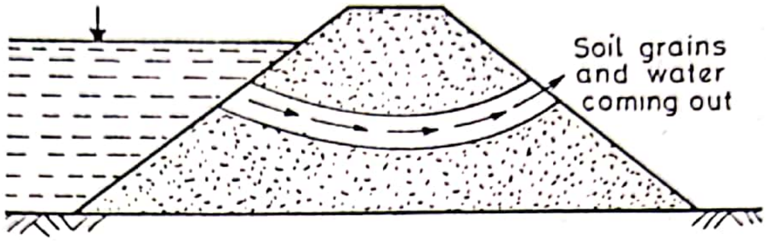


Fig. 20.7. Piping through the dam body.

(c) *Sloughing of D/S Toe.* The process behind the sloughing of the toe is somewhat similar to that of piping. The process of failure due to sloughing starts when the downstream toe becomes saturated and get eroded, producing a small slump or a miniature slide. The miniature slide leaves a relatively steep face which becomes saturated by the seepage from the reservoir and slumps again, forming a more unstable surface. The process continues till the remaining portion of the dam is too thin to withstand the horizontal water pressure, leading to the sudden failure of the dam.

20.7.3. Structural failures. About 25% of the dam failures have been attributed to structural failures. Structural failures are generally caused by shear failures, causing slides.

(a) *Foundation slide.* (*i.e.* overall stability of the dam). When the foundation of the earth dams are made of soft soils, such as fine silt, soft clay, etc., the entire dam may slide over the foundation. Sometimes, seams of fissured rocks, shales or soft clay, etc. may exits under the foundation, and the dam may slide over some of them, causing its failure. In this type of failure, the top of embankment gets cracked and subsides, the lower slope moves outward forming large mud waves near the heel, as shown in Fig. 20.8:

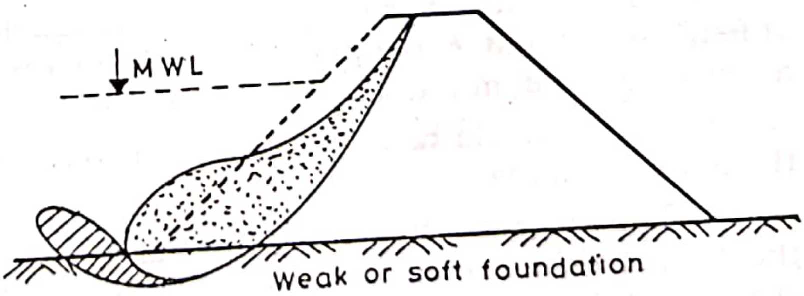


Fig. 20.8. Sliding due to soft or weak foundation.

Excessive pore water pressure in confined seams of sand and silt, artesian pressure in abutments, or hydrostatic excess developed due to consideration of clay seams embedded between sands or silts, etc. may reduce the shear strength of the soil, until it becomes incapable of resisting the induced shear stresses, leading to the failure of the dam foundation whithout warning. Loose sand foundations may fail by the liquefaction or flow slides.

(b) *Slide in Embankments.* When the embankment slopes are too steep for the strength of the soil, they may slide causing dam failure.

20.8. Design Criteria for Earth Dams

(1) A fill of sufficiently low permeability should be developed out of the available materials, so as to best serve the intended purpose with minimum cost. Borrow pits should be as close to the dam site as possible, so as to reduce the leads.

(2) Sufficient spillway and outlets capacities should be provided so as to avoid the possibility of overtopping during design flood.

(3) Sufficient freeboard must be provided for wind set-up, wave action, frost action and earthquake motions.

(4) The seepage line (*i.e.* phreatic line) should remain well within the downstream face of the dam, so that no sloughing of the face occurs.

(5) There is little harm in seepage through a flood control dam. if the stability of foundations and embankments is not impaired, by piping, sloughing, etc. : but a conservation dam must be as watertight as possible.

(6) There should be no possibility of free flow of water from the upstream to the downstream face.

(7) The upstream face should be properly protected against wave action, and the downstream face against rains and against waves upto tail water. Provisions of horizontal berms at suitable intervals in the d/s face may be thought of, so as to reduce the erosion due to flow of rain water. Ripraps should be provided on the entire u/s slope and also on the d/s slope near the toe and up to slightly above the tail water so as to avoid erosion.

(8) The portion of the dam, downstream of the impervious core, should be properly drained by providing suitable horizontal filter drain, or toe drain, or chimney drain, etc.

(9) The upstream and downstream slopes should be so designed as to be stable under worst conditions of loading. These critical conditions occur for the u/s slope during sudden drawdown of the reservoir, and for the d/s slope during steady seepage under full reservoir.

SEEPAGE ANALYSIS

Seepage occurs through the body of all earthen dams and also through their pervious foundations. The amount of seepage has to be controlled in all conservation dams and the effects of seepage (*i.e.* position of phreatic line) has to be controlled for all dams, in order to avoid their failures.

The seepage through a pervious soil material, for two dimensional flow, is given by Laplacian equation

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0 \qquad \text{...(20.14)}$$

where $\phi = K \cdot h$ = Velocity potential
K = Permeability of the soil
h = Head causing flow.

The above equation is based on the following assumptions :

(*i*) Water is incompressible.

(*ii*) The soil is incompressible and porous. The size of the pore space do not change with time regardless of water pressure.

(*iii*) The quantity of water entering the soil in any given time is the same as the quantity flowing out of the soil.

(*iv*) Darcy's law is valid for the given soils.

(*v*) The hydraulic boundary conditions at the entry and exit are known.

A graphical solution of the above equation, (*i.e.* Eq. 20.11) suggests that the flow through the soil, following the above assumptions, can be represented by a *flow-net* ; which consists of two sets of curves, known as '*Equipotential lines*' (*i.e.* lines of equal energy) and '*stream lines*' (*i.e.* flow lines), mutually perpendicular to each other, as shown in Fig. 20.11.

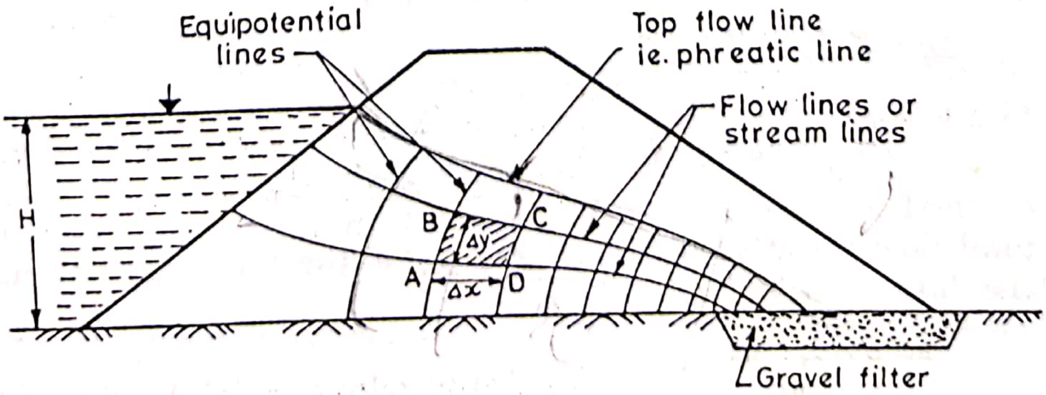


Fig. 20.11. Flow Net.

Scanned with CamScanner

20.13.4. Stability of Upstream Slope During Sudden Drawdown. When the reservoir is full, the critical region is near the downstream face. If no drainage arrangement is made and the d/s slope is also steep, the phreatic line may intersect the d/s slope creating serious conditions there. This can be avoided by providing drainage filter or drainage toe, etc., or by broadening the base of the dam so that the head loss is great enough to bring the line of saturation beneath the d/s toe of the dam.

For the u/s slope, the critical condition can occur, when the reservoir is suddenly emptied. In such a case, the water level within the soil will remain as it was when the soil pores were full of water. The weight of this water within the soil, now tends to slide the u/s slope along a circular arc.

The tangential component of the saturated soil lying over the arc, will create a disturbing force ; while the normal component minus the pore pressure shall supply the shear strength of the soil. High pore pressures shall be developed in this case and although a true solution can be obtained from the flow net and pressure net, an approximate solution can be easily obtained, by considering the soil resting over the failure arc as saturated, for calculating T's ; and as submerged for calculating N's.

The factor of safety (F.S.) is finally obtained from the equation

$$\text{F.S.} = \frac{c \cdot AB + \tan\phi\, \Sigma N'}{\Sigma T}$$

N's represent normal components on submerged density and T's represent tangential components on saturated unit weight of soil. The maximum factor of safety obtained for the critical slip circle should be 1.5, for safe designs.

The seeping water below the phreatic line, exerts a pore pressure on the soil mass which lies below the phreatic line. Hence, if the slices of the critical arc, happen to include this submerged soil, [Fig. 20.25 (*a*)], the shear strength developed on those slices shall be correspondingly reduced. The net shear strength on such a slice shall be $= c\,\Delta L + (N - U)\tan\phi$, where U is the pore pressure.

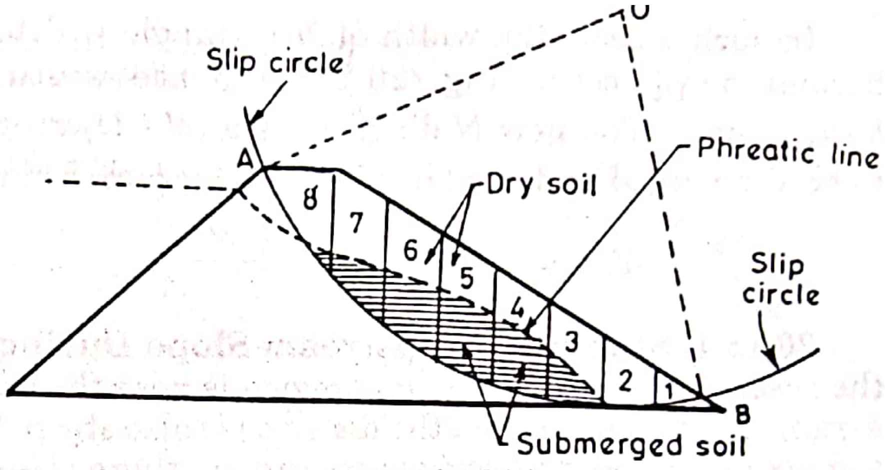


Fig. 20.25 (*a*)

The factor of safety (S.F.) for the entire slip circle is then given by the equation.

$$\text{F.S.} = \frac{c \cdot AB. + \tan\phi\,(\Sigma N - \Sigma U)}{\Sigma T} \qquad ...(20.25)$$

where ΣU is the total pore pressure on the slip circle.

The pore pressure at a point is represented by the piezometric head at that point as explained earlier. The variation of the pore pressure along a failure arc is, therefore, obtained as explained below :

First of all, draw a flow net and thus determine the points of intersections of equipotential lines with the failure arc. At each point of intersection, measure the vertical ordinate from that intersection to the level at which that particular equipotential line cuts the phreatic line. The pore pressures represented by the vertical heights so obtained, are then plotted to a scale in a direction perpendicular to the sliding surface at the respective points of intersection.

The pore pressure distribution is thus shown in Fig. 20.25(*b*) (shaded area).

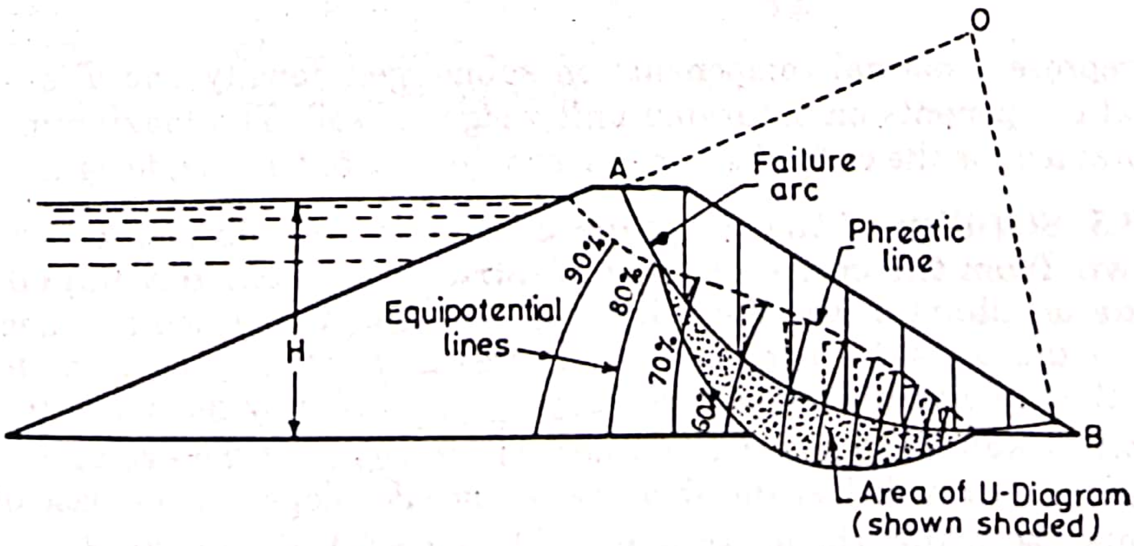


Fig. 20.25 (*b*)

The area of this diagram can be measured by a planimeter. The area of this diagram can also be calculated by ordinate method as was done for N and T cases taking the unit weight of water as 9.81 kN/m^3 ($\approx$ 10 kN/m^3). Knowing ΣN, ΣU and ΣT, F.S. can be calculated easily by using equation 20.25.

SEEPAGE CONTROL IN EARTH DAMS

The water seeping through the body of the earthen dam or through the foundation of the earthen dam, may prove harmful to the stability of the dam by causing softening and sloughing of the slopes due to development of pore pressures. It may also cause piping either through the body or through the foundation, and thus resulting in the failure of the dam.

20.14. Seepage Control Through Embankments

Drainage filters called 'Drains' are generally provided in the form of (*a*) *rock toe* (*b*) *horizontal blanket* (*c*) *chimney drain*, etc. in order to control the seepage water. The provision of such filters reduces the pore pressure in the downstream portion of the dam and thus increases the stability of the dam, permitting steep slopes and thus affecting economy in construction. It also checks piping by migration of particles. These drains, consist of graded coarse material in which the seepage is collected and moved to a point where it can be safely discharged. In order to prevent movement of the fine material from the dam into the drain, the drain or filter material is graded from relatively fine on the periphery of the drain to coarse near the centre. A multi-layered filter, generally called **inverted filter** or *reverse filter* is provided as per the criteria suggested by Terzaghi for the design of such filters.

The various kinds of drains, which are commonly used are shown and described below :

20.14.1. Rock Toe or Toe Filter [Fig. 20.31 (*a*)]. The 'rock toe' consists of stones of size usually varying from 15 to 20 cm. A toe filter (graded in layers) is provided as a transition zone, between the homogeneous embankment fill and rock toe. Toe filter generally consists of three layers of fine sand, coarse sand, and gravel ;

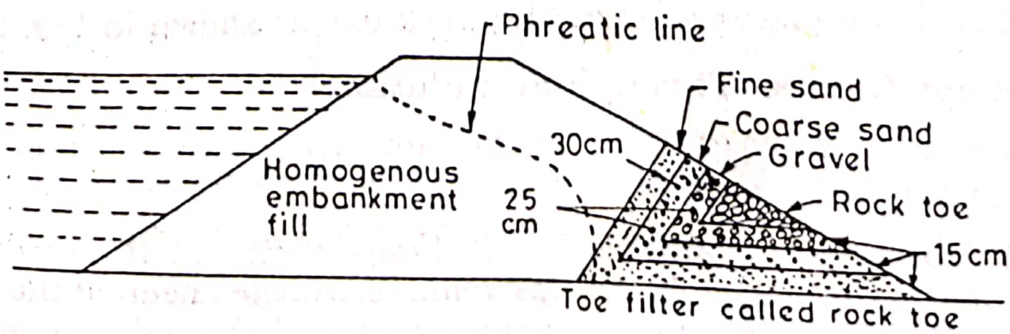


Fig. 20.31 (*a*). Rock Toe.

as per the filter criteria requirements. The height of the rock toe is generally kept between 25 to 35% of reservoir head. The top of the rock toe must be sufficiently higher than the tail water depth, so as to prevent the wave action of the tail water.

20.14.2. Horizontal Blanket or Horizontal Filter. [Fig. 20.31 (*b*) and (*c*)]. The horizontal filter extends from the toe (d/s end) of the dam, inwards, upto a distance varying from 25 to 100% of the distance of the toe from the centre line of the dam. Generally, a length equal to three times the height of the dam is sufficient. The blanket should be properly designed as per the filter criteria, and should be sufficiently pervious to drain off effectively.

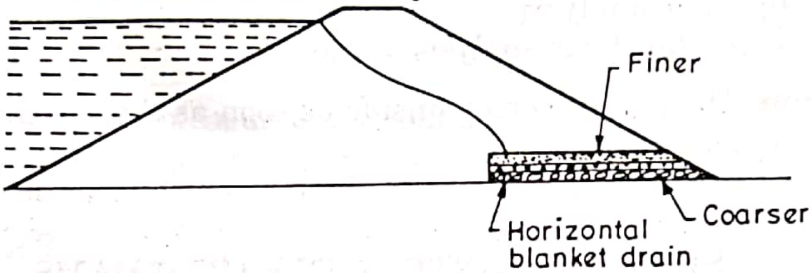


Fig. 20.31 (*b*). Horizontal Filter.

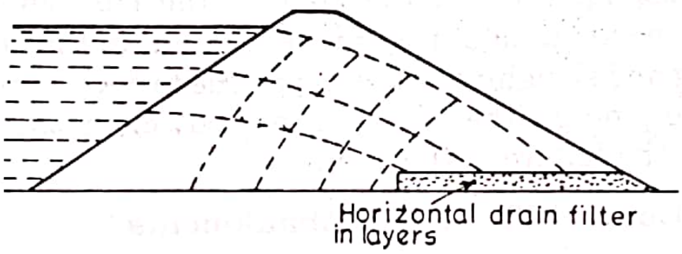


Fig. 20.31 (*c*). Inefficient 'Horizontal drain' in stratified embankments.

20.14.3. Chimney Drain. [Fig. 20.31 (*d*)]. The horizontal filter, not only helps in bringing the phreatic line down in the body of the dam but also provides drainage of the foundation and helps in rapid consolidation. But, the horizontal filter tries to make the soil more pervious in the horizontal direction and thus causes stratification. When large scale stratification occurs, such a filter becomes inefficient as shown in Fig. 20.31 (*c*). In such a possible case, a vertical filter (or inclined u/s or d/s) is placed along with the horizontal filter, so as to intercept the seeping water effectively, as shown in Fig. 20.31 (*d*). Such an arrangement is termed as *chimney drain.* Sometimes a horizontal filter is combined and placed along with a rock toe, as shown in Fig. 20.31 (*e*).

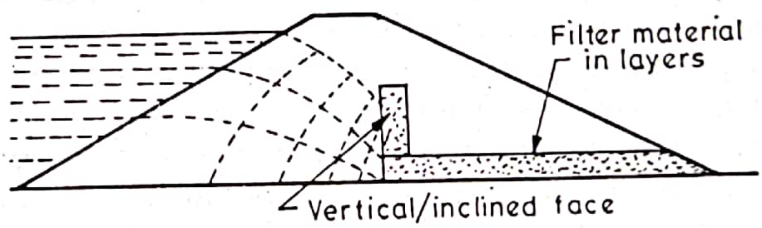


Fig. 20.31 (*d*). 'Chimney Drain' in Stratified Embankments.

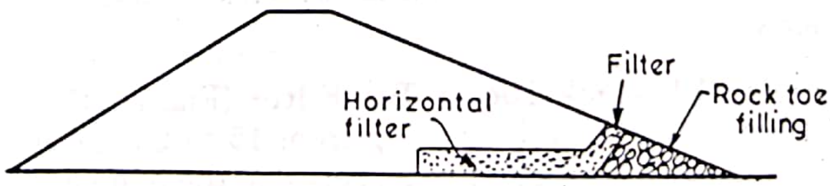


Fig. 20.31 (*e*). Horizontal filter combined with rock toe.

20.15. Seepage Control Through Foundations

The amount of water entering the pervious foundations, can be controlled by adopting the following measures :

20.15.1. Impervious Cutoffs. Vertical impervious cutoffs made of concrete or sheet piles may be provided at the upstream end (*i.e.* at heel) of the earthen dam (Fig. 20.32). These cutoffs should, generally, extend through the entire depth of the pervious foundation, so as to achieve effective control on the seeping water.

When the depth of the pervious foundation strata is very large, a cutoff, up to a lesser depth, called a *partial cutoff* may be provided. Such a cutoff reduces the seepage discharge by a smaller amount. So much so, that a 50% depth reduces the discharge by 25%, and 90% depth reduces the discharge by 65% or so.

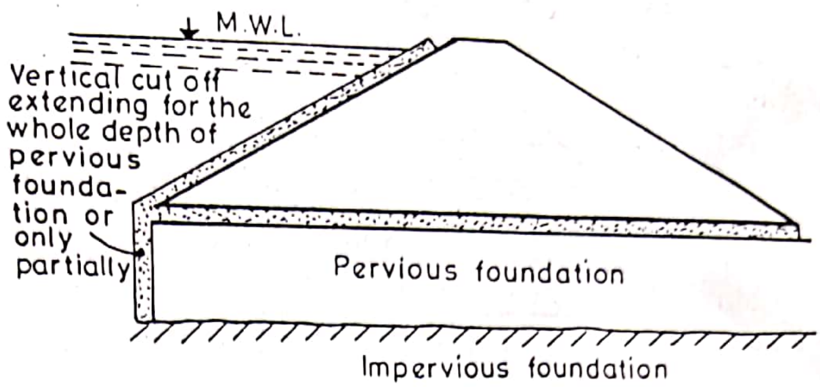


Fig. 20.32

20.15.2. Relief Wells and Drain Trenches. When large scale seepage takes place through the pervious foundation, overlain by a thin less pervious layer, there is a possibility that the water may boil up near the toe of the dam, as shown in Fig. 20.33 (*a*).

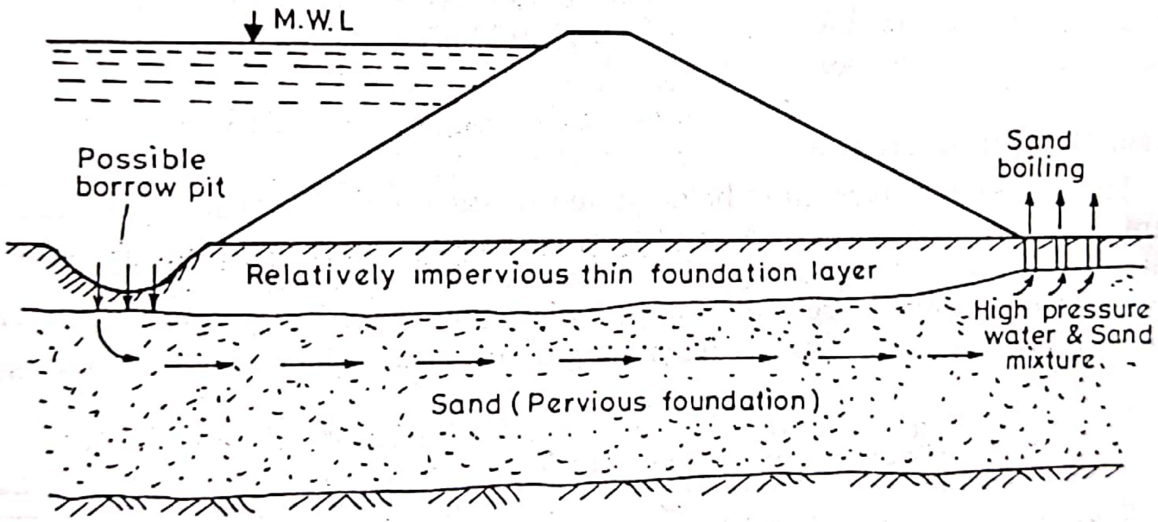


Fig. 20.33 (*a*). Sand Boiling Phenomenon.

Such a possibility, can be controlled by constructing relief wells or drain trenches through the upper impervious layer, as shown in Fig. 20.33 (*b*) and (*c*),

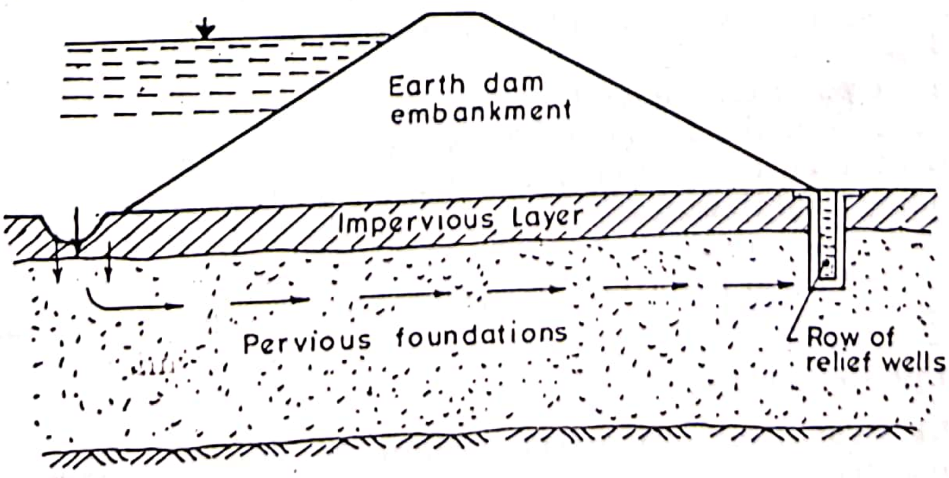


Fig. 20.33 (*b*). Provision of Relief Wells.

so as to permit escape of seeping water. The possibility of sand boiling may also be controlled by providing d/s berms beyond the toe of the dam as shown in Fig.

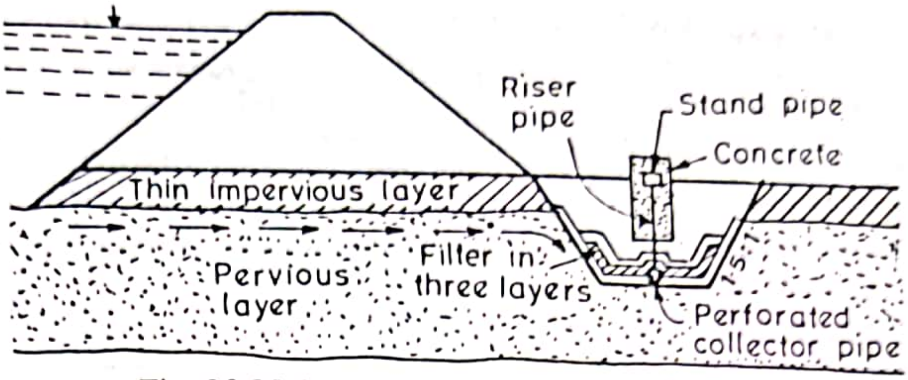


Fig. 20.33 (c). Enlarged View of Drain Trench.

20.33 (*d*). The weight of the overlying material, in such a case, is sufficient to resist the upward pressure and thus preventing the possibility of sand boiling. The provision of such berms, also protects the d/s toe from possible sloughing due to seepage.

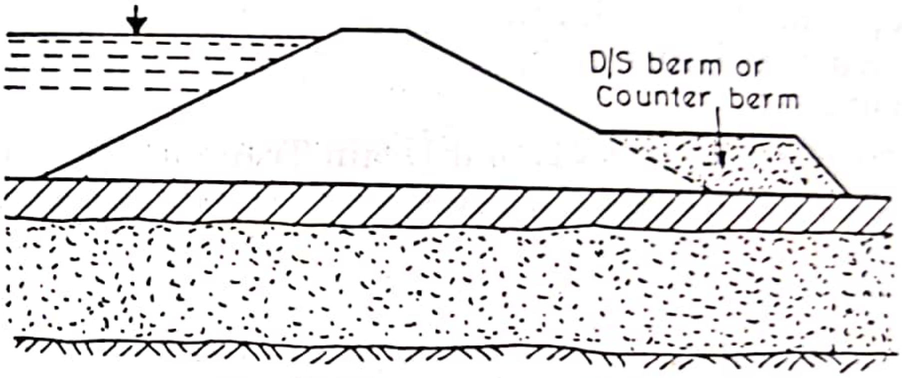


Fig. 20.33 (*d*). Provision of d/s Berms.

20.16. Design of Filters

The drainage filters must be designed in such a way that neither the embankment nor the foundation material can penetrate and clog the filters. The permeability or size of filter material should also be sufficient to carry the anticipated flow with an ample margin of safety. A rational approach to the design of filters has been provided by Terzaghi. According to him, the following filter criteria should be satisfied.

$$\frac{D_{15} \text{ of filter}}{D_{85} \text{ of base materials}} < 4 \text{ to } 5 < \frac{D_{15} \text{ of filter}}{D_{15} \text{ of base material}} \qquad ...(20.50)$$

The embankment soil or the foundation soil surrounding the filter, is known as base material.

When the ratio of D_{15} of filter to D_{85} of base material does not exceed 4 to 5, base material is prevented from passing through the pores of the filter. Similarly, when the ratio of D_{15} of filter to D_{15} of base material is more than 5 (between 5 to 40), the seepage forces within the filter are controlled up to permissible small magnitudes.

Multilayered filters (generally 3 layers) consisting of materials of increasing permeabilities from the bottom to top are, many a times, provided and are known as **inverted filters.** These filters are costly and should be avoided where possible. The minimum total thickness of filter is 1 m. However, if sufficient quantities of filter material are available at reasonable costs, thicker layers of filter may be provided. The thicker the layer, the greater the permissible deviation from the filter requirements.

Spillways, Energy Dissipators, and Spillway Gates

21.1. Introduction

A spillway is a structure constructed at a dam site, for *effectively disposing of the surplus water from upstream to downstream.* Just after the reservoir gets filled up, up to the normal pool level, water starts flowing over the top of the spillway crest (which is generally kept at normal pool level). Depending upon the inflow rate, water will start rising above the normal pool level, and at the same time, it will be let off over the spillway. The water can rise over the spillway crest, upto the maximum reservoir level, which can be estimated from the inflow flood hydrograph and the spillway characteristics, by the process of flood routing, explained earlier. Therefore, it is only the spillway, which will dispose of the surplus water and will not let the water rise above the maximum reservoir level. Had there been no such structure, over which the water would have overflown, the water level must have exceeded maximum reservoir level, and ultimately would have crossed the freeboard and thus overtopped the dam, causing the failure of the dam. *Hence, a spillway is essentially a safety valve for a dam.* It must be properly designed and must have adequate capacity to dispose of the entire surplus water at the time of the arrival of the worst design flood.

Many dams have failed (especially the earthen dams) because of the improperly designed or inadequate spillways.

21.2. Location of a Spillway

A spillway can be located either within the body of the dam, or at one end of it or entirely away from it, independently in a saddle. If a deep narrow gorge with steep banks, separated from a flank by a hillock with its level above the top of the dam (such as shown in Fig. 21.1), is available, the spillway can be best built independently of the dam.

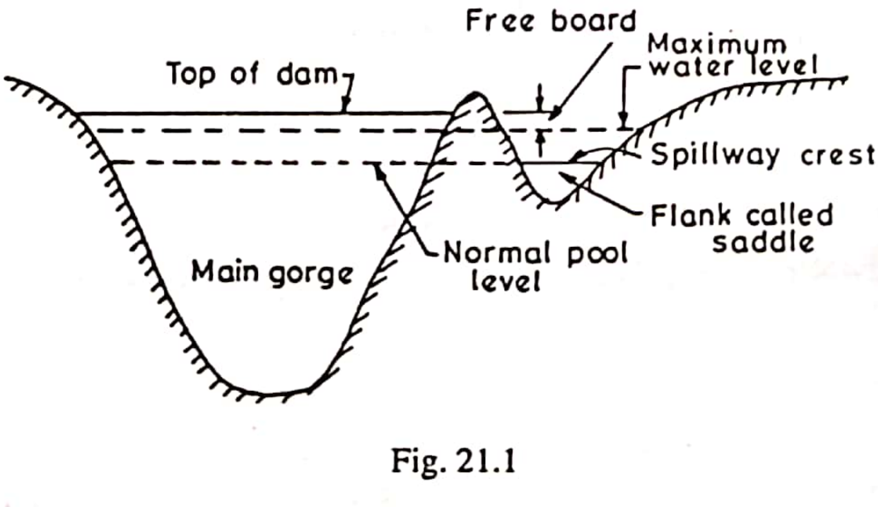


Fig. 21.1

Under such circumstances, a concrete or an earthen dam can be constructed across the main valley and a spillway can be constructed independently into the saddle. Sometimes, a concrete or a masonry dam along with its spillway can be constructed in the main valley, while the flank or flanks are closed by earthen controlled

VARIOUS TYPES OF SPILLWAYS

Depending upon the type of the structure constructed for disposing of the surplus water, the spillways can be of the following major types :

(1) *Straight Drop Spillway.*

(2) *Overflow Spillway* generally called *Ogee Spillway.*

(3) *Chute Spillway* often called *Trough Spillway* or Open channel Spillway.

(4) *Side Channel Spillway.*

(5) *Shaft Spillway.*

(6) *Syphon Spillway.*

The various types of spillways enumerated above are described below along with the design details of 'Ogee Spillway' and 'Chute Spillway'.

21.5. Straight Drop Spillway or Overfall Spillway

This is the simplest type of spillway and may be constructed on small bunds or on thin arch dams, etc. It is a low weir and simple vertical fall type structure, as shown in Fig. 21.3. The downstream face of the structure may be kept vertical or slightly inclined. The crest is sometimes extended in the form of an overhanging lip, which keeps small discharges away from the face of the overfall section. The water falls freely from the crest under the action of gravity. 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. The design of such a spillway is done as that of a weir which was explained in the chapter on weirs. Sometimes, a secondary dam of low

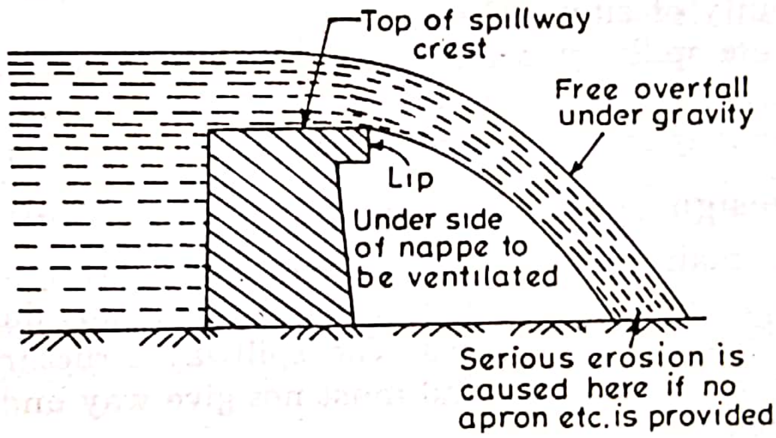


Fig. 21.3. (*a*) Straight drop spillway without d/s protection.

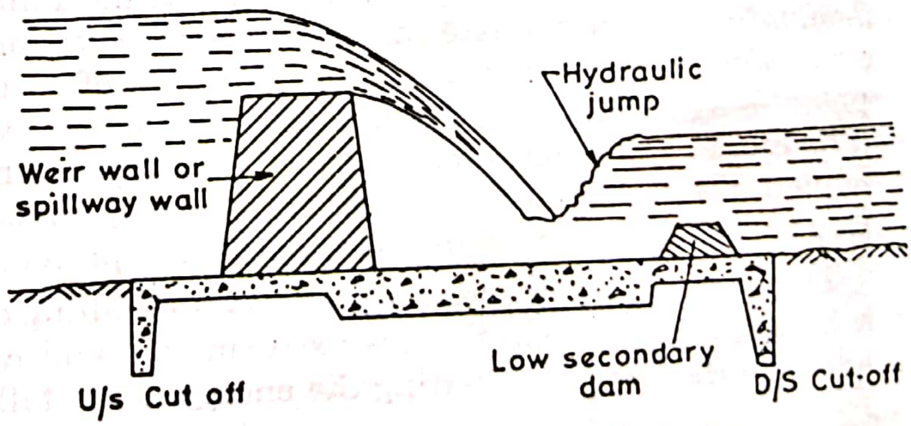


Fig. 21.3. (*b*) Straight drop spillway with d/s protection works.

height is constructed on the downstream side to create an artificial pool of water so as to dissipate the energy of the falling water.

21.6. Ogee Spillway or Overflow Spillway

Ogee spillway is an improvement upon the 'free overfall spillway, and is widely used with concrete, masonry, arch and buttress dams. Such a spillway can be easily used on valleys where the width of the river is sufficient to provide the required

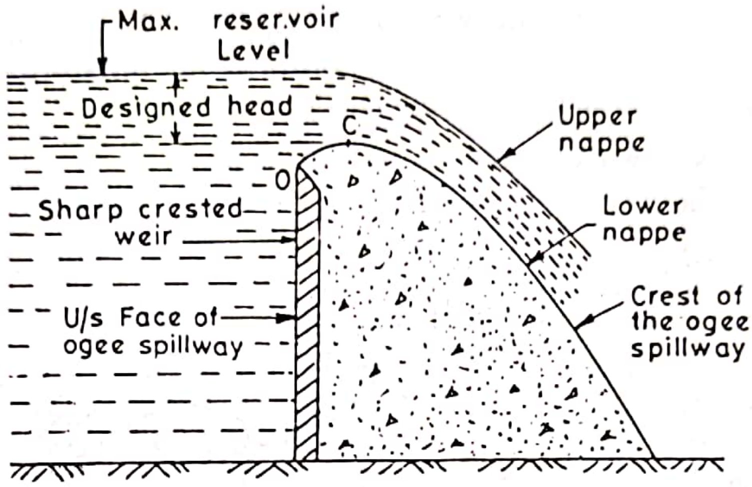


Fig. 21.4. (*a*) Section of an ogee spillway with vertical u/s face.

crest length and the river bed below can be protected from scour at moderate costs. *The profile of this spillway is made in accordance with the shape of the lower nappe of a free falling jet, over a duely ventilated sharp crested weir,* as shown in Fig. 21.4 (*a*).

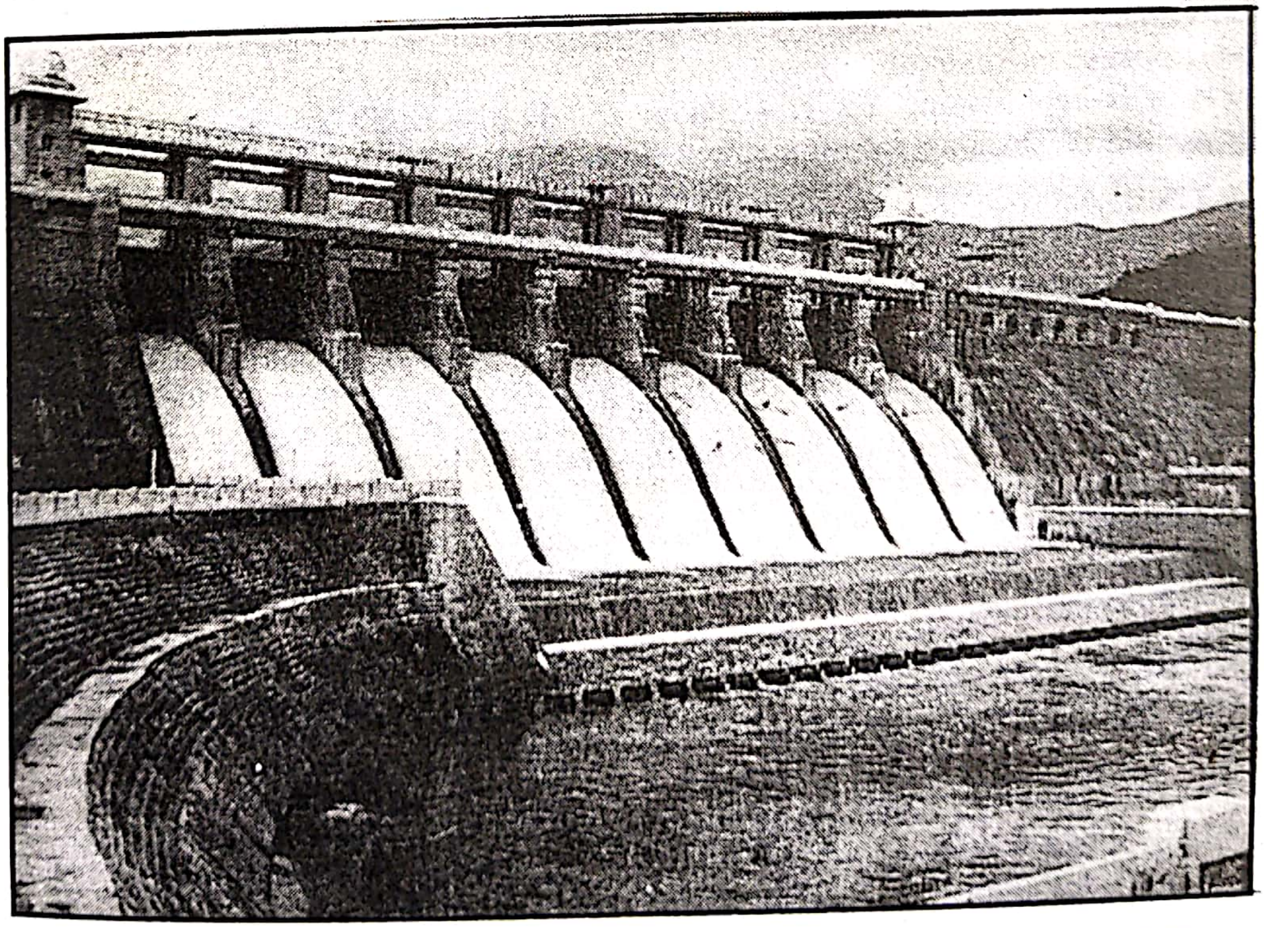


Fig. 21.4. (*b*) Photoview of an Ogee Spillway of Amaravathi dam (earthern cum masonry dam) located on river Amaravathi (a tributary of river Cauvery) in Coimbatore District in Tamil Nadu state.

The shape of the lower nappe of freely falling jet over a sharp crested weir can be determined by the principle of projectile. It generally rises slightly (to point C) as it originates from the crest (O) of a sharp crested weir and then falls to make a parabolic form. Now, if the space between the sharp crested weir and the lower nappe is filled with concrete or masonry, the weir so formed will have a profile similar to an 'ogee' (S-shaped curve in section), and hence called an 'ogee weir' or an 'ogee spillway'. This lower nappe, will then become the crest of the spillway. Since the lower nappe of the free falling jet will be different for different heads over the crest of the sharp crested weir, *the profile of the ogee weir is generally confined to the lower nappe that would be obtained for maximum head over the spillway (i.e. upto the maximum reservoir level).*

In a free overfall spillway, the water jet falls clearly away from the face of the spillway, and the gap between the jet and the face is kept ventilated. While in an ogee spillway, the falling water glides over the curved profile of the spillway, and there is no space between water and crest of the spillway, under normal design conditions.

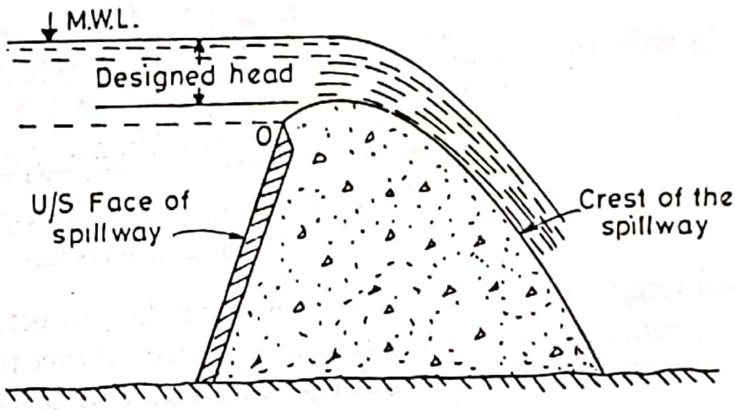


Fig. 21.5. Ogee spillway with inclined u/s face.

Normally, the upstream face of the spillway is kept vertical and the crest shape confirms to the lower nappe of a vertical sharp crested weir under maximum head. But if the upstream face of the spillway is kept sloping, the crest shape should also confirm to the lower nappe that would be obtained for an inclined sharp crested weir (Fig. 21.5).

21.6.1. Cavitation. The crest of the ogee spillway can be made to confirm only to one particular nappe that would be obtained at one particular head. This head is called the *designed head* and represented by, say, H_d. But in practice, the actual head of water on the spillway crest, called the *operating head,* may be less or more than the designed head. *If this operating head on the spillway is more than the designed head, the lower nappe of the falling jet may leave the ogee profile, thereby generating negative pressure at the point of separation.* The generation of vacuum or negative pressure (*i.e.* pressure below the atmospheric pressure) may lead to formation of bubbles or cavities in the water. These cavities or bubbles filled with air, vapour and other gases are formed in a liquid, whenever the absolute pressure (*i.e.* atmospheric pressure—vacuum pressure) of the liquid is close to its vapour pressure, so as to commence evaporation. Such a condition may arise when the head of water is more than the designed head and the consequent high velocity jet causes reduced pressures or negative pressures in the lower region of the water jet.

Such cavities, on moving downstream, may enter a region where the absolute pressure is much higher (*i.e.* more vacuum). This causes the vapour in the cavity to condense and return to liquid with a resulting implosion or collapse of the cavity. When the cavity collapses, extremely high pressures are generated. The continuous bombardment of these implosions will thus take place near the surface of the spillway, causing fatigue failure of its material. The small particles of concrete or masonry are thus broken away, causing formation of pits on its surface and giving the surface a spongy appearance. This damaging action of cavitation is called 'pitting'. The cavitation plus the vibrations from the alternate making and breaking of contact between the water and face of the spillway, may thus result in serious structural damages to the spillway crest. *Hence, it can be concluded that if the head of water over the spillway is more than the designed head, cavitation may occur. On the other hand, if the head of water over the spillway is less than the designed head, the falling jet would adhere to the crest of the ogee spillway, creating positive hydrostatic pressures and thereby reducing the discharge coefficient of the weir.*

21.6.2. Designing the Crest of the Ogee Spillway. The ogee spillways were being designed in the earlier periods, in accordance with the theoretical profile obtained for the lower nappe of a free falling jet. The profile was known as Bazin's profile. Theoretically, the adoption of such a profile, should cause no negative pressures on the crest under designed head. But in practice, there exists a lot of friction due to roughness on the surface of the spillway. Hence, negative pressure on such a profile seems inevitable. The presence of negative pressure causes the danger of cavitation and sometimes fluctuations and pulsations of the nappe. Hence, while adopting a profile for the spillway crest, the avoidance of negative pressures must be an objective along with consideration of other factors such as practicability, hydraulic efficiency, stability and economy. Depending upon research work based on these objectives, various modified profiles have been proposed these days.

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 d/s profile can be represented by the equation

$$x^n = K \cdot H_d^{\,n-1} \cdot y \qquad \text{...(21.1)}$$

where (x, y) are the co-ordinates of the points on the crest profile with the origin at the highest point C of the crest, called the apex

H_d is the *design head including the velocity head.*

K and n are constants depending upon the slope of the upstream face. The values of K and n are tabulated in Table 21.1.

Table 21.1

Slope of the u/s face of the spillway	*K*	*n*
Vertical	2.0	1.85
1 : 3 (1H : 3V)	1.936	1.836
1 : $1\frac{1}{2}$ (1H : $1\frac{1}{2}V$)	1.939	1.810

21.6.4. Discharge Formula for the Ogee Spillway. The discharge passing over the ogee spillway is given by the equation :

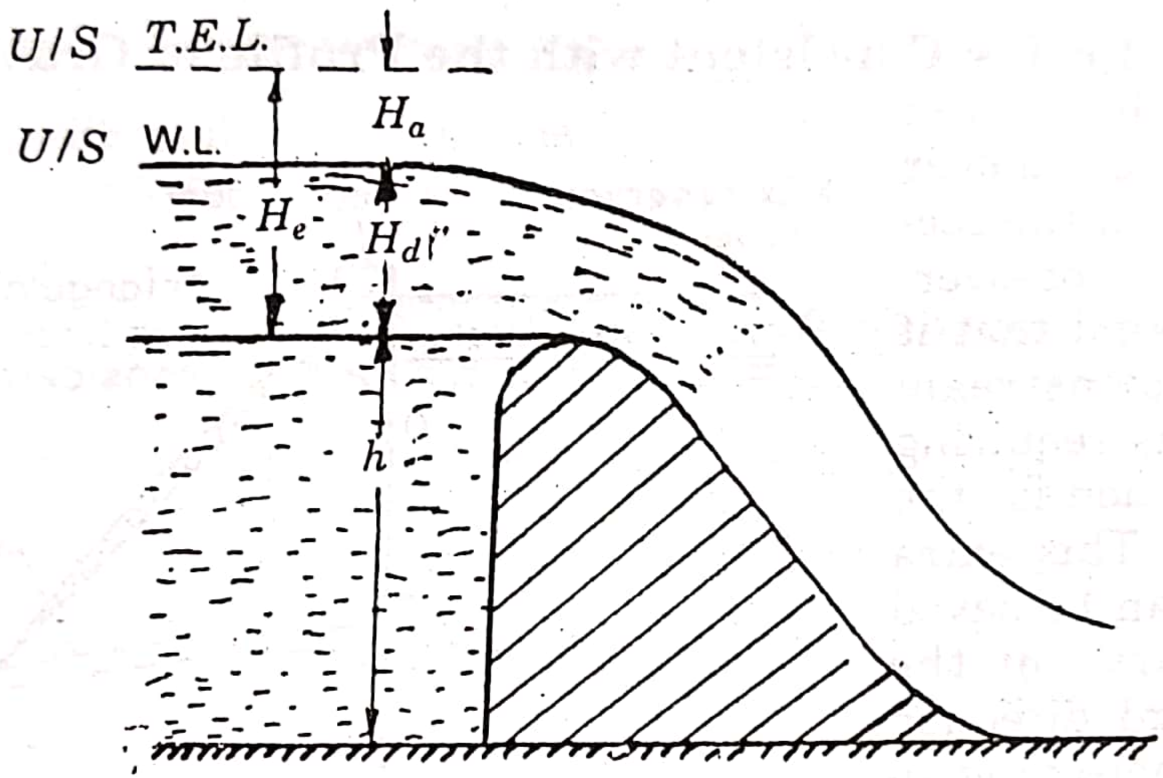


Fig. 21.11

$$Q = C \cdot L_e H_e^{3/2} \qquad \text{...(21.4)}$$

where Q = Discharge

L_e = Effective length of the spillway crest

C = Coefficient of discharge which depends upon various factors such as relative depth of approach, [*i.e.* d/H_d ratio (Fig. 21.11), relation of actual crest shape to the ideal nappe shape, slope of upstream face, downstream apron interference, and submergence, etc.

$H_e =$ Total head over the crest including the velocity head.

If the discharge Q is used as the design discharge in Equation (21.4), then the term H_e will be the corresponding design head (H_d) plus the velocity head (H_a). In such a case, $H_e = H_d + H_a$. For high ogee spillways, the velocity head is very small, and $H_e \approx H_d$.

21.7. Chute Spillway or the Trough Spillway

An ogee spillway is mostly suitable for concrete gravity dams especially when the spillway is located within the dam body in the same valley. But for earthen and rockfill dams, a separate spillway is generally constructed in a flank or a saddle, away from the main valley, as explained earlier. Sometimes, even for gravity dams, a separate spillway is required because of the narrowness of the main valley. In all such circumstances, a separate spillway may have to be provided. The *Trough Spillway* or *Chute Spillway* is the simplest type of a spillway which can be easily provided independently and at low costs. It is lighter and adaptable to any type of foundations ; and hence provided easily on earth and rockfill dams. A chute spillway is sometimes known as a **waste weir**. If it is

21.8. Side Channel Spillway

The side channel spillway (Fig. 21.22) differs from the chute spillway in the sense that while in a chute spillway, the water flows at right angles to the weir crest after spilling over it, whereas in a side channel spillway the flow of water after spilling over the crest, is turned by 90° such that it flows parallel to the weir crest (*AB*), as shown in Fig. 21.22 (*a*).

This type of spillway is provided in narrow valleys where no side flanks of sufficient width to accommodate a chute spillway are available. If a crest length equal to *AB* is provided along *AC* (*i.e.*, along axis of a chute spillway), heavy cutting shall be required. In such topographies, a chute spillway may be replaced by a side channel spillway.

The design of side channel, required for diverting the flow, is beyond the scope of this book. However, it may be mentioned that the analysis of flow in the side channel, is made by the application of the momentum principle in the direction of flow. The water entering the side channel has no momentum in the direction in which it has to move. The slope of the side channel should, therefore, be sufficient to overcome friction losses as well as to provide acceleration in the direction of flow against the mass of incoming water.

After the end of the crest *A*, the water is taken away as in an ordinary chute channel, till it joins the river downstream.

Many other spillways may be constructed somewhere in between the chute spillway and the side channel spillway. In such cases, the direction of water after passing over the crest is changed somewhere between 0° and 90°.

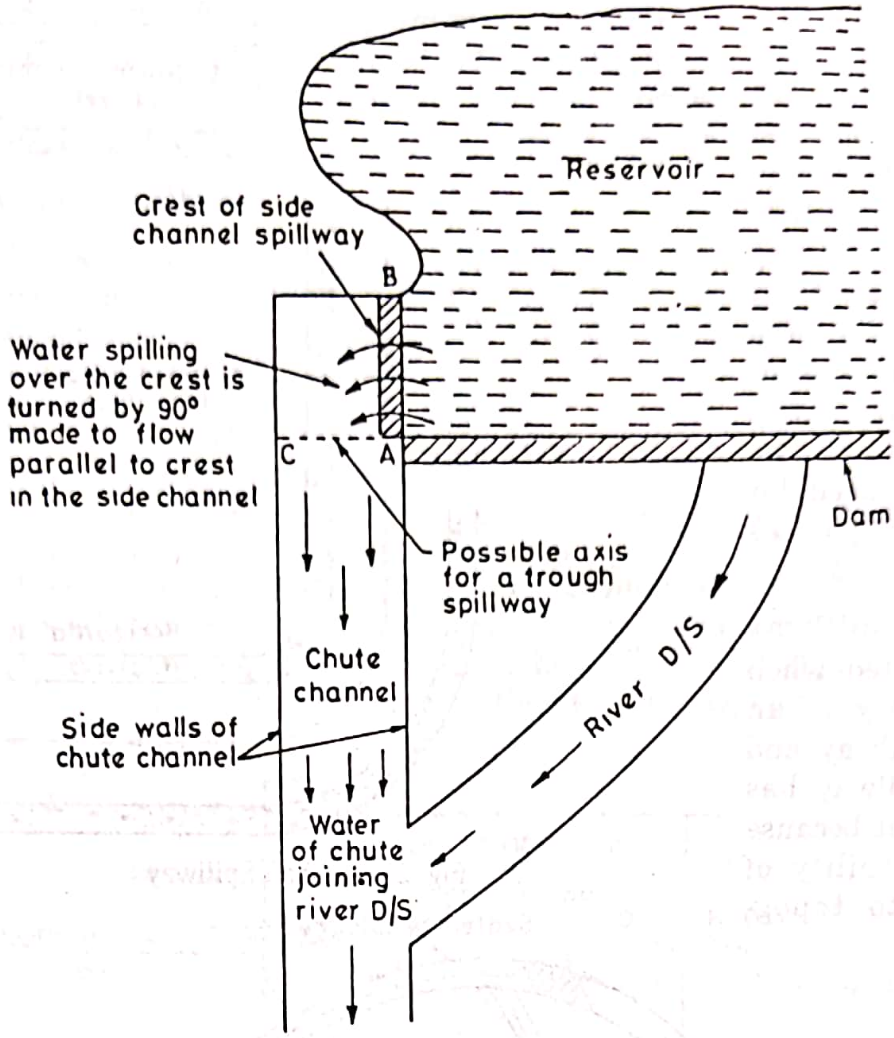


Fig. 21.22. (*a*) Simplified line sketch of a Side Channel Spillway.

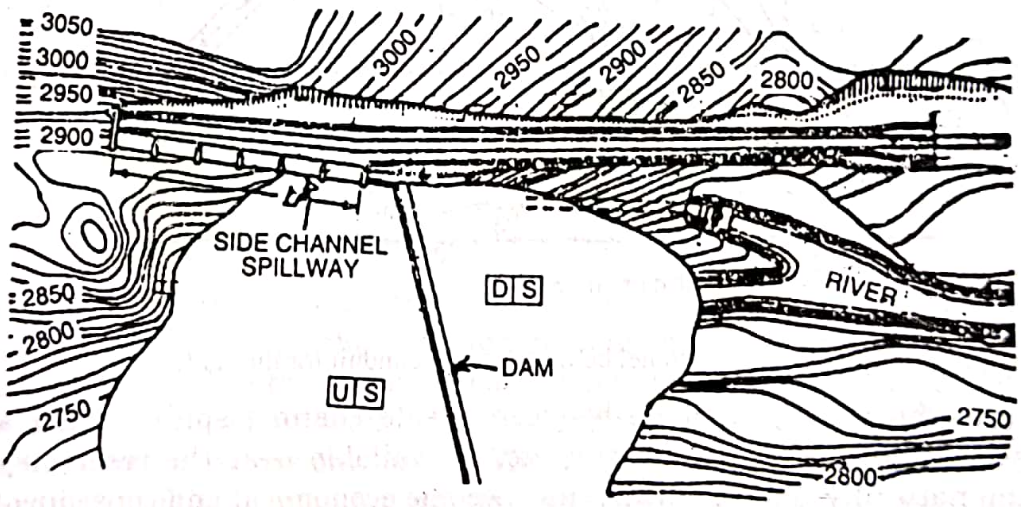


Fig. 21.22. (*b*) Layout plan of a Side Channel Spillway.

21.9. Shaft Spillway

In a shaft spillway (Fig. 21.23), the water from the reservoir enters into a vertical shaft which conveys this water into a horizontal tunnel which finally discharges the water into the river downstream. Sometimes, the vertical shaft may be excavated through some natural rocky island or rocky spur existing on the u/s of the river near the dam. Sometimes, artificial shafts may be constructed. For small heights, the shafts may be constructed entirely of metal or concrete, or clay tiles. But for larger heights, reinforced cement concrete may be used. For smaller heights, no special inlet design is necessary, but on large projects, a flared inlet called **morning glory** is often used.

The horizontal tunnel or the conduit may be taken either through the body of the dam (as may be done in concrete gravity dams) or below the foundations (as may be done in earthen dams). The diversion tunnels constructed for diversion of the river, may sometimes be planned and used for shaft spillways, as shown in Fig. 21.24.

A shaft spillway may be adopted when the possibility of an over-flow spillway and a trough spillway has been ruled out because of non-availability of space due to topog-

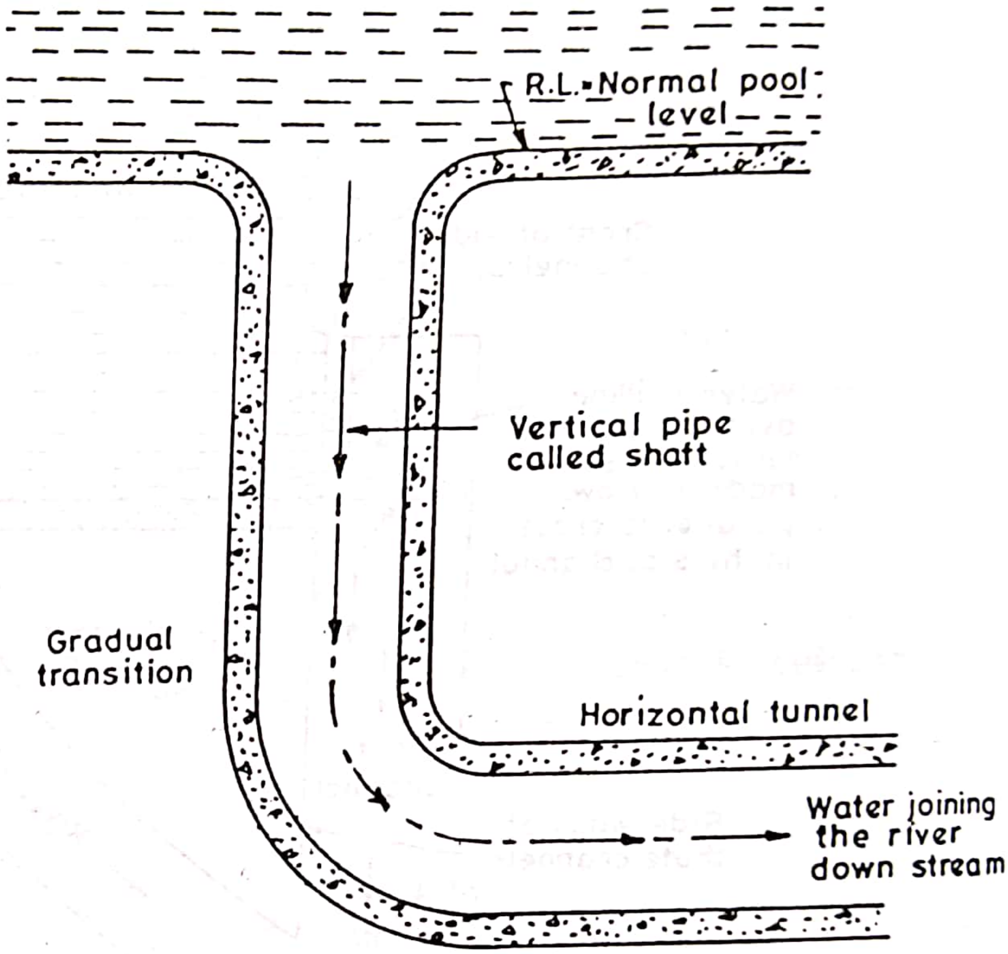


Fig. 21.23. Shaft Spillway.

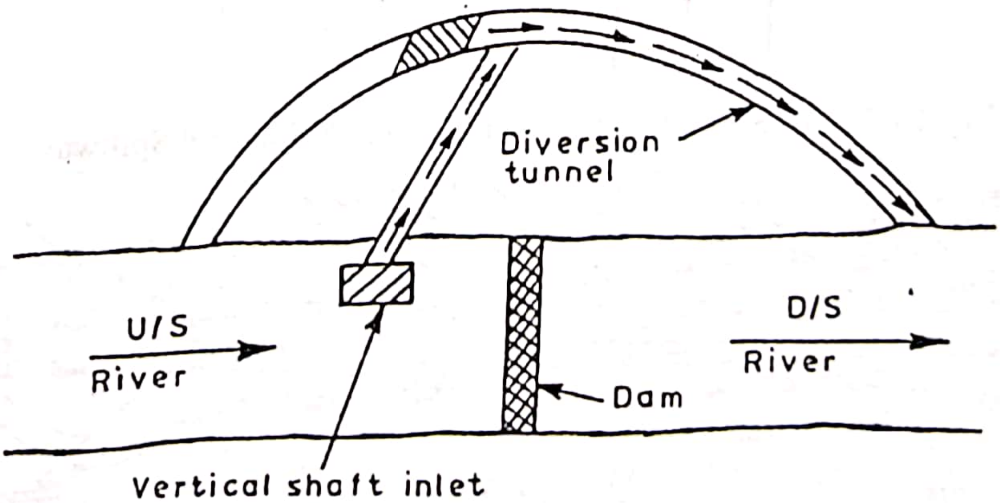


Fig. 21.24. Diversion Tunnel being used as a conduit for the shaft spillway.

21.10. Syphon Spillway

A siphon spillway essentially consists of a siphon pipe, one end of which is kept on the upstream side and is in contact with the reservoir, while the other end discharges water on the downstream side. Two typical installations of siphon pipes are shown in Figs. 21.25 and 21.26.

21.10.1. Tilted Outlet Type of a Syphon Spillway. The siphon pipe in Fig. 21.25 has been installed within the body of the dam. When the valley is very narrow and no space is available for constructing a separate spillway, the siphon pipes can be installed within the dam body, as shown in Fig. 21.25. An air vent may be connected with the siphon pipe. The level of the air vent may be kept at normal pool level, while the entry point of the siphon pipe may be kept still lower so as to prevent the entry of debris, etc. in the siphon. The outlet of the siphon may be submerged so as to prevent the entry of the air in the siphon from its d/s end.

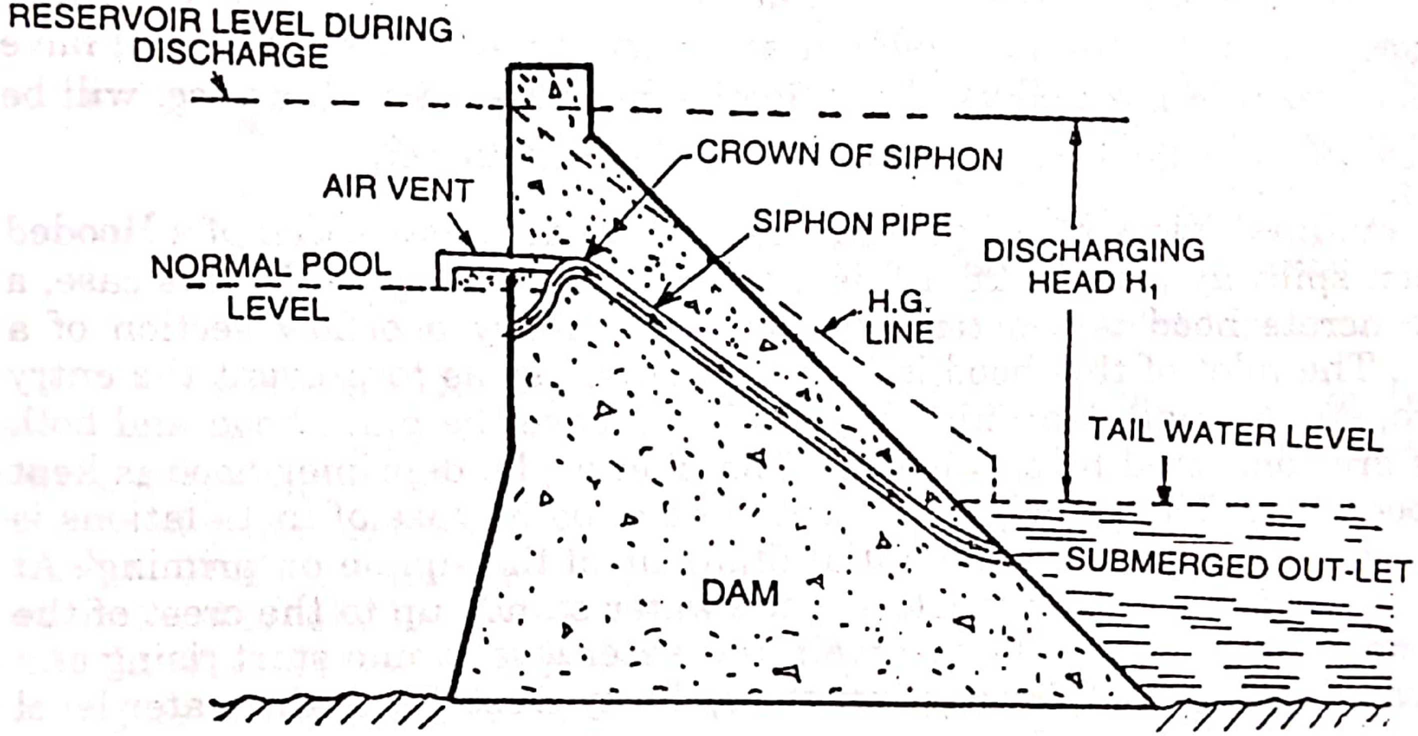


Fig. 21.25. Siphon pipe installed within the gravity dam.

ENERGY DISSIPATORS

21.11. Energy Dissipation below Overflow Spillways

The water flowing over the spillway acquires a lot of kinetic energy by the time it reaches near the toe of the spillway (because of conversion of potential energy into kinetic energy). If arrangements are not made to dissipate this huge kinetic energy of water, and if the velocity of 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*.

In general, the kinetic energy of this super-critical flow can be dissipated in two ways :

(*i*) By converting the super critical flow into sub-critical flow by *hydraulic jump*.

(*ii*) By directing the flow of water into air and then making it fall away from the toe of the structure. The energy is dissipated by the *aeration of jet and impact* of water on the river bed. Though some scour will take place, but it is too small or too far away from the dam to endanger it. Bucket type energy dissipators work on this principle.

21.11.1. Hydraulic Jump Formation. The phenomenon of hydraulic jump has already been explained in details in Chapter 10. It was mentioned therein, that a 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).

$$y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}} \quad \text{i.e. Eq. (10.4)}$$

where q is the discharge intensity.

For a given discharge intensity over a spillway, the depth y_1 is equal to q/V_1 ; and V_1 is determined by the drop H_1, being equal to $\sqrt{2gH_1}$.

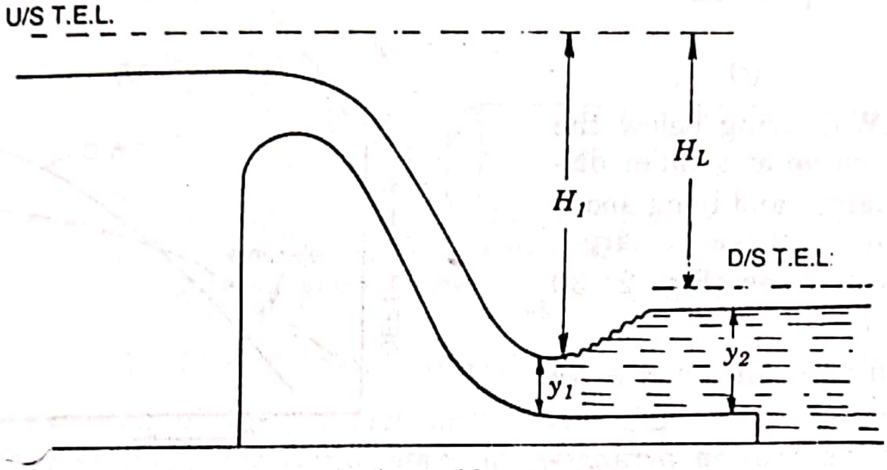


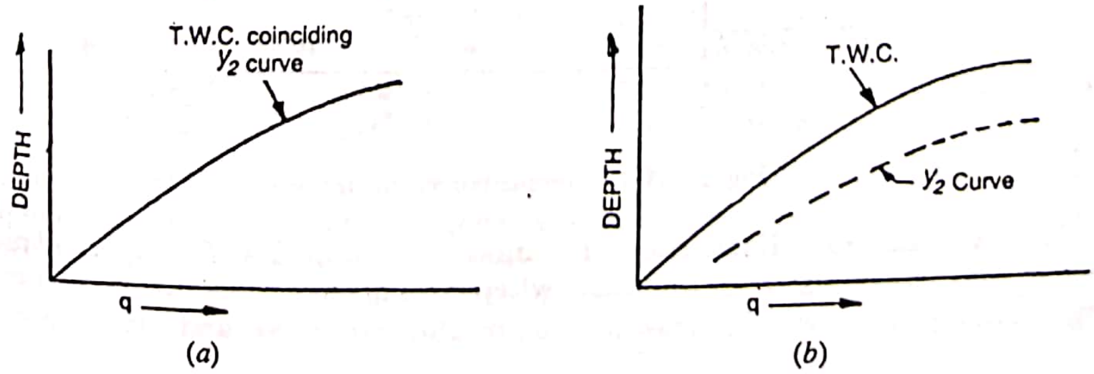
Fig. 21.29

Hence, for a given discharge intensity and given height of spillway, y_1 is fixed and thus y_2 (*i.e.* the depth required for the formation of hydraulic jump) is also fixed. But the availability of a depth equal to y_2 in the channel on the d/s cannot be guaranteed as it depends upon the tail water level, which depends upon the hydraulic dimensions and slope of the river channel below. The problem should, therefore, be analysed before any solution can be found. Hence, for different discharges, the tail water depth is found by actual gauge discharge observations and by hydraulic computations. The post jump depths (y_2) for all those discharges, are also computed from equation (10.4). If a graph is now plotted between q and tail water depth, the curve obtained is known as the *Tail Water Curve (T.W.C.)*. Similarly, if a curve is plotted on the same graph, between q and y_2, the curve obtained is known as the *Jump Height Curve (J.H.C.)* or *y_2 curve.*

Now there are five possibilities

(*a*) T.W.C. coinciding with y_2 curve at all discharges [Fig. 21.30 (*a*)].

(*b*) T.W.C. lying above the y_2 curve at all discharges [Fig. 21.30 (*b*)].



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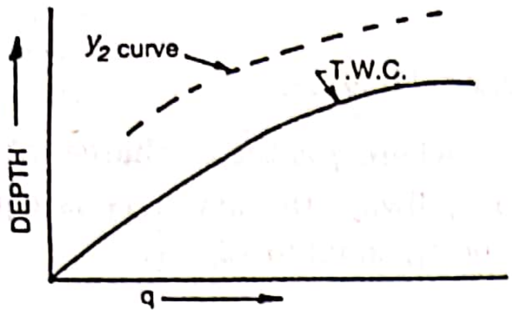
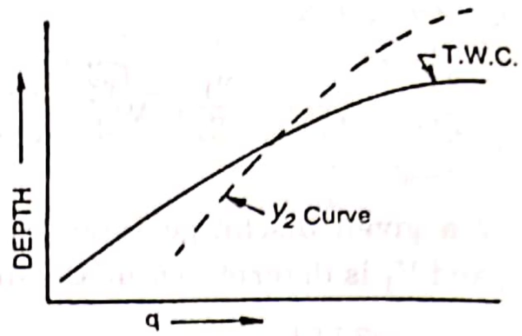
(c) T.W.C. lying below the y_2 curve at all discharges [Fig. 21.30 (c)].

(d) T.W.C. lying above the y_2 curve at smaller discharges and lying below the y_2 curve at larger discharges [Fig. 21.30 (d)].

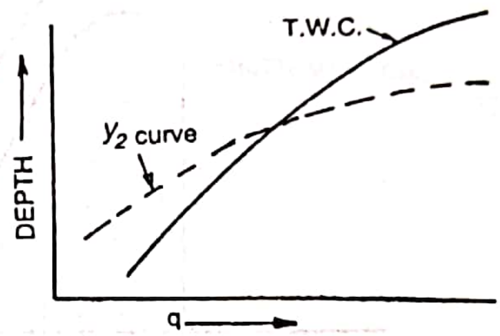


(c)



(d)

(e) T.W.C. lying below the y_2 curve at smaller discharges and lying above the y_2 curve at larger discharges [Fig. 21.30 (e)].



(e)

Fig. 21.30

Depending upon the relative positions of T.W.C. and y_2 curve, the energy dissipation arrangements can be provided below the spillway, as explained below for all these five cases.

21.11.1.1. Energy dissipators for case (a) : *When T.W.C. coincides with y_2 curve at all discharges.* This is the most ideal condition for jump formation. The hydraulic jump will form at the toe of the spillway at all discharges. In such a case, a simple concrete apron of length $5(y_2 - y_1)$ is generally sufficient to provide protection in the region of hydraulic jump, as shown in Fig. 21.31 (a).

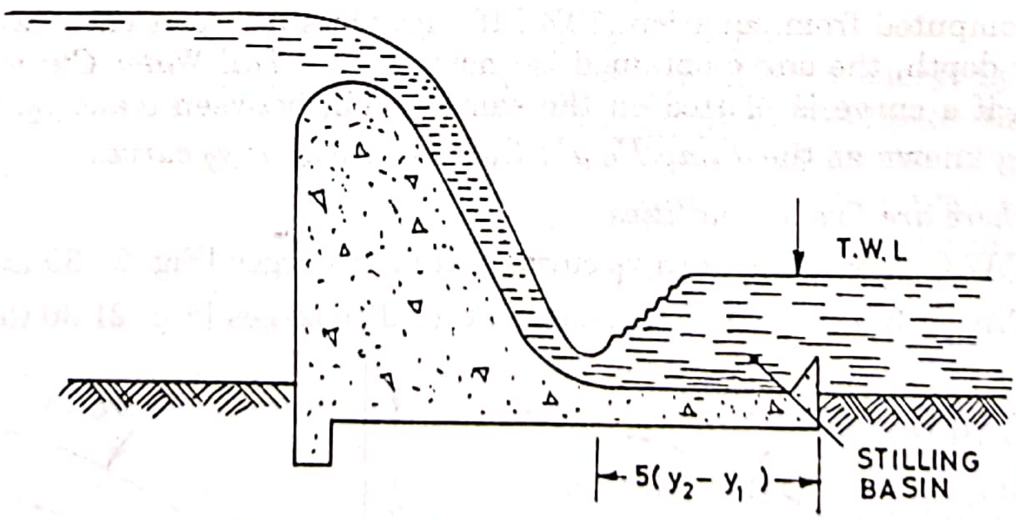


Fig. 21.31. (a) Simple horizontal apron.

21.11.1.2. Energy dissipators for case (b) : *When T.W.C. is lying above the y_2 curve at all discharges.* In this case, when y_2 is always below the tail water, the jump forming at toe will be drowned out by the tail water, and little energy will be dissipated. 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 above the river bed** level as shown in Fig. 21.31 (b_1). The jump will form on the sloping apron where depth equal to y_2 (lesser than the tail water depth at toe) is available. The slope of the apron is made in such a way that proper conditions for a jump will occur somewhere on the apron at all discharges. A lot of extra concreting is required to be done, as shown.

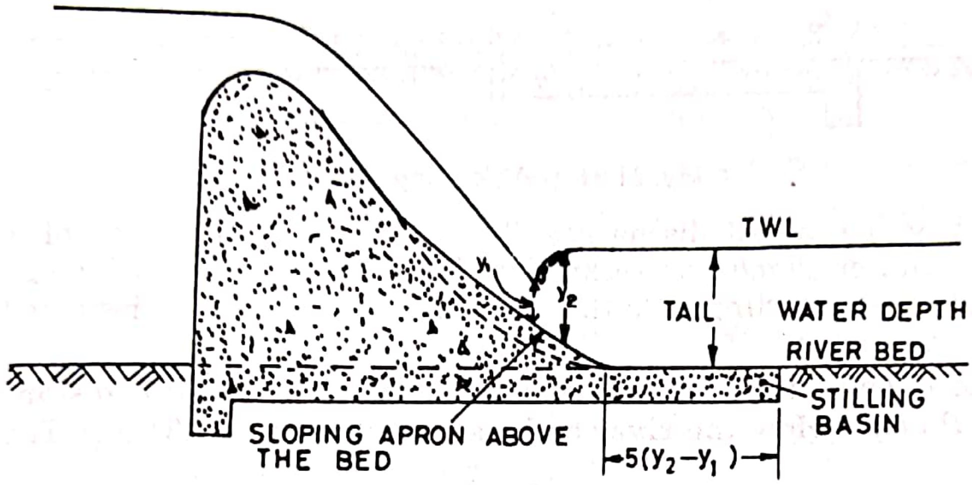


Fig. 21.31. (b_1) Sloping apron above the bed.

(*ii*) A second solution of this problem can be in the form of providing a **roller bucket** type of energy dissipator. It consists of an apron, which is upturned sharply at ends, as shown as in Fig. 21.31 (b_2).

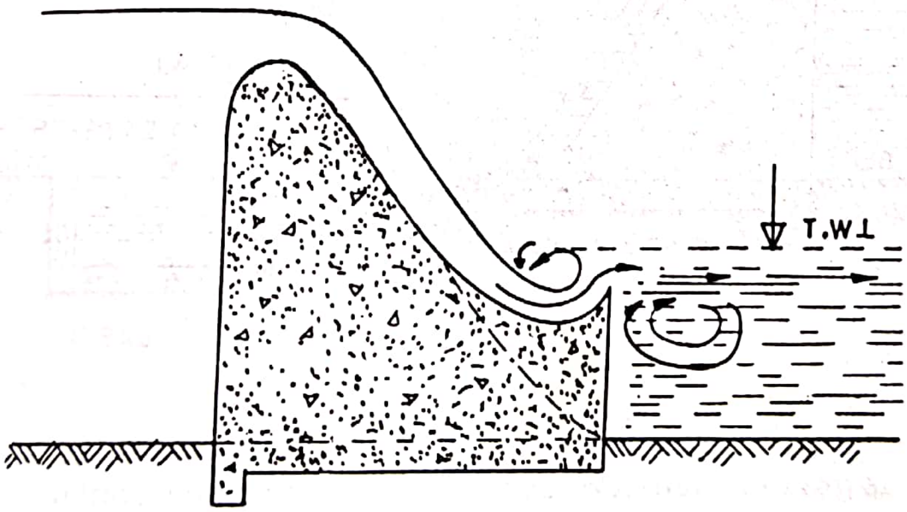


Fig. 21.31. (b_2) Roller Bucket.

Two main rollers are formed which dissipate the energy due to internal turbulence.

The roller which is formed downstream of the bucket, tends to move the scoured bed material towards the dam, thus, preventing serious scour at toe of the dam. Sometimes, the scoured material may enter the bucket under the action of u/s roller, and may cause severe abrasion. A *dentated bucket lip* may, therefore, have to be provided, so as to permit removal of material caught in the bucket.

21.11.1.3. Energy dissipators for case (*c*) : *When T.W.C. lies below the y_2 curve at all dischages.* (*i*) If the tail water is very low, the water may shoot up out of the above bucket, and fall harmlessly into the river at some, distance downstream of the bucket. *This bucket is then known as* **ski jump bucket** and can be used for energy dissipation in case (*c*) : *i.e.* when the tail water depth is

21.15. Standard Stilling Basins

Various types of stilling basins have been generalised for use on different types of works, by various agencies. The designs of these basins have been developed on the basis of long experience and on model studies, keeping in view the protection obtained consistent with economy. These basins are not simple concrete aprons but are generally provided with **auxiliary devices** such as chute blocks, sills, baffle walls, etc. These devices can help in dissipating the energy of flow by offering resistance to flow and may stabilise the flow in a shorter length of the basin, thus affecting economy.

In general, a stilling basin may be defined, as a structure in which the energy dissipating action is confined. If the phenomenon of hydraulic jump is basically used for dissipating this energy, it may be called a hydraulic jump type of stilling basin. The auxiliary devices may be used as additional measures for controlling the jump, etc.

Before we reproduce a few standard stilling basins, let us first describe, in brief, the effects produced by auxiliary devices.

Chute Blocks. Chute blocks are a kind of serrated device (*i.e.* row of small projections like teeth of saw) and provided at the entrance of the stilling basin. The incoming jet of water is furrowed and partly lifted from the floor, producing a shorter length of jump than what would have been without them. They also help in stabilising the flow and thus improve the jump performance (Fig. 21.32).

Sills and Dentated Sills. Sill or more preferably dentated sill is generally provided at the end of the stilling basin. The dentated sill diffuses the residual portion of high velocity jet reaching the end of the basin. They, therefore, help in dissipating residual energy and to reduce the length of the jump or the basin (Refer Fig. 21.32).

Baffle Piers. They are the blocks placed within the basin, across the basin floor. They help in breaking the flow and dissipate energy mostly by impact. These baffle piers, sometimes called **friction blocks,** are very useful in small structures, such as low spillways and weirs, etc. They, however, give way due to cavitation, under the influence of high velocity jets, and hence are unsuitable for large works.

21.15.1. U.S.B.R. Basins. U.S.B.R. has standardised stilling basins for different ranges of Froude numbers. The important of these basins, are :

(1) **U.S.B.R. stilling basin II.** This is recommended for use on large structures, such as dam spillways, large canal structures, etc., when the incoming *Froude number* (F_1) *is more than* 4.5. The dimensions of the chute blocks, dentated sill, etc. are shown in Fig. 21.32. The length of the basin is related to the Froude number (F_1) as given in Table 21.15.

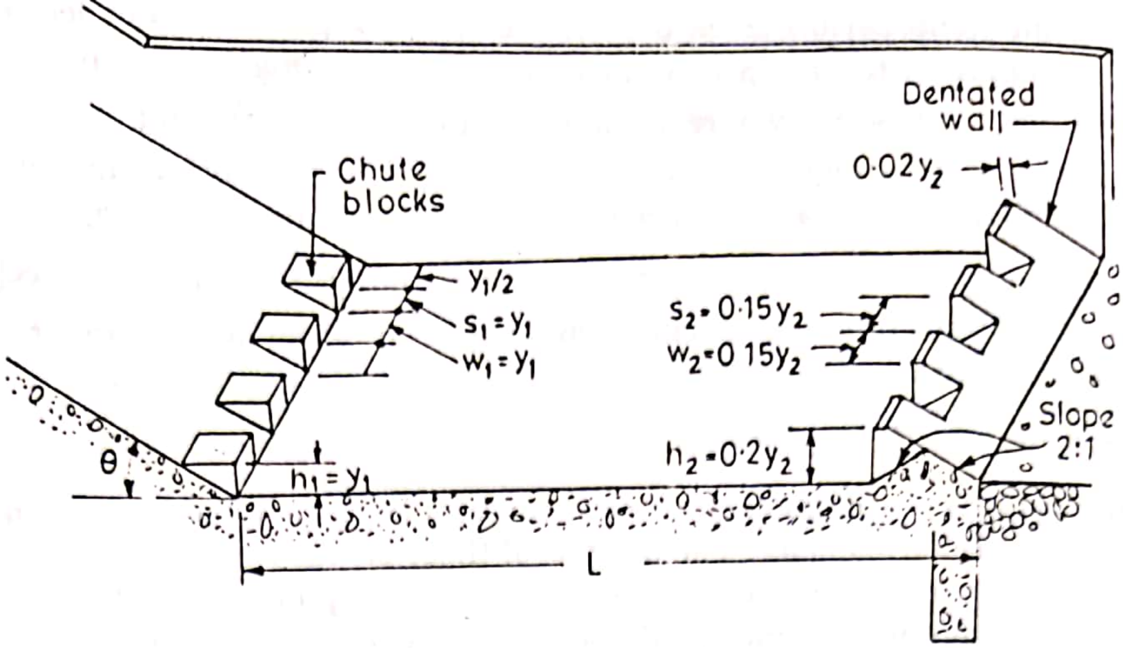


Fig. 21.32. U.S.B.R. Stilling basin II ($F_1 > 4.5$).

Table 21.15

F_1	*Length of the basin*
4	$3.6\,y_2$
6	$4\,y_2$
8	$4.2\,y_2$
10 or more	$4.3\,y_2$

An economy in the length of the basin up to about 35% ($4.3\,y_2$ in place of $6y_2$) is thus obtained with auxiliary devices. The floor of the basin should be set at such a level as to provide 5% more water depth than y_2.

(2) **U.B.S.R. stilling basin IV.** This type of stilling basin is shown in Fig. 21.33. It is used for Froude number varying between 2.5 and 4.5, which generally occurs in canal weirs, canal falls, diversion dams, etc. This basin is applicable only to rectangular cross sections. Since oscillating waves are generated in this range of Froude number, they are tried to be controlled at source by providing large chute

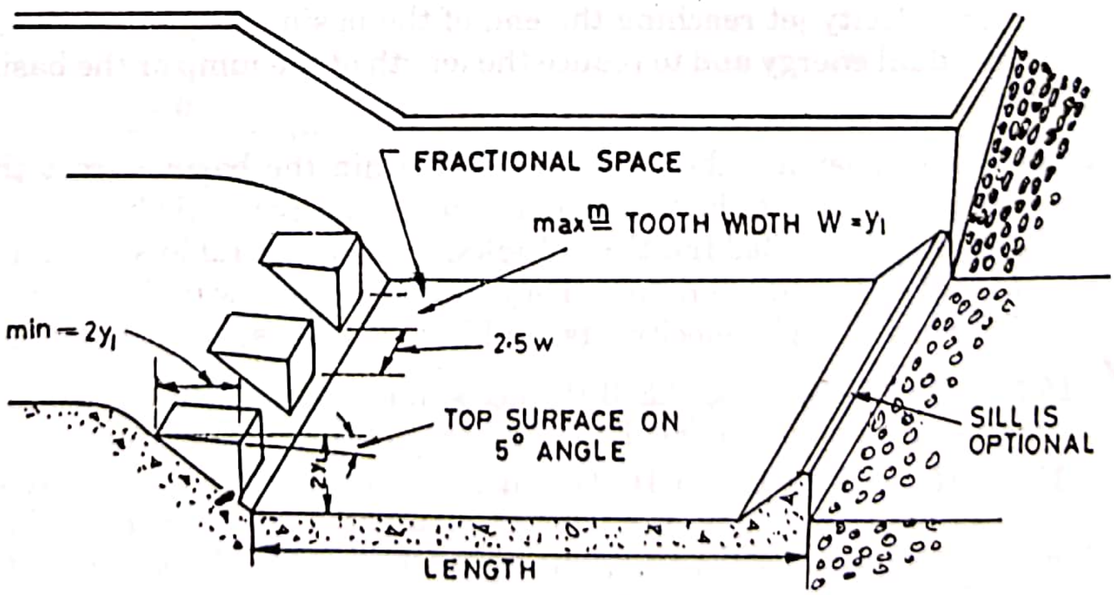


Fig. 21.33. U.S.B.R. Stilling basin IV (F_1 lies between 2.5 and 4.5).

Fig. 21.34 (*c*)

If a temporary barrier can be installed over the permanent raised crest of a spillway, additional water can be stored between the spillway crest and the top of the barrier during the fag-end of the rainy season. The small flows in excess of the barrier top level, may be permitted to pass over the barrier. If, however, large flood occurs, the barrier may be removed and full spillway capacity made available for the outflow.

Sometimes on large dams, regular gates may be installed over the permanent crest, so as to function like a movable additional crest. In such a case, the height of the permanent raised crest can be reduced and the balance provided by the movable crest (*i.e.* gate). If there is a permanent raised crest up to the gate top, the storage, of course would be equal to that of a gated crest ; but in times of serious floods, the rise in flood level would be much more as compared to what would have been in a gated crest. This is because, the gates would be opened during serious floods so as to provide more head and hence larger discharge and consequent lesser rise in flood levels. Hence, the top level of the non-overflow section and the value of land acquisition for the reservoir which has to be determined by the maximum rise of flood above the spillway crest, can be reduced by providing gated crest or controlled crests. In other words, the dam height can be reduced for the same useful storage, or more useful storage can be obtained for the same height, provided the dam spillway is controlled by gates, etc.

This saving in the dam height and land acquisition will be more, if more height of the gates is provided. This saving is, however, counter-balanced by the cost of the gates. The cost of the gates would be more if their height is increased. (The gate cost include the principal cost and OMR, *i.e.* operational, maintenance and repair costs.) An economic balance between these two factors must be worked out and the cheapest combination found before deciding the height of the permanent crest and the height of the temporary crest (*i.e.* gates). This is also governed by the limitations of maximum available gate heights.

Gates can be provided on all types of spillways except siphon spillways. In siphon spillways, the gates are not required as the rise in flood level is already small compared to other types of spillways. The gates for earthen dams should be provided with caution, since the faulty operation or failure of their operation may lead to serious rise in flood levels, causing overtopping and failure of dam.

21.17. Types of Spillway Gates

The various types of spillway gates are described below in brief :

21.17.1. Dropping Shutters or Permanent Flash Boards. They consist of wooden panels usually 1.0 to 1.25 m high. They are hinged at the bottom and are supported against the water pressure by struts (Fig. 21.35). The shutters fall flat on the crest when the downstream supporting struts are tripped. Hence they are not suitable for curved crests.

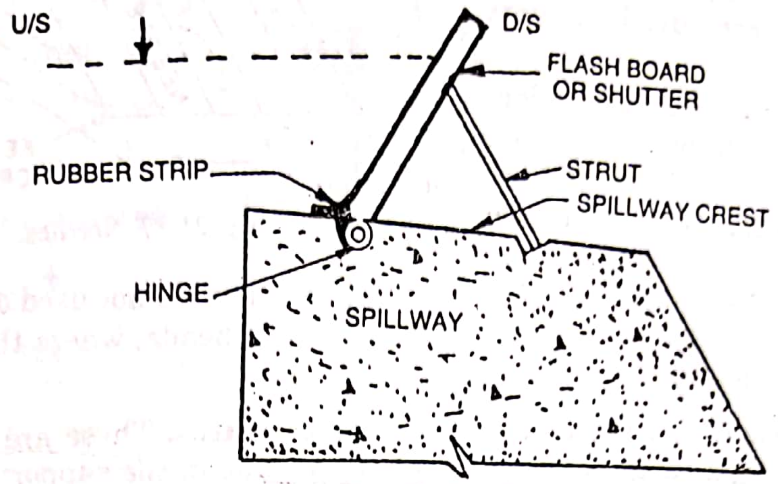


Fig. 21.35. Flash board or Dropping shutter.

These shutters can be raised or lowered from an overhead cableway or a bridge. Various types of shutters which drop and hoist themselves automatically, have been designed these days. These automatic shutters work on the principle of counter weights acting against the water pressure. Automatic shutters do not function well when interferred by floating debris, ice, etc.

Sometimes temporary flash boards, which shall fall as soon as overtopped by water, may be used for very minor works.

All kinds of flash-boards do have some disadvantages and hence used only on small spillways of minor importance.

21.17.2. Stop Logs and Needles. Stop logs consist of wooden beams or planks placed one upon the other and spanning in the grooves between the spillway piers (Fig. 21.36). They can be placed and removed either by hand or with hoisting

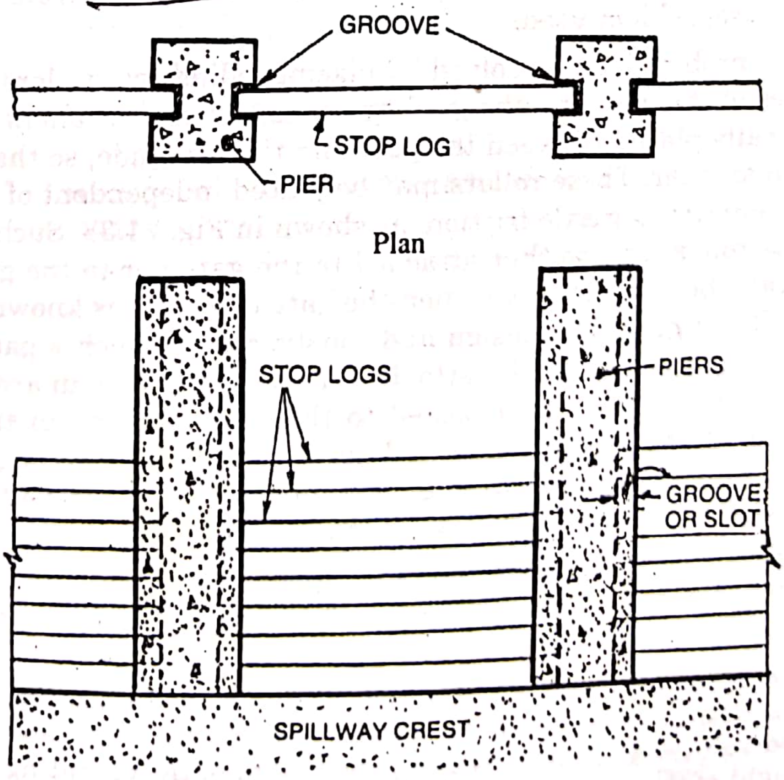


Fig. 21.36. Stop logs.

mechanism. Considerable time may get wasted in removing them, if they become jammed in the slots. Leakage between the logs is also a big problem. They are, therefore, used on very minor works.

Needles. Needles are wooden logs kept side by side with their lower ends resting in a keyway on the spillway and upper ends supported by a bridge (Fig. 21.37). It is very difficult to handle these needles at the time of flow and hence they are not used on any major works. They are sometimes used for emergency bulk heads, where they need not be replaced until the flow has stopped.

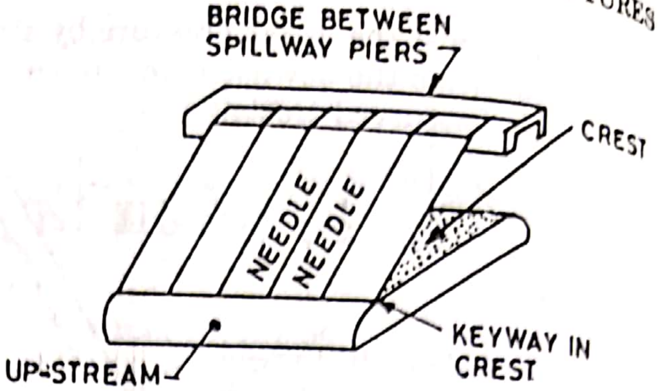


Fig. 21.37. Needles.

21.17.3. Vertical Lift Gates or Rectangular Gates. These are rectangular gates spanning horizontally between the grooves made in the supporting spillway piers (Fig.21.38). The grooves are generally lined with rolled steel channel sections of appropriate size, so as to provide a smooth bearing surface having sufficient bearing strength and are known as *grove guides.* These rectangular gates move between the groove guides, and can be raised or lowered by a hoisting mechanism at the top.

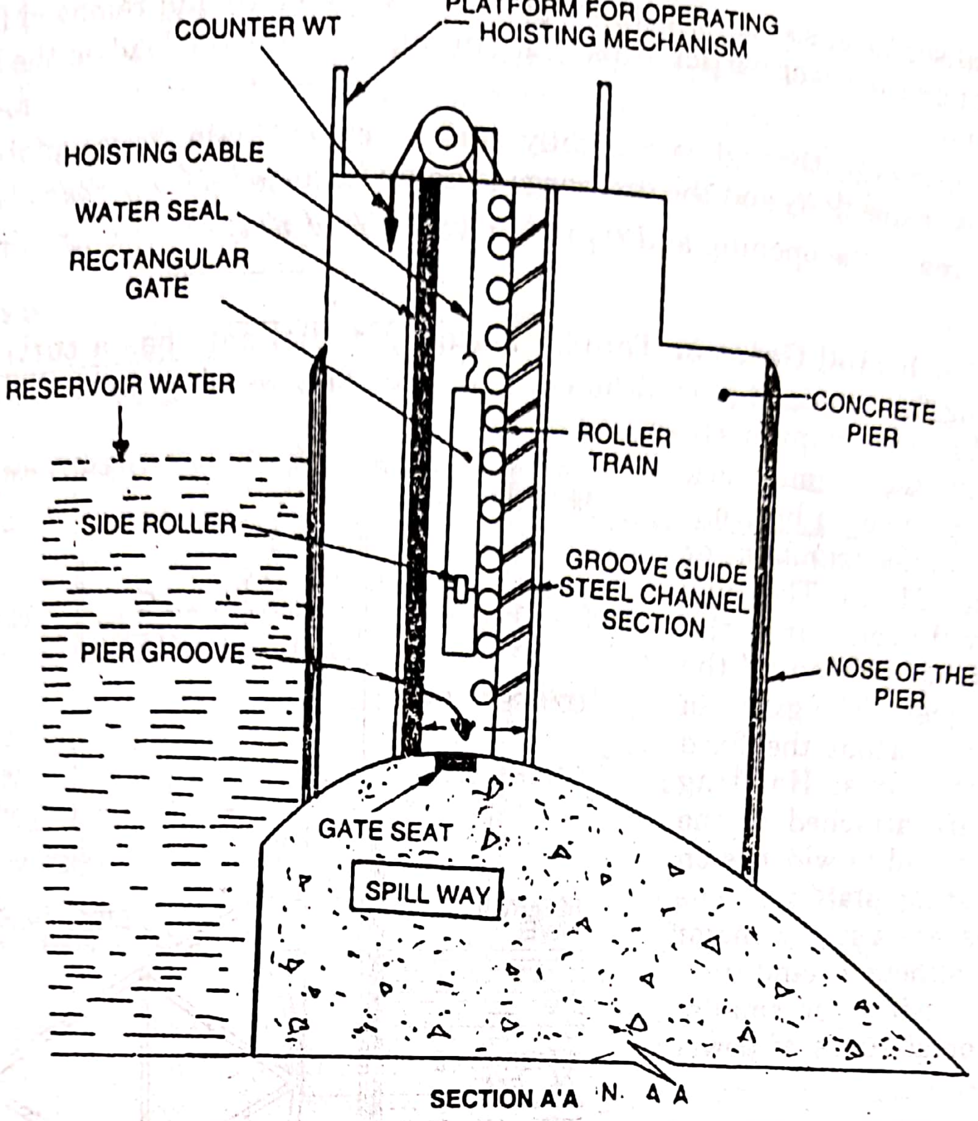
The gates are often made of steel, although they may be made of concrete or wood. They are generally placed vertical, although they may be kept slightly inclined downstream.

Because of the hydrostatic force caused by the upstream water standing against the gate, large friction is developed between the gate and the downstream groove guides. Hence, if the gate is in direct contact with the guides, as is there in a *sliding gate*, large friction will be developed, and it will be very difficult to move the gate. Hence, in a *sliding gate* relatively larger hoisting capacity is required to operate the gate because of the sliding friction that has to be overcome. The sliding gates are, therefore, seldom used.

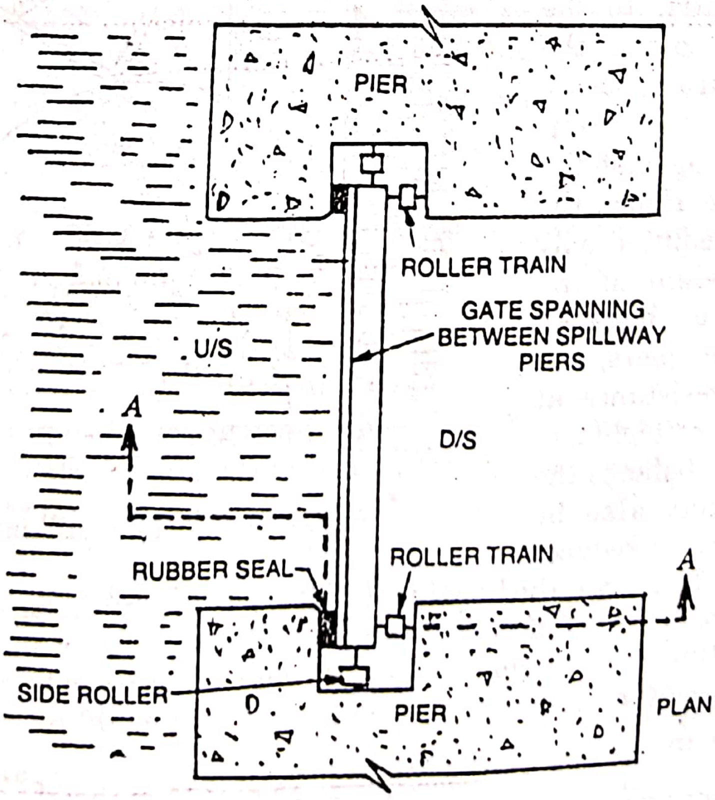
This friction problem can be solved by placing cylindrical rollers between the bearing surfaces of the gate and the guide grooves. A train of rollers or wheels is, therefore, generally placed between the gate and the d/s guide, so that the sliding friction is much smaller. These rollers may be placed independent of the gate and the guide, thus eliminating axle friction, as shown in Fig. 21.38. Such an arrangement, when the rollers are neither attached to the gate nor to the guide grooves but rolls vertically between the two when the gate is moved, is known as a *Stoney gate* or a *Free Roller gate*. The design and construction of such a gate is difficult and rollers are, therefore, generally attached to the gate. Such an arrangement in which the rollers or wheels are attached to the gate and ride in tracks on the downstream side of the groove guide, is known as a *Fixed wheels gate* or *Fixed roller type gate*. Rubber seals are used to seal the openings between the upstream leaf plate and the sides of the pier grooves, as shown.

Large vertical lift gates may be counter balanced by a counter weight beam, which is loaded to balance the self-weight of the gate. Hence, hoisting force is required only to balance the frictional resistance.

Vertical lift gates have been used in size 15 m × 15 m (height and span). If the gate height is larger, head room is required for lifting the gate clear of the maximum reservoir level ; thus increasing the height of the operating platform. To reduce the height of the operating platform, high gates may be broken up into two



(a)



(b)

Fig. 21.38. 'Vertical stoney gate' or 'Free roller gate'.

horizontal sections, so that the upper portion may be lifted and removed from the guides before the lower portion is moved. This also reduces the load on the hoisting mechanism.

The discharge through a partially raised vertical gate, takes place by an undershot orifice flow, and the discharge formula is given by $C_d A \sqrt{2g \cdot H_1}$, where A is the area of the opening and H_1 is the water head above the centre line of the opening.

21.17.4. Radial Gates or Tainter Gates. A radial gate has a curved water supporting face made of steel. The curved water face which is in the shape of a sector of a circle is properly braced by steel frame work which is pivoted on horizontal shafts called *trunnions* or *pins* (Fig. 21.39). The pins are anchored in the downstream portion of the spillway piers. The gate can thus rotate about the fixed horizontal axis. Hoisting cables are attached to the gate and lead to winches on the hoisting platform. The winches are usually motor driven, although hand driving is possible for smaller works or at times of power failures.

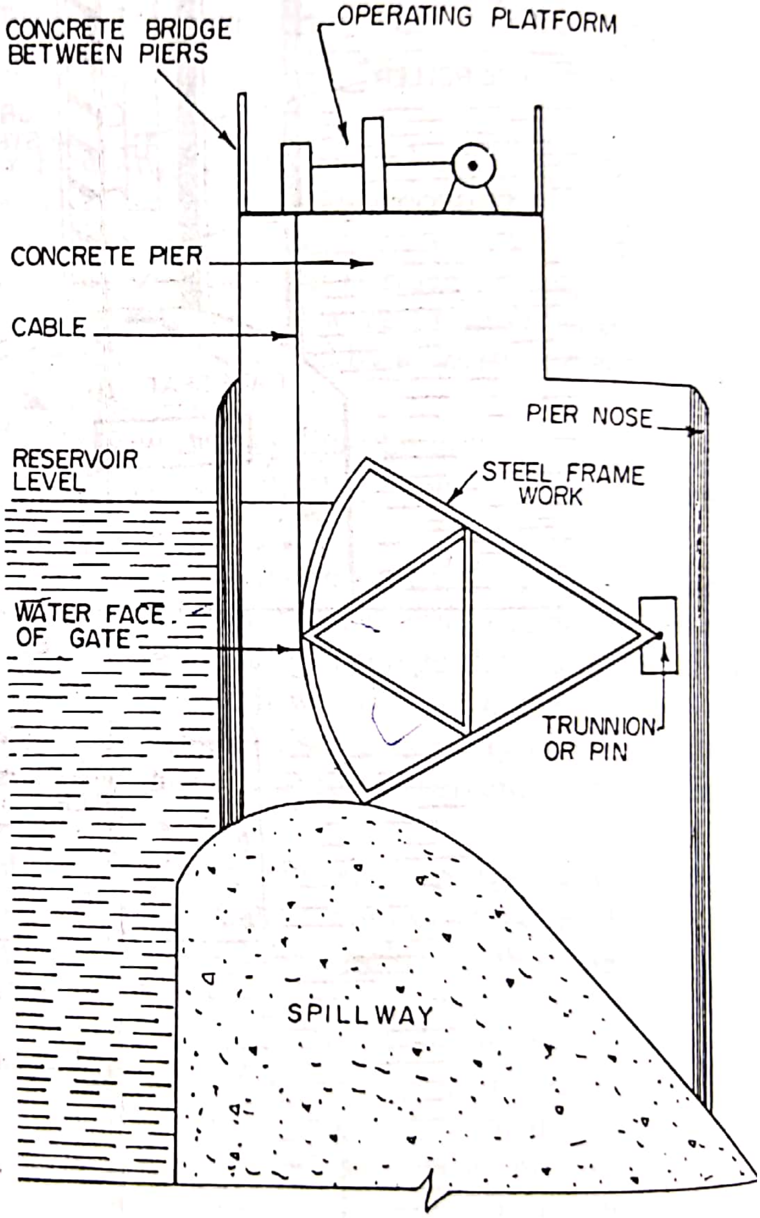


Fig. 21.39. Radial gate or Tainter gate.

The water-face segment is made concentric to the supporting pins so that the entire water thrust passes through the pins, thus creating no moment against the lifting of the gate. Hence, the lifting force is required only against the weight of the gate, the friction between the seals and the piers, and the frictional resistance at the pins. Counter weights, in order to counter balance the self weight, may also be used, which further reduces the lifting force. Moreover, the hoisting load is nearly constant for all gate openings. Hence, radial gates can be used with smaller lifting force for all heads, and hand operating hoisting mechanism may suffice for smaller works ; whereas in the vertical lift gates of the same size, power mechanism might be needed.

21.17.5. Drum Gates. Drum gates are useful for longer spans of the order of 40 m or so and medium heights say 10 m or so. The drum gate consists of a segment of a cylinder which may be raised above the spillway crest or may be lowered into the *recess* made into the top of spillway.

The U.S.B.R. drum gate (Fig. 21.40) is completely enclosed and is hinged at the upstream end. The buoyant forces due to head water pressure underneath the drum, aid in its lifting. In this type of dam gate, the drum is enclosed on all the three sides as well as on the ends, thus forming a water tight vessel. When the drum is lowered, it fits into the recess in such a way that the surface becomes coincident with the designed ogee shape of the crest.

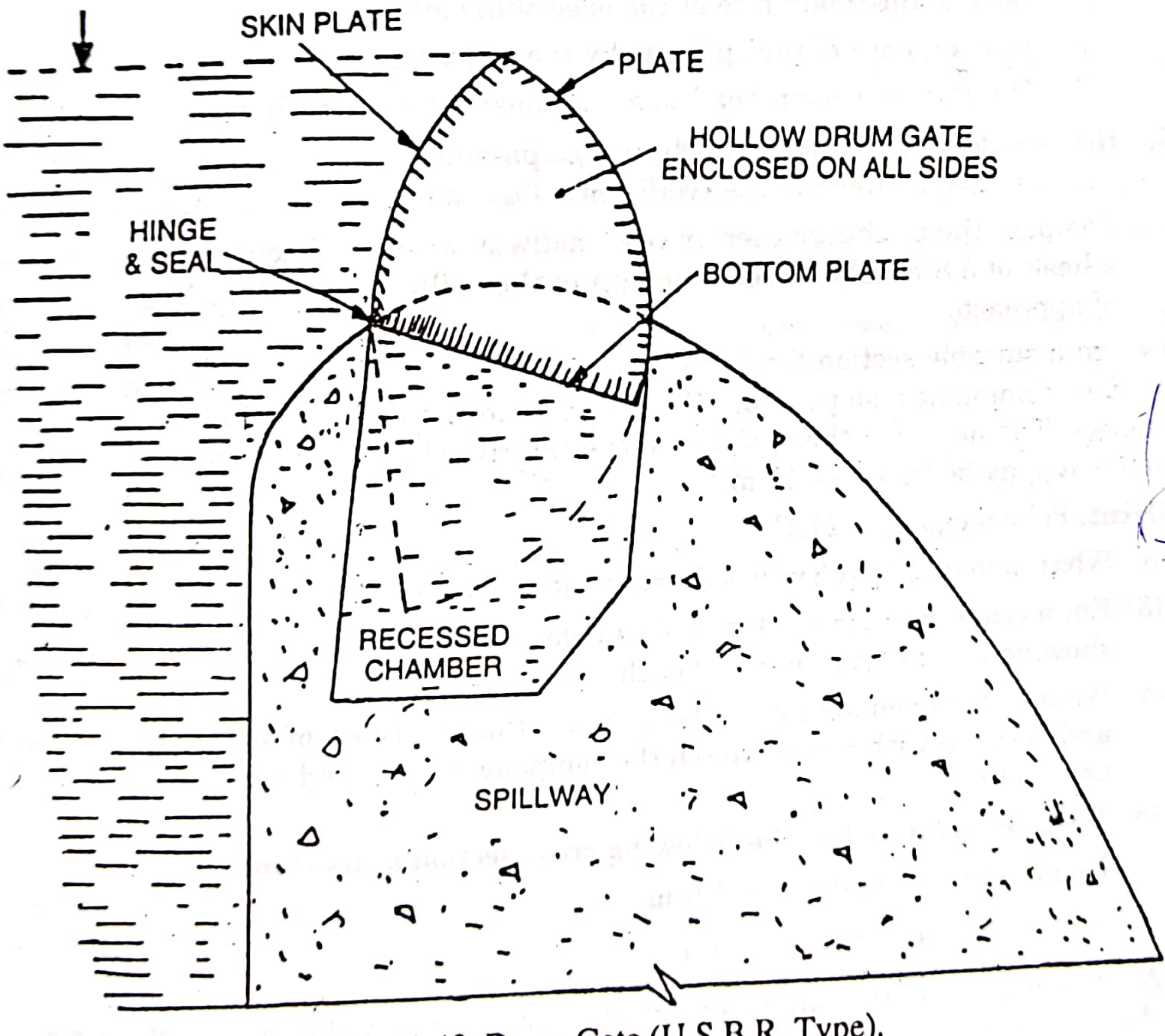


Fig. 21.40. Drum Gate (U.S.B.R. Type).

The other type of drum gate may have no bottom plate and shall be raised only by the buoyant action of water entering the recess, underneath the skin plate of drum.

The drum gates require large recess and hence, are not suitable for smaller spillways.