Syllabus:

3.3 Irrigation

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1. Chapter 1: Irrigation General Definition and Background

The science of artificial application of water to the land, in accordance to the crop requirements of the land thought the crop period for full-fledged nourishment of crops is called irrigation.

The rainfall in Nepal occurs mainly in four monsoon months. The distribution of rainfall is non uniform and the rainwater can't fulfill year round demand of the crops. To fulfill the water demand of the Crops, water has to be supplied artificially which is called irrigation.

1.1. Advantages of Irrigation:

1. Increase in Food Production: Irrigation increases crop yield and helps to attain self sufficiency in food.
2. Optimum Benefits: By optimum utilization of water, benefit from irrigation will be maximized.
3. Elimination of Mixed Cropping: When water is not available in sufficient, two crops are sown together so that the one crop for which the condition becomes favorable will grow. By ensuring the required amount of water through irrigation, mixed cropping can be eliminated.
4. General Prosperity: The revenue generated due to increase in production of crops will bring prosperity.
5. Generation of Hydroelectricity: If some of the head is available for the production of hydropower, electricity can also be generated. E. g; 48 MW power from Bheri-Babai diversion, power from Gandak Barrage, etc.
6. Domestic Water Supply: Irrigation water can also be supplied for domestic uses for sanitation, agriculture, livestocks.
7. Facilities of Communication: The embankments and the barrages can be used for the physical communication (transportation).
8. Inland Navigation: The larger irrigation canals can be used for the inland navigation.
9. Afforestation: Trees if planted along the canal banks aid in afforestation works.

1.2. Disadvantages of Ill Effects of Irrigation:

1. Water Pollution: The chemicals used for irrigation may join the ground water and the surface sources of water causing the water pollution.
2. Irrigation may result in the colder and damper climate resulting the marshy land and breeding of mosquitoes causing out breakage of the diseases like Malaria and Dengu.
3. Over irrigation may lead to water logging and may lead to reduction in crop yields
4. The huge cost is needed to develop irrigation projects but it is difficult to charge the farmers for water fee. So irrigation increases Government's investments.

1.3. Status of Irrigation Development in Nepal:

Majority of Nepalese population is directly and indirectly connected with the irrigation. There are small to large irrigation systems in Terai but only few irrigation systems are developed in hills. Based on the annual report 2076/77 of Department of Water Resources and Irrigation, the status of irrigation development is as follows.

Total cultivable area = 2.64 million ha

Total irrigable land = 1.76 million ha (due to the difficult topography of the country, only this much land can be irrigated.

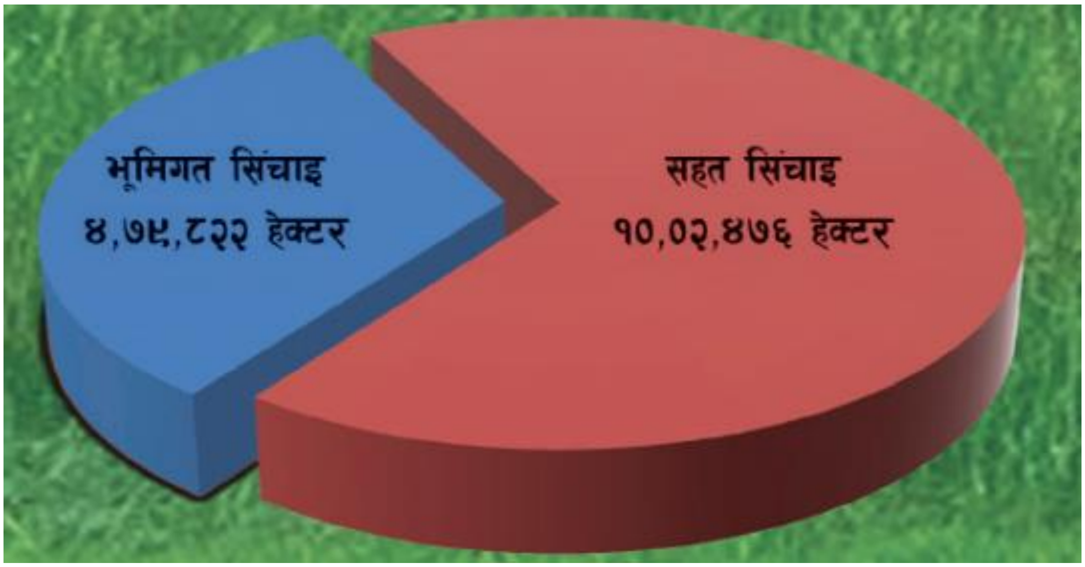
Upto the end of the fiscal year, 2076/77, the status of the irrigation facilities is presented below.

Surface irrigation = 10,02,746 ha = 10.027 Million ha

Underground irrigation = 4,79,882 ha = 0.479 Million Ha

Total irrigated land = 14,46,731 ha = 1.446 Million Ha

In the annual report of DWRI, the area irrigated due to farmer watercourses and surface irrigation are merged.



Source: Annual Book of Department of Water Resources and Irrigation (2076/77)

1.4. Types of Irrigation and Their Suitability

Irrigation may be broadly classified into two categories, surface and sub-surface irrigation. The generally used irrigation techniques are:

1. **Surface Irrigation**

   Surface irrigation can be further classified into

   - Free Flooding or Ordinary Flooding
   - Boarder Flooding
   - Check Flooding
   - Basin Flooding
   - Furrow Irrigation

2. **Sprinkler Irrigation**
3. **Lift Irrigation**
4. **Drip of Trickle Irrigation**

   The brief description of these methods is given below.

1. Surface Irrigation

In this method, the water is applied on the field by flooding.

The following are the techniques of surface flooding.

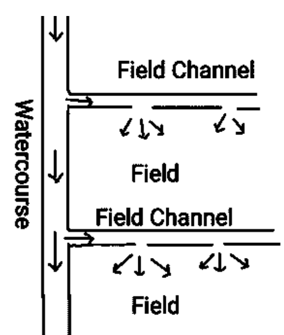


Fig: Free flooding

i. Free Flooding or Ordinary Flooding:

In this method, water is applied to field without any control structures (control structures are levees or boarders – आलीहरू). There are no levees (आलिहरू) to control the flow of water and this method is also called wild flooding. The application efficiency is low for this type of irrigation and is suitable for close growing crops, pastures (गौचरण) and steep slope.

ii. Border Flooding: In this method, the land is divided into a number of strips by the low levees called borders.

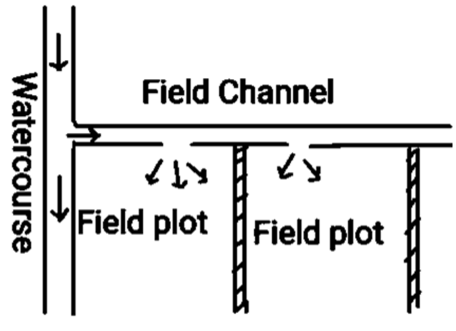


Fig: Boarder flooding

The water is supplied to each strips through the ditches. The field plots are provided with longitudinal slope of 0.5 to 1.5 %. This method is becoming more popular these days. The field plots are usually 10 to 20 cm in width and 100 to 400 m in length.

iii. Check Flooding: In check flooding, water is controlled by low and flat levees (आलीहरू).

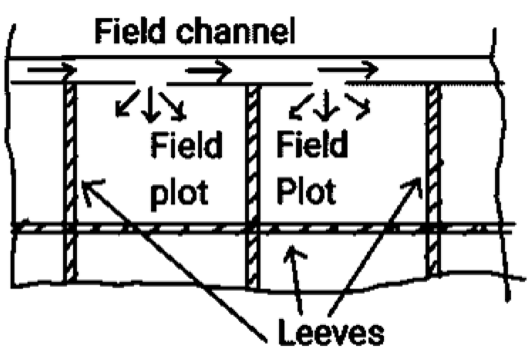


Fig: Check flooding

Levees are generally constructed along the contours, having vertical interval of 5 to 10 cm. This method is suitable for all types of soils (fine and coarse). This method is adopted for cereal crops (अन्नबाली) and the crops that can resist inundation for certain time.

iii. Basin Flooding: The basin is a check area formed around a tree or a group of trees. This method is special type of check flooding and is adopted specially for orchard trees. One or more trees are generally placed in the basin, and the surface is flooded as in check method by ditch water. This method is suitable for orchard trees and nursery.

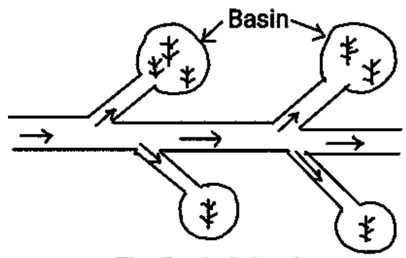


Fig: Basin Irrigation

v. **Furrow Irrigation Method:** Furrows are the field ditches, excavated between the rows of plants and carry irrigation water through them. Furrows have depth from 8 cm to 30 cm and may be upto 400m long. Water may be supplied to the furrows and water seeps laterally to the soil containing the crops. The crops are grown on ridges.

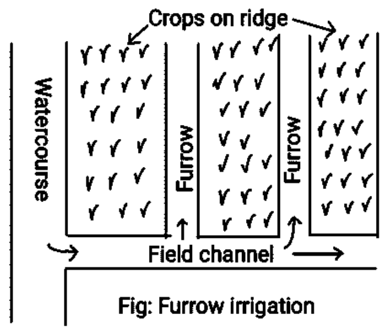


Fig: Furrow irrigation

2. **Sprinkler Method:** Water is applied to the soil from the sprinklers in the form of artificial rains. It consists of network of main, lateral and sprinklers. The loss of soil due to the flow of the water above it is avoided. Water is supplied to the sprinklers from the network of the pipes.

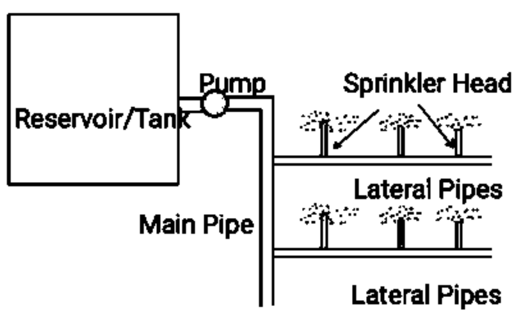


Fig: Sprinkler Irrigation

Favorable Conditions for the Sprinkler System

- When the land topography is irregular and hence unsuitable for surface irrigation (flooding irrigation).
- When the land gradient is steeper and the soil is easily erodible.
- When the soil is excessively permeable or highly impermeable.
- When there is scarcity of water resources.

Advantages of the Sprinkler System

- The seepage losses in the canals is eliminated.
- No cultivation area is lost for making ditches, levees.
- Land levelling is not required.

- Fertilizers can be applied uniformly by applying with the irrigation water.
- Loss of water due to over flooding, deep percolation is avoided.

Disadvantages of Sprinkler System:

- High wind may distort sprinkler pattern, causing non-uniform spreading of water on the crops.
- In areas of high temperature and high wind velocity, considerable loss of water may take place.
- Not suitable for the crops requiring large depth of water like paddy.
- Initial cost of the system is high.
- Requires larger electrical power.
- There is risk of abrasion and clogging of the nozzles of sprinkler.

3. Lift Irrigation

Lift irrigation is the system of irrigation where the irrigation water is raised to certain elevation by lifting through the pumps. When it is difficult to construct large headworks in the river for water diversion or if the command area is at higher elevation than the source, lift irrigation is preferred. The major difference of lift irrigation with other schemes is that the water is raised upto the lift chamber/reservoir from the source with the help of pumps. After that, water flows under gravity. If electricity is available at low cost, this method of irrigation becomes more economical.

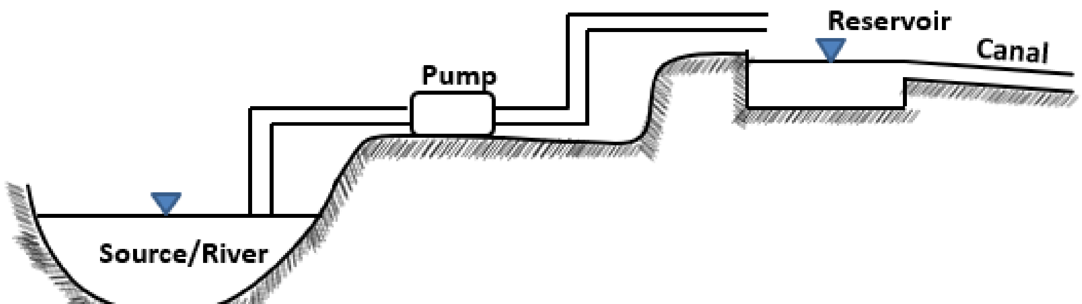


Fig: Lift Irrigation

4 Drip Irrigation: Drip irrigation method is the method is irrigation in which the irrigation water is directly applied to the root zone of crops with the help of the network of pipes and nozzles. The pipes and nozzles may be laid on the ground surface or below the ground surface. It is specially adopted in the areas where there is huge scarcity of water. The evaporation and the percolation loss is eliminated by this method.

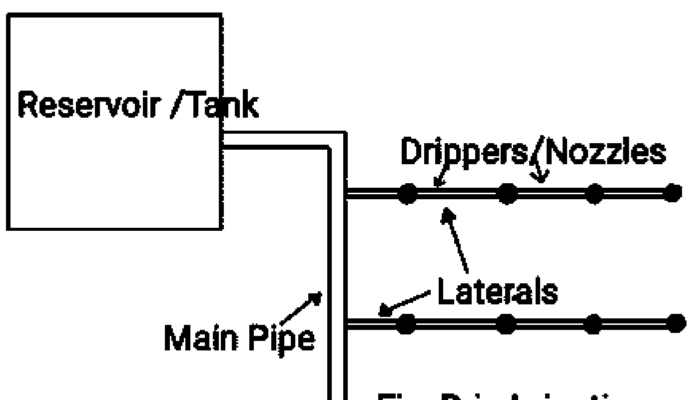


Fig: Drip Irrigation

Advantages:

- Economical use of water, so application efficiency is high.
- Suitable if there is scarcity of water.
- No evaporation and percolation loss.
- Fertilizer can be mixed with water and applied uniformly in the crops.
- No loss of land in the construction of field ditches and watercourses.

Disadvantages:

- Requires large initial investment. This is the costliest method of irrigation water distribution.
- There is chances of clogging (blocking) of the nozzles.
- Not suitable for irrigating the large area.
- Required skilled manpower.
- Not suitable for low value crops.

1.5. Need of Irrigation in Nepal:

Nepal is agriculture dependent country. A large fraction of Nepalese economy is shared by irrigation. Irrigation development is needed in Nepal due to following reasons

a. About 75 to 80 % of rainfall in Nepal occurs in 4 monsoon months. The rain water cannot satisfy water requirement of crops and rainfall is non uniform as well. So, to meet water requirement of crops which is not satisfied by rainfall, artificial irrigation is required.
b. A large amount of culturable area is still unirrigated. If irrigation facilities are provided in this area, the crop yield would increase.
c. For maintaining self-dependency in crop and food.
d. To raise the economic status of the country.
e. To enhance the financial status of the farmers.
f. The term 'Water requirements 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. Water requirement, will vary with the crop as well as with the place. In other words, different crops will have different water requirements, and the same crop may have different water requirements at different places of the same country depending upon the variations in climates, type of soils, methods of cultivation, and useful rainfalls, etc.
g. The area where irrigation is a must for agriculture is called the arid region, while the area in which inferior crops can be grown without irrigation is called a semi-arid region.

2. Chapter 2: Irrigation Water Requirement

2.1. Crop Period, Base Period, Delta and Duty

Crop Period and Base Period

The time period that elapses from the instant of its sowing to the instant of its harvesting is called the crop-period. The time between the first watering of a crop at the time of its sowing to its last watering before harvesting is called the Base period of the Base of the crop, Crop period is slightly more than the base period, but for all practical purposes, they are taken as one and the same thing, and generally expressed in days.

Delta (∆)

The depth of water required during the base period of the crop for it's full growth is called delta of water. It is the sum of total water applied during the base period of crops.

The delta of certain important crops are given below.

S. No. (1)	*Crop* (2)	*Delta on field* (3)
1.	Sugarcane	120 cm (48″)
2.	Rice	120 cm (48″)
3.	Tobacco	75 cm (30″)
4.	Garden fruits	60 cm (24″)
5.	Cotton	50 cm (22″)
6.	Vegetables	45 cm (18″)
7.	Wheat	40 cm (16″)
8.	Barley	30 cm (12″)
9.	Maize	25 cm (10″)
10.	Fodder	22.5 cm (9″)
11.	Peas	15 cm (6″)

Duty (D):

Duty of water is defined as the area of land in hectares that can be irrigated by supplying 1 m^3/sec water throughout the base period of the crop. If duty of water for a crop is 864 hectares/cumecs, it means that 1 m^3/sec water can irrigate 864 hectares of that crop. Note that m^3/sec and cumecs are same

Unit of duty is hectares/cumecs and the relation of duty with area and discharge is:

$$\text{Discharge (Q)} = \frac{\text{Area (A)}}{\text{Duty (D)}}$$

So, by knowing duty, we can calculate the discharge required.

Relationship Between Duty and Delta:

From the definition of duty, let us assume that 1 m3/sec water can irrigate D hectares land. The base period of the crop if B and the water requirement of crop is ∆ meters.

D = duty in hectares/cumec

∆ = total depth of water supplied in metres

B = base period in days

Volume of water required to irrigate D hecratres land with water requirement ∆

V1 = Area*Depth

= D Hectares * Δ meters

= D * 104 m^2 * Δ m

= 104 D* Δ m^3 ………….. (1)

Again for the same field of D hectares, one cumec of water is required to flow during the entire base period. Hence, water supplied to this field.

V2 = Discharge *Time

= 1 m3/sec * B Days

= (1)×(B×24×60×60) m^3

V2 = 86400*B m^3 ... (2)

Equating Equations (1) and (2), we get

104 D* Δ = 86400*B

or, D = $\frac{86400*B}{10^4*\Delta} = \frac{86400*B}{10000*\Delta} = \frac{8.64B}{\Delta}$, where B is in days and Δ is in meters.

If B is in days and Δ is in centimeters, then duty is obtained by:

D = $\frac{864*B}{\Delta}$…………… (3)

In conclusion,

Duty, D = $\frac{8.64B}{\Delta}$ if B is base period in days and Δ is in m and

D = $\frac{864*B}{\Delta}$ if B is base period in days and Δ is in cm.

Note that if Base period increases, duty increases and if delta increases duty decreases.

2.2. *Cropping Season and Principal Crops*

From agricultural point of view, the cropping season is mainly divided into Rabi season and kharif season. The type of crops in each season may be different however, some crops grow in both seasons.

a. Rabi Season/Rabi Crops

- Generally Rabi season starts from October 1st October and ends at March 31st.
- Rabi crops are also called winter crops
- Rabi crops consume less water than the kharif crops
- Some examples of Rabi crops are wheat, barley (जौं), gram, mustard, potato, tobacco, etc. are the Rabi crops.

b. Kharif Season/Kharif Crops

- Generally, Kharif season starts from 1st April to September 30th.
- Kharif crops are also called summer crops.
- Kharif crops consume more water than Rabi Crops.
- Some examples of Kharif crops are rice, bajra, jowar, cotton, groundnut, etc.

c. Perennial Crops

- The base period of perennial crops lies in both Rabi and Kharif seasons.
- Examples of perennial crops are Sugarcane and Garden Crops.

2.3. Kor Watering

The first watering which is given to a crop when the crop is few centimeters high is called Kor – Watering. It is the maximum single watering. Other waterings are done after Kor watering usually at regular intervals. To understand Kor watering more clearly, Kor – Depth and Kor period must be understood. While fixing design discharge of canals, we consider Kor Watering.

a. Kor Depth: The depth of water required for Kor Watering is called Kor depth. Kor depth is different for different crops. Kor depth for rice is 19.00 cm, kor depth for wheat is 13.5 cm and kor depth for sugarcane is 16.5 cm.

b. Kor Period: The time period during which water demand of crops is more than average demand is called Kor – Period. It is also defined as the time period during which Kordepth is applied. Kor period of rice is 2 to 4 weeks (generally taken as 2 weeks) and KorPeriod of wheat is 3 – 8 weeks (generally 4 weeks).

Kor Duty: $D_{Kor} = 864*\frac{B_{Kor}}{\Delta_{Kor}}$ if the value of Δ is in cm

Also: $D_{Kor} = 8.64*\frac{B_{Kor}}{\Delta_{Kor}}$ if the value of Δ is in meter.

2.4. Some Important Terms

a. Crop Ratio

The area to be irrigated for Rabi crop is generally more than that irrigated for Kharif crop. This ratio of proposed areas to be irrigated in RabiSeason to that in the Rabi Season is called Crop Ratio. This ratio le generally 2:1.

Crop Ratio = $\frac{Area\ Irrigated\ in\ Rabi\ Season}{Area\ Irrigated\ in\ Kharif\ Season}$ = 2:1

b. Paleo Irrigation

Sometimes, in initial stages before de cop is sown, the land is very dry. This particularly happens at the time of swing of Rabi crops because of hot September, when the soil may be too dry to be sown easily. In such case, the soil is moistened with water, so as to help in sowing of the crops. This is known as Paleo irrigation. Thus paleo irrigation is defined as the first watering applied to the soil before the crop is grown so as to bring the soil to required moisture level.

c. Cash Crops

A cash crop may be defined as a crop which has to be encashed in the market for processing, etc. as it cannot be consumed directly by the cultivators. All non-food crops, are thus, included in cash crops. Crops like jute, tea, cotton, tobacco, sugarcane, etc. are, therefore, called cash crops. The food crops like wheat, rice, barley. maize, etc. are excluded from the list of cash crops

d. Outlet Discharge Factor

The duty at outlet of the canal or head of watercourse is called outlet discharge factor. Outlet discharge factor is useful to calculate the discharge of a watercourse.

2.5. Irrigation Efficiencies

Irrigation Efficiencies Efficiency is the ratio of the water output to the water input.Various types of efficiencies are given below.

a. Efficiency of Water-Conveyance (η_c)

It is the ratio of the water delivered in fields from the outlet point of the channel, to the water entering into the channel a starting point.

η_c = conveyance efficiency = $\frac{Water\ delivered\ at\ the\ end\ of\ channel}{water\ entering\ the\ starting\ point\ of\ channel}$

The water lost in the channels is called conveyance loss.

b. Efficiency of water application (η_a)

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 also be called on farm efficiency, as it takes consideration the water lost in the farm. The water lost during application of water to fields is called application loss.

$$\eta_a = \text{Application efficiency} = \frac{Water\ stored\ in\ the\ root\ zone\ of\ crops}{water\ delivered\ to\ the\ field}$$

c. Efficiency of water-storage (η_s)

It is the ratio of the water stored in the mod during irrigation to the water needed in the root zone prior to irrigation capacity-existing moisture content).

Efficiency of Water Storage = $\eta_s = \frac{Water\ Stored\ in\ the\ root\ zone}{Water\ needed\ in\ the\ root\ zone} = \frac{Water\ Stored\ in\ the\ root\ zone}{Field\ Capacity - Existing\ Moinsure\ Content}$

d. Efficiency of water use (η_u)

It is the ratio of the water beneficially used, including leaching water, to the quantity of water delivered.

2.6. Irrigation Water Requirement

Irrigation water requirement is defined as the amount water required for the crops that should be provided by irrigation.

a. Consumptive Use (C_u):

The amount of water utilized by the plant in transpiration (building plant tissues, etc.) and evaporation from adjacent soils or plant leaves is called consumptive use. This is also called evapotranspiration.

b. Effective Rainfall (Re):

Precipitation (rainfall) falling during the growing period of the crop that is available to meet water requirement of crop is called effective rainfall.

c. Land Preparation and Leaching Water Requirement (L.P and Le.):

Before seeding (बिउ छर्ने)and transplanting (रोप्ने)certain crops some additional water is required. This is called land preparation water requirement. Also some more water may be needed in certain soils to remove harmful salts. This amount of water required is called leaching water requirement. land preparation water requirement is also defined as the amount of water required to bring the soil to required moisture level.

d. Irrigation Water Requirement:

The amount of water that is to be supplied by irrigation for fulfilling the water requirements of crops is called irrigation water requirement. The irrigation water requirement is different from crop water requirement. Crop water requirement means how much water is required to the crops and irrigation water requirement means how much water is to be supplied by irrigation.

1. **Consumptive Irrigation Requirement (CIR):** The amount of water to be supplied through irrigation to meet the consumptive use demand of crops is called consumptive irrigation requirement, CIR. Mathematically**,** **CIR = C_u – R_e**, Where, C_u = Consumptive Use and R_e = Effective rainfall.

2. **Net Irrigation Requirement (NIR):** The amount of water to be supplied at the root zone to meet water requirements of the crops is called Net Irrigation Requirement, NIR.

Mathemetically, NIR = C_u – R_e + L. P. and Le.

Where, L. P. and Le.= amount of water required for land preparation and leaching.

If Land Preparation and Leaching Water is not required , then, NIR = $C_u - R_e$

3. **Field Irrigation Requirement:** The amount of water required to be supplied to the field to meet water requirements of the crop is called Field Irrigation Requirement, FIR.

Mathematically, FIR = Consumptive Use - Effective rainfall + Field Losses. = Net Irrigation Requirement + Field Loss/Application Loss = NIR + Field Loss/Application Loss

Also, the field irrigation requirement can be calculated from,

$\eta_a = \frac{Net\ Irrigation\ Requirement,\ NIR}{Field\ Irrigation\ Requirement,\ FIR}$

or, Field Irrigation Requirement (FIR) = $\frac{Net\ Irrigation\ Requirement, NIR}{\eta_a}$

Where, η_a = Application efficiency**= water available at root zone/water supplied to field**

4. **Gross Irrigation Requirement (GIR):** The amount of water to be supplied from the canal outlet to meet water requirements of crops is called gross irrigation requirements, GIR.

Mathematically, GIR = Consumptive Use - Effective rainfall + Field Losses + Conveyance Losses.

GIR = Filed Irrigation Requirement + Conveyance loss

The conveyance losses include the amount of water that is lost as seepage, evaporation, etc, as it is flowing in canal.

Also, gross irrigation water requirement can be estimated by,

$\eta_c = \frac{Field\ Irrigation\ Requirement,\ FIR}{Gross\ Irrigation\ Requirement,\ GIR}$

or, Gross Irrigation Requirement (GIR) = $\frac{Field\ Irrigation\ Requirement, FIR}{\eta_c} = \frac{NIR}{\eta_a * \eta_c}$

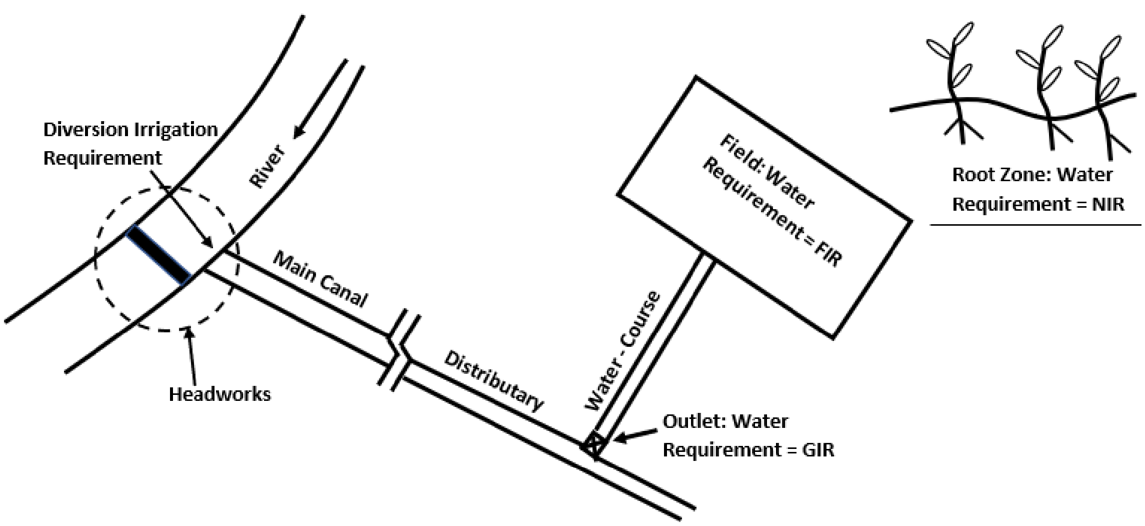
Where, η_c = Conveyance efficiency.

5. **Diversion Irrigation Requirement**

The amount of water required to be diverted from the hewadworks is called diversion irrigation requirement.

Diversion Irrigation Requirement = $\sum \frac{GIR}{\eta_{combined}}$

Where, $\eta_{combined}$ = combined efficiency of main canal, branch canal and distributaries



2.7. Command Area

Command area of any irrigation project or a canalis defined as the area irrigated by that project or the canal. The command area may be expressed in following ways.

a. Gross Command Area (GCA)

It is defined as the area bounded within the boundary of an irrigation project, which can be irrigated economically without considering the limitation of quantity of available water. It includes cultivable as well as uncultivable area. Ponds, reserved forests, roads, residential areas are the uncultivable areas.

b. Cultivable or Culturable Command Area (CCA)

The part of Gross Command Area on which cultivable is possible. It does not include the uncultivable area. The CCA may be taken as 80 % of GCA in absence of data.

CCA = GCA – Uncultivable area

c. Net Command Area (NCA)

The command area which is obtained by deducting (घटाउने) the area occupied by canal and canal structures from Cultivable Command Area.

Therefore, NCA = CCA – Area Occupied by Canal and Canal Structures

d. Intensity of Irrigation:

Intensity of irrigation of a particular season and a particular crop is the ratio of the area of that crop actually irrigated in that season to the cultivable command area.

Irrigation intensity = $\frac{Area\ Actually\ Irrigated}{Cultivable\ Command\ Area}$

The annual irrigation intensity is the sum of irrigation intensity of the all seasons throughout the year. The annual irrigation intensity can be more than 100%.

2.8. Soil Moisture Irrigation Relationship

The amount of moisture present in the soil is called soil moisture. The types of water contained by soil may be classified as

Saturation Moisture Content: The amount of moisture that is present in the soil when all of its pores has been filled by water is called saturation moisture content. This is the maximum moisture content in the soil.

Gravity Water: The amount of water which will drain out if the soil is allowed to drain under gravity is called gravity water.

Field Capacity: The amount of moisture that is retained by the soil after drainage has taken for sufficient time is called field capacity.

Field Capacity = Saturation moisture content – gravity water

Capillary Water: The moisture that is held in pores of soil due to surface tension is called capillary water. This moisture is useful to plants. This can be extracted by the crops through capillary action.

Hygroscopic Water or Adsorbed Water: The amount of water that is contained in the soil by chemical bonds is called hygroscopic water or adsorbed water. This water is not available to the plants. This water can be removed by heating.

Permanent Wilting Point, PWP: The moisture content below which the plant can not extract moisture from the soil is called permanent wilting point.

Optimum Moisture Content, OMC: The moisture content below which the moisture level is not allowed to fall for the proper growth of the plant is called optimum moisture content.

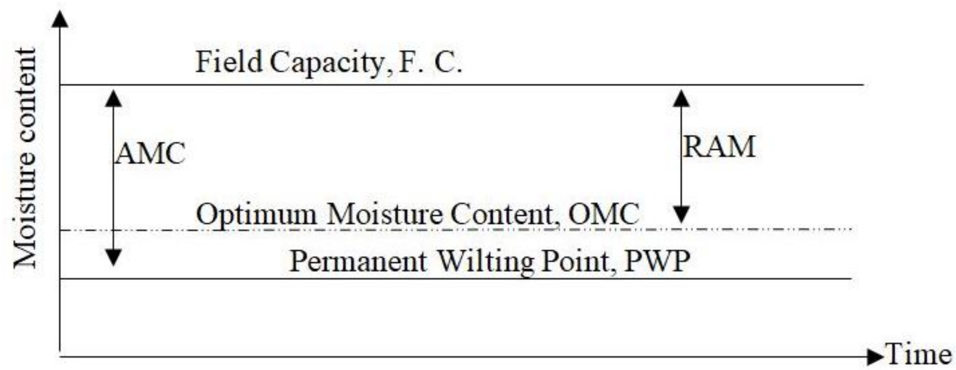
Available Moisture Content, AMC: The difference between Field Capacity and Permanent Wilting Point is called available moisture content.

Readily Available Moisture, RAM: The difference between Field Capacity and Optimum Moisture Content is called Readily Available Moisture. Normally, RAM = (75 to 80 %) × AMC.

Irrigation Interval: The time interval after which irrigation is done is called irrigation interval. Irrigation interval is calculated by:

Irrigation Interval = $\frac{\text{Extractable Moisture from Soil}}{\text{Daily Water Requirement of Crops (Cu)}}$

Soil Moisture Deficiency: The difference between the field capacity and existing moisture content is called soil moisture deficiency.



2.9. Types of Soils Based on Irrigation:

Soil type helps farmers decide what crops will grow best because some crops do better with particular types of soil. The best soil has a balance of nutrients, such as potassium, magnesium, and others. Certain crops may need more of one nutrient, so farmers put back those ingredients into the soil for better crops. Here are six types of soil and what to grow in each:

Sandy soil is not good for plants. However, melon and coconut grow in sandy soil. If water is available for irrigation then crops such as maize, millets, barley can be grown in desert soil. Cactus also grows in this soil

Clay soil is also not good for many plants. It is only good for crops like paddy, which require a lot of water.

Loamy soil is ideal for growing crops such as wheat, sugarcane, cotton, jute, pulses, and oilseeds. Vegetables also grow well in this soil.

Red soil is suitable for growing groundnuts, pulses, millet, cotton and tobacco

Black soil is ideal for growing crops such as cotton, sugarcane, tobacco, wheat, millets, and oilseeds.

Q. How do you fix design discharge of a distributary

1. Find the area irrigated in each season.

   Area Irrigated $= (I.I.) * CCA$ ← for each season.

2. Find the duty of irrigation for each season.

   $$\text{Duty } (D) = \frac{864\, B}{\Delta} \leftarrow \text{Average demand}$$

   $$\text{Kor Duty } (D_{kor}) = \frac{864 * B_{kor}}{\Delta_{kor}} \leftarrow \text{peak demand.}$$

   * we have to adopt kor duty.

3. Find the required discharge for each season/crops.

   $$Q_{Rabi} = \frac{A_{Rabi}}{(D_{kor})_{Rabi}} \leftarrow \text{Rabi crops only}$$

   $$Q_{kharif} = \frac{A_{kharif}}{(D_{kor})_{kharif}} \leftarrow \text{Kharif crops only}$$

   $$Q_{perenial} = \frac{A_{perenial}}{(D_{kor})_{perenial}} \leftarrow \text{perenial}$$

4. Find the required discharge of each season.

   $(Q)_{Rabi\ season} = Q_{Rabi} + Q_{perenial}$

   $(Q)_{kharif\ season} = Q_{kharif} + Q_{perenial}$ crops.

5. The maximum discharge among Rabi & Kharif season is the required design discharge.

3. Chapter 3: Canal and Canal Design

Definition

Canal is defined as the open conduit either lined or unlined which is used to convey water. A large number of canals are used to supply water to the field that is diverted from the source.

3.1. Types of Canals/Classification of Canals

Canals can be classified into following categories

a. Based on Function

i. Irrigation Canal: An irrigation canal is constructed to supply water for irrigation.

ii. Power Canal: A power canal is constructed for hydropower generation.

iii. Feeder Canal: A feeder canal is constructed to feed two or more other canals or branch canals.

iv. Navigation canal:A navigation canal is constructed for water transportation (जल यातायात).

v. Drainage Canal: A drainage canal is excavated to drain water of water logged area.

b. Based On Discharge

i. Main Canal: The main canal takes off directly from a river or reservoir. It carries water in large amounts to feed the branch and distributary canals. Due to conveying of very high discharge through the main canal it is not recommended to do direct irrigation from it.

ii. Branch Canal: The branch canal takes off from main canals at regular intervals. These canals supply water to major and minor distributary canals. The discharge of the branch canal is generally more than30 m^3 /sec. In the case of branch canals also, direct irrigation is not recommended unless their water carrying capacity is very low.

iii. Major Distributary: Major distributary canal takes off from the branch canal or in some cases from the main canal. They supply water to minor distributaries, watercourses and field channels. A major distributary has discharge capacity less than 30 m^3/sec.

iv. Minor Distributary/Minor: Minor distributary canal takes off from major distributaries and sometimes directly from branch canals depending upon the discharge of canals. Their discharge is generally below 2.5 m^3 /sec. These canals supply water to the field channels.

v. Watercourses: Watercourses in some cases also called Field channels are small channels excavated by cultivators in the irrigation field. These channels are fed by the distributary canals and branch canals through canal outlets.

c. Based on Source

i. Perennial Canal: A Perennial canal is a type of canal in which water is available throughout the year. This type of canal is generally directed from a perennial source of supply water bodies. Several Permanent hydraulic structures are constructed in this type of canal for water regulation and distribution. A Perennial canal can also be called as a permanent canal.

ii. Inundation Canal: Inundation canal is a type of canal in which water is available only during the flood periods. These type of canals are taken off from rivers to control the water level in rivers during floods. A canal head regulator is provided to regulate the flow into the canal.

d. Based On Alignment

i. Watershed/Ridge canal

A canal aligned along the ridgeline or watershed line of an area is said to be ridge canal or watershed canal. Since it is running at the peak altitude of the area, irrigation on both sides of the canal up to a larger extent of the area is possible. There is no interception of natural drains on ridge lines hence, no cross drainage works are required for this type of canal.

- Most preferred canal alignment
- Canal is aligned along watershed/ridge line
- It is suitable for plain areas, where slopes are relatively flat and uniform
- This type alignment ensures gravity irrigation on both sides of the canal
- Cross drainage structures are not required

ii. Contour Canal

A canal aligned roughly parallel to the contours of the area is called a contour canal. This type of canal can be seen in hilly regions. Since it is parallel to the contour line, the ground on one side of the canal is higher and hence irrigation is possible only on the other side of the canal. A contour canal has to pass the natural drainages (खोलाहरू/नदीहरू) and hence cross drainage works are required to be provided.

- Canal aligned parallel to the contour line is called contour canal
- They are aligned generally when canals take off from river.
- Can irrigate only on one side of canal as one of banks on the higher side.
- Sometime it is called single bank canal.
- Suitable for hilly area
- Cross drainage structures are required.

iii. Side Slope Canal

A side slope canal is that which is aligned at right angles to the contours; i.e. along the side slopes.

It is a canal wl ... ershed or valley. The canal thus run ... ainage works are not required.

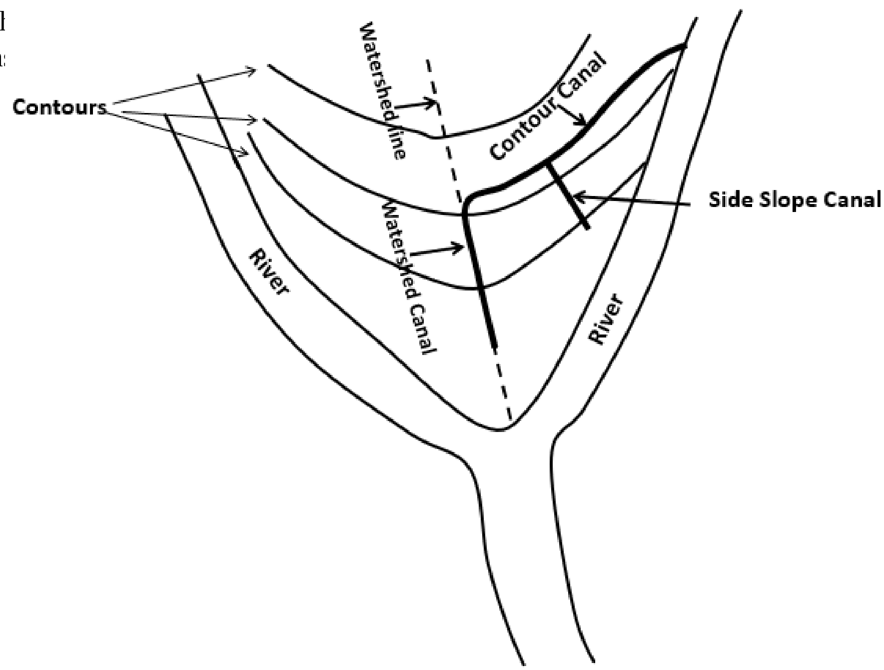


Fig: Different Alignments of Canal

e. Based on Boundary Surface of Channel

i. Alluvial Canal: The soil which is formed by continuous deposition of silt from water flowing through a given area is called alluvial soil and the canal formed on such soil is called alluvial canal. Alluvial canals are unlined canals.

ii. Non Alluvial Canal: If the boundary surface of the canal is of non-alluvial soils such as loam, clay, rock, etc. then it is said to be a non-alluvial canal.

iii. Rigid Surface canal: Rigid surface canals also come under non-alluvial canals but here the boundary surface of the canal is lined artificially with a hard layer of lining material such as cement, concrete, stones, etc.

3.2. Losses in Canals

During the passage of water from the main canal to the outlet at the head of the watercourse, water may be lost either by evaporation from the surface or by seepage through the peripheries of the channels. These losses are sometimes very high, of the order of 25 to 50% of the water diverted into the main canal. In determining the designed channel capacity, a provision for these water losses must be made. The provision for the water lost in the watercourses and in the fields is however, already made in the outlet discharge factor, and hence, no extra provision is made on that account. Evaporation and seepage losses of channels are discussed below:

a. Evaporation: The water lost by evaporation is generally very small, as compared to the water lost by seepage in certain channels. Evaporation losses are generally of the order of 2 to 3 per cent of the total losses. They depend upon all those factors, on which the evaporation depends, such as temperature, wind velocity, humidity, etc. In summer season, these losses may be more but seldom exceed about 7% of the total water diverted into the main canal.

b. Seepage: There may be two different conditions of seepage Absorption and Percolation.

i. Percolation: In percolation, there exists zone of continuous saturation from canal to the water-table and a direct flow is established. Almost all the water lost from canaljoins the ground water reservoir or ground water table.

ii. Absorption: In absorption, a small saturated soil zone exists round the canal section, and is surrounded by zone of decreasing saturation. A certain zone just above the water-table is saturated by capillarity. Thus, there exists an unsaturated soil zone between the two saturated zones, as shown in figure.

The seepage losses depend upon the following factors:

(i) Type of seepage, i.e. whether 'percolation' or 'absorption".

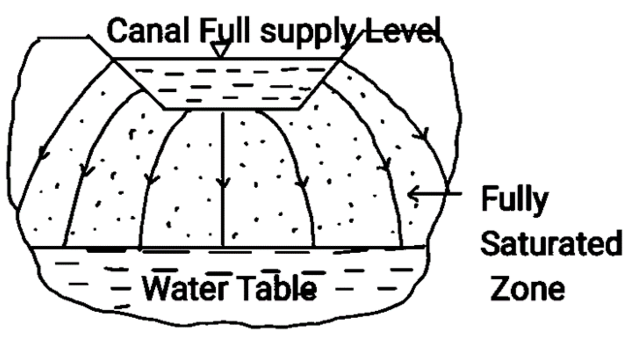
(ii) Soil permeability.

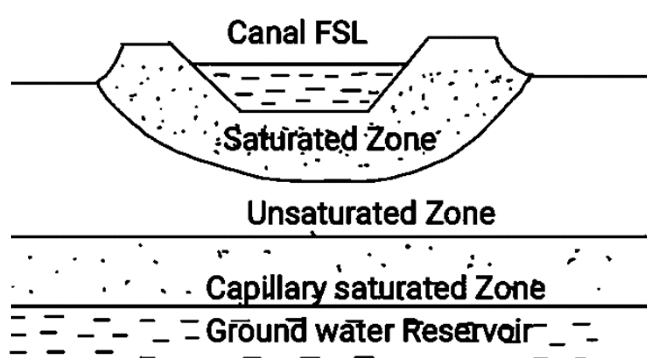
(iii) The condition of the canal: the seepage through a silted canal is less than that from a new canal.

(iv) Amount of silt carried by the canal: the more the silt, lesser are the losses.

(v) Velocity of canal water; the more the velocity, the lesser will be the losses.

(vi) Cross-section of the canal and its wetted perimeter.





3.3. Minimization of Losses in Canals

Technological measures to reduce seepage, leakage and percolation losses in irrigation include the lining of canals and watercourses, and promoting modern irrigation technologies such as pipe, sprinkler and drip systems.

a. Lining of Irrigation Canals: Lining of canal reduces seepage loss. It also reduces evaporation loss by reducing surface area of the channel. Canal lining materials may be concrete, shortcrete, brick, compacted/hard clay, etc.

b. Low pressure pipes can be used as conveyance structure. This reduces seepage and evaporation loss.

c. Sprinkler irrigation systems can be used as distribution systems. It reduces seepage and evaporation loss as well as the water use can be controlled.

d. Drip Irrigation systems can be used as distribution systems to canals. Water lost by seepage and evaporation can be minimized to large extent in drip irrigation system.

e. Maintenance of irrigation canals reduces loss from canals

f. Removal of vegetation and weeds from canals.

g. Using short length canals

h. Scheduled use of water helps to reduce loss from canals.

3.4. Lining of Canals

The process of treating the canal with non-erodible materials like concrete, bricks, stones and boulders is called canal lining. Canal lining is done to avoid the erosion and scouring of the canal cross section.

a. hard Surface Lining:

i. cement or concrete lining

ii. Shortcrete or plaster lining

iii. Tile or brick lining

iv. Asphaltic concrete lining

v. Boulder lining

b. Earth Type Lining:

i. Compact earth lining

ii. Soil cement lining

3.5. Advantages of Lining

1. **Water Conservation**: Lining a canal results in reduction in water losses, as water losses in unlined irrigation canals can be high.
2. **No seepage of water into adjacent land or roads**: If canal banks are highly permeable, the seepage of water will cause very wet or waterlogged conditions, or even standing water on adjacent fields or roads. Lining of such a canal can solve this problem.
3. **Reduced canal dimensions:** The resistance to flow of a lined canal is less than that of an unlined canal, and thus the flow velocity will be higher in the lined canal. Therefore, with the higher velocity, the canal cross-section for a lined canal can be smaller than that of an unlined canal.
4. **Reduced maintenance**: Maintenance costs for the following issues are eliminated using lining

of canals.

- Periodical removal of silt deposited on the beds and sides of canals.
- Removal of weeds and water canals.
- Minor repairs like plugging of cracks, uneven settlements of banks, etc.

5. Prevents water seepage through surface of the canal.
6. Helps in preventing Water-Logging
7. Increases discharge carrying capacity of the channel
8. Increases channel life and reduce the maintenance cost.
9. Increases Gross area under cultivation.
10. Silting is prevented as velocity is increased.
11. Prevents or Reduces Weed Growth.
12. Increase available head for power generation due to flatter gradient.

FINANCIAL JUSTIFICATION AND ECONOMICS OF CANAL LINING

In certain cases, the lining of a channel may be required from purely technical considerations. For example, a canal constructed on a high fill or a canal founded partly on rock and partly on permeable strata, may be unsafe, unless it is lined. Sometimes a hard lined surface may be required to withstand the high flow velocities, as in power channels. Apart from these special circumstances, the engineer is required to produce a good economic justification for the capital outlay that is likely to be invested on lining. In considering the economy of canal lining, it is necessary to evaluate the tangible (which can be measured in terms of money) and additional benefits, and then to compare these with the cost of lining. *Benefit cost ratio* can, therefore, be worked out, so as to justify the necessity of lining.

Mathematically speaking, expenditure on a project is justified if the resultant annual benefits exceed the annual costs (including interest on the capital expenditure) *i.e. Benefit cost ratio is more than one.* The justification for lining the existing channels is different from that of constructing new lined channels in a new project. It is because of the fact that a large number of additional advantages ; such as lesser earth-work-handling, lesser land acquisition, lesser impounding reservoir capacities, etc., are obtained in a new project, by adopting lining for new canals.

5.3. Justification for Lining the Existing Canals

(*i*) **Annual Benefits.** Irrigation water is sold to the cultivators at a certain rate. Let this rate be rupees R_1 per cumec. If m cumecs of water is saved by lining the canal, annually, then the money saved by lining $= mR_1$ rupees.

Lining will also reduce maintenance cost. The average cost of annual upkeep of unlined channel can be worked out from previous records. Let it be Rs. R_2. If p is the percentage fraction of the saving achieved in maintenance cost by lining the canal, then the amount saved $= p{\cdot}R_2$ rupees.

$\therefore$ *The total annual benefits* $= mR_1 + p{\cdot}R_2$ ...(5.1)

(The value of p is generally taken as 0.4)

(*ii*) **Annual Costs.** If the capital expenditure required on lining is C rupees, and the lining has a life of say Y years, then the annual depreciation charges will be $\frac{C}{Y}$ rupees. If r percent is the rate of annual simple interest, then a locked up capital of C rupees would earn, annually $C\left(\frac{r}{100}\right)$ rupees as interest charges, and since the capital value of the asset decreases from C to zero in Y years, the average annual interest cost may be taken as $\frac{C}{2}\left(\frac{r}{100}\right)$ rupees.

$\therefore$ The total *annual costs of lining*

$$= \frac{C}{Y} + \frac{C}{2} \times \frac{r}{100} \qquad \text{...(5.2)}$$

Benefit cost ratio $= \frac{\text{Annual Benefits}}{\text{Annual Costs}}$

$$= \left[\frac{m \cdot R_1 + p \cdot R_2}{\frac{C}{Y} + \frac{C}{2} \times \frac{r}{100}}\right] \qquad \text{...(5.3)}$$

If p is taken as 0.4, then

$$\text{Benefit cost ratio} = \left\{\frac{m \cdot R_1 + 0.4\, R_2}{\frac{C}{Y} + \frac{C}{2} \times \frac{r}{100}}\right] \qquad \text{...(5.4)}$$

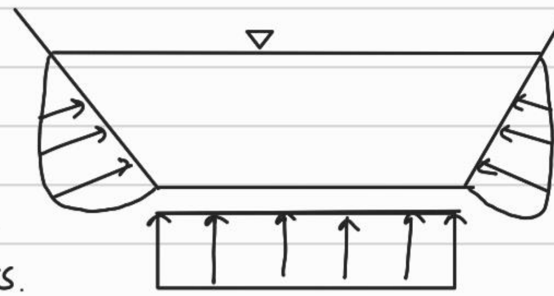
For project justification, benefit cost ratio must be greater than unity.

In addition to the benefits grouped above *i.e.* (water saving and reduction in maintenance cost) there may be benefits, like prevention of water–logging, reduced cost of drainage for adjoining lands, reduced risk of breaching, reduced incidence of malaria and other diseases in damp areas. Actual evaluation of these benefits is very difficult and may be approximated, based on experience, and may be taken into account for evaluating the annual benefit cost ratio.

Design of Unlined Canals:-

1. Tractive force Approach of Canal design (Stability कोताणी)

Tractive Stress:-



* The stress on channel cross-section due to flowing water is called tractive stress.
* Average tractive stress on bottom, $\tau = \gamma_w RS$
* Average tractive stress on side slopes, $\tau' = 0.75\gamma_w RS$

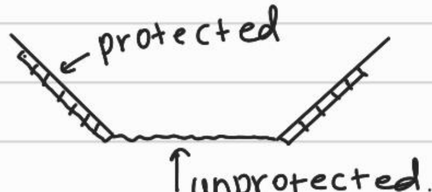
Concept of tractive stress design approach

* tractive stress (लाग्ने) $\leq$ critical shear stress (धान्ने)

Impending Motion Condition:-

* When tractive stress is equal to critical shear stress, the particle will be on the verge of motion. This is called impending motion condition.

a. Shield's method of Design of Channels with protected side slopes and Unprotected Bed



* for a particle at channel bottom, following two forces act

a. force due to flowing water = drag force

$= F_1 = \frac{1}{2} * C_D * \rho * A * V^2$

C_D = drag coefficient

ρ = density of water

A = Area (projected area) of particle

V = Velocity of flow.

b. Resisting force = frictional force

$= F_2 = W - U$

W = wt. of particle ; U = upthrust

or $F_2 = \frac{1}{6} * \pi d^3 * (\gamma - \gamma_w)$

For the particle to be in equilibrium,

$F_1 = F_2$ ..... i)

Target from eq^n i) shield found that

$$\frac{\tau_c}{\gamma_w (G-1) * d} = f(R_e^*) \quad ....\text{ ii)}$$

τ_c = critical shear stress (छान्ने) of bed particles

G = specific gravity of bed particles

d = diameter of particles.

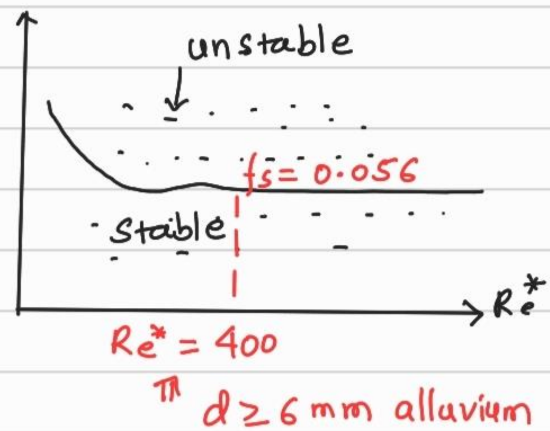
R_e^* = particle Reynold's no $= \frac{\rho * V^* * d}{\mu}$

V^* = shear friction velocity $= \sqrt{\frac{\tau}{\rho}}$

μ = coefficient of viscosity

Also; $\frac{\tau_c}{\gamma_w (G-1) * d} = f_s$ = shield's entrainment function .... iii)

* Shield provided a graph and showed that for alluviums with diameter ≥6mm and $Re^* \geq 400$, $f_s = 0.056$.



$\therefore \quad f_s = 0.056$

$$\Rightarrow \frac{\tau_c}{\gamma_w (G-1) d} = 0.056$$

or $\tau_c = \gamma_w * (2.65-1) * d * 0.056 \quad (G = 2.65)$

or $\boxed{\tau_c = \frac{\gamma_w d}{11}}$ .... iv)

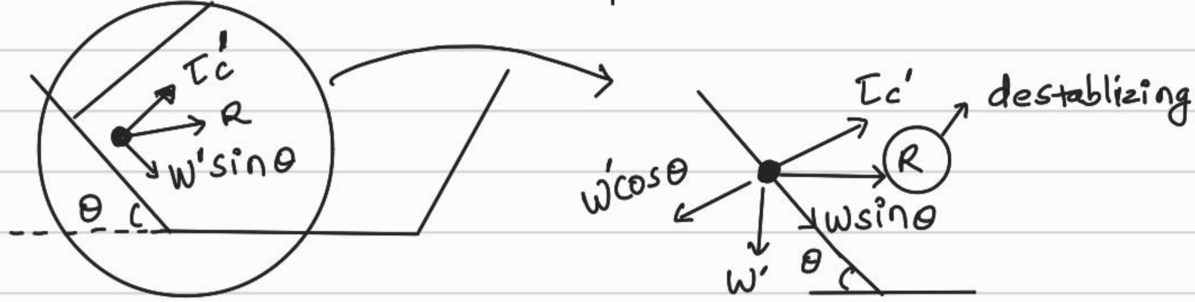
* from basic concept of tractive force approach theory: $\tau \leq \tau_c$ (tractive stress ≤ critical shear stress)

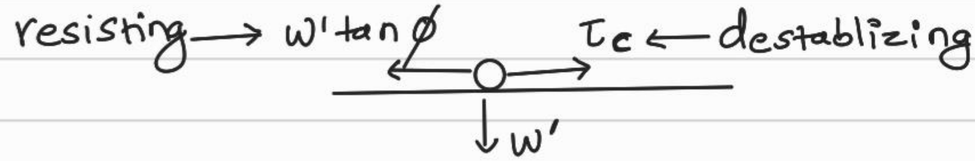
$$\Rightarrow \quad \gamma_w R S \leq \frac{\gamma_w d}{11} \quad (\because \tau = \gamma_w R S \text{ for bottom})$$

$$\Rightarrow \boxed{d \geq 11 RS}$$

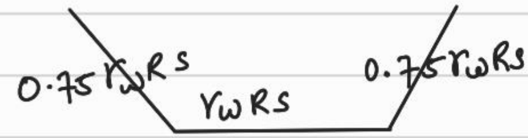
* Conclusion:- particle size/diameter ≥ 11RS, otherwise particle will not be stable.

b. Design of Canals with Unprotected side and Bed





let, τ = shear stress acting at bottom



τ' = shear stress acting at sides

τ_c = critical shear stress of bottom

τ_c' = critical shear stress of side slopes

w' = submerged weight of particle

θ = side slope angle

ϕ = angle of friction

<u>For bottom particle</u>

Resisting force = force causing motion

$\mu * w' = \tau_c$

$\Rightarrow \quad w' * \tan\phi = \tau_c \quad \ldots\ldots \text{i>}$

For side slope particles,

Resisting force = force causing motion.

$\mu * w\cos\theta = R$

$\Rightarrow \quad w'\cos\theta * \tan\phi = \sqrt{\tau_c'^2 + w'^2 \sin^2\theta} \quad \ldots \text{ii>}$

from i> & ii>

$$\frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} \quad \ldots\ldots \text{iii>}$$

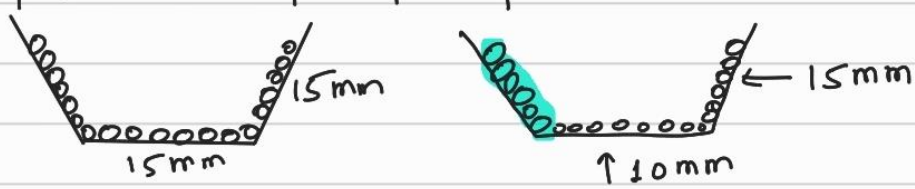
Here; $\dfrac{\tau_c'}{\tau_c} = K$ = tractive stress ratio

from iii> we see that $K < 1$. $\Rightarrow \tau_c' < \tau_c$

$\Rightarrow$ It shows that side slope particle is weaker than bed particles.

for example → side मा ठुलो चाहीन्छ ।

→ { particle size required for bed :- $d = 10mm$
particle size required for slope :- $d = 15mm$



option 1 option 2

➘ Therefore, since side slope particle is more critical, design shall be done for considering stability of side slopes or side slope particles should be larger than bed particles.

3.6. Design of Regime Channels:

The channels in which there is neither silting, nor scouring are called regime channels. For a channel to be in regime, the sediment carried by the canal water should be just carried by it and the canal water should not scour the canal bed.

Kenedy Theory:

According to kenedy, the non silting and non scouring condition can be achieved if the flow velocity is such that water can hold the sediments carried by it in suspension. The silt supporting power of the water is due to the eddies or turbulence generated in the canal by due to friction of water with the canal bed. So, he introduced the critical velocity and suggested that this velocity should be maintained in the canal to avoid silting and scouring. According to Lacey,

Critical velocity $V_c = o.55 * y^{0.64}$

Where, y = flow depth

For different soils, critical velocity will be different and so he introduced the critical velocity ratio, m.

Therefore, for any soil, $V_c = o.55 *m* y^{0.64}$, where m = critical velocity ratio.

Design Steps:

1. Assume appropriate depth
2. Calculate critical velocity, V_c = o.55 * $y^{0.64}$
3. Calculate the sectional area, A = Q/V, where Q = design discharge of the canal
4. Calculate the width of the canal from A = B*y + y^2*Z, where Z = side slope of the canal (z=0.5)
5. Check the critical velocity with the actual velocity, v. If the actual velocity and critical velocity match, the designed section is ok, else change the depth and follow the steps from 2 to 5 until critical and actual velocity are equal.
   Actual velocity, V $=\frac{\frac{1}{n}+(23+\frac{0.00155}{s})}{1+\left(23+\frac{0.00155}{s}\right)*\frac{n}{\sqrt{R}}}*\sqrt{RS}$
   Where, s = longitudinal or bed slope, R = hydraulic radius = A/P, P = wetted perimeter, A = Area

Lacey's Theory:

He after his research work suggested that the canal will not be in regime only because there is no silting and scouring. He suggested that the canal that is in true regime anf final regime can only be regime canal. Lacey has suggested three types of regime canals.

i. Initial Regime

When only bed slope of a channel changes due to dropping of silt and it's cross section or wetted perimeter remains unaffected, even the channel can exhibit "no silting and no scouring" properties, the channel is said to be in initial regime. In suchb channel, the side slopes of channels are protected and the side slopes can not change but bed slope changes and the channel shows non silting and non scouring bahaviour due to change in its bed slope. Such a channel looks regime channel outwardly but is not actually a regime section.

ii. Final Regime

If there is no resistance from sides and bed of the channel and channel cross section can change freely, the channel cross section gets adjusted according to the design discharge and flow condition. Such a channel is said to be in final regime and lacey theory is applicable to this.

iii. True Regime

For a canal to be in true regime, following conditions should be met because a channel that is regime at a slope will not be regime at some another slope. The channel which is regime at a discharge will not be regime at some another discharge bd so on.

- Discharge is constant
- Flow is uniform
- Silt charge is constsnt
- Silt grade is constant
- Channel is fowing through a medium which can be as easily deposited as easily it can be scoured.

If the above conditions are not met and there is no silting and scouring, the canal may be in initial regime or final regime but not in true regime.

The basic equations of lacey are:

$Af^2 = 140*V^5$ ………………………………. i)

$R = \frac{5}{2}*\frac{V^2}{f}$ ………………………………. ii)

$V = 10.8*R^{2/3}*S^{1/3}$ ………………………………. iii)

$f = 1.76\sqrt{d_{mm}}$

The derrived formula are:

$P = 4.75*\sqrt{Q}$

$V = (\frac{Q*f^2}{140})^{1/6}$

$R = 0.47*(\frac{Q}{f})^{1/3} = 1.35*(\frac{q^2}{f})^{1/3}$

$S = \frac{f^{5/3}}{3340*Q^{1/6}}$

Where,

V = velocity, P = wetted perimeter, R = Hydraulic Mean Depth = Lacey's Scour depth, Q = Discharge, f = silt factor, d_{mm} = mean particle diameter, S = bed slope

Design Procedure:

i. Calculate the velocity $V = (\frac{Q*f^2}{140})^{1/6}$
   Where Q = design discharge,
   f = silt factor $=1.76\sqrt{d_{mm}}$, where d_{mm} = diameter of the particles in mm.
ii. Calculate the flow area by A =Q/V
iii. Calculate the wetted perimeter by $P = 4.75*\sqrt{Q}$
iv. Solve the equations A = B*y + y2z and P = B +2y$\sqrt{1+Z^2}$, (where Z = side slope = 0.5) to get B and Y
v. Calculate the bottom slope by: $S = \frac{f^{5/3}}{3340*Q^{1/6}}$

Kenedy Numerical:

Design a stable channel that carries discharge of 50 m^3/sec through a channel of slope 1 in 4000 using Kenedy's theory. Assume Kutter's rougisity coefficient (n) = 0.023 and the critical velocity ratio, m =1.1

Solution:

Given

Q	50	cumec
n =	0.023	
m =	1.1	
s=	0.00025	

Assume

Z =	0.5	

Trial 1

Assume,

Y=	2.5	m
$\boxed{V_c =}$	1.09	m/sec
A=	45.976195	m^2

$$A = b * Y + Y^2$$

Formulas are

$$P = B+2*Y\sqrt{1+Z^2}$$

from the relation of A,	B=	17.14	m
	P=	22.73	m
	R=	2.02	m
Actual Velocity	V=	1.11	m/sec

Since, V is not equal to V_c , The calculated section should be revised.

Since V is less then V_c , increase Y to increase V. So take Y=3m

Trial No.	Y (m)	V_c (m/sec)	A (m^2)	B (m)	P (m)	R (m)	V (m/sec)	Remarks
1	2.5	1.09	45.98	17.14	22.73	2.02	1.11	Increase Y
2	3	1.22	40.91	14.57	21.27	1.92	1.07	Decrease Y
3	2.6	1.12	44.84	16.58	22.40	2.00	1.10	Adopted

Try to match at least two digits and stop the trial at Y=2.6 m.

Hence, the final section is:

Y=2.6m, Z =0.5 m, 16.58 m, V=1.1 m/sec

Lacey Numerical:

Design a suitable channel in alluvium with silt factor of 1.1 and the design discharge of 50 m^3/sec.

Solution:

Solution:

Given that:

Q	50	m^3/sec
f	1.1	

Step 1

v=	0.87	m/sec

Step 2

A= 57.50 m^2
Step 3
P= 33.59 m

We know,

P = B+2*Y$\sqrt{1+Z^2}$ Or, B = P - 2*Y$\sqrt{1+Z^2}$

$A = b * Y + Y^2$ or A =(P - 2*Y√(1+Z^2))*Y-ZY^2

Solving from the equation of A for B and Y,
Y= 1.90 m
B= 29.34 m
Step 4
Also, from Lacey's Formula, slope of the channel is:
S = 0.00018
Hence, the final Section is Y=1.9 m, B=29.34 m, s=0.00018

Comparsion of Lacey Theory and Kenedy theory.

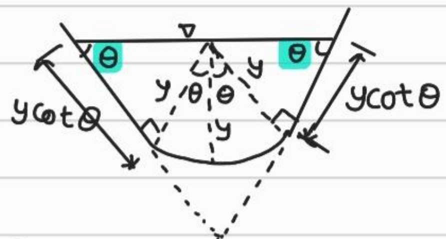
Kenedy	Lacey
1. All non silting and non-scouring channels are regime channels.	1. All non-silting and non-scouring channels can't be considered as regime channels.
2. Applicable for all regime channels.	2. Applicable for true regime and final regime
3. Particle size is not included in design parameter	3. Particle size is included in design parameter.
4. Turbulence is due to friction with channel bed only.	4. Turbulence is due to friction with wetted perimeter.
5. Based on trial and error approach	5. It gives final section. Trial & error is not required.
6. Requires bed slope/bed slope should be given.	6. Channel maintains required bed slope itself.

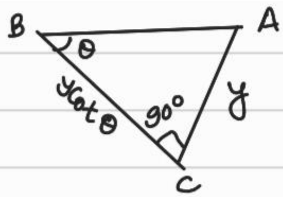
Design of Lined Channels:-

* lined channels should be designed to achieve economic sections.
* Bottom corners of lined channels are rounded to reduce wetted perimeter.

1. Triangular Channel:-

* lower corners of channel are rounded to achieve economical section.





from fig:

$$\tan\theta = \frac{AC}{BC} \Rightarrow BC = \frac{AC}{\tan\theta}$$

$$\Rightarrow BC = AC \cot\theta = y\cot\theta$$

* Area of section, A = Area of two Δ^s + Area of sector

Area of sector $= \frac{1}{2} y^2 * 2\theta$

($\because$ Area of sector $= \frac{1}{2} r^2\theta$)

$\therefore$ Area of sector $= y^2\theta$

Area of two triangles

$= 2 * \frac{1}{2} * y * y\cot\theta = y^2\cot\theta$

$\therefore$ Area of section $= y^2\cot\theta + y^2\theta = y^2(\theta + \cot\theta)$

Wetted perimeter, $P = 2y\cot\theta$ + Arc length

$\Rightarrow P = 2y\cot\theta + y * 2\theta$ ($\because$ arc length $= r*\theta$)

$= 2y(\theta + \cot\theta)$

$\therefore$ Hydraulic mean radius $= R = \frac{A}{P} = \frac{y^2 * (\theta + \cot\theta)}{2y(\theta + \cot\theta)}$

$$\boxed{\Rightarrow R = \frac{y}{2}}$$

Now; from manning's formula, $Q = \frac{A}{n} R^{2/3} S^{1/2}$

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

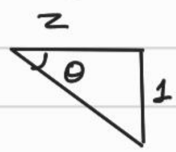
Note :- Velocity in canal should not be more than critical velocity

:- Discharge capacity of canal should not be less than design discharge.

Permissible Velocities in Lined Canals :-

Type of lining	Permissible velocity
PCC lining	2 to 2.5 ms^{-1}
Burnt clay lining	1.8 ms^{-1}
Boulder lining	1.5 ms^{-1}

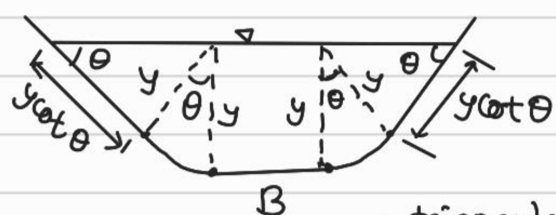
Side slope angle θ is found by assuming $z = 1.25$ to 1.5



$\tan\theta = \frac{1}{z} \Rightarrow \theta = \tan^{-1}\left(\frac{1}{z}\right)$

Trapezoidal Channel :-

* lower corners are rounded to achieve economical cross section or to reduce wetted perimeter.



triangular section

* Area = $\boxed{\text{two triangles + two sectors}}$ + one rectangle

$= y^2(\theta + \cot\theta) + By$

* Wetted perimeter $= y\cot\theta + y\cot\theta + B + 2 \times \text{Arc length}$

$= 2y\cot\theta + B + 2 \times y\theta$

* $R = \frac{A}{P} = \frac{y^2(\theta + \cot\theta) + By}{2y\cot\theta + B + 2y\theta}$

* from manning's formula, $Q = \frac{A}{n} R^{2/3} S^{1/2}$

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

Example 5.3. *Design a lined channel to carry a discharge of 15 cumecs. The available and accepted country slope is 1 in 9000. Assume suitable values of side slopes and good brick work in lining.*

Solution. Let us first of all assume that the side slopes of the channel be $1\frac{1}{4} : 1$ (*i.e.* $1\frac{1}{4} H : 1 V$) and the value of Manning's rugosity coefficient be 0.015 for good brick work. The channel section may be designed as triangular (as given in Fig. 5.3) because the discharge is small.

Considering Fig. 5.3, we have

$$\tan\theta = \frac{1}{1\frac{1}{4}} = \frac{1}{1.25}$$

or $$\cot\theta = 1.25$$

$\therefore$ $$\theta = 0.675 \text{ radians}$$

Using $$A = y^2(\theta + \cot\theta) \quad \text{...(5.5)}$$

and $$P = 2y(\theta + \cot\theta) \quad \text{...(5.6)}$$

We get $$A = y^2(0.675 + 1.25) = 1.925\, y^2$$

$$P = 2y(0.675 + 1.25) = 3.85\, y$$

$$R = \frac{y}{2} = 0.5\, y \quad \text{...(5.7)}$$

Now, using Manning's formula

$$Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot S^{1/2}$$

Q is given to be 15 cumecs.

$\therefore$ $$15 = \frac{1}{0.015} \times (1.925y^2) \cdot (0.5y)^{2/3} \frac{1}{\sqrt{9000}}$$

or $$15 = \frac{1.925}{0.015 \times 94.8} y^2 (0.63y^{2/3})$$

$$= 0.852\, y^{8/3}$$

$\therefore$ $$y^{8/3} = 17.6$$

or $$y = (17.6)^{3/8} = \mathbf{2.93.\ metres.}$$

Hence, use the section shown in Fig. 5.3 with 2.93 m depth and $1\frac{1}{4} : 1$ side slopes.

Example 5.4. *Design a concrete lined channel to carry a discharge of 350 cumecs at a slope of 1 in 5,000. The side slopes of the channel may be taken as $1\frac{1}{2}$: 1. The value of n for lining is 0.014. Assume limiting velocity in the channel as 2m/sec.*

Solution.
$$V = \frac{1}{n} \cdot R^{2/3} \cdot S^{1/2}$$

$\therefore$
$$2 = \frac{1}{0.014} R^{2/3} \cdot \frac{1}{\sqrt{5000}}$$

or
$$(2 \times 0.014 \times 70.8) = R^{2/3}$$

or
$$R = (1.98)^{3/2} = \mathbf{2.79\ m.} \qquad ...(i)$$

The channel section is assumed to be trapezoidal as shown in Fig. 5.4. For $1\frac{1}{2}$: 1 slopes ; we have, $\cot\theta = 1.5$ or $\theta = 34.1° = 0.59$ radians.

Using
$$A = y\,(B + y\theta + y\cot\theta) \qquad ...(5.8)$$
$$P = B + 2y\theta + 2y\cot\theta \qquad ...(5.9)$$

we get
$$A = y\,(B + 0.59\,y + 1.5\,y) = y\,(B + 2.09\,y)$$
$$P = B + 1.18\,y + 3y \qquad = B + 4.18\,y$$

But
$$A = \frac{Q}{V} = \frac{350}{2} = 175 \text{ sq.m.}$$

$\therefore$
$$175 = y\,(B + 2.09\,y)$$

or
$$\frac{175}{y} = B + 2.09\,y$$

or
$$\left(B = \frac{175}{y} - 2.09\,y\right) \qquad ...(ii)$$

From (*i*), we get
$$2.79 = R = \frac{A}{P} = \frac{175}{B + 4.18\,y}$$

or
$$2.79 = \frac{175}{\left(\frac{175}{y} - 2.09y\right) + 4.18\,y}$$

or
$$\left(\frac{175}{y} - 2.09\,y\right) + 4.18\,y = \frac{175}{2.79} = 62.7$$

or
$$175 - 2.09y^2 + 4.18y^2 = 62.7y$$

or
$$2.09\,y^2 - 62.7\,y + 175 = 0$$

or
$$y^2 - 30\,y + 83.7 = 0$$

or
$$y = \frac{30 \pm \sqrt{900 - 334.8}}{2} = \frac{30 \pm 23.8}{2} = \frac{30 - 23.8}{2}$$

(ignoring unfeasible + ve sign)

$$= \mathbf{3.2\ metres.}$$

But
$$B = \frac{175}{y} - 2.09\,y = \frac{175}{3.2} - 2.09 \times 3.2 = 54.7 - 6.7 = \mathbf{48\ m.}$$

So use,
$$\left.\begin{aligned} \text{Bed width} &= \mathbf{48\ m} \\ \text{Depth} &= \mathbf{3.2\ m} \end{aligned}\right] \textbf{Ans.}$$

3.7. Cross Section of An Irrigation Canal

The cross section of an irrigation canal consists of following components.

a. Side Slopes
b. Berms
c. Freeboard
d. Banks
e. Service Roads
f. Back Berms or Counter berms
g. Spoil Banks
h. Borrow Pits

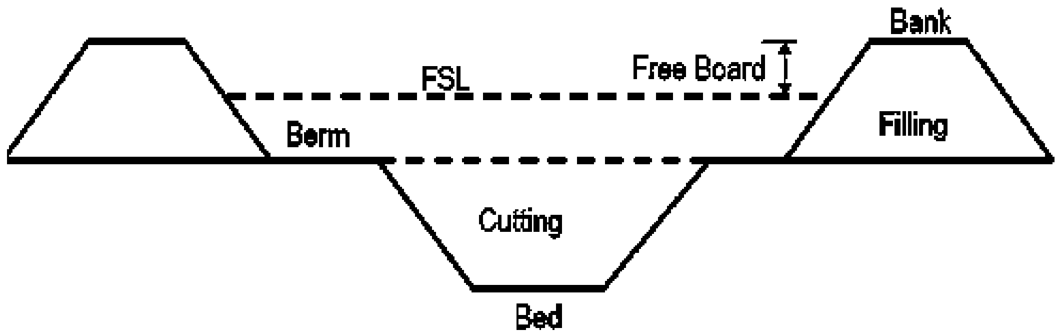


Fig: *Components of Canal Cross Section*

The cross section shown in above figure shows canal in partial cutting and partial filling. The cross section of canal should be balanced. That means, volume of cutting should be equal to volume of filling.

a. Side Slopes: The side slope in cutting is slightly more than side slope in filling. The side slope in cutting is 1H:1V to 1.5H : 1V and side slope in filling is 1.5H : 1V to 2H : 1V. In case of channel with silt laden water, the side slope is finally adjusted to 0.5 H: 1V.

b. Berms: The horizontal distance left at the ground level between the toe of bank and top edge of cutting. The purpose of berm is as shown below.

i. The silt deposited on the side is very fine and impervious. Hence, it serves as a good lining for reducing loss, leakage and consequent breaches.
ii. It helps to maintain wider waterway so that the fluctuation of discharge does not produce much fluctuation in depth because of wider waterway.
iii. They give additional strength to the banks and provide protection against erosion and breaches.
iv. They provide a scope for future widening of the canal.
v. Berms can be used as borrow pits for excavating soil to be used for filling.

c. Free Board: The margin between FSL and Bank Level is called free board. The amount of free board depends on the size of the channel. The free board in any canal depends on the discharge that flows in the canal. The free board for different discharges is given below

Discharge (m^3/Sec)	**Free Board (m)**
Below 0.30	0.3
0.3 to 1.00	0.4
1.00 to 5.00	0.5
5.00 to 10.00	0.6
10.00 to 30.00	0.75
30.00 to 150.00	0.90

d. **Banks:** Primary purpose of the banks is to retain water. The saturation line should be kept away from downstream face of the banks.

e. **Service Roads:** Service roads are provided for the inspection purpose and may serve the purpose of communication as well. They are provided above FSL of the the channel.

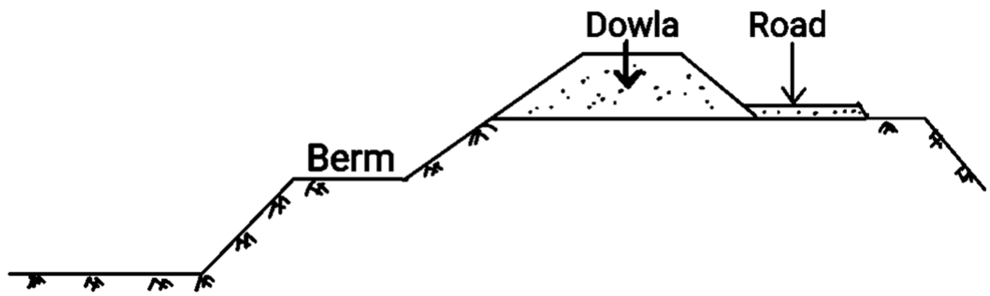


***Fig:** Cross Section of Canal Showing Service Road and Dowla*

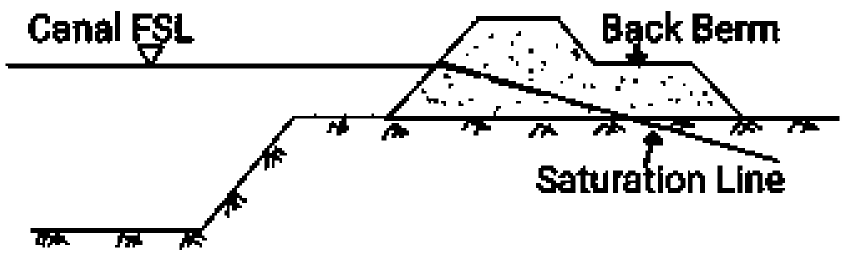


***Fig:** Cross Section of Canal Showing Counter Berm/Back Berm*

f. **Dowlas:** As a measure of safety in driving, dowlas are provided above the inspection road. They are generally provided 0.3 m high and 0.6 m wide at the top.

g. **Back Berms or Counter Berms:** The back berms or counter berms are provided so that the saturation gradient line does not cut the downstream end of slope. They are provided on the outside slope of the bank. The saturation gradient line should be at least 0.5 m away from the banks.

h. **Spoil Banks:** If the volume of cutting is more than the volume of filling, the extra earth has to be disposed of by means of mechanical transport. But this may be costly. To avoid this, the extra earth is disposed in the earthen embankments called spoil banks.

i. **Borrow Pits:** When the volume of earthwork in filling exceeds the earthwork in excavation, the earth has to be brought from somewhere. The extra soil is taken from borrow pits.

1. **External Borrow Pits:** Borrow pits excavated outside the canal is called external borrow pits.
2. **Internal Borrow pits:** The borrow pits excavated inside the canal section are called internal borrow pits. Internal borrow pits are dug at the center half of bottom of the channel.

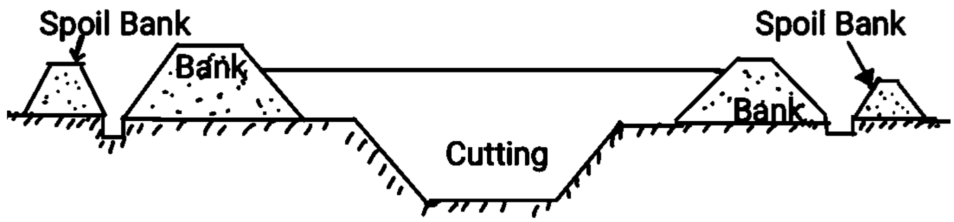


Fig: *Cross Section of Canal Showing Spoil Banks*

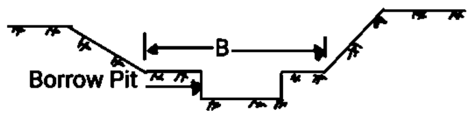


Fig: Cross Section of Canal Bed Showing Borrow Pit

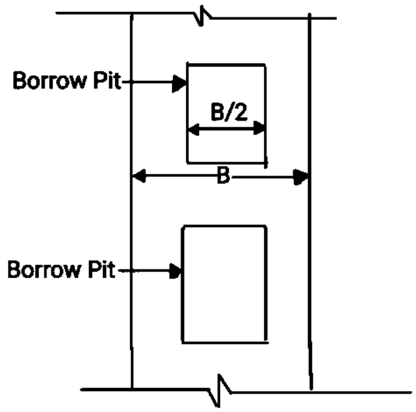


Fig: *Cross Section and L-Section of Canal Showing Borrow Pit*

4. Chapter 4: Headworks for Irrigation Projects

The basic purpose of an irrigation system or an irrigation project is to divert water from river or a source and to supply the water to fields.

Headworks: The structures constructed at the head of canal to divert required amount of silt free water or to store and divert water are called headworks.

Storage Headworks: The structures constructed to store water of wet season and supply the stored water during dry season are called storage headworks. Components of Storage Headworks are: dam, spillway, stilling basin, intake, under sluice or bottom outlets and other auxiliary components.

Diversion Headworks: The works, which are constructed at the head of the canal, in order to divert the water towards the canal so as to ensure regular supply of silt free water with minimum head are known as diversion headworks.

❖ **Functions of Diversion Headworks**

a. To divert the river water into canal.
b. To raise the level of the supply so that it can irrigate the area more efficiently.
c. To withdraw required amount of water from the river.
d. To control the silt entry into the canal.
e. To pass the flood.
f. To reduce the fluctuations of the level of the river.

❖ **Components of Diversion Headworks**

The components of diversion headworks include

a. Weir or barrage
b. Under sluices
c. Fish Ladder
d. Canal Head Regulator
e. Silt excluder
f. Silt Ejector
g. River training and river protection works such as marginal bunds, guide bunds, spurs

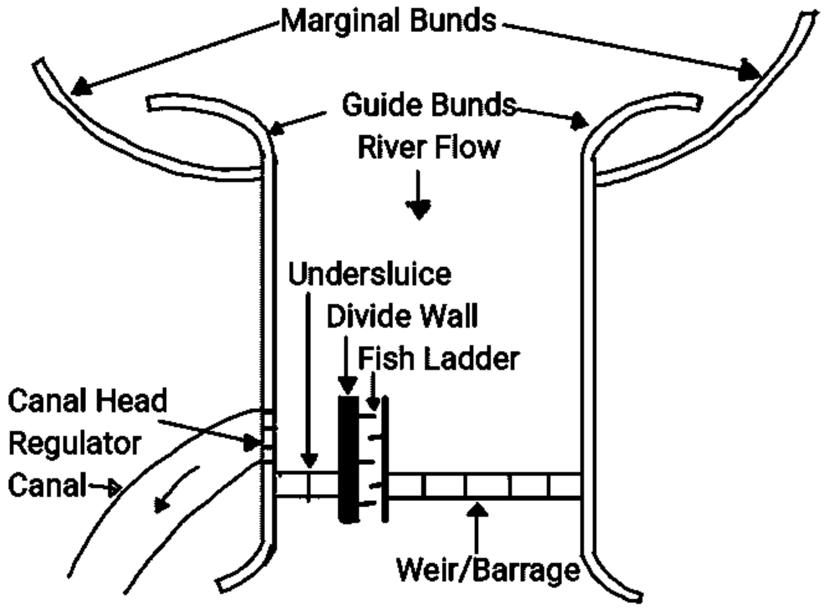


***Fig:** Components of Diversion Headworks*

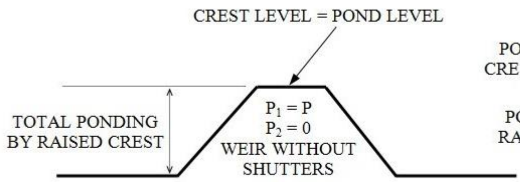
The description of these components is given below.

a. Weir and Barrage

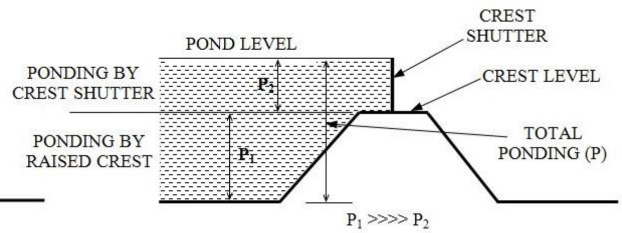
Weir	Barrage
Ponding is achieved mainly by the raised crest.	Ponding is achieved mainly by the gates.
Cost of construction is low.	Cost of construction is high
The afflux (rise in water level due to the construction of structure) structure is high.	The afflux (rise in water level due to the construction of structure) structure is low.
The silting in upstream is high.	The silting in upstream is low.
It can’t be connected with the roadway.	It can be connected with the roadway (e. g. Koshi barrage)

Weir: Weir is defined as the structure constructed transverse to the river flow in which the ponding of water achieved mainly by the raised crest. There may be gates on the weir crest, however the gates are small and ponding is mainly due to the raised crest.

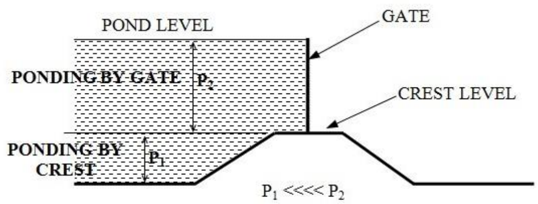
Barrage: Barrage is defined as the hydraulic structure constructed transverse to river flow in which ponding is achieved mainly by the gates. A barrage may or may not have raised crest. But in practice, barrages are provided with low gates and high crest.



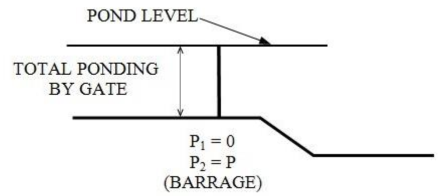
(a) Fig: Weir without shutters



(b) Fig: Weir with shutters



(c) Fig: Barrage with a small raised crest



(d) Fig: Barrage without any raised crest

b. Under-Sluice: Under-Sluice is the part of the diversion structure which is mainly constructed to flush the sediment deposited near the head regulator. The crest level of the under-sluice is kept at deepest river bed level. When under-sluice gate is opened, water flows at high velocity through the gate and the it scours the sediments deposited in front of the regulator. Since the level of the under-sluice portion is lower than the barrage portion, water is drawn towards the regulator side during dry season when discharge in the river is low.

The design discharge of under sluice is maximum of

i. 10 to 15 % of design flood discharge
ii. 2 times canal discharge
iii. Dry season food discharge

c. Divide Wall

The divide wall is the concrete or masonry wall which is constructed perpendicular to the weir axis and which separates weir proper and under sluice portion.

Functions:

- Separates the under-sluices from weir proper.
- Helps to provide comparatively less turbulent pocket of water near the head regulator, resulting in the deposition of silt in this pocket.
- Prevents the progression of cross currents and thus erosion of the weir.

d. Fish Ladder:

Fish ladder is constructed to allow the movement of fish along the river. When barrage or weir is constructed, then it becomes difficult for the fish to migrate along the river across that structure. So, a fish ladder is constructed to allow the migration of the fish from upstream to downstream during winter and upstream to downstream during summer.

e. Canal Head Regulator:

- A gated structure which is constructed at the head of the canal to regulate the flow of water into the canal is known as canal head regulator. It also maintains the head that is required for causing the flow of water into the canal.

- The crest level of the canal head regulator should be provided 1.2 to 1.5 m above the river bed level.

A Canal Head Regulator Serves following functions

- It regulates the supply of water entering the canal
- It controls the entry of silt into the canal
- It prevents the river floods from entering the canal

f. Silt Excluder (सिल्टजानैनदिने)

- It is the structure constructed at the river bed to avoid the entry of silt or sediments in the main canal.
- It consists of parallel tunnels along the river length. The tunnels carry the sediment laden flow directly to downstream of the river and hence the silt is not allowed to enter the main canal through the head regulator.
- This structure is provided in the river upstream of the canal head regulator.

g. Silt Ejector or Extractor (क्यानलमा गईसकेको सिल्टलाई फाल्ने)

- These are also called silt extractors.
- These are the structures which are constructed in the main canal to extract the silt from the canal.
- These are constructed on the bed of the canal little downstream of the canal head regulator.
- The width and depth of the silt ejector is more than the canal and hence the velocity of flow is less in the silt ejector. Due to less velocity, the silts settle down at the bottom. The silt laden water is ejected through the tunnels located at the bottom of the silt ejector.

h. River Training Structures:

River Training Structures are required at the headworks site to avoid the change in path of river, to make river water flow in specific path, to save river banks from cutting and many other purposes. Generally, following river training structures are used.

i. **Marginal Bunds/Embankments:** Marginal bunds/embankments are provided to avoid inundation of the surrounding area from high flood and to avoid outflanking of the structures due to flood.

ii. **Guide Bunds:** Guide bunds are provided to guide the river water flow in a specified path.

iii. **Spurs/Groynes:** Spurs/Groynes are provided to avoid cutting of the river bank due to flood.

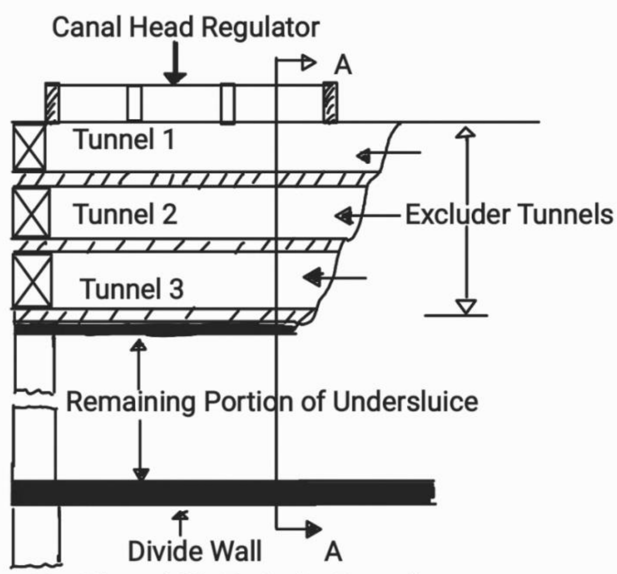


Fig: Plan of Silt Excluder Tunnels

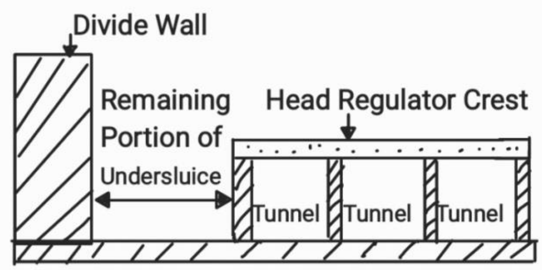


Fig: Section of Excluder tunnels at AA

Fig: *Cross Section and L Section of Silt Excluder*

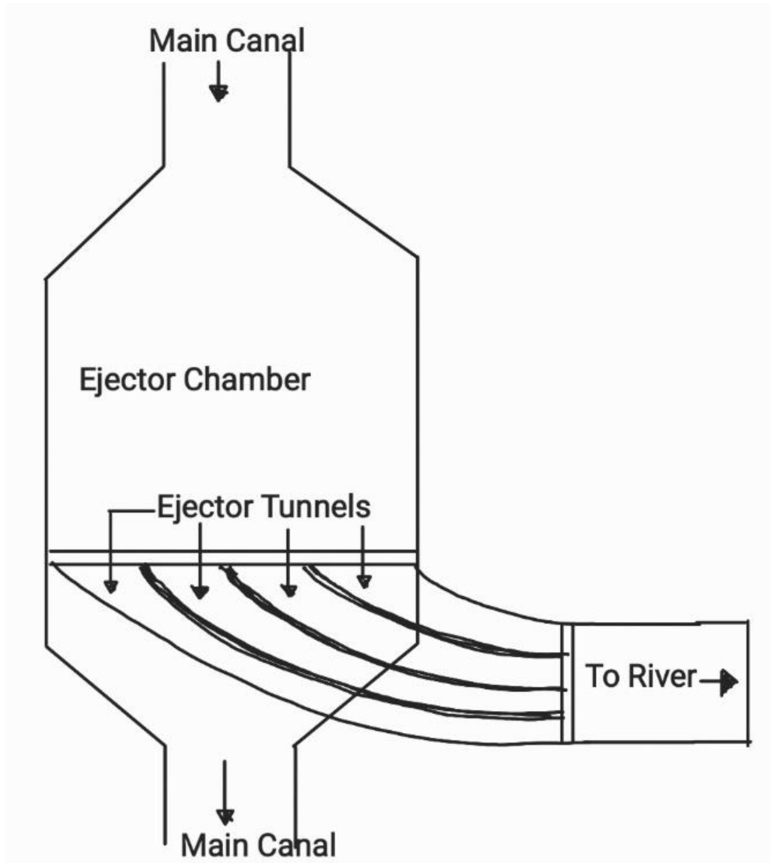


Fig: *Plan of a Silt Ejector*

5. Chapter 5: Planning and Management of Irrigation Systems

5.1. GENERAL IRRIGATION SYSTEM PLANNING

Proper system planning and design is essential to Irrigation Water Management (IWM) and requires the thoughtful consideration of many elements.

A small irrigation project can be developed in a relatively short time. For example, a farmer can develop his own tube well irrigation system by securing bank loan and, soon after, getting the engineering works constructed.

However, development of a large irrigation project is more complicated and time-consuming due to the associated organizational, financial, legal, administrative, environmental and engineering problems.

The main stages of a large irrigation project are:

i. The promotional stage,
ii. The planning stage,
iii. The construction stage, and
iv. The settlement stage.

The planning stage itself consists of three sub-stages:

i. Identification Study
ii. Prefeasibility Study
iii. Detail Feasibility Study
iv. Detail Design & Estimate

i. Identification Study

After project request from the concerned area, identification study may be carried. The identification study includes field visit, preparation of report and recommendation for further studies. The identification study shall cover following aspects

a. Water Availability
b. Type of Diversion works
c. Length of main canal
d. Size and type of command area (terrace/plain)
e. Water right problems
f. Number of households, existing cropping pattern,
g. Nature of soil in canal alignment & command area
h. Major technical difficulties (cross drainage/landslide/unstable zones)
i. Poor farmer's presence
j. Farmer's interest

ii. Prefeasibility Study

If the project is selected for further studies after identification study, prefeasibility study shall be conducted. Based on the data and information collected during the field visit the team needs to analyze the project findings and finalize the prefeasibility study. The analysis should be based on technical, economical and social aspects of project implementation. The analysis should cover the following aspects:

a. Length of main canal;
b. Size and type of command area (terrace/plain);
c. Water right problems;
d. Type of soil;
e. Major technical difficulties (cross drainage/landslide/unstable zones);
f. Poor farmer's presence;
g. Farmer's interest;
h. Estimate water requirement of crop
i. Make Assessment of tentative cost of the project
j. Evaluate Cost per ha
k. Make Assessment of tentative benefit of the project
l. Evaluate Internal Rate of Return and Benefit Cost Ratio
m. Recommendation whether the further study shall be carried out or not.

iii. Detailed Feasibility Study, Detail Design and Estimate

The main objective of feasibility study is to assess whether the project is technically, economically, environmentally feasible or not. Detail Design and Estimate is conducted if the project is found to be feasible. Detail Design and Estimate is conducted after the Detailed Feasibility for large irrigation projects, whereas they are conducted together for medium irrigation projects. The following works need to be done under Detailed Feasibility Study and Detail Design and Estimate.

a. Conduct and Prepare Desk study: Prepare a desk study report by review previously prepared identification/prefeasibility report.
b. Conduct field work covering following:
    i) Intake/Head Work site survey
    ii) Discharge measurement
    iii) Canal alignment survey
    iv) Canal Profile Survey
    v) Work Inventory
    vi) Major structure site survey
    vii) Command area survey (GPS Survey)
c. Conduct Geological Study & Geomorphologic study (Optional)
d. Conduct Seismological Study (Optional)

e. Conduct Subsurface Exploration (Optional)
f. Conduct Agricultural survey covering following
   1. Existing cropping pattern,
   2. Existing crop yields
   3. Inputs used and its availability
   4. Marketing facility and labor situation
   5. Food Security
   6. Existing & Anticipated Irrigation/Water Management Practices
   7. Accessibility

g. Conduct EIA study and incorporate the recommendations of EIA in design.
h. Conduct alternative analysis that includes alternatives for water source, canal alignment, headworks locations.
i. Calculate Irrigation water requirement and diversion irrigation requirement based on command area, cropping pattern and cropping intensity.
j. Complete design of headworks, can and structures.
k. Prepare drawings (working drawings for detail design type study)
l. Quantity Estimation
m. Cost estimation
n. Benefit estimation
o. Conduct financial and economic analysis and evaluate financial parameters like B/C ratio. IRR, payback period
p. Prepare implementation schedule
q. Prepare BoQ and tender documents.

5.2. ORGANIZATION AND IRRIGATION MANAGEMENT

Irrigation Water Management (IWM) is the practice of monitoring and managing the rate, volume, and timing of water application according to the seasonal crop needs, giving consideration to the soil intake and water holding capacities. Soil moisture should be managed to obtain optimum yields, without deep percolation losses or runoff.

Objectives of Irrigation Water Management

1. To maximize utilization of water
2. To optimize utilization of water
3. To maximize crop production
4. To minimize the losses
5. To distribute uniformly as per the requirements of crop
6. Assessment on Water Users

Irrigation water management will help irrigators determine the effectiveness of irrigation practices, make good water management decisions, and justify making irrigation adjustment in existing systems. Tools are available to assist the irrigator with irrigation water management:

1. "Checkbook" method to monitor and balance soil moisture in irrigated cropland.
2. Flow meters to record instantaneous flow rates and total volume usage.
3. Soil moisture meters and sensors to monitor soil water deficit.
4. Soil moisture data loggers to record soil moisture history throughout the growing season.

Methods of water management and control:

1. Land development: size, shape and proper grading of land, proper cultivation at appropriate time
2. Field application of water: it should be selected according to type of soil, topography, type of crop, water availability, effective rainfall etc.
3. Irrigation by rotation: When available water is lower than demand.
4. Method of charging: On area basis, volumetric basis (best for controlling water), composite rate basis, seasonal basis
5. Education and training to farmers: to develop scientific and technical skill, for participation in construction, operation and maintenance of field channels in best way.

OPERATION AND MAINTENANCE OF IRRIGATION SYSTEM

Operation of Irrigation system

Operation of irrigation systems implies a package of organizational, economic and technical arrangements that ensure planned distribution and full use of water resources for maximum yield of agricultural crops of good quality under irrigation conditions.

Objective of operation is to distribute right amount of water to the crop at the right time and right location in least cost and most efficient way.

Operation is done by Water User Association (WUA), GoN, Jointly by WUA and GoN and by private sectors as well.

Operational measures include the following:

1. Implementation of scheduled water use practice to provide the irrigation regime required under specific meteorological conditions on certain land areas under efficient water use;
2. Keeping irrigation, drain and other canals, pipelines and buildings in good working order by guarding, supervision, maintenance and repair of irrigation systems;
3. Prevention of inflow of excess water into irrigation system and diversion of excess water;
4. Control of water losses in canals and improvement of system efficiency;
5. Organization of irrigation water accounting;
6. Control over proper water use and groundwater conditions;
7. Management of forest vegetation along canals;
8. Control over crop management practice on irrigated lands;

9. Liquidation of salinization and water logging on irrigated lands.

5.3. Maintenance of Irrigation system

Maintenance of irrigation system is necessary to supply rich water to communities for better production. After construction, it becomes essential to maintain for its proper and efficient functioning.

Types of maintenance:

i. Routine / regular maintenance: greasing and oiling, vegetation removing etc
ii. Periodic maintenance: (painting, desilting, repair of weir and stilling basin
iii. Emergency maintenance: after disaster
iv. Special / specific maintenance: road network, drainage network, flood protection bank etc

1. Routine/ Regular Maintenance

i. The maintenance activities planned and implemented regularly with a view to increase day to day canal operation efficiency is called regular maintenance.
ii. It saves amount of maintenance cost in future.
iii. The regular maintenance is highly cost beneficial and does not require high technology and skill. E.g.: greasing and oiling, vegetation removing, embankment repairing etc.

2. Periodic Maintenance

i. It is called preventive maintenance as it is done to prevent the major possible damages to irrigation project.
ii. The maintenance works carried out once a year to prevent any possible major damages to irrigation projects called periodic maintenance.
iii. Periodic Maintenance need detail technical planning, preparation and supervision of works. E.g.: Removal of large volume of silt, repairs of weirs, stilling basin, removal of large trees and debris.

3. Emergency Maintenance

i. It is generally done after disasters or to prevent disaster for future in rapid way. E.g.: Leakage, blockages etc

4. Specific/ Special Maintenance

The maintenance works which are grouped according to major physical components of irrigation system like: irrigation channel maintenance, irrigation structures maintenance, drainage network, flood protection works etc.

5.4. INSTITUTIONAL ASPECTS OF IRRIGATION SYSTEM MANAGEMENT

On the basis of management, government of Nepal has following practices for management of irrigation system:

1. Agency managed irrigation systems (AMIS) such as those operated by the Government through the Department of Water Resources and Irrigation (DWRI). The government official continues to manage the system after completion. (In past most of the irrigation system were managed like this, but nowadays it is being transferred to the farmers).
2. Farmers managed irrigation system (FMIS) such as traditional system (see detail in next chapter)
3. To turn system over to farmers to manage them. (By creating WUAs transfer project to users. Need management training for leaders of WUAs). The belief is that transferred program has better chance of success and provides more opportunities to change the course if required.
4. Jointly managed irrigation systems (JMIS) run by Government and Users through mutual cooperation and understanding. Manage the system jointly. Some parts of the physical system (Main, Branch canal, headworks) are managed by government and distributaries, field channels, water courses managed by farmers. (WUA shave some O&M responsibilities, while agency has bulk responsibilities for O&M. Full water users control including regulation is rarely found in practice except in isolated small system. Greater degree of agency control is generally found in this system
5. Private irrigation Systems (PIS) which are operated and maintained by big farmers: It is also a challenge to be success such irrigation management in developing country like Nepal where irrigation is more focus to solve hand to mouth problem and not profit oriented.

5.5. farmers Managed Irrigation System:

- In Farmers Managed Irrigation System(FMIS), irrigation facilities are owned, developed and managed by the farmers.
- They form a group of water users, develop rules and norms by assigning the rights and responsibilities among themselves.
- There is long history of FMIS in Nepal.
- Donor agencies such as Asian Development Bank (ADB), World Bank (WB) are providing loan for the betterment of the FMIS.

Salient Features of FMIS

1. A Self organized system
   - Farmers collectively construct and govern their systems.
   - Make decisions on service area, water allocation rules and other necessary rules collectively.
2. Equitable and judicious allocation of irrigation water
3. Good Governance
4. Governing of FMIS
   - Irrigation infrastructures

- Water sharing system
- The emphasis of the government in developing and managing irrigation system is paving the way for greater role of local user organizations.
- The centralized approach of agency managed system has proven unsuitable because of poorly managed systems and users refuse to pay for the services.

FMIS May Be

a. Farmer initiated, constructed and managed.
b. Agency supported and handed to farmers for management.
c. Agency developed and farmers responsible for the management partly or fully.

In FMIS, the farmers feel the real sense of ownership.

Advantages of FMIS

- Increased ownership over the system.
- System operated well.
- Cost of irrigation can be reduced as the Water Users Associations take more responsibilities than the agency managed system.
- Increased fee collection by the local rules.
- Reduced negative environmental impact.
- More area can be irrigated due to the improved services.
- Good co-ordination between head and tail users.

6. Chapter 6: Waterlogging

Waterlogging id the process due to which the productivity of the crops gets decreased due to rise in water table. Plant needs different nutrients like nitrates, phosphate for their growth and development. Nitrates are provided by bacteria under a process called nitrification. These bacteria need oxygen for their survival. The supply of oxygen gets cut off when land becomes ill aerated resulting in death of bacteria and fall in the production of nitrates. As a result, the growth of plant reduces, which reduces crop yield.

Salinity: When water table rises up to the root zone of plant, there is evaporation by capillarity, the salt present in water gets deposited in the root zone of crops. The concentration of these alkali salt present in the root zone of crop has corroding effect on the plant which prevent growth and ultimately plant fades away. Such soil is called saline soil and the phenomenon is called salinity.

Causes of Water Logging

1. **Over and Intensive Irrigation:** Maximum irrigation over a small area is called intensive irrigation. The intensive irrigation causes excessive percolation and subsequent rise in water table.
2. **Seepage of Water through adjoining high lands:** Water from adjoining high lands may seep into sub soil of affected land and may rise water table.
3. **Seepage of Water through Canals:** Water may seep from the sides and beds of adjoining canal, reservoir, situated at high level land as a result, water table of affected area rises.
4. **Excessive Rains:** Excessive rainfall may create temporary water logging and in the absence of good drainage, it may lead to continuous water logging.
5. **Impervious Obstruction:** If water seeping below the soil moves horizontally, but finds an impervious obstruction, it will cause rising of water table on the upstream side of obstruction.
6. **Inadequate Natural Drainage:** Soils having less permeable soil below the top layer of pervious soils will not be able to drain the water deep into the ground and hence resulting in high water table.
7. **Submergence due to floods:** If and continuously remains submerged by floods, water loving plants like weeds, grasses grow which obstruct natural drainage of the soil thus increasing the chances of water logging.
8. **Irregular or Flat Topography:** In steep terrains, the water is drained quickly. On a flat or irregular terrain having depression etc, the draiage is poor and water will be percolated deep resulting rise of water table.

Preventive Measures of Water Logging:

Water logging can be controlled if the quantity of water into the soil below is checked and reduced. To achieve this, the inflow of water into the underground reservoir should be reduced and the outflow from this reservoir should be increased to keep the water level in the ground as below as possible. The various measures adopted for controlling the water logging are:

a. Reducing Intensity of Irrigation:

In the areas where there is a chance of water logging, intensity of irrigation should be reduced.

b. Adopting Crop rotation:
   Sowing of crop water in every season should be discouraged. A wet crop should be followed by a dry crop, i. e; crop rotation should be adopted.
c. Optimum Use of Water:
   Farmers should be educated and trained in using just the required quantity of water, which will give optimum yield.
d. Providing Intercepting:
   To collect water seeping through the sides of canal intercepting drains shall be provided in the banks of canal.
e. Improving Natural Drainage of the Area:
   The rain water and the excess irrigation water should be effectively drained off to reduce their percolation into ground water reservoir. This is achieved improving the natural drainage of the area by measures like clearing the obstruction in the path of the flow, desilting, etc.
f. Provision of Artificial Drainage System:
   An artificial drain in the form of surface drain shall be provide to drain off storm water and surplus irrigation water.
g. Lining Canals and Watercourses:
   To control the seepage of the water passing through the bed and sides of the canals, the canal should be lined.
h. Introduction to Special Irrigation Methods:
   Special irrigation methods like lift irrigation, sprinkler irrigation and the drip irrigation reduce the problems of water logging.

Remedial Measures of Water Logging:

The remedy against water logging is the saving the water logged land from the ill effects of water logging.

a. Construction of Surface Drains: The surface drains remove the excess water deposited on the agricultural lands. This is especially adopted in the areas where there is no sufficient slope for draining the water.
b. Construction of Underground Drains: The underground drains remove the excess water in the soils and the duration of wetting due to water logging can be reduced.
c. Constructing the outlets: The outlets should be constructed to drain off excess water.
d. Leaching: In this method, land is flooded with the adequate depth of water. The alkali salts present in the soil get dissolved in the water which percolate down to join the water table or drained away by surface or sub surface drains. This removes the unwanted salts in the soil and the soil becomes fit for the irrigation.

7. Chapter 7: River Training Works

River Training is defined as the engineering works which are constructed in the river so as to guide and confine the flow to river channel, and to control and regulate the river bed configuration, thus ensuring safe and effective disposal of floods and sediment loads in the river.

Objectives of River Training

The river training works may serve the following objectives or advantages:

i. To prevent the river from changing its course and to avoid outflanking of structures like bridges, weirs, aqueducts, etc.
ii. To prevent flooding of the surrounding countries by providing a safe passage for the flood waters without overtopping the banks.
iii. To protect the river banks by deflecting the river away from the attacked banks.
iv. To ensure effective disposal of sediment load.
v. To provide minimum water depth required for navigation.

Classification of River Training

Depending on the purpose for which river training program is undertaken, river training works may be classified into following three categories.

a. High Water Training for Discharge: High water training is undertaken with the primary purpose of flood control. It therefore aims at providing sufficient river cross-section for the safe passage of maximum flood, and concerned with making the adjoining area flood-proof by construction of dykes, levees, marginal bunds etc.

b. Low water training or Training for depth: Low water training is done for purpose of providing sufficient water depth in the channels during low water periods. It may be accomplished by concentrating and enhancing the flow in the desired channel by closing other channels, process of bandailing by contracting the width of the channel with the help groynes, etc

c. Mean water training or Training for sediment: Mean water training aims at efficient disposal of suspended load and bed load, and thus, to preserve the channel in good shape. The maximum accretion capacity of a river occurs in the vicinity of mean water or dominant discharge. Therefore, the changes in the river bed are attempted in accordance with that stage of flood flow. The mean water training is the most important type and forms the basis on which the former two river trainings (i,. e; high water training and low water training) are planned.

Methods of River Training

The following are the structures constructed for river training.

a. Marginal embankments or levees or dykes
b. Guide banks or bell's bund
c. Groynes or Spurs
d. Artificial cut-offs
e. Pitching of banks and provision of launching aprons
f. Pitched islands

The detail description of these methods is given below

a. Marginal Embankments or Levees or Dykes

It is an embankment structure which runs parallel to the river either on both sides or on one side for some suitable length. These embankments prevent the entry of the flood water to the adjoining towns, important land, settlement (वस्ती). A levee or dyke is mainly used for flood control mainly by controlling the river and not by training the river.

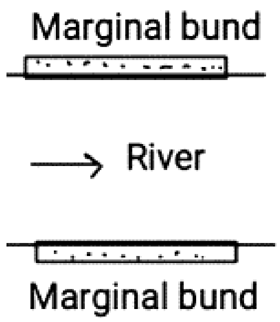


Fig: Plan of Marginal bund

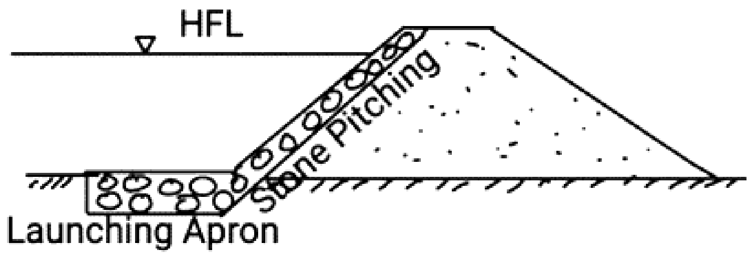


Fig: Cross Section of Marginal Bund

b. Guide Banks or Guide Bunds or Bell's Bunds

The guide banks are the embankment structures constructed along the river to guide the river flow in the axial path. Guide banks prevent the meandering of the river and they do not allow the river flow to cut the river banks. They are constructed upstream of every hydraulic structures like bridge, weir, barrage, etc. The guide banks are constructed in pairs and are symmetrical in the plan. Guide bund is also called bell's bund.

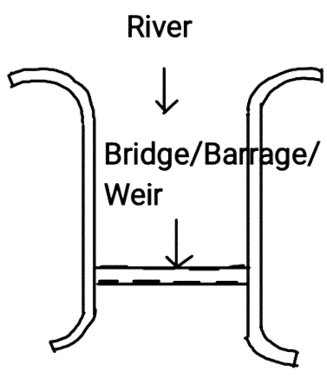


Fig: Plan of Guide Bund

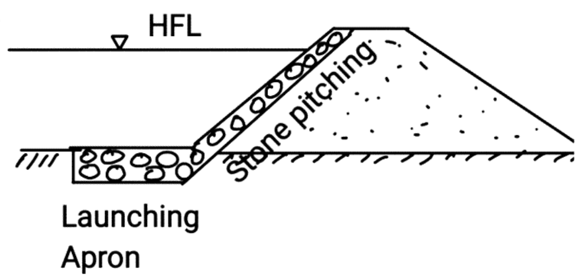


Fig: Section of guide bund

c. Groynes or Spurs

A spur or groyne is the structure which is constructed at the bank of the river and is projected towards the river. It is constructed to keep the flow away from the river bank and to avoid the bank cutting.

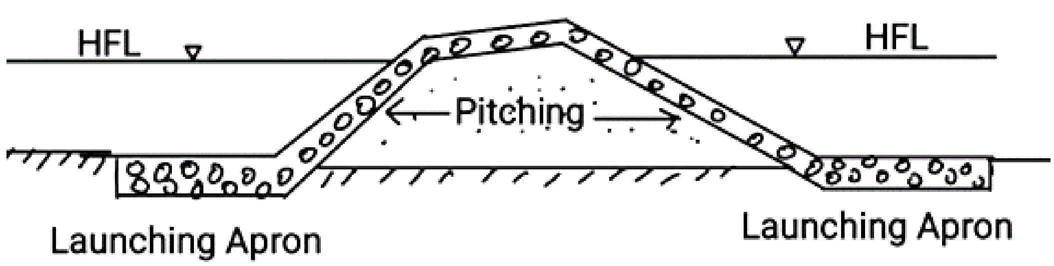


***Fig:** Cross Section of spurs/groynes*

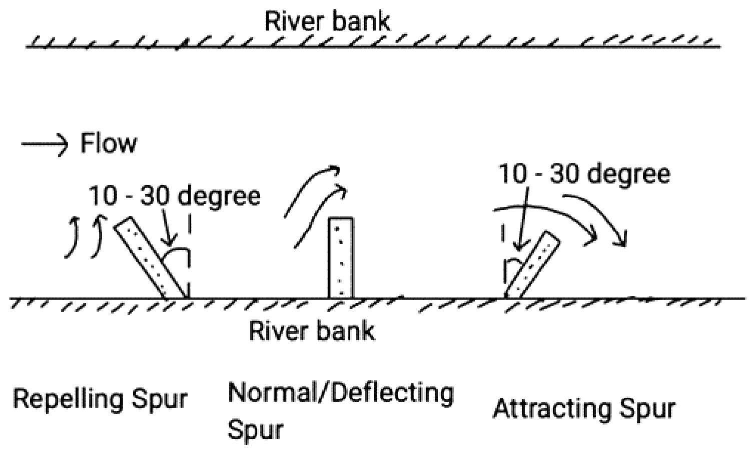


Fig: *Plan Showing layout of spurs*

Objective of Groynes Construction:

- To train the river along the desired course by attracting or deflecting or repelling the flow in the desired direction.
- To reduce concentration of flow at a particular point of attack.
- To create slack zone for silting up the area.
- To protect a bank by keeping the flow away from it.
- To contract a wide river for the purpose of increasing the river width for navigation.

Classification of Groynes

1. **Based on the methods and the materials of construction:**
   i. **Impermeable Groynes:** These spurs do not allow the flow of water through them. They consist of either rock fill, or core of sand or sand and gravel or the alluvial soil available in the river. Their top and the sides are protected by the pitching of stones and concrete.
   ii. **Permeable Groynes:** They permit restricted flow of water through them. They obstruct the flow and allow the deposition of the sediments carried by the rivers. They are also called the sedimentation groynes. The accumulated sediments work as the protection to these types of groynes.
2. **On the basis of Height of Groynes:**
   i. **Submerged Groynes:** If the depth of water is sufficient in the river and the groynes are in the submerged condition, they are called submerged groynes.
   ii. **Unsubmerged Groynes:** If the groynes are large enough and are not submerged, they are called unsubmerged groynes.
3. **On the basis of Function Served**
   i. **Attracting groyenes:** These types of groynes are pointing downstream of the river and they tend to attract the flow of water. They create the scour holes on the downstream. These types of groynes should be provided on the opposite side of the bank which is to be protected. They make acute angle with direction of flow.
   ii. **Normal Groynes:** They are aigned perpendicular to the river. They deflect the water away from the bank. They are also called as deflecting groynes. These are the mostly used type of groynes. They make right angle with flow direction.
   iii. **RepelingGroynes:** They are pointing upstream and they repel the flow away from the bank. Such groynes create still water pocket on upstream side and suspended sediment gets deposited in the pocket. They are more effective and do not cause any problems as compared to the attracting groynes. They make obtuse angle with flow direction.

d. **Artificial Cut off:** An artificial cutoff is constructed in the meandering river to divert the flow along the fixed watercourse. It is constructed when the meander is regularly increasing and the valuable land and property is about to be wasted. It is used for straightening the river approach to a structure. Artificial cutoff decreases length of a river.

e. **Pitching of the banks and the provision of launching Aprons:** Banks of the river are directly protected by the stone pitching or by concrete blocks or by brick lining or by vegetative cover. The slope of the pitching may vary from 1V:1H to 1V:2H. The launching apron may also be provided to prevent the scour at the toe.

f. **Pitched Island:** A pitched island is the artificially constructed island in the river bed and is protected by the stone pitching. Due to the turbulence constructed in the vicinity, the river channel around the island gets deepened and thus, attracting the river water towards itself and holding the water permanently.

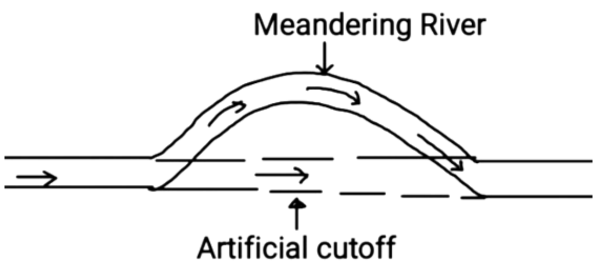


Fig: Artificial Cutoff

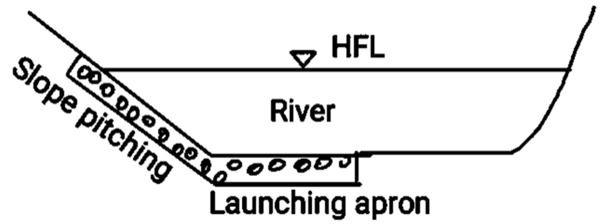


Fig: Bank pitching with launching aprons

6 DESIGN CRITERIA AND PROCEDURES

6.1 STRUCTURAL MEASURES FOR RIVER CONTROL

There are various river engineering works that are used, either singly or in combination, to provide flood protection and to control riverbank erosion to protect life and property of the people. These are summarized in Table 6.1. Design of these structures is presented in the subsequent headings.

Table 6-1: Common River Training and Riverbank Erosion Control Measures

S.N.	Category	Problems	Facilities / Structural Measures
1	High water training	Over flow / Inundation / Flooding	Flood embankment/Dike/Levee/ flood walls Widening of waterway/river Dredging/Excavation Combination of above
2	Reduction/control of the peak discharge of flood	Flooding down stream	Dam Retarding basin or Detention basin
3	River Bank erosion, River training.	Bank cutting, River shifting	Revetment Spurs Change of waterway/cut-off channel
4	Prevention of riverbed degradation	River bed erosion and bed lowering	Ground sill / bed bar, drops, check dams.
5	Prevention of sediment and debris to maintain the flow uninterrupted	Surface erosion and land slides	Sabo works / debris arresting dam (for sediment and debris control) Regular maintenance of channel (excavation/dredging)
6	Protection of river structures confining river flow	Afflux (Rise of water level Up Stream)	Guide bund

Source: (River Training Mannual by WECS Nepal, 2020)

8. Chapter 8: Special Considerations for Hill Irrigation Systems:

Hill irrigation are the small irrigation systems especially designed and constructed for small command areas in hill. They are small in size as compared to the large irrigation systems in Terai. Design of Hill Irrigation consists of more than the technical aspects of irrigation design. The wider aspects are as follows.

a. General

i. Location of Headworks
- Should be located on straight portion
- Should be located on geologically stable section
- Should be accessible
- Elevation should be such that water reaches the whole command area

ii. Diversion Structure
- Alternatives may be concrete weir, masonry weir and gabion weir.
- The diversion structure shall be selected carefully considering durability, impact of flood and sediments.

iii. Intakes:
- Two types of intakes are generally used, side intake or bank intake and bottom intake or trench intake.
- Side intake is used in most of rivers due to sediment problem.
- Bottom intake is used in the steep rivers that carries boulders.
- The size of intake should be such that it can withdraw required discharge even with minimum water level in the river.

iv. Sediment control structures:

The sediment control structures are different for hill irrigation. Silt excluders and silt ejectors are not used in hill irrigation. Following structures are generally used.
- ✓ Gravel trap: for removing sediment larger than 5 mm size.
- ✓ Settling basin
  - Target particle size should be 0.5 mm (i. e; the particles with size greater than 0.5 mm should be removed.
  - The sediment particles larger than 0.8 mm should be flushed hydraulically.
  - Bottom slope of the basin should be sufficient to develop flushing velocity.

v. Canal
- Stabilize the canal section with civil engineering structures and bioengineering structures
- If there are problems of landslide in a particular area, canal excavation may further trigger it. So, it may be better to provide pipe alignment for a certain length.
- The canal should be lined to avoid seepage and to avoid problems of landslide triggering.
- The cut and fill slope shall be protected to avoid any erosion and landslides.

b. Scheme Objectives:

The overall objective of hill irrigation development is to improve agricultural production, and within this wider aim there will be a number of general and scheme specific objectives. Scheme specific objectives need to be set jointly by farmers and the co-operating irrigation agency and agreed by both parties so that all persons involved in the planning, design, implementation and management of the scheme have a clear idea of what they are trying to achieve.

General objectives might include:

- increasing crop production in the project area,
- diversification of crops,
- improving social welfare of the community,
- achieving self sufficiency in food grain production,
- reducing food imports into the project area,
- increasing food production to sell in neighboring villages,
- promoting farmer/community involvement in operation and maintenance of irrigation.

Specific project objectives might include:

- strengthening farmers' institutions to manage irrigation systems and handing over of schemes to farmers,
- strengthening agricultural extension support to the project,
- improving agricultural and water management practices in the project area to increase production,
- ensuring equitable and timely water supplies to farmers,
- improving reliability of water supplies to promote farmer confidence and encourage adoption of better varieties and higher levels of farm inputs,
- increasing command area,
- increasing cultivation intensity.

c. *Agricultural considerations*

Following agricultural considerations should be considered during the design of hill irrigation schemes.

1. Soil: Crops should be grown according to the soil condition. If the soil is light and poorly suited for the rice, other crops should be grown. The methods of irrigation should also be selected according to the soil type.
2. Access: Access to the scheme is also important in terms of the provision of operation and maintenance. The crops in excess to the local demand should be marketed.
3. Future Crop Diversification: The provision of diversification of the crop in the recent future may affect the design of the distribution of the distribution system.
4. Agro-processing facilities and water for domestic use: The facilities for the processing like mills can also be combined with the irrigation schemes. Also, the water for the domestic use should be supplied to the households.

d. Managerial and Institutional Considerations:

The farmer user group should be enforced where it is already existing and new user group should be formed where there is no such group. The institutional capability of the users group should be increased for the effective operation and maintenance of the system. The level of managerial and institutional capability of the farmers will vary from the project to project and will depend upon

- History of irrigation in this project
- Technical and social complexities of the project
- The number of years the institutions have been active.
- The nature of social and technical issues the institution has helped to solve.

e. Social Arrangements:

The following factors affect the design and distribution of the hill irrigation schemes.

- Senior and Junior water rights
- Village boundaries
- Ethnic groups

The hydraulic efficiency and the cost should be sacrificed considering the existing social arrangements. Long distribution canals may be constructed to meet the demands of each farmers and avoid the conflicts between them. The water should be supplied to the farmers at their preferred time. (day time will be preferred by the farmers to night time)

f. Financial Arrangements

- Farmers must be informed about what is required of them. The limited objectives of the farmers may be fulfilled due to the budget limitation.
- The choice of technology and the materials will be affected by the financial arrangements.
- The contribution from the farmers may be either in cash or by working in the system.
- Farmers should be encouraged to make financial arrangements for the maintenance of the irrigation scheme.

g. Engineering:

Engineering design is the main key of the hill irrigation system. Engineers should take into considerations of the following facts for good design.

- ***Farmers participation in Engineering Design***
  Farmers participation in the design process will help the engineers avoid to make costly mistakes. The farmers have a good knowledge of local conditions and the materials. They can suggest the best alternatives to the designers and the farmers will understand the system which will ultimately help them during the maintenance stage.
- ***Field Based Design***
  The alignment of the scheme, location of the structures and selection of the structures should be based on the field. The designers should spend more time on field and take necessary inputs from the farmers.
- ***Canal Design:***
  The canals pass through the steep slope and should be designed carefully. The number of distribution canals should be increased if possible so that the different command areas get different distribution network and conflict among the farmers will be reduced. Multiple number of smaller canals should be constructed rather than a single large canal. If there are stability problems for canal alignment, following solutions may be adopted

1. Stabilize the canal section with civil engineering structures and bioengineering structures
2. If there are problems of landslide in a particular area, canal excavation may further trigger it. So, it may be better to provide pipe alignment for a certain length.
3. The canal should be lined to avoid seepage and to avoid problems of landslide triggering.
4. The cut and fill slope shall be protected to avoid any erosion and landslides.

- ***Engineering Design Standards:***
  It is good to adopt high design standards like heavy structures, large foundation depths, good and sophisticated materials but since the irrigation structures are constructed in hills, there is a potential risk of failure of the structures. So, the structures should be designed with minimum and adequate standards. Also, the local materials should be used wherever possible. The following alternates may be used.
  1. Gabion diversion structure or masonry (stone masonry) weir shall be used in alternate to barrage or concrete weirs.
  2. Gravel trap and settling basin shall be used in alternate to the silt excluder and silt ejectors.
  3. Side intake or bank intake shall be provided to minimize sediment entry.
  4. For distribution of water in different canals, for example from main canal to branch canal locally constructed structures such as Sanchos shall be used in place *of regulator.*

h. *Construction Materials*

- Use of the imported materials like cement, steel reinforcement should be reduced to maintain the cost of the system.
- Local materials like clay, stones should be combined with the imported materials to achieve reasonable cost and performance.
- Bio-engineering techniques in combination with the minor structural support should be encouraged to be used.
- Farmers should be trained about the use of cement mortar structures, gabion structures so that they can effectively be involved in the maintenance activities.

9. Chapter 9: Watershed Management:

Watershed: Watershed is the area of land draining into a stream at a given location. Another names for watershed are: catchment area, drainage area, basin area. Divide is a line which separates catchment from its neighboring catchments.
Soil conservation: Soil conservation is the conservation of soil against atmospheric agents such as water and wind.
Watershed management: Watershed management refers to the management of land, vegetation and water of the watershed to achieve maximum benefit with minimum risk.
Soil erosion: Soil erosion is the detachment, transport and deposition of soil particles from one place to another under the influence of wind and water or gravity force.
Problems within watershed

- Soil erosion
- Floods
- Drought
- Deforestation
- Landslides
- Silting of reservoirs and agricultural lands
- Desertification
- Decrease of recharge to groundwater

Reasons of soil erosion
a. Natural factors

- Precipitation and wind
- Steep Topography
- Loose soil formations

b. Human induced factors

- Deforestation
- Overgrazing
- Faulty cultivation
- Shifting cultivation
- Carelessly built roads

Problems caused by erosion

- Reduction in the crop production
- Siltation of rivers, canals and reservoirs
- Disease and public health hazard due to deterioration in water quality
- Meandering of rivers

Need of soil and water conservation

- Need of soil conservation
- To prevent deterioration of land
- To increase crop production
- To reduce siltation in rivers, lakes, canals
- Objectives of watershed management
- To conserve soil and water
- To reduce flood, droughts, landslides
- To improve the ability of land to hold water

- Rainwater harvesting
- Recharging groundwater
- Growing greeneries (development of forest and grassland)
- Use of conserved water for Electricity, water supply, irrigation, wildlife, fisheries and livestock farming
- To develop rural areas
- To improve the quality of life

Watershed Management Approaches

Watershed management is the combination of all the activities and approaches conducted on a watershed to reduce soil erosion, increase infiltration and reducing run off. Also the water within the watershed is conserved so that it can be used to fulfill all the needs of water users in the watershed like water supply, irrigation, power production and others. There are different approaches adopted and different activities involve in watershed management. The approaches of watershed management are:

- Watershed is considered as a unit for development.
- Conservation of land, water and vegetation is done within a watershed.
- Higher productivity through the management of natural resources within a watershed is achieved.
- Community participation is focused.
- Dealing with common and private properties is done to acquire necessary lands.
- Minimization of natural disasters like flood, drought and landslide is achieved.
- Development of rural areas is focused.
- Utilization of runoff for useful purposes like water supply, irrigation, hydropower and other used like recreational uses.
- Integrated multi-disciplinary approach (integrated inputs of various disciplines).
- Vegetative and engineering measures are adopted.

Different Methods of Watershed Management:

1. Agronomic Measures: These include the agricultural measures like
   a. Contour Cropping
   b. Strip Cropping
   c. Conservation Farming
   d. Grassland farming
   e. Horticulture (फलफूल तथा बगैचा खेति)
2. Bio-Engineering Measures:
   a. Jute Netting
   b. Palisades construction
   c. Fascine Construction
   d. Horizontal and vertical grass plantation
   e. Wattle fences, etc.
3. Engineering Measures:
   a. Bunding (Contour and graded bunding)
   b. Terraces construction
   c. Construction of drainage structures and grassed waterways
   d. Contour and Staggered Trenching
   e. Gully control structures like construction of temporary and permanent check dams.

f. Construction of different spillway structures.
g. Construction of soil and water retaining structures.

The following structures are constricted with canal for conveyance and distribution of irrigation water to farmer fields

a. Cross Drainage Structures
b. Regulators
c. Falls/Drops
d. Escapes
e. Canal Outlets

9. Chapter 9: Cross Drainage Structures

In an Irrigation project, when the network of main canals, branch canals, distributaries, etc. are provided, then these canals may have to cross the natural drainages like rivers, streams, nallahs, etc. at different points within the command area of the project. The crossing of the canals with such obstacle cannot be avoided. So, suitable structures must be constructed at the crossing point for the easy flow of water of the canal and drainage in the respective directions. These structures are known as cross-drainage works. Thus cross drainage structure is defined as the structure constructed at the crossing of canal and natural drain so as to pass the canal water and drain water safely.

Types of Cross Drainage Structures

1. Irrigation canal passes over the drainage
   a. Aqueduct
   b. Siphon Aqueduct
2. Drainage passes over the irrigation canal
   a. Super passage
   b. Siphon super passage
3. Drainage and canal intersection each other at same level
4. Inlet and outlet

The description of these structures is given below

1. Irrigation Canal Passes Over Drain

The irrigation canal passes over the drain if Full Supply Level (FSL) of the canal is above high flood level (HFL) of the drain. These types of structures are aqueduct. This is of further two types.

a. Aqueduct: In aqueduct, irrigation canal is taken over the drainage (such as river, stream etc..). The Full Supply Level (FSL) of canal is above High Flood Level (HFL) of the drain and Canal bed level is above the high flood level of drain. In this case, the drainage water passes clearly below the canal and drainage water is in open channel flow.

b. Siphon Aqueduct: In siphon aqueduct, irrigation canal is taken over the drainage (such as river, stream etc..). The Full Supply Level (FSL) of canal is above High Flood Level (HFL) of the drain and Canal bed level is below the high flood level of drain. In this case, the drainage water touches the canal bottom and drainage water flows under pressure.

If required, the bed level of drain may be lowered to increase height of siphon barrels and decrease velocity of flow.

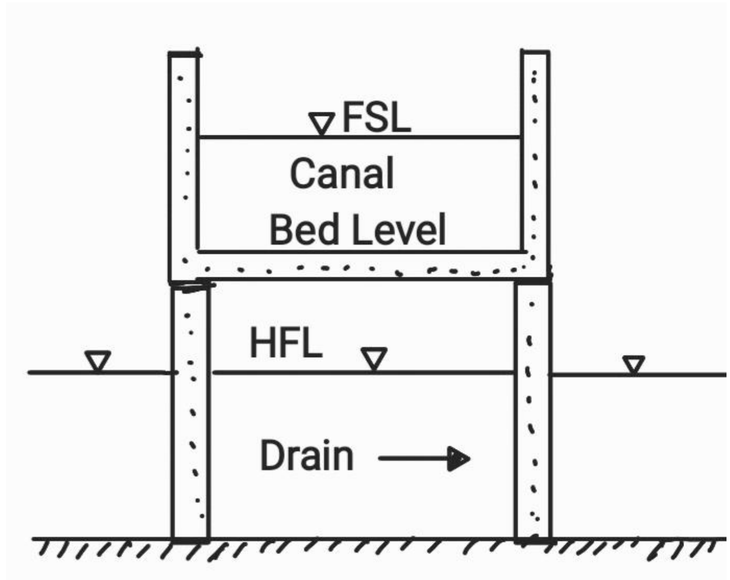


Fig: *Aqueduct*

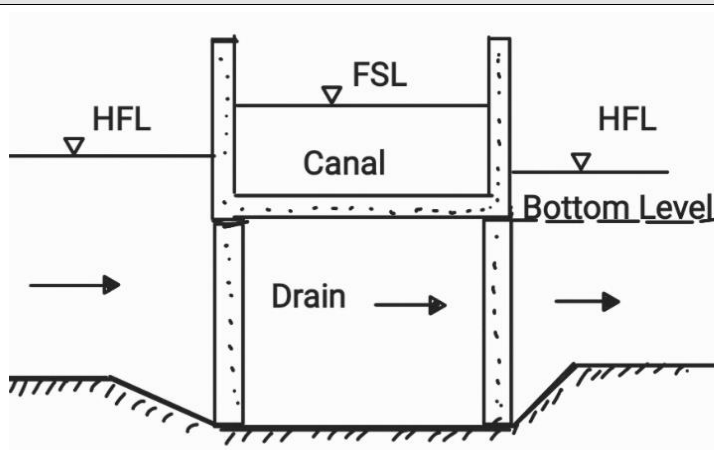


Fig: *Siphon Aqueduct*

2. Drainage Passes Over the irrigation Canal.

These structures are provided if the High Flood Level of river is above Full Supply Level of canal. This type of structure is called super passage. If the

a. Super Passage: In super passage, the Bottom level of drain is above Full Supply level of canal so that the canal water does not touch bed level of drain.

The canal water passes clearly below the drainage and the canal water is in open channel flow.

b. Siphon Super Passage/Canal Siphon: In siphon super passage, the Full Supply level of canal is above Bottom level of drain so that the canal water touches the drain bottom.

The canal watertouches the bottom of drain and canal water flows under pressure. This structure is also called canal siphon or simply siphon.

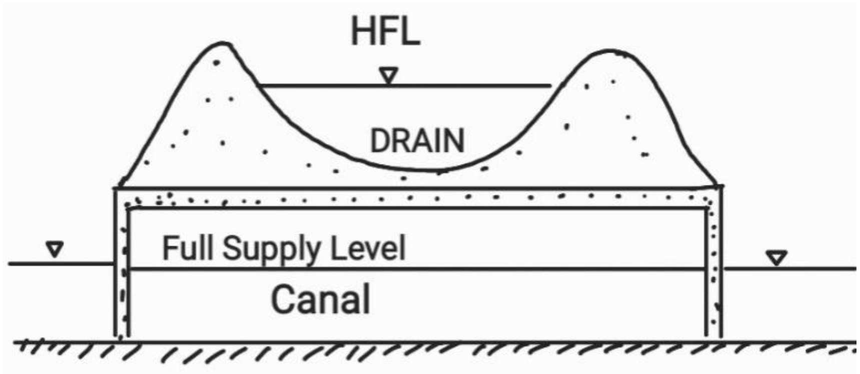
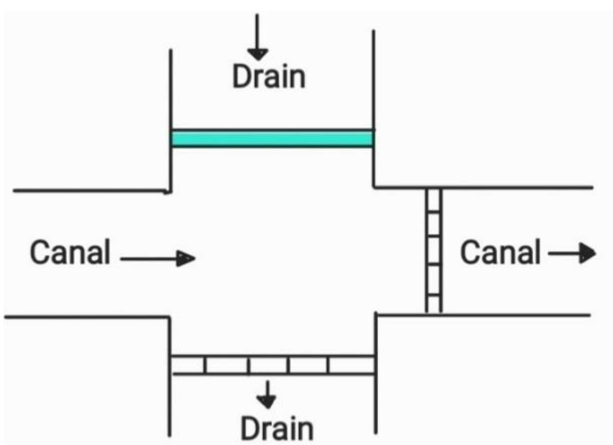
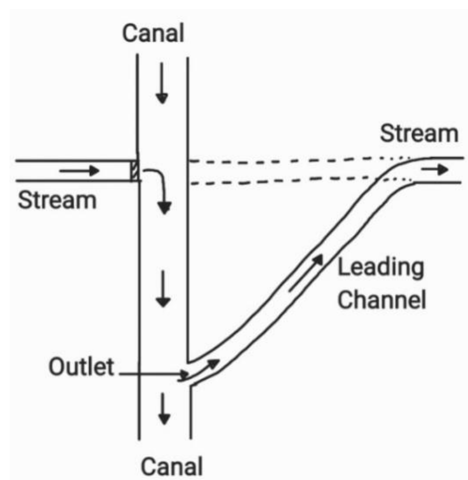


Fig: Super - Passage

3. **Drainage and canal intersection each other at same level:** When water level of canal and the stream are approximately the same and quality of water in canal and stream is not much different, the cross drainage work constructed is called level crossing where water of canal and stream is allowed to mix. With the help of regulators both in canal and stream, water is disposed through canal and stream in required quantity.

4. **Inlet and outlet:** When a small drain meets the irrigationcanal at a level equal or slightly higher than Full Supply Level of canal, drain water and canal water are allowed to mix. The drain water enters the canal through inlet structure. The water carried by drain can be added to canal water. However, if the canal cannot carry the drain water or if the canal water is enough to meet water demand, then drain water is released back to the drain. For this an outlet structure is constructed at some point away from the inlet point to release the water into drain.





10. Chapter 10: Regulators

Regulators are constructed to distribute the water to branch canal, distributaries to provide the required discharge at the required time. Simply regulators are provided to regulate the discharge in main, branch canal and distributaries. Whereas, canal outlets/modules are provided to regulate the discharge in the watercourses and field channels. Generally, there are three types of regulators.

a. Canal Head Regulator: Canal head regulator is provided at the head of main canal to withdraw the required amount of discharge from the river. It also maintains head to supply water to main canal. There is only one canal head regulator in a canal system. One should not be confused about canal head regulator and head regulator (distributary head regulator) as both are different structures.

b. Distributary Head Regulator or Head Regulator: Regulators Constructed at the off taking point are called head regulator. It is constructed at the head of distributary or off-taking canal. It is called distributary head regulator or simply head regulator.

Functions:

i. To control discharge of the off taking canal or branch canal.
ii. To control the entry of silt into off taking canal.
iii. To serve as a meter for measuring discharge of off-taking or branch canal.

c. Distributary Cross Regulators or Cross Regulators: A Regulator Constructed in parent canal downstream of an off-take canal is called distributary cross-regulator or simply cross regulator.

Functions:

iv. To Control the flow of water in entire canal irrigation system
v. When water level in the main canal is low, it helps in heading up water on the upstream and to feed off-take channels to their full demand in rotation.
vi. They help in absorbing fluctuations in various sections of canal system and in preventing possibilities of breaches in the tail reaches.
vii. To act as discharge meter to measure discharge flowing in the parent canal.
viii. Cross regulator is often combined with a road bridge, so as to carry the road which may cross the irrigation channel near the site of cross regulator.

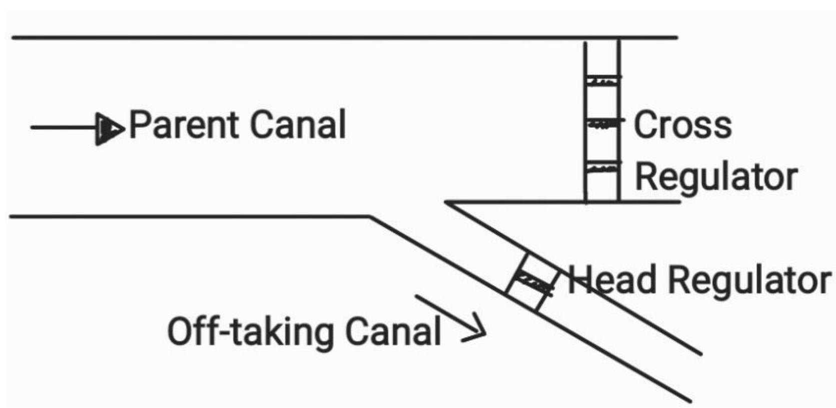


Fig: *Distributary Cross Regulator and Head Regulator*

11. Chapter 11: Drop/Fall Structures

Whenever the available natural ground slope is steeper than the designed bed slope of the channel, the difference is adjusted by constructing vertical 'falls' or 'drops' in the canal bed at suitable intervals, as shown in figure below.

Such a drop in a natural canal bed without any structure will not be stable and, therefore, in order to retain this drop, a masonry/concrete structure is constructed. Such a pucca structure is called a Canal Fall or a Canal drop.

Canal falls should be provided with energy dissipating structure/mechanism.

Falls or drop structures are more required in watershed/ridge and side slope canals as compared to contour canals.

Drop structures are also required if there is sudden change in elevation of ground over which canal flows.

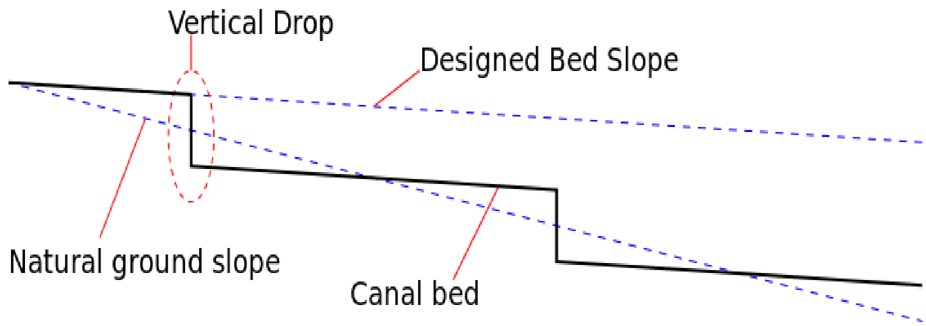


Fig: *Symbolic Representation of Fall/Drop Structure*

Types of Falls/Drops

Depending on the ground level conditions and shape of the fall the various types of fall are:

a. **Ogee Fall:** This type of fall has gradual convex and concave surfaces i.e. in the ogee form. The gradual convex and concave surface is provided with an aim to provide smooth transition and to reduce disturbance and impact. A hydraulic jump is formed which dissipates a part of kinetic energy.

b. **Vertical Fall (Sarda Fall):** In the Sarda fall/vertical fall, the water drops vertically.The energy dissipation is achieved by impact of the drop on the floor. Water Cushion is provided to absorb the impact of energy. This type of fall is Suitable for a discharge upto 15 m3/sec and is suitable for a drop height up to 1.5 m. For discharge up to 14 m3/sec, rectangular crest is provided. For discharge more than 14 m3/sec, trapezoidal crest is provided.

c. **Rapid Fall:** When the natural ground level is uniform and rapid, this rapid fall is suitable. It consists of long sloping glacis. The energy dissipation is achieved by formation of hydraulic jump.This type of fall is costly due to formation of strong jump and hence has become obsolete.

d. **Stepped Fall/Cascade fall:** It consists of a series of vertical drops in the form of steps. This steps is suitable in places where sloping ground is very long and require a long glacis to connect the higher bed level u/s with lower bed level d/s. It is practically a modification of rapid fall. The bed of the canal within the fall is protected by rubble masonry with surface finishing by rich cement mortar.

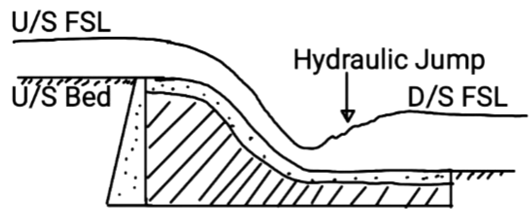


Fig: Ogee Fall

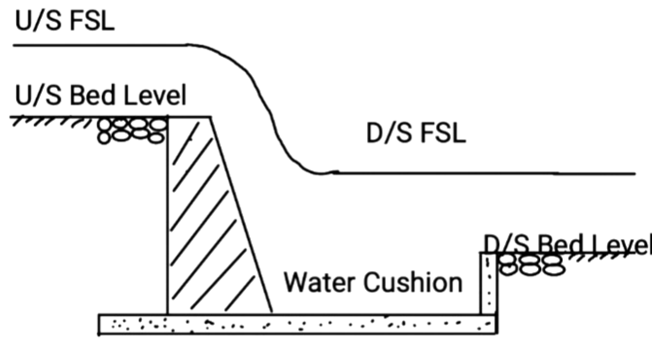


Fig: Vertical Drop Fall

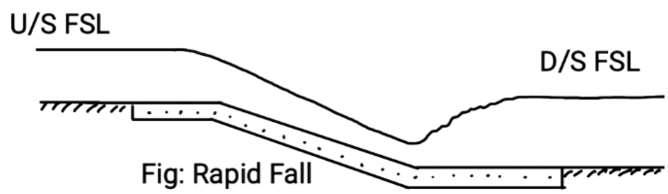


Fig: Rapid Fall

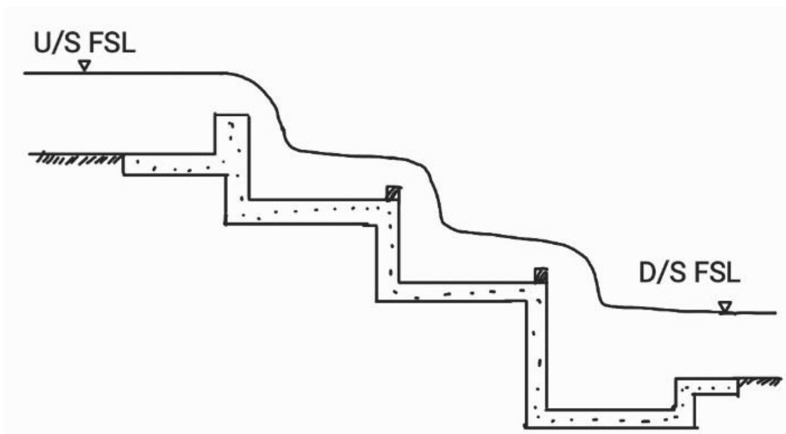


Fig: Cascade/Stepped Fall

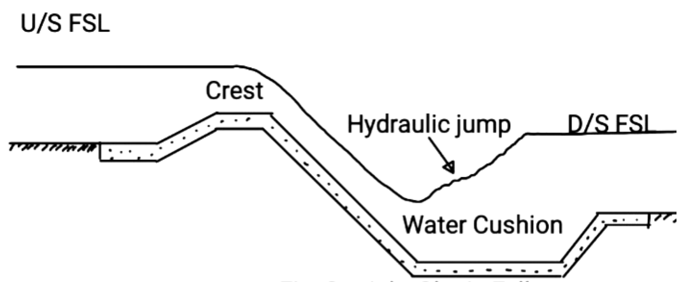


Fig: Straight Glacis Fall

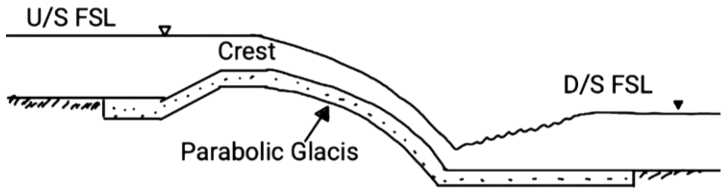


Fig: Montague Fall

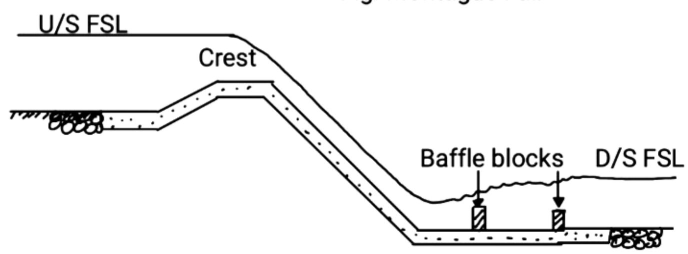


Fig: Baffle/Inglis Fall

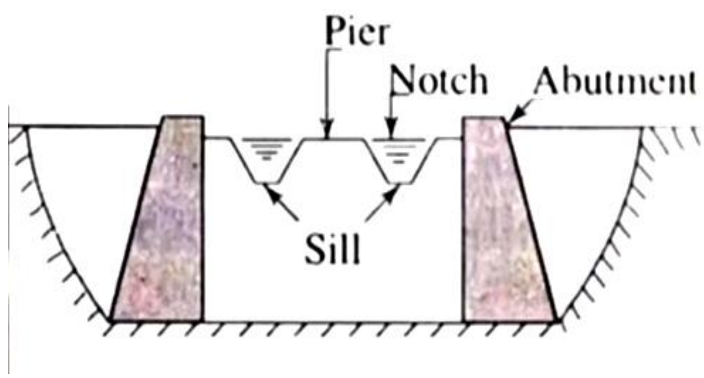


***Fig:** Trapezoidal Notch Fall*

e. **Straight Glacis Fall:** It consists of a straight glacis provided with a crest wall. For dissipation of energy of flowing water, a water cushion is provided where hydraulic jump is formed and energy is dissipated. This type of fall is suitable for a discharge upto 60 m3/sec and drop height of 1.5 m.

f. **Montague Fall:** In the straight glacis type fall, the energy dissipation is not complete. Therefore, in montague type fall, the profile is slightly modified to parabolic type for better energy dissipation. The parabolic surface is difficult to construct and this type of fall becomes costly.

g. **Englis Fall or Baffle Fall:** A straight glacis type fall when added with baffle platform and a baffle wall as shown in the figure is called Englis Fall or Baffle Fall. They are quite suitable for all discharges and for drops more than 1.5 m. They can be flumed easily so as to affect economy. Note: Fluming means reducing width of structure. Fluming is done to reduce cost of structure. The baffle walls are of calculated height and placed at calculated distance, which help in the formation of hydraulic jump and energy dissipation.

h. **Trapezoidal Notch Fall:** The trapezoidal fall consists of number of trapezoidal notches constructed on a high crest wall across the channel with a smooth entrance and flat circular lip projecting downstream from each notch to spread out the falling jet. These falls do not affect the depth of water in the canal on upstream of the fall

12. Chapter 12: Canal Escapes

It is a side channel constructed to remove surplus water from an irrigation channel (main canal, branch canal, or distributary etc.) into a natural drain. The water in the irrigation channel may become surplus due to

i. Mistake in design or operation.
ii. Difficulty in regulation at the head
iii. Excessive rainfall in the upper reaches
iv. Outlets being closed by cultivators as they find the demand of water is over

Types of Escapes:

a. **Surplus Escape:** It is also called regulator type. In this type sill of the escape is kept at canal bed level and the flow is controlled by a gate. This type of escapes are preferred now-a-days as they give better control and can be used for employing the canal for maintenance.

b. **Weir Escape (Tail Escape):** In a weir type escape, the crest of the weir wall is kept at the same level as Full Supply Level (F.S.L.) of the canal so that when water level in the channel becomes more than designed FSL, the water spills out. If a tail escape is provided at the tail end of the canal and is useful in maintaining the required FSL in the tail reaches of the canal and hence, it is called tail escape

c. **Scouring Escape:** This escape is constructed for the purpose of scouring of excess silt deposited in the head reaches from time to time. Hence, it is called scouring escape. Here the sill of the regulator is kept at about 0.3 m below the canal bed level at escape site. When deposited silt is to be scoured, a higher discharge than the Full Supply Discharge is allowed to enter the canal from the head works. The gate of the escape is raised so as to produce scouring velocity which remove the deposited silt. This type of Escape has become obsolete as silt ejector provided in the canal can produce better efficiency.

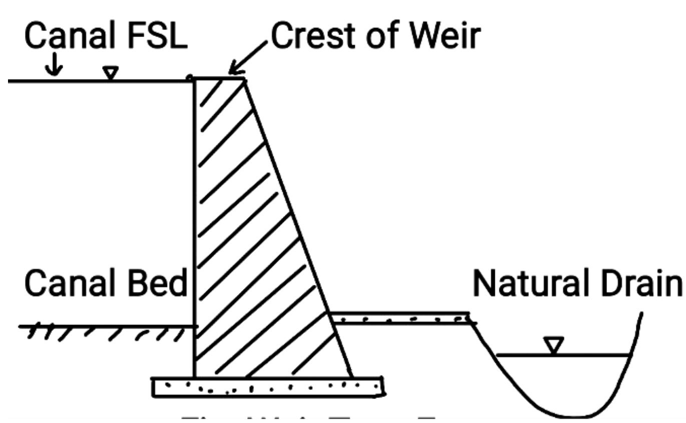


***Fig:** Weir Type Escape*

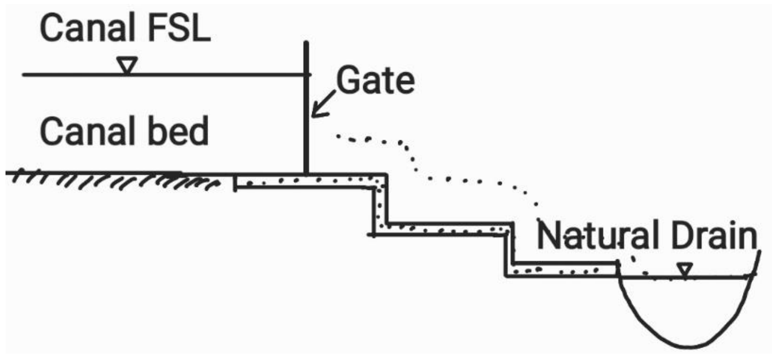


Fig: *Regulator Type (Surplus) Escape*

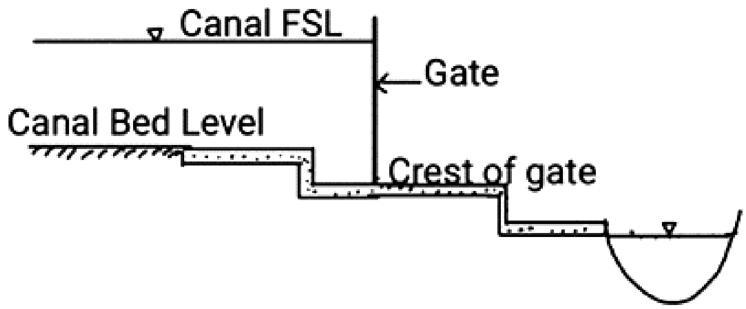


Fig: *Scouring Type Escape*

13. Chapter 14: Outlets/Modules

A canal outlet or a module is a small structure built at the head of the water course. It is provided to supply water to the watercourses from the distributary.

a. Non-Modular Outlets: Non-modular modules are those through which the discharge depends upon the water level of the distributary and the water course. The discharge through the outlet varies with change in water level of watercourse and distributary both. **Common examples are:**

   a. Open sluice

   b. Drowned pipe outlet

b. Semi Modular Outlets or Flexible Modules: In these types of outlets, the discharge depends only upon the water level in the distributary and not upon the water level in the water course. These types of outlets are also called flexible outlets. The discharge changes with the change in water level of distributary only. Examples are pipe outlet, open flume type etc.

c. Modular Outlets or Rigid Modules: Modular outlets or rigid modules or modular outlets are those through which discharge is constant and does not depend on the water level of distributary and of the water course. Examples of such outlets are Gibb's Module, Khanna Module, Foote Module.

14.5. Design Considerations for Cross Drainage Works

The following steps may be involved in the design of an aqueduct or a syphon-aqueduct. The design of a superpassage and a syphon is done on the same lines as for aqueducts and syphon aqueducts, respectively, since hydraulically there is not much difference between them, except that the canal and the drainage are interchanged by each other.

14.5.1. Determination of Maximum Flood Discharge. The high flood discharge for smaller drains may be worked out by using empirical formulas ; and for large drains, other reliable methods such as Hydrograph analysis, Rational formula, etc. may be used.

14.5.2. Fixing the Waterway Requirements for Aqueducts and Syphon-Aqueducts. An approximate value of required waterway for the drain may be obtained by using the Lacey's equation, given by

$$P = 4.75 \cdot \sqrt{Q}$$

where P = is the watted perimeter in metres

Q = Total discharge in cumecs.

For wide drains, the wetted perimeter may be approximately taken equal to the width of the drain and hence, equal to the waterway required. However, no extra provision is generally made for the space occupied by piers. Hence, if the total waterway provided is equal to P, the effective or clear waterway will be less than P by as much extent as is occupied by pier widths. For smaller drains, a smaller figure for the waterway than that given by Lacey's regime perimeter, may be chosen. The maximum permissible reduction in waterway from Lacey's perimeter is 20%. Hence, for smaller drains, the width of the waterway provided should be so adjusted as to provide this required perimeter (minimum value $= 0.8\,P$). The decided clear water way width is provided in suitable number of bays (spans).

Size of the Barrels. After having fixed the waterway width & number of compartments (bays), the height of the drain barrels has to be fixed. In case of an aqueduct, the canal trough is carried clear above the drain HFL, and drain bed is not to be depressed. Hence, the height of bay openings is automatically fixed in aqueducts, as equal to the difference between HFL and DBL of drain.

However, in syphon-aqueducts, the required area of the drainage waterway can be obtained by dividing the drainage discharge by the permissible velocity through the barrels. This velocity through the barrels is generally limited to 2 to 3 m/sec. The waterway area is then divided by the decided waterway width of the drain openings, to compute the height of the openings, and the extent of depressed floor.

Due to the reduction in the width of the drainage, afflux is produced near the work site. The afflux will increase more and more, if the waterway is reduced more and more. The value of afflux is limited, so that there is no flooding of the country-side. The afflux may be calculated by using Unwin's formula as explained below in the following article.

14.5.3. Afflux and Head Loss through Syphon Barrels. It was stated earlier that the velocity through syphon barrels is limited to a scouring value of about 2 to 3 m/sec. A higher velocity may cause quick abrasion of the barrel surfaces by rolling grit, etc. and shall definitely result in higher amount of afflux on the upstream side of the syphon or syphon-aqueduct, and thus, requiring higher and longer marginal banks.

The head loss (h) through syphon barrels and the velocity (V) through them are generally related by Unwin's formula*, given as :

$$h = \left[1 + f_1 + f_2 \frac{L}{R}\right] \frac{V^2}{2g} - \frac{V_a^2}{2g} \quad \text{...(14.1)}$$

where L = Length of the barrel.

R = Hydraulic mean radius of the barrel.

V = Velocity of flow through the barrel.

V_a = Velocity of approach and is often neglected.

f_1 = Coefficient of head loss at entry

= 0.505 for unshaped mouth

= 0.08 for bell mouth.

f_2 = is a coefficient such that the loss of head through the barrel due to surface friction is given by

$f_2 \cdot \frac{L}{R} \cdot \frac{V^2}{2g}$; where f_2 is given as :

$$f_2 = a\left(1 + \frac{b}{R}\right) \quad \text{...(14.2)}$$

where the values of a and b for different materials may be taken as given in Table 14.1.

Table 14.1

Material of the surface of barrel	*a*	*b*
Smooth iron pipe	0.00497	0.025
Encrusted pipe	0.00996	0.025
Smooth cement plaster	0.00316	0.030
Ashlar or brick work	0.00401	0.070
Rubble masonry or stone pitching	0.00507	0.250

*The total head loss consists of three losses, *i.e.*

Entry loss $= f_1 \frac{V^2}{2g}$, (*ii*) Friction loss $= \frac{f_2 L V^2}{2gR}$, (*iii*) Exit loss $= \frac{V^2}{2g}$.

After having fixed the velocity (V) through the barrels, the head (h) required to generate that much velocity can be found by using the equation (14.1).

The d/s HFL of the drain remains unchanged by the construction of works, and thus the u/s HFL can be obtained by adding h to the d/s HFL. The u/s HFL, therefore, gets headed up by an amount equal to h and is known as afflux. The amount of afflux is limited because the top of guide banks and marginal bunds, etc. are governed by this raised HFL. So a limit placed on afflux will limit the velocity through the barrels and *vice versa*. Hence, by premitting a higher afflux and, therefore, a higher velocity through the barrels, the cross-sectional area of syphon barrels can be reduced, but there is a corresponding increase in the cost of guide banks and marginal bunds and also the length of d/s protection is increased. Hence, an economic balance should be worked out and a compromise obtained between the barrel area and afflux. Moreover, in order to reduce the afflux for the same velocity, the entry is made smooth by providing bell mouthed piers and surface friction is reduced by keeping the inside surface of the barrels as smooth as possible.

14.5.4. Fluming of the Canal. The contraction in the waterway of the canal (*i.e.* fluming of the canal) will reduce the length of barrels or the width of the aqueduct. This is likely to produce economy in many cases. The fluming of the canal is generally not done when the canal section is in earthen banks. Hence, the canal is generally not flumed in works of Type I and Type II. However, fluming is generally done in all the works of Type III.

The maximum fluming is generally governed by the extent that the velocity in the trough should remain subcritical (of the order of 3 m/sec). Because, if supercritical velocities are generated, then the transition back to the normal section on the downstream side of the work may involve the possibility of the formation of a hydraulic jump. This hydraulic jump, where not specifically required and designed for, would lead to undue loss of head and large stresses on the work. The extent of fluming is further governed by the economy and permissible loss of head. The greater is the fluming, the greater is the length of transition wings upstream as well as downstream. This extra cost of transition wings is balanced by the saving obtained due to the reduction in the width of the aqueduct. Hence, an economic balance has to be worked out for any proposed design.

After deciding the normal canal section and the flumed canal section, the transition has to be designed so as to provide a smooth change from one stage to the other, so as to avoid sudden transition and the formation of eddies, etc. For this reason, the u/s or approach wings should not be steeper than $26\frac{1}{2}^{\circ}$ (*i.e.* 2 : 1 splay) and the d/s or departure wings should not be steeper than $18\frac{1}{2}^{\circ}$ (*i.e.* 3 : 1 splay). Generally, the normal earthen canal section is trapezoidal, while the flumed pucca canal section is rectangular. It is also not necessary to keep the same depth in the normal and flumed sections. Rather, it may sometimes be economical to increase the depth and still further reduce the channel width in cases where a channel encounters a reach of rocky terrain and has to be flumed to curtail rock excavation. But an increase in the water depth in the canal trough will certainly increase the uplift pressures on the roof as well as on the floor of the culvert, thus requiring larger roof and floor sections and lower foundations. Due to these reasons, no appreciable economy may be obtained by increasing the depth.

The following methods may be used for designing the channel transitions :

(*i*) Mitra's method of design of transitions (when water depth remains constant).

(*ii*) Chaturvedi's method of design of transitions (when water depth remains constant).

(*iii*) Hind's method of design of transitions (when water depth may or may not vary).

(*i*) **Mitra's Hyperbolic Transition when water depth remains constant.** Shri A.C. Mitra, Chief Engineer, U.P. Irrigation Deptt. (Retd.), has proposed a hyperbolic transition for the design of channel transitions. According to him, the channel width at any section *X–X*, at a distance *x* from the flumed section (Fig. 14.13) is given by

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{L_f B_n - (B_n - B_f)\, x} \qquad ...(14.3)$$

where B_n = Bed width of the normal channel section.

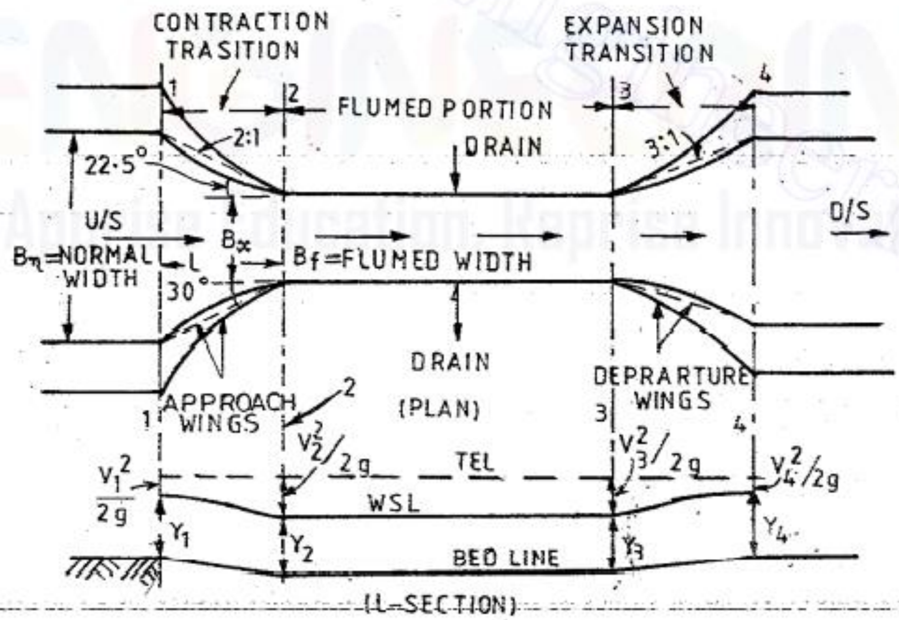
B_f = Bed width of the flumed channel section.

B_x = Bed width at any distance x from the flumed section.

L_f = Length of transition.

Derivation of equation (14.3) is given below :

The above transition equation (*i.e.* equation 14.3) was derived on the basis that the rate of change of velocity per unit length of the transition remains constant throughout



In Fig. 14.13, the contraction transition (*i.e.* the approach transition) starts at section 1-1 and finishes at section 2-2. The flumed section continues from section 2-2 to section 3-3. The expansion transition starts at section 3-3 and finishes at section 4-4. From

section 4-4 onwards, the channel flows in its normal cross-section and the conditions at this section are completely known. Let V and y with appropriate subscripts refer to velocities and depths at different sections.

The FSL at section $4\text{–}4$ = Bed level at section $4\text{–}4 + y_4$ = (known)

$\therefore$ TEL at section $4\text{–}4$ = FSL at section $4\text{–}4 + \dfrac{V_4^2}{2g}$ = (known)

Between section 3-3 and 4-4, there is an energy loss in the expansion, which is generally taken as equal to $0.3\left(\dfrac{V_3^2 - V_4^2}{2g}\right)$.

$\therefore$ TEL at section $3\text{–}3$ = TEL at section $4\text{–}4$ (known) $+ 0.3\left(\dfrac{V_3^2 - V_4^2}{2g}\right)$

As the trough dimensions at section 3-3 are known, V_3 is also known, and hence, TEL at section 3-3 can be computed. Knowing TEL at 3-3 ; FSL at 3-3 can be calculated by subtracting $\dfrac{V_3^2}{2g}$ from TEL. Similarly, bed level at 3-3 can also be computed by substracting y_3 from FSL at 3-3.

Between sections 2-2 and 3-3, the channel flows in a trough of constant cross-section. The only loss in the trough (H_L) is the friction loss which can be computed with Manning's formula, *i.e.*, Manning's formula, *i.e.*,

$$\left(Q = \frac{1}{n} A \cdot R^{2/3} \cdot S^{1/2}\right)$$

or $$Q = \frac{1}{n} A \cdot R^{2/3} \cdot \sqrt{\frac{H_L}{L}}$$

or $$H_L = \frac{Q^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}}.$$

Adding this head loss H_L to TEL of section 3-3, the TEL at section 2-2 is obtained. The FSL at section 2-2 can then be obtained by subtracting $\frac{V_2^2}{2g}$ from TEL of 2-2. Similarly, the bed level at section 2-2 can be easily obtained by further substracting y_2 from FSL at 2-2. Since the depth and velocity are constant in the trough, the TEL, FSL and bed lines are all parallel to each other from section 2-2 to 3-3.

Between section 1-1 and 2-2, there is a loss of energy due to contraction. This loss is generally taken as equal to $0.2\left[\frac{V_2^2 - V_1^2}{2g}\right]$.

Thus the TEL at section 1-1

$$= \text{TEL at section } 2\text{–}2 + 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right) = \text{(known)}.$$

Knowing TEL at section 1-1, FSL at 1-1 can be obtained by subtracting $\frac{V_1^2}{2g}$ from TEL at 1-1. Similarly, bed level at 1-1 can be obtained by subtracting y_1 from FSL at 1-1.

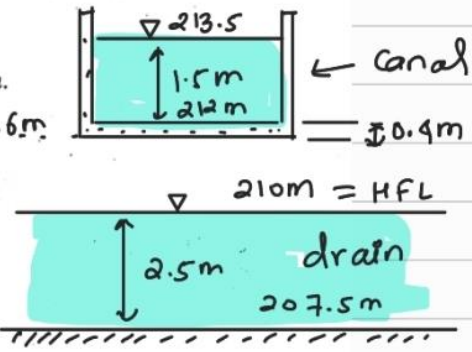
The bed level, FSL and TEL having been determined at all the four sections, the total energy line may be drawn by assuming it to be a straight line between adjacent sections. The bed line may also be drawn straight between adjacent sections, provided, the rise or fall in bed is small. The corners should, however, be rounded off in this case. However, if the change in bed level is considerable, the bed line in the transition section should be drawn as a smooth reverse curve, tangential to the bed lines at ends.

Design of Aqueduct and Siphon Aqueduct:

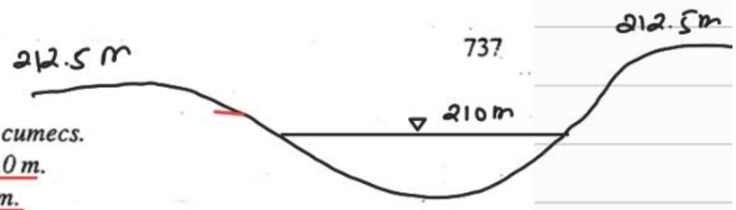
Example 14.1. *Design a suitable cross-drainage work, given the following data at the crossing of a canal and a drainage.*

Canal

Full supply discharge	= *32 cumecs*
Full supply level	= *R.L. 213.5*
Canal bed level	= *R.L. 212.0 m.*
Canal bed width	= *20.*
Trapezoidal canal section with $1\frac{1}{2}$ *H : 1 V slopes.*	
Canal water depth	= *1.5 m.*



CROSS DRAINAGE WORKS 737



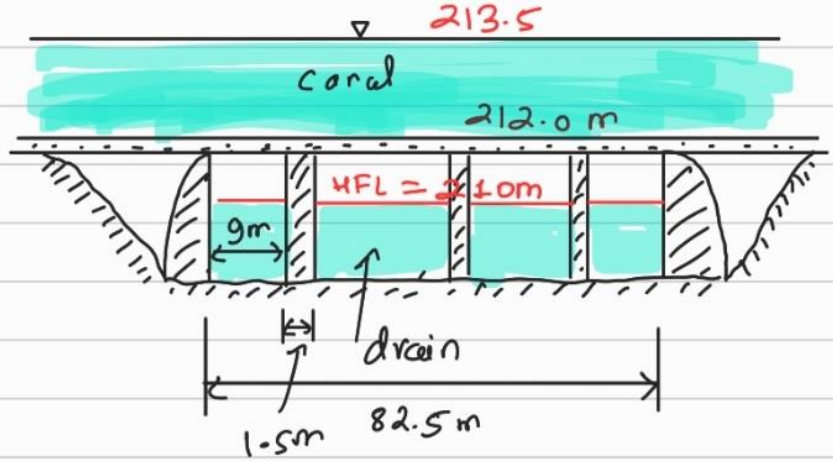
Drainage

High flood discharge	= *300 cumecs.*
High flood level	= *210.0 m.*
High flood depth	= *2.5 m.*
General ground level	= *212.5 m.*

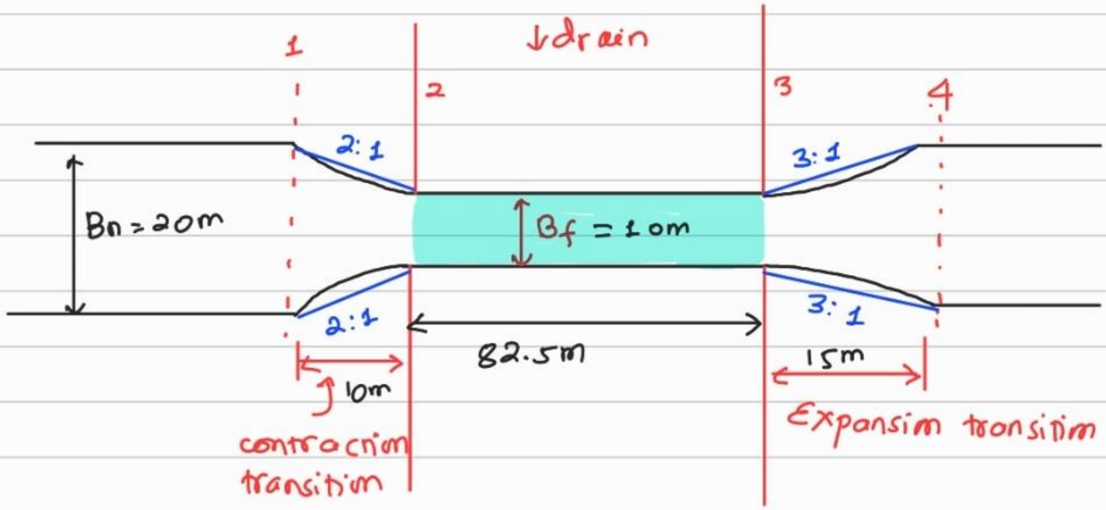
1. Selection of suitable CDS

* Since HFL of drain is lower than bottom level of canal including it's floor (211.6m), provide aqueduct.

2.



Note: no of piers is indicative.



2. Waterway: Lacey's waterway, $P = 4.75\sqrt{Q}$ → for drainage

or $P = 4.75\sqrt{300} = 82.27$

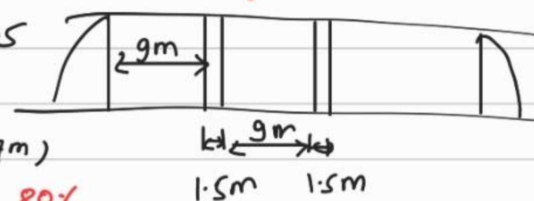
Provide piers at 8 to 10m clear spacing and pier width of 1.5m. Provide clear spacing of 9m.

No of clear spacing $= \dfrac{82.27}{9+1.5} = 7.83 \simeq 8$

So; provide 8 clear spacings and 7 piers.

∴ Actual provided → overall waterway.

waterway, $L = 8*9 + 7*1.5$
$= 82.5m >$



required waterway. (82.27m)

clear waterway should be 80% of required waterway. Clear waterway $= 8*9 = 72m$

80% of required waterway $= 0.8 * 82.27 = 66m$ ⇒ Safe.

③ fluming of Canal: fluming → reduction of width → contraction.

flumed width, $B_f = \dfrac{\text{Normal width } (B_n)}{2} = \dfrac{20}{2} = 10m$

Check for super-critical flow at section 2 or 3.

$$V_2 = \frac{Q}{A_2} = \frac{32}{B_f * y_2}$$

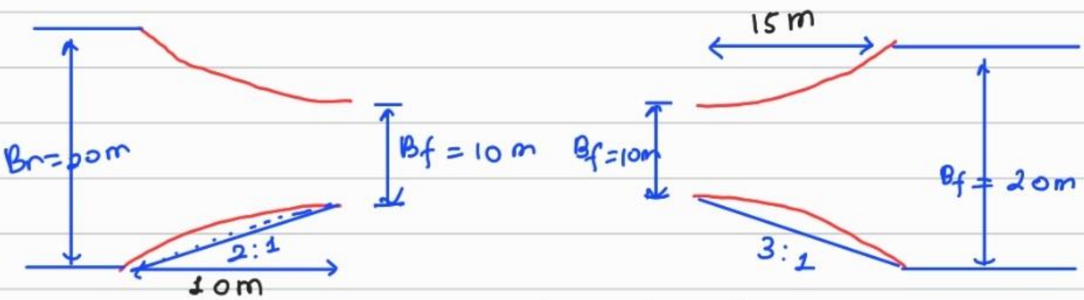
{ Note depth is same at every point in the aqueduct trough $\Rightarrow y_1 = y_2 = y_3 = y_4 = 1.5m$

$$\therefore V_2 = \frac{32}{10 * 1.5}$$

$$= 2.13 \text{ m/sec.}$$

$$Fr_2 = \frac{V_2}{\sqrt{g * y_2}} = \frac{2.13}{\sqrt{9.81 * 1.5}} = 0.55 < 1 \Rightarrow \text{subcritical} \Rightarrow \text{ok.}$$

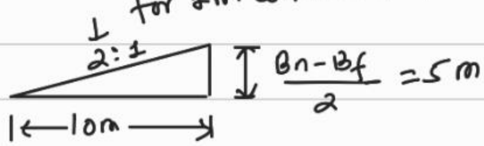
4. length of transition:



a. length of Contraction transition (2:1)

$$= \frac{B_n - B_f}{2} * 2 = \frac{20-10}{2} * 2 = 10m$$

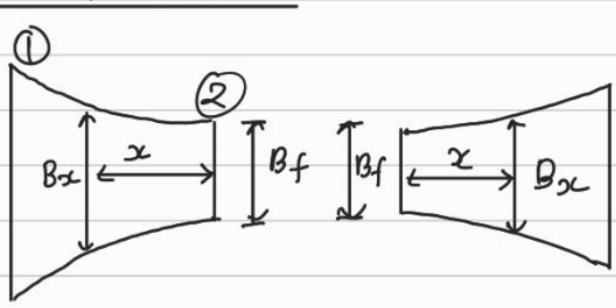
for 1m contraction, 2m horizontal



b. length of expansion transition (3:1)

$$= \frac{B_n - B_f}{2} * 3 = 15 m$$

5. Width of transition:



width of transition at any distance x from flumed portion; $B_x = \frac{B_n \cdot B_f \cdot L_f}{B_n \cdot L_f - (B_n - B_f)x}$ (mitra's hyperbolic transition)

$B_n = 20m$; $B_f = 10m$; L_f = length of transition

for contraction transition; $L_f = 10m$

x	0	2	4	6	8	10
B_x	10	11.11	12.5	14.28	16.67	20

$$B_x = \frac{20*10*10}{20*10 - (20-10)x}$$

for expansion transition; $L_f = 15m$

x	0	3	6	9	12	15
B_x	10	11.11	12.5	14.28	16.67	20

$$B_x = \frac{20*10*15}{20*15 - (20-10)x}$$

6. Calculation of Bed levels:

$$TEL = z + \frac{P}{\gamma} + \frac{V^2}{2g}$$

In open channel flow; z = Bed level

$\frac{P}{\gamma}$ = pressure head = y = water depth

$$\therefore TEL = \text{Bed level} + y + \frac{V^2}{2g}$$

Assume given bed level as bed level at 4.

Note that; $y_1 = y_2 = y_3 = y_4 = 1.5m$

$$V_4 = \frac{Q}{A_4} = \frac{Q}{B_n * y_4 + y_4^2 * z}$$ {side slope given trapezoidal Canal}

side slope = 1.5 H : 1 V $\Rightarrow z = 1.5$

$$\therefore V_4 = \frac{32}{20*1.5 + 1.5^2*1.5} = 0.95 m/sec$$

$\therefore (TEL)_4 = z_4 + y_4 + \frac{v_4^2}{2g} = 212 + 1.5 + \frac{0.95^2}{2*9.81}$

$= 213.54$ m

Also; $(TEL)_3 = (TEL)_4 + (h_f)_{3-4}$

$(h_f)_{3-4} = 0.3 * \frac{v_3^2 - v_4^2}{2g}$

$v_3 = v_2 = 2.13$ m/sec { Calculated in previous step }

$\therefore (h_f)_{3-4} =$

$0.3 * \frac{2.13^2 - 0.95^2}{2*9.81} = 0.055$m

$\therefore (TEL)_3 = 213.54 + 0.055 = 213.595$ m

$z_3 = (TEL)_3 - y_3 - \frac{v_3^2}{2g} = 211.863$ m $\Rightarrow$ bed level at 3

$(TEL)_2 = (TEL)_3 + (h_f)_{2-3}$

$(h_f)_{2-3}$ = frictional head loss

$= \frac{V^2 n^2}{R^{4/3}} * L$

$V = V_2 = V_3 = 2.13$ m/sec; assume $n = 0.016$

L = provided waterway = 82.5m.

Note that; section bet^n 2 to 3 is rectangular.

$R = \frac{A}{P} = \frac{10 * 1.5}{10 + 2*1.5} = 1.153$m

$\therefore (h_f)_{2-3} = \frac{2.13^2 * 0.016^2}{1.153^{4/3}} * 82.5 = 0.079$

$(TEL)_2 = 213.595 + 0.079 = 213.674$

$z_2 = (TEL)_2 - y_2 - \frac{V_2^2}{2g} = 211.94$ m

$(TEL)_1 = (TEL)_2 + (h_f)_{1-2}$

$(h_f)_{1-2}$ = Contraction loss $= 0.2 * \frac{V_2^2 - V_1^2}{2g}$

Note: $V_1 = V_4 = 0.95$ m/sec

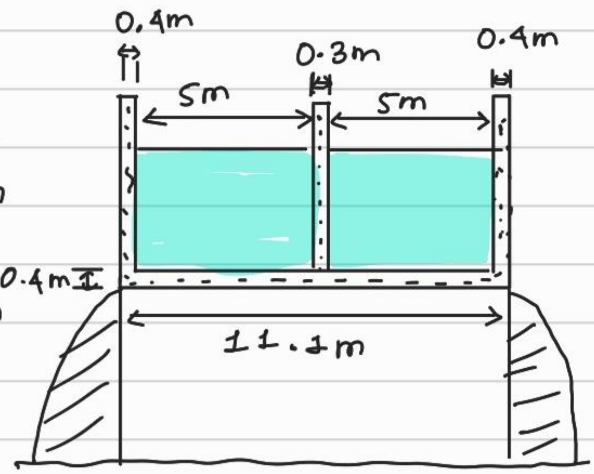
$$\therefore (h_f)_{1-2} = 0.2 \times \frac{2.13^2 - 0.95^2}{2 \times 9.81} = 0.037\,m$$

(contraction loss)

$$\therefore (TEL)_1 = 213.674 + 0.037 = 213.711\,m$$

$$z_1 = (TEL)_1 - y_1 - \frac{V_1^2}{2g} = 212.165\,m$$

7. Design of Canal trough



* Divide the rectangular portion into bays of 5m each.
* Outer wall thickness = 0.4m
* inner wall thickness = 0.3m
* bottom slab thickness = 0.4m
* Total width of trough
  = 5+5+0.4+0.4+0.3
  = 11.1m

Design of Cross - Regulator and Head Regulator:

Example 13.1. *Design a cross regulator and a head regulator for a channel which takes off from the parent channel with the following data :*

Discharge of parent channel	= 140 cumecs
Discharge of distributary	= 15 cumecs
FSL of the parent channel, u/s	= 210.0 m
FSL of the parent channel, d/s	= 209.8 m
Bed width of parent channel, u/s	= 52 m
Bed width of parent channel, d/s	= 46 m
Depth of water in the parent channel d/s and u/s	= 2.5 m

u/s bed level = 207.5 m

Crest level of HR = Head Regulator

= 207.5 + 0.6

= 208.1

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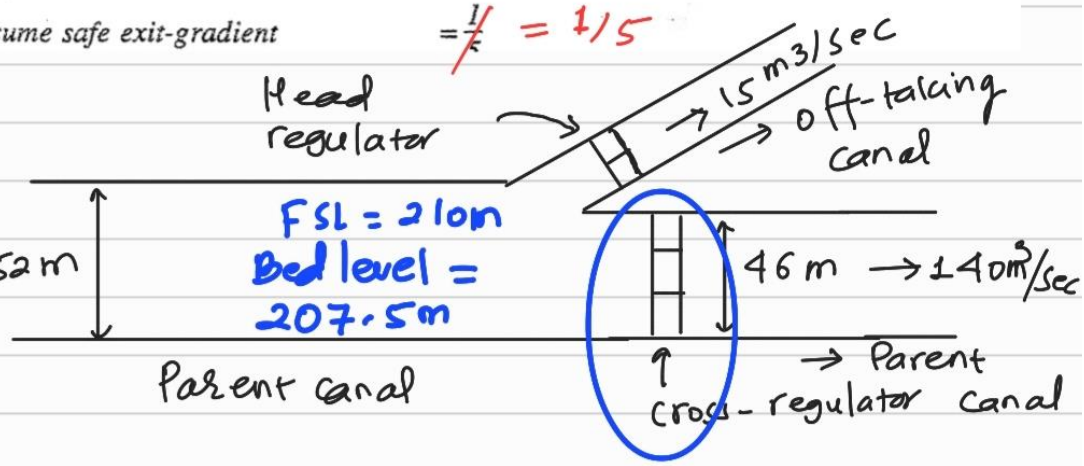
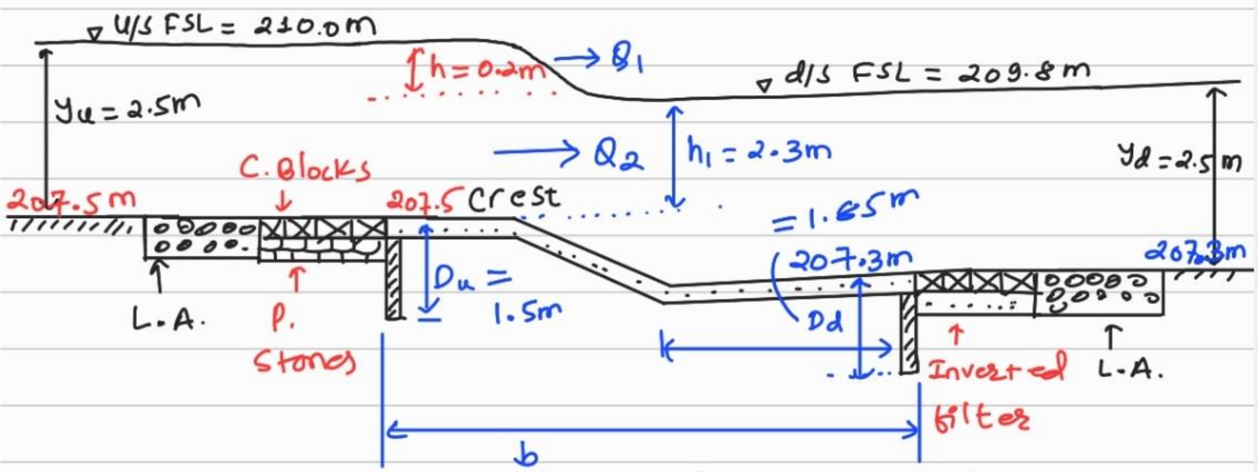
FSL of distributary	= 209.1 m
Silt factor	= 0.8 m
Assume safe exit-gradient	= ~~1/5~~ = 1/5



Design of Cross-Regulator



L-Section of Cross-Regulator

1. Crest levels:

Crest level of cross regulator = u/s bed level

Crest level of head regulator = u/s bed level + (0.3 to 1 m)

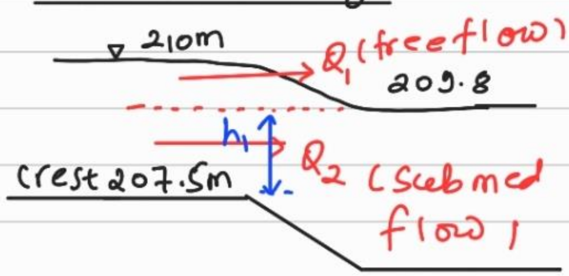
In above example, u/s bed level = u/s FSL − water depth on u/s

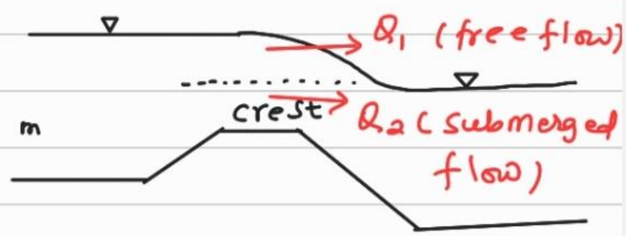
$= 210 - 2.5$

$= 207.5\text{m}$

∴ Crest level of cross-regulator = 207.5m

2. Waterway:





The waterway should be such that it can carry design discharge.

Total discharge; $Q = Q_1 + Q_2$

$Q_1 = \frac{2}{3} Cd_1 * \sqrt{2g} * B * h^{3/2}$; $Cd_1 = 0.577$

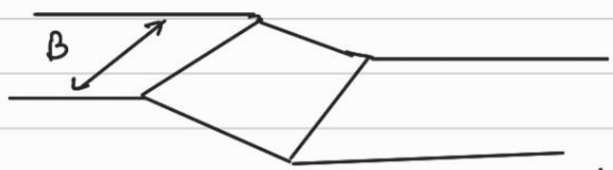
$\therefore Q_1 = 1.69 * Bh^{3/2}$

$Q_2 =$ submerged part discharge

$= Cd_2 *$ Area $*$ velocity

$= Cd_2 * B * h_1 * \sqrt{2gh}$ $\{Cd_2 = 0.8\}$

$\therefore Q_2 = 3.54 * B * h_1 * h^{1/2}$



$\therefore Q = 1.69 * B * h^{3/2} + 3.54 * B * h_1 * h^{1/2}$

$h =$ u/s FSL $-$ d/s FSL $= 210 - 209.8 = 0.2m$

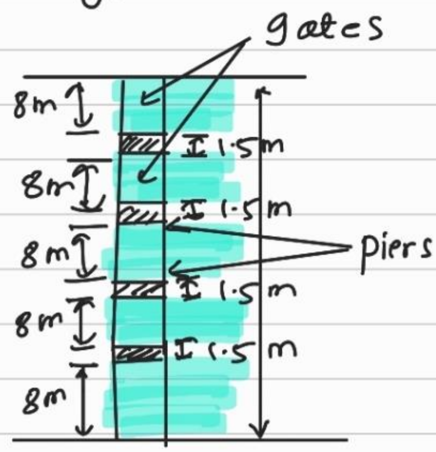
$h_1 =$ d/s FSL $-$ crest level $= 209.8 - 207.5$
$= 2.3m$

$Q = 140 m^3/sec.$

$140 = 1.69 * B * 0.2^{3/2} + 3.54 * B * 2.3 * 0.2^{1/2}$

$\Rightarrow B = 37m$ (clear waterway)

let us provide 5 gates with 8m clear spacings and 4 piers with 1.5 width.



$\therefore$ Provided waterway
$= 8*5 + 4*1.5$
$= 46m$ (total)

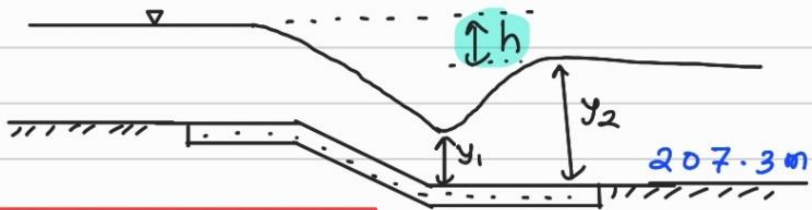
Clear waterway $= 8*5$
$= 40m$

3. D/s floor level :

d/s floor level required $=$ d/s FSL $- Ef_2$

$Ef_2 =$ specific energy after jump $= y_2 + \frac{V_2^2}{2g}$

85/110



$$\boxed{\because \text{specific energy} = y + \frac{v^2}{2g}}$$

To solve for y_1 & y_2 ⇒

$$h_f = \frac{(y_2 - y_1)^3}{4y_1y_2} \; ; \quad \ldots\ldots\ldots \text{i)}$$

$$\frac{2q^2}{g} = y_1y_2(y_1 + y_2) \quad \ldots\ldots\ldots \text{ii)}$$

For simplification; $\frac{y_2}{y_1} = x \Rightarrow y_2 = y_1x$

$h_f = h =$ head loss due to jump $= h =$ u/s FSL − d/s FSL

$= 0.2m$

$$q = \frac{Q}{\text{width (clear waterway)}} = \frac{140}{40} = 3.5\,m^3/sec/m$$

from eq^n i);

$$\frac{(y_1x - y_1)^3}{4y_1 * y_1x} = 0.2 \quad (h_f = 0.2)$$

$$\Rightarrow \frac{y_1^3(x-1)^3}{4y_1^2x} = 0.2$$

$$\Rightarrow y_1 = \boxed{\frac{0.8x}{(x-1)^3}} \quad \ldots\ldots\ldots \text{iii)}$$

Substituting this value in eq^n ii);

$$2 * \frac{q^2}{g} = y_1y_2(y_1 + y_2)$$

or $\dfrac{2 * 3.5^2}{9.81} = y_1 * y_1 x * (y_1 + y_1 x)$

$= y_1^3 * x(1+x)$

$\Rightarrow 2.49 = \left[\dfrac{0.8x}{(x-1)^3}\right]^3 * x * (1+x)$

$\Rightarrow x = 2.43$

$\therefore y_1 = \dfrac{0.8 * 2.43}{(2.43-1)^3} \quad \left\{ \dfrac{0.8x}{(x-1)^3} \right\}$

$= 0.66m$

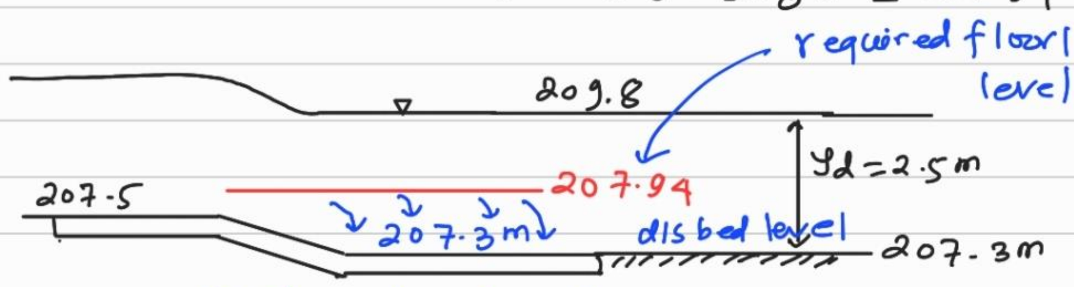
$y_2 = y_1 x = 0.66 * 2.43 = 1.62m$

$V_2 = \dfrac{Q}{A_2} = \dfrac{Q}{B * y_2} = \dfrac{140}{40 * 1.62} = 2.16 \text{ m/sec}$

$\therefore E_{f2} = y_2 + \dfrac{V_2^2}{2g} = 1.62 + \dfrac{2.16^2}{2 * 9.81} = 1.86m$

$\therefore$ Required d/s floor level $=$ d/s FSL $- E_{f2}$

$= 209.8 - 1.86 = 207.94$



d/s floor level $=$ d/s FSL $- E_{f2}$

$= 207.94$

from given data; d/s bed level $=$ d/s FSL $- y_d$

$= 209.8 - 2.5 = 207.3m$

Note: If required d/s floor level comes out to be more than d/s bed level, provide the floor at downstream bed level.

So, downstream floor level $= 207.3m$

4. Depth of cutoff

Depth of u/s cutoff $= D_u = \frac{y_u}{3} + 0.6$

y_u = u/s water depth = 2.5m

$\therefore D_u = \frac{2.5}{3} + 0.6$ | = 1.43m

$\simeq 1.5$m

Depth of d/s cutoff = $D_d = \frac{y_d}{2} + 0.6$ m

$= \frac{2.5}{2} + 0.6$

$= 1.85$m

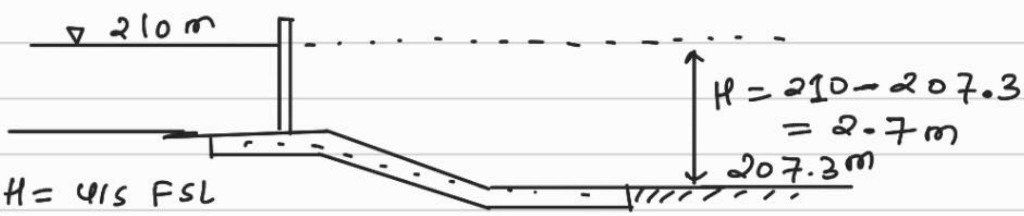
5. Design of floor:

a. length of floor:-

from khosla; (since exit gradient is given)

$G_E = \frac{H}{d} * \frac{1}{\pi\sqrt{\lambda}} \Rightarrow G_E = \frac{1}{5}$; d = depth of d/s cutoff = 1.85m

H = maximum head diff between upstream and downstream.



H = u/s FSL − d/s bed level

= 210 − 207.3 = 2.7 m

$\therefore \frac{2.7}{1.85} * \frac{1}{\pi\sqrt{\lambda}} = \frac{1}{5}$

$\Rightarrow \lambda = 5.4$

But; $\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} \Rightarrow \alpha = 9.75$

Also: $\alpha = b/d \Rightarrow b = \alpha d = 9.75 \times 1.85 = 18.03$ m

$= 18 m.$

$\Rightarrow$ Total floor length, $b = 18 m.$

b. d/s floor length

$$= \max \begin{cases} \frac{2}{3} b = \frac{2}{3} * 18 = 12 m \\ 5(y_2 - y_1) = 5(1.62 - 0.66) \\ \qquad = 4.8 m \end{cases}$$

$= 12 m$

6. Protection Works:

a. Upstream:

i> launching apron:

length $= 1.5 * Du = 1.5 * 1.5 = 2.25 m$

thickness, $T = 1.5 * t$

t = thickness after launching

= 2 * diameter of stones used.

$= 2 * 0.3 = 0.6$ m {dia of stones used = 0.3m}

$\therefore T = 1.5 * 0.6 = 0.9 m$

ii> Concrete blocks:-

Total length $= 1.5 * Du = 1.5 * 1.5 = 2.25 m$

ii> Packed stones

Total length $= 1.5 * Du = 1.5 * 1.5 = 2.25 m$

Note:- thickness of concrete blocks + packed stones = $T = 0.9 m$ (thickness of launching apron)

b. downstream:

i> launching apron:

* length $= 1.5 * Dd = 1.5 * 1.85$
$= 2.775 m \simeq 2.8 m$

* thickness $= 0.9 m$ (same as u/s)

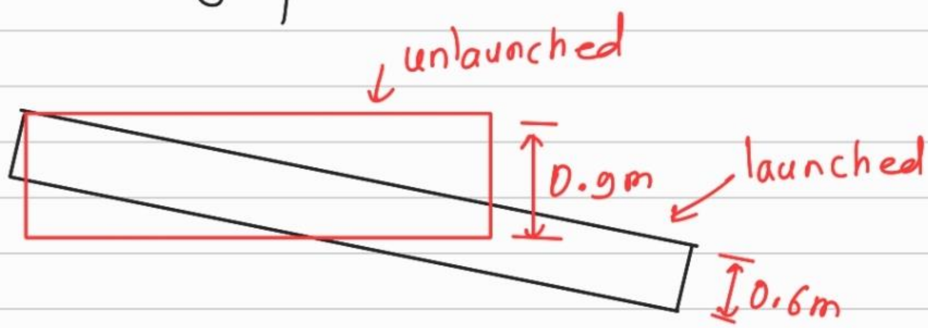
ii) Concrete blocks :-

Total length = $1.5 * D_d \simeq 2.8$ m

iii) Inverted filter:

* Total length = $1.5 * D_d \simeq 2.8$ m

Note :- Thickness of concrete blocks + inverted filter = 0.9m (thickness of launching apron).



Design of Head Regulator

1. Crest level : Provide crest at 0.6 m above upstream canal bed.

Crest level = u/s canal bed level + (0.3 to 1 m)

= 207.5 + 0.6 = 208.1 m

(for head regulator, parent canal is on upstream and branch canal is on downstream.)

2. Design of canal section:

We have:, velocity, $V = \left(\frac{Qf^2}{140}\right)^{1/6} = \left(\frac{15 * 0.8^2}{140}\right)^{1/6}$

$= 0.64$ m/sec

wetted perimeter, $P = 4.75\sqrt{Q} = 18.4$ m

As this is not wide rectangular channel, so assume

$B = 0.8P = 0.8 * 18.4 = 14.72$ m $\simeq 15$ m

So; assuming rectangular canal, $y_d = y = \frac{Q}{V * B} = \frac{15}{15 * 0.64}$

$= 1.56m$

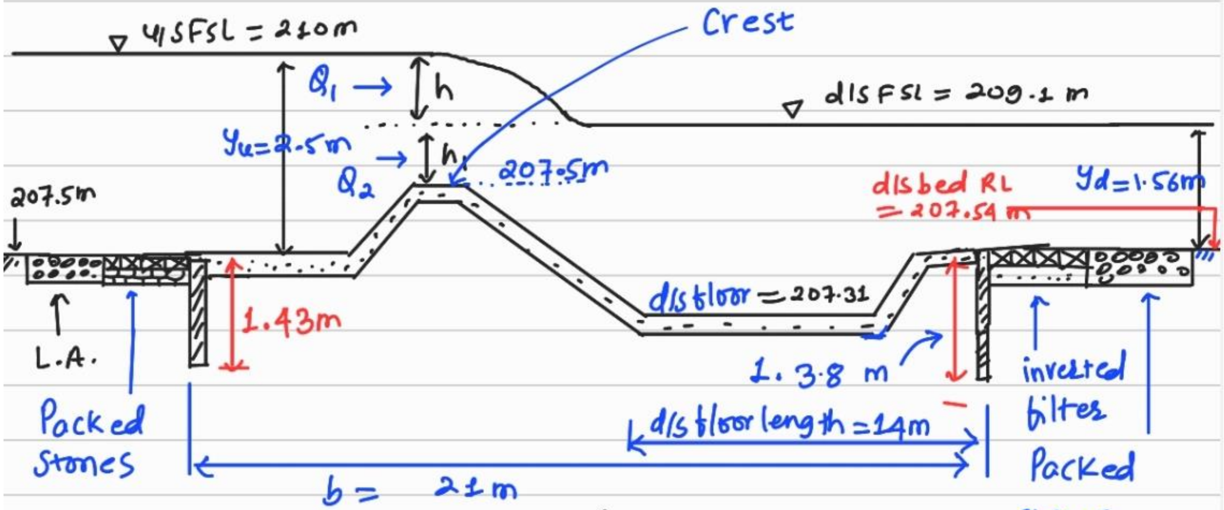


Fig:- L-Section of Head Regulator

3. Waterway:

The water way is calculated by:

$$Q = 1.69 * B * h^{3/2} + 3.54 * B * h_1 * h^{1/2}$$

where; $Q = 15\ m^3/sec$; B = required clear waterway.

$h = u/s\ FSL - d/s\ FSL = 210 - 209.1 = 0.9m$

$h_1 = d/s\ FSL - \text{Crest level} = 209.1 - 208.1$

$$\therefore B = \frac{Q}{1.69 * h^{3/2} + 3.54 * h_1 * h^{1/2}} = 3.12m$$

Since, available canal width is 15 meter and required water way (clear) is 3.12 meter only, for safety, we provide 2 bays of 3.0 meter each and one pier of 1.0 meter width.

$\therefore$ Clear waterway $= 2 * 3 = 6m$

overall waterway $= 2 * 3 + 1 = 7m$.

So, the canal must be flumed near regulator portion.

4. D/S floor level

d/s floor level required = d/s FSL $- E_{f2}$

E_{f2} = specific energy after jump formation $= y_2 + v_2^2/2g$

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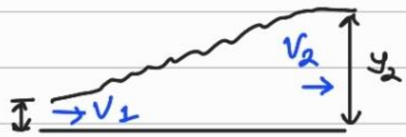
To solve for y_2:



use jump equation:

$$y_1 y_2 * (y_1 + y_2) = \frac{2q^2}{g} \quad \ldots\ldots i)$$

$$h_f = \frac{(y_2 - y_1)^3}{4 y_1 y_2} \quad \ldots\ldots\ldots ii)$$

where q = specific discharge $= \frac{Q}{\text{clear waterway}}$

$= \frac{15}{6} = 2.5\ m^3/sec/m$; $h_f = h = 0.9m$

let; $\frac{y_2}{y_1} = x$; then; from eqn. ii); we write:

$$0.9 = \frac{(y_1 * x - y_1)^3}{4 y_1 * y_1 x} \Rightarrow \frac{y_1^3 * (x-1)^3}{4 y_1^2} = 0.9$$

$$\Rightarrow y_1 = \frac{3.6 * x}{(x-1)^3} \quad \ldots\ldots\ldots iii)$$

Substitute for y_1 in eqn i). Then;

$$y_1 y_2 * (y_1 + y_2) = \frac{2q^2}{g} \Rightarrow y_1 * y_1 x * (y_1 + y_1 x) = \frac{2 * 2.5^2}{9.81}$$

$$\Rightarrow y_1^3 * x * (1+x) = 1.27$$

$$\Rightarrow \left[\frac{3.6 * x}{(x-1)^3}\right]^3 * x * (1+x) = 1.27$$

solving; $x = 4.53$; $y_1 = \frac{3.6 * 4.53}{(4.53-1)^3} = 0.37m$

$y_2 = y_1 x = 1.68m$

Now; $V_2 = \frac{Q}{A_2} = \frac{Q}{B * y_2} = \frac{15}{6 * 1.68} = 1.49\ m/sec$

$$\therefore E_{f2} = y_2 + \frac{V_2^2}{2g} = 1.68 + \frac{1.49^2}{2 * 9.81} = 1.79m$$

$\therefore$ d/s floor level = d/s FSL − E_{f2} = 209.1 − 1.79
= 207.31m

Here; d/s bed level = d/s FSL − water depth in distributary
= 209.1 − 1.56 = 207.54 m

Since; required d/s floor level is below d/s bed level, provide d/s floor at 207.31 m

5. Cutoff depths: Depth of u/s cutoff; $D_u = \frac{y_u}{3} + 0.6$
$= \frac{2.5}{6} + 0.6 = 1.43m$

Depth of d/s cutoff, $D_d = \frac{y_d}{2} + 0.6 = \frac{1.56}{2} + 0.6 = 1.38m$

6. Floor lengths

a. Total floor length from Khosla; $G_E = \frac{H}{d} * \frac{1}{\pi\sqrt{\lambda}}$

Here, H = maximum head difference between u/s and d/s
= u/FSL − d/s bed level = 210 − 207.54 = 2.46m

$d = D_d = 1.38m$

$\therefore \frac{1}{5} = \frac{2.46}{1.38} * \frac{1}{\pi\sqrt{\lambda}} \Rightarrow \lambda = 8.03$

But; $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \Rightarrow \alpha = 15.03$

But; $\alpha = \frac{b}{d}$
$\Rightarrow b = \alpha * d$
$= 15.03 * 1.38$
$= 20.73m$
$\simeq 21.00m$

b. d/s floor length: maximum of $\{\frac{2}{3} * b$ and $5(y_2 - y_1)$
$2/3b$ = max {14, 6.55} = 14m

7. Protection works:

a. Protection works on u/s

i> launching apron length = $1.5 * D_u$ = 2.15m

* launching apron thickness = same as cross regulator = 0.9m

ii> Total depth of concrete blocks & packed stone combined = 0.9m

iii> Total length of concrete blocks and launching apron each = 2.15m

b. Protection Works on d/s i> launching apron length $= 1.5\,Dd = 2.07\,m$

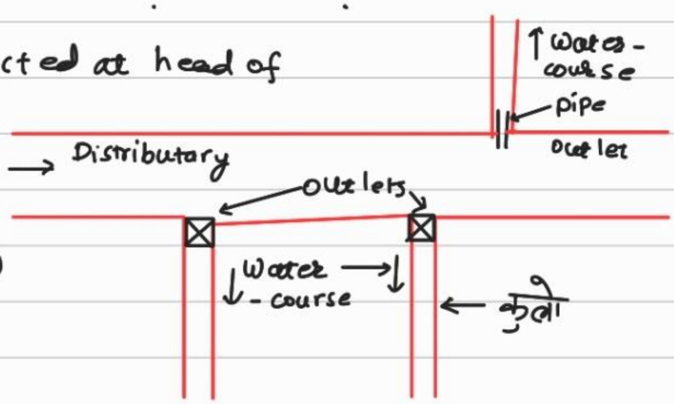
* launching apron thickness = Same as u/s = 0.9 m

ii> Total depth of concrete blocks & packed stone combined = 0.9

iii> Total length of concrete blocks and launching apron each = 2.07

Canal Outlets.

* The structures constructed at head of the water-course to provide water to the water-courses (कुलाहरु कुलो) are called outlets.



Types of Outlets:

a. Non-modular Outlet

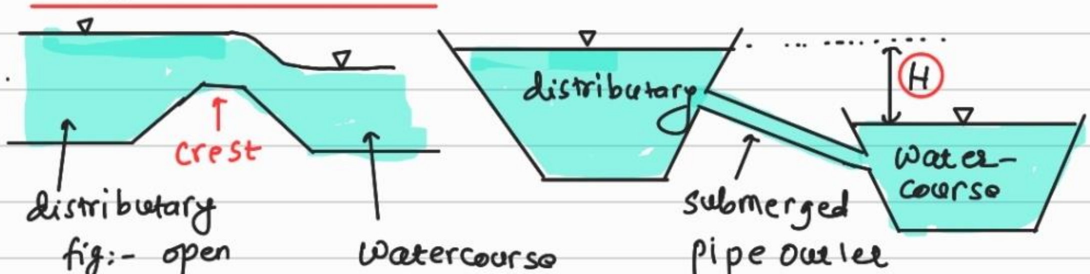


fig:- open sluice

* In this type of outlet, the discharge through outlet depends on water level of both distributary and water-course.

* e.g; open sluice, submerged/drowned pipe outlet.

Design of submerged pipe outlet

Total Head loss = H

⇒ entry loss + friction loss + exit loss = H

$$K_e \times \frac{V^2}{2g} + \frac{f l V^2}{2gd} + \frac{V^2}{2g} = H$$

K_e = coefficient of entry loss = 0.5

$$\Rightarrow 0.5 * \frac{V^2}{2g} + \frac{fl}{d} * \frac{V^2}{2g} + \frac{V^2}{2g} = H$$

$$\Rightarrow \left(1.5 + \frac{fl}{d}\right) * \frac{V^2}{2g} = H \quad \ldots\ldots\ i>$$

from this we can get V.

If V is known; $Q = A * V$

C.W.C. Guidelines has recommended following formula for discharge;

$$Q = Cd * A * \sqrt{2gH}$$

H = water level difference

$Cd = 0.73$

$$A = \frac{\pi d^2}{4}$$

b. Semi modular outlet / flexible outlet

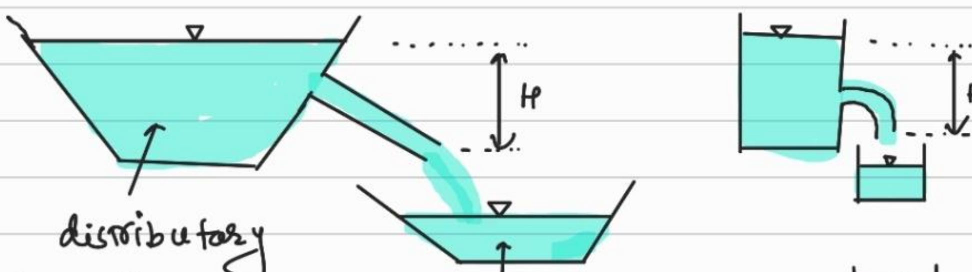


Fig:- free Pipe outlet

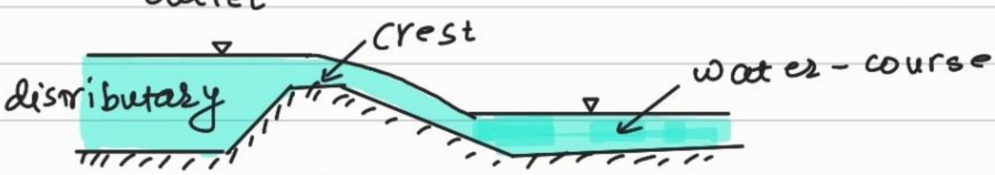


Fig:- open flume

* In this type of outlet, the discharge depends on water level of distributary only and not on the water level of water-courses.
* e.g; free pipe outlet, open flume.

Design of freepipe outlet:

$Q = Cd * A * \sqrt{2gH}$

Cd = coefficient of discharge = 0.62

H = water level of distributary – elevation of end of pipe.

$A = \frac{\pi d^2}{4}$

c. Modular Outlet / Rigid module

* In this type outlet, discharge remains constant, i.e., discharge doesnot depend on water level of both distributary and water-course.
* e.g; Gibb's module, Khanna module, Foote module

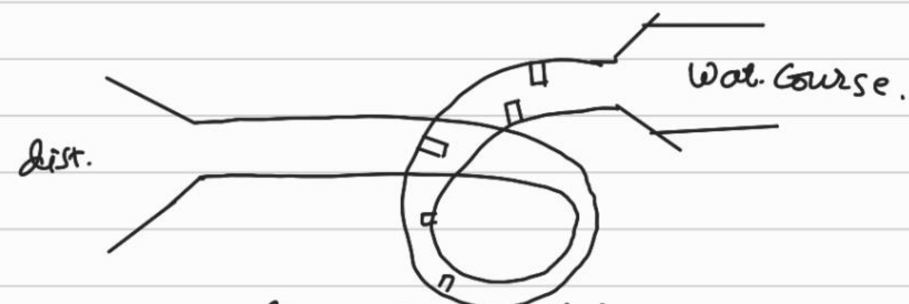


fig:- Gibb's module

Q1: What are the factors to be considered while selecting the alignment of canals

Following factors should be considered while fixing the canal alignment:

1. The canal should be straight, as a result, the length of the canal will be minimum, which helps to reduce the loss of water due to percolation and evaporation.

2. It should be spotted that the canal does not pass through a village or town but by the side of it.

3. Cross-drainage works should be avoided since such works are very costly.

4. Deep cuttings and high filling should be avoided.

5. The alignment should avoid fissure and brackish formations. Fissured formations cause loss of water through fissures, and brackish formations render water useless.

6. To get off excessive percolation losses, the alignment of the canal should avoid passing through sandy tracks.

7. The alignment should avoid rock formation. Required a lot of labor for the construction of canals in the rocky pathways.

8. Idle length of the canal should be minimum.

9. Suitable foundations and construction materials should be available for the works like falls, cross regulators, head regulators etc.

10. Unnecessary curves in the canals should be avoided, if the curves are necessary, it should be as large radius as possible.

11. The cutting and filling should be such that(i.e. cutting should be equal to filling as possible), it will be most economical.

Q2. Factors Affecting The Water Requirement of Crops

Following are the five factors affecting the water requirement of crops:

Water Table

Water requirements for the crop depend on the position of the water table. Water requirement will be less if the **water table** is nearer to the ground surface. But if the **water table** is much below to the ground surface, therefore water requirement will be more.

Ground Slope

If the **ground slope is steep**. The water flows down quickly along the slope of the land, and the soil gets less time to absorb required moisture for their growth, that means the water loss is more for steep ground. Hence, the water requirement will be more.

But if the **ground is flat**, the water flows gently and the soil gets enough time to absorb the required moisture. so the water requirement is less.

Weather

In **hot weather**, the loss of water is more due to the increase of evaporation and also increase the transpiration through leaves, therefore, the water requirement will be more.

But in **cold weather**, the evaporation loss is very low, therefore the water requirement will be less.

Method of Application of Water

In **surface method**, evaporation loss is more. Hence, more water required to overcome the loss of water due to evaporation.

But in **sub-surface method**, evaporation loss is less, so water requirement will be less.

Method of Ploughing

For **deep ploughing**, the water requirement is less because the soil can hold moisture for a long period and vice versa.

Factors Affecting Duty of Water

Method and system of irrigation

(i) Furrow method ⇢ High duty
(ii) flood irrigation ⇢less duty than the furrow system
(iii) Perennial system ⇢ high duty
(iv) Inundation system ⇢ low duty than the perennial system
(v) basin irrigation ⇢ low duty
(vi) Flow system ⇢ low duty
(vii) Lift irrigation ⇢ High duty
(viii) Un-controlled flooding ⇢ low duty

(ix) The duty of water is high for sprinkler and drip irrigation method, as compared to the surface irrigation method

Types of Crops

Different crops need completely different quantities of water. So, the duty of water varies from crop to crop. The crops that need a large volume of water have a lower duty of water than for the crops which need less amount of water.

Quality of irrigation water

(i) Harmful salt and alkali content leads to a lower duty of water due to the wastage of a considerable amount of water.
(ii) Fertilizing matter in water increase duty

Method of Cultivation

If the land is correctly ploughed up to the specified depth and created quite loose before irrigating, the soil can have high water holding capability within the root zone of the plants. This may cut back the quantity of watering and hence result in a higher duty of water.

Time of irrigation

(i) within the initial stages, the land to be cultivated might, not be properly levelled arid hence more than the required amount of water may be applied, which leads to a lower duty of water.

(ii) The gradual rise of the water table with time because of continuous irrigation will make water accessible in the root zone of the plants, therefore comparatively less amount of water is going to be needed to be applied, which will result in a higher duty of water.

Canal Condition

(i) Earthen canal ⇢ percolation loss is high ⇢which leads to a lower duty of water.

(ii) Lined canal⇢ percolation loss will be less ⇢hence, the duty of water will be high.

The climatic condition of the area

The climatic condition which affects the duty of water is
(i) Temperature ⇢ high temperature ⇢loss of water will be more due to Evapotranspiration ⇢so, low duty.

(ii) wind-higher wind velocity-loss of water will be more due to evaporation -low duty.

(iii) Humidity⇢ high humidity⇢ loss of water will be less due to Evapotranspiration⇢ so, high duty.

(iv) Rainfall⇢ increase in duty

Base periods of crops

When the base period of a crop is long, more water may be required so leading to a lower duty of water.

The character of soil and subsoil of the canal

(i) Coarse-grained soil⇢ seepage and percolation losses are high⇢ so less duty.

(ii) Fined grained soil⇢ Percolation losses are less ⇢so, high duty.

Method of assessment

(i) Volumetric method⇢ more duty (economical use of water).

(ii)crop rate or area basis⇢ less duty (more water to be used).

Methods of Improving Duty of Water

Proper Ploughing

The duty of water is high in case of deep and proper ploughing because it helps to increase moisture retaining capacity of the soil for a long period. So, the duty of water should be improved by avoiding shallow ploughing and adopting deep ploughing.

Crop Rotation

The rotation of crop must be practiced to increase the fertility of the soil as well as to increase the moisture-retaining capacity of the soil.

Method of Irrigation System

The duty of water is high in the case of perennial irrigation system because, in this system, the head regulator is used. So, by using this type of irrigation system, the duty of water can be increased.

Implementation of Tax

The water tax should be implemented based on the water consumption volume. As a result, the farmers will use the water economically, and thus the duty of water will be high.

Frequently Cultivation

The land should be cultivated frequently because frequent cultivation minimizes the loss of moisture from the soil.

The canal lining should be done such a way that it reduces the percolation loss of water.

Reduce Transmission Loss

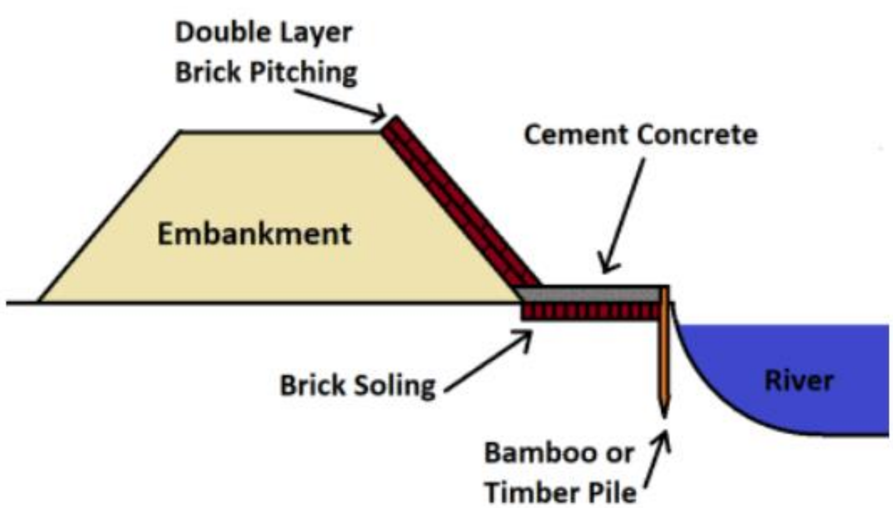
To overcome transmission loss of the water, the canals should be as possible as near to the cultivated land.

Methods of River Bank Protection

1. Brick Pitching
2. Stone Riprap
3. Boulder Pitching
4. Concrete Slab Lining
5. Groynes/Spurs

In this method of river bank protection, first of all, around 3m length of bamboo or timber piles are driven along a line about 1 m away from the toe of the river embankment, and the center to center distance between one pile and the next pile is kept 15 cm.

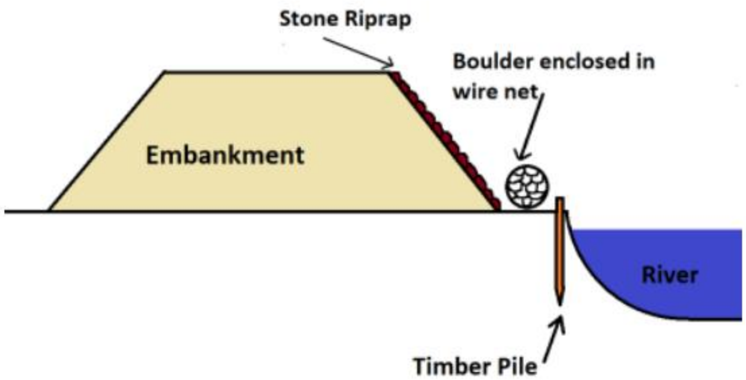
The layer of brick flat soling is provided on the space between the toe and the pile line. Then usually, 15 cm thickness of cement concrete (1:3:6) is laid over the brick flat soling on the room between the toe of the embankment and the pile line.



The sloping side of the bank is protected by double-layer brick pitching with ce

ment mortar of 1:6 ratio.

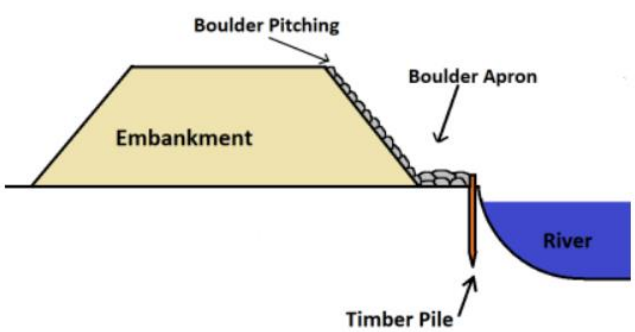
In this method of bank protection, Around 3 m length of timber piles are driven at 1 m center to center along the line about 1 m away from the toe of the embankment. The piles are projected about 45 cm above the ground surface.



Then the boulders enclosed in wire net is provided along the space between the toe and the pile line. And for the protection of the sloping side of the embankment, stone riprap finished with cement mortar is provided.

In this method of river bank protection, around 4 m to 5 m length of timber piles are driven at 1 m center to center, along the line about 1 m away from the toe of the embankment. The piles are projected about 50 cm above the ground surface.

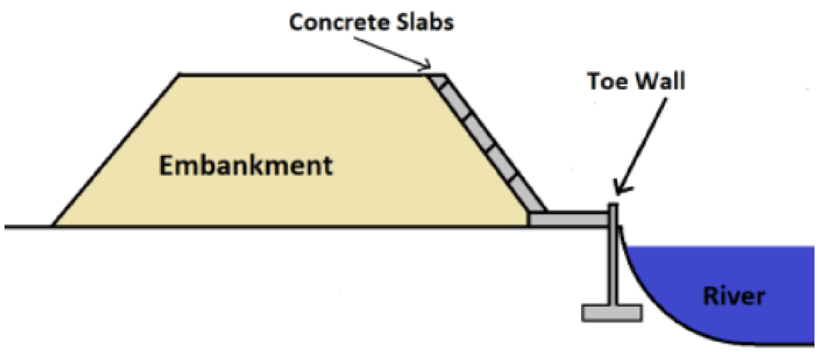
Then, within the space between the toe and the pile line, two layers of boulder apron are provided. The sloping side is lined with boulder pitching which is finished with cement mortar.



In this method of bank protection, A wall is constructed along the bank of the river. This type of wall is called the toe wall.

Then the concrete slabs are placed on the space between the toe of the embankment and toe wall, and it is set by using cement mortar. The sloping side of the embankment is lined with concrete slabs, and joints are finished with cement mortar.

Depending on the site condition, concrete slabs may be of different sizes. Usually, the size of the concrete slab (50 cm X 50 cm X 10 cm) is used.



Standards of canal cross section

- Usual dimensions of canal cross-section elements have been given here and there inappropriate articles.
- The standards as suggest by CWPC are given as follows.

Element	*Discharge of canal in cumecs*					
	Below 0.3	*0.3 to 1*	*1.0–5*	*5–10*	*10–30*	*30–150*
1. Minimum width of bank	0.6 in	1.0 m	1.5 m	2.0 m	3.5 m	5 m
2. Free board	0.3 in	0.4 in	0.5 in	0.6 in	0.75 m	0.90 m
3. Width of road way	nil	3.5 m	3.5 m	5 m	5 m	6 m
4. Depth of cover over saturation gradient	0.5 m	0.5 m	0.5 m	0.5 m	0.75 m	1 m

Indian standard criteria for design of unlined canals in alluvial soil (IS: 7112—1973)

General.

- A trapezoidal section is recommended for the canal.

- The longitudinal slope of the canal is determined depending upon the average slope.
- The natural ground along the proposed alignment.
- This is the maximum average slope that can be provided on the canal.

Side Slopes.

- These are dependent on the local soil characteristics and are designed to withstand.
- The following conditions during the operation of the canal.

(a) The sudden drawdown condition for inner slopes.

(b) The canal running full with banks saturated due to rainfall.

- Canal in the filling will generally have side slopes of 1.5: 1.
- For canals in cutting, the side slope should be between 1: 1 and 1.5: 1 depending upon the type of soil.

Free Board.

- Freeboard in a canal is governed by consideration of the canal size and location, rainwater inflow, water surface.
- Fluctuations caused by regulators, wind action, soil characteristics, hydraulic gradients.
- Service road requirements, and availability of excavated material.
- A minimum freeboard of 0.5 m for discharge less than 10 cumecs and 0.75 m for discharge greater than 10 cumecs is recommend.
- The freeboard shall be measured from the FSL to the level of the top of the bank.
- The height of the dowel portion should not be used for freeboard purposes.

Bank Top Width.

The minimum values recommended for top width of the bank are as follows.

Discharge in m^3/sec	*Minimum bank top width*	
	Inspection bank (m)	*Non-inspection bank (m)*
0.15 to 7.5	5	1.5
7.5 to 10.0	5.0	2.5
10.0 to 15.0	6.0	2.5
15.0 to 30.0	7.0	3.5
30.0 and above	8.0	5.0

The remaining Portion

Q. The culturable commanded area for a distributary is 15,000 hectares. The intensity of irrigation (I. I.) for Rabi (wheat) is 40% and for Kharif (rice) is 15%. If the total water requirement of the two crops are 37.5 cm and 120 cm and their periods of growth are 160 days and 140 days respectively: (a) Determine the outlet discharge from average demand considerations; (b) Also determine the peak demand discharge, assuming that the kor water depth for two crops are 13.5 cm and 19 cm and their kor periods are 4 weeks and 2 weeks respectively.

Sol^n: C.C.A. = 15,000 ha

I.I. of Rabi = 40%

I.I. of Kharif = 15%

Δ for Rabi = 37.5 cm

Δ for Kharif = 120 cm

Base period for Rabi = 160 days

Base period for Kharif = 140 days

Kor depth for Rabi = 13.5 cm

Kor depth for Kharif = 19 cm

Kor period for Rabi = 4 weeks

Kor period for Kharif = 2 weeks

$Q_{average}$ = ?

Q_{peak} = ?

a. Average Demand Consideration

Rabi

Area irrigated, $A = 40\%$ of CCA

$= 0.4 * 15,000 = 6000\,ha$

Duty, $D = 864 * \frac{B}{\Delta} = 864 * \frac{160}{37.5}$

or $D = 3686.4$ ha/cumec

$\therefore$ Required discharge in Rabi season

$= \frac{\text{Area irrigated}}{\text{duty}} = \frac{6000}{3686.4}$

$\therefore Q = 1.63\ m^3/sec$

Kharif Season:

Area irrigated $= 15\%$ of C.C.A.

$= 0.15 \times 15000$

$= 2250\,ha$

Duty, $D = 864 * \frac{B}{\Delta} = 864 * \frac{140}{120}$

or $D = 1008$ ha/cumecs

$\therefore$ Discharge required for Kharif season,

$Q = \frac{\text{Area}}{\text{Duty}} = \frac{2250}{1008} = 2.23\ m^3/sec$

$\therefore$ design discharge based on average demand is $Q = 2.23\ m^3/sec$

b. Peak Demand

Kharif season

Area irrigated, $A = 2250$ ha

Duty considering peak demand, $D = 864 \times \frac{B_{kor}}{\Delta_{kor}}$

$\therefore D = \frac{864 \times 14}{19}$ ($B_{kor} = 2$ weeks $= 14$ days)

$\therefore D = 636.63$ ha/cumecs

$\therefore$ Discharge required, $Q = \frac{\text{Area}}{\text{duty}}$

$\Rightarrow Q = \frac{2250}{636.63} = 3.53\ m^3/sec$

Rabi season

Area irrigated, $A = 6000$ ha

Duty, $D = \frac{864 \times B_{kor}}{\Delta_{kor}} = \frac{864 \times 28}{13.5}$

or $D = 1792$ ha/cumecs

$\therefore$ Required discharge in Rabi season,

$Q = \frac{\text{Area}}{\text{duty}} = \frac{6000}{1792} = 3.35\ m^3/sec$

$\therefore$ design discharge $=$ maximum $= 3.53\ m^3/sec$

turbulence.

* Critical velocity, $V_c = 0.55 * m * y^{0.64}$ ..... i)

  here, m = critical velocity ratio

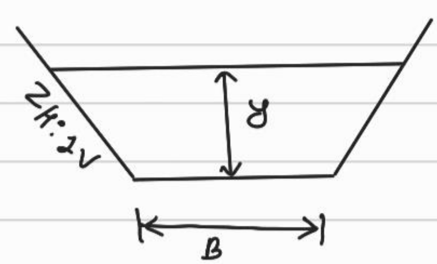
  y = flow depth

  V_c = critical velocity

Q. Design an irrigation canal to carry $50\ m^3/sec$ discharge. The channel is to be laid at a slope of $\boxed{1 \text{ in } 4000.}$ The critical velocity ratio for the soil is $\boxed{1.1.}$ Use Kutter's rougosity coefficient as 0.023

Soln: $Q = 50\ m^3/sec$, slope, $s = 1/4000$,
$CVR = m = 1.1$, $n = 0.23$

1. Assume depth, $y = 2m$
2. Critical velocity, $V_c = 0.55 * m * y^{0.64}$
   $= 0.55 * 1.1 * 2^{0.64}$
   $= 0.94\ ms^{-1}$
3. Area, $A = \frac{Q}{V} = \frac{50}{0.94} = 53.1\ m^2$
4. Calculate B



   We know, $A = By + y^2 z$

Assume $z = 0.5$

$\therefore 53.1 = B * 2 + 2^2 * 0.5$

$\Rightarrow B = 25.55\ m$

5. Check for velocity:-

from Kutter's

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

Wetted perimeter, $P = B + 2y\sqrt{1+z^2}$

$= 25.55 + 2 \times 2 \sqrt{1+0.5^2}$

or $P = 30.03\ m$

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

$$\therefore V = \frac{\frac{1}{0.023} + 23 + 0.00155 * 4000}{1 + (23 + 0.00155 * 4000) * \frac{0.023}{\sqrt{1.77}}} * \sqrt{\frac{1.77 \times 1}{4000}}$$

$= 1.016\ ms^{-1}$ → This is actual velocity

Here $V_c = 0.942m$

$V = 1.016$

If $V_c = 0.55\ m\ y^{0.64} < V$ ⇒ then increase y.

If $V_c > V$, then decrease y.

Assume $y = 2.65\ m$. Then

1> Critical velocity, $V_c = 0.55 * m * y^{0.64}$

$= 0.55 * 1.1 * 2.65^{0.64}$

or $V_c = 1.12\ ms^{-1}$

2> Area, $A = \frac{Q}{V} = \frac{50}{1.12} = 44.64\ m^2$

3. Calculate B

$A = By + y^2 z \Rightarrow 44.64 = B*2.65 + 2.65^2*0.5$

(Assume $z = 0.5$)

$\Rightarrow B = 15.52\,m$

4. check for velocity

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

Wetted perimeter, $P = B + 2y\sqrt{1+z^2}$

or $P = 15.52 + 2*2.65*\sqrt{1+0.5^2}$

or $P = 21.44\,m$

$R = A/P = \frac{44.64}{21.44} = 2.08\,m$

∴ Actual velocity,

$$V = \frac{\frac{1}{0.023} + 23 + \frac{0.00155}{1/4000}}{1 + (23 + 0.00155*4000) * \frac{0.023}{\sqrt{2.08}}} * \sqrt{2.08*\frac{1}{4000}}$$

or $V = 1.13\,ms^{-1}$

Here, $V \simeq V_c$. So the designed section is ok.

Adopt $y = 2.65\,m$, $B = 15.2\,m$, $z = 0.5$, $S = \frac{1}{4000}$.

<u>Lacey Theory:</u>

* It is applicable for true regime and final regime channels.

Q. Design a canal section by using Lacey theory. Assume silt factor is 1.1.

Soln: $Q = 50\ m^3/sec$; $f = 1.1$

i) velocity, $v = \left(\frac{Qf^2}{140}\right)^{1/6}$

$\Rightarrow v = \left(\frac{50 * 1.1^2}{140}\right)^{1/6}$

$\Rightarrow v = 0.87\ ms^{-1}$

ii) Area of flow, $A = \frac{Q}{V} = \frac{50}{0.87} = 57.47\ m^2$

iii) Wetted perimeter, $P = 4.75\sqrt{Q}$

$= 4.75 * \sqrt{50}$

or $P = 33.58\ m$

iv) Calculate B and y.

We know; $A = By + y^2 z$ (Assume $z = 0.5$)

$\Rightarrow 57.47 = B*y + 0.5y^2$ ⌒ i)

$P = B + 2y\sqrt{1+z^2}$

or $33.58 = B + 2*y*\sqrt{1+0.5^2}$

$\Rightarrow B = 33.58 - 2.236y$ ..... ii)

substituting value of B in eqn i) from ii);

$57.47 = (33.58 - 2.236y)*y + 0.5y^2$

$\Rightarrow y = 1.901\ m \simeq 1.9\ m$

$B = 33.58 - 2.236 * 1.9$

or, $B = 29.34\ m$

v) Calculate longitudinal slope or bed slope.

$S = \frac{f^{5/3}}{3340 * Q^{1/6}} = \frac{1.1^{5/3}}{3340 * 50^{1/6}}$

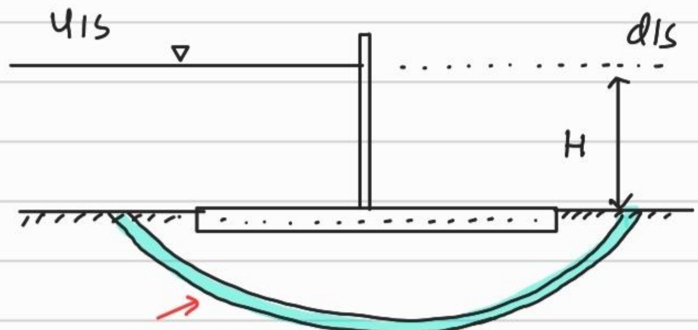
or $S = 1.83*10^{-4} = \frac{1}{5470}$

Seepage Theories and Related Failure of Hydraulic structures founded on Permeable foundation (or due to seepage)

Hydraulic structure :- Dam, weir, barrage, regulators, cross-drainage structure, fall/drop structure, escape structure.

* When a hydraulic structure is founded on permeable soil, the seepage occurs below the foundation/floor.
* Due to seepage, structure may fail by piping or uplift

a. Piping



Suppose; H = 10 m
Head loss = 6m
residual head = 4m

* When the water emerging at downstream end posseses residual energy, it may lift the soil particles at exit end. The progressive removal of soil particles causes the formation of sub-surface pipe and structure may be subside. This is called failure by piping.

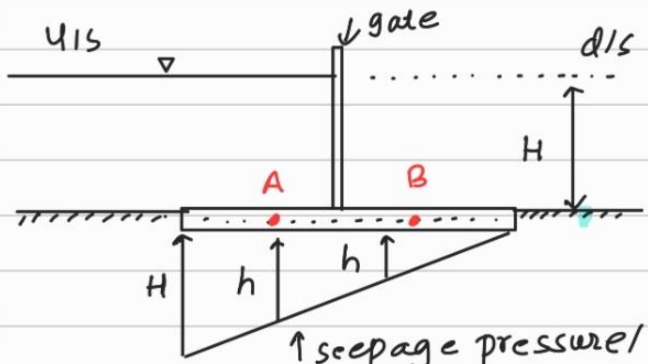
b. Failure by uplift

At point A

$\gamma_w * H + \gamma_c' * t$

t

$\gamma_w * h$



At point B

$\gamma_c' * t$

$\gamma_w * h$

At A, the downward force is more than upward force → No risk of uplift

At B, there may be chances of uplift

* If the seepage pressure/uplift pressure/residual pressure is more than self weight of floor, the uplift pressure may tend to lift the floor and floor may crack. This is called failure by uplift.
* Required thickness at any point for no uplift is calculated by:

Take example of point B.

uplift pressure = self wt.

$\Rightarrow \gamma_w * h = \gamma_c' * t$ { h = residual head,

t = thickness of floor

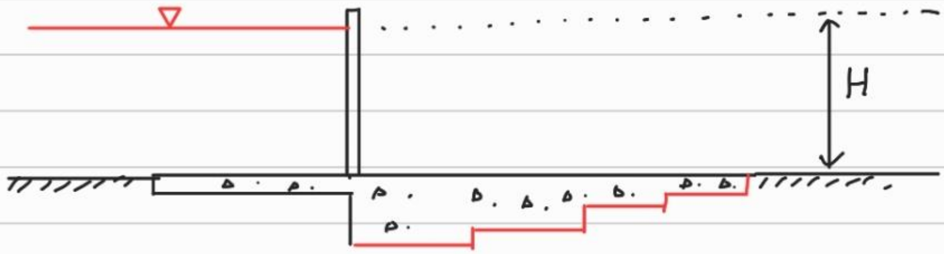
γ_c = unit wt. of floor material

γ_c' = submerged unit wt. of floor material $= \gamma_c - \gamma_w$

$\therefore \gamma_w * h = (\gamma_c - \gamma_w) * t$

$$\Rightarrow t = \frac{\gamma_w}{\gamma_c - \gamma_w} * h = \frac{h}{\frac{\gamma_c}{\gamma_w} - \frac{\gamma_w}{\gamma_w}}$$

$\Rightarrow \boxed{t = \frac{h}{G-1}}$; $G = \frac{\gamma_c}{\gamma_w}$ = specific gravity of floor

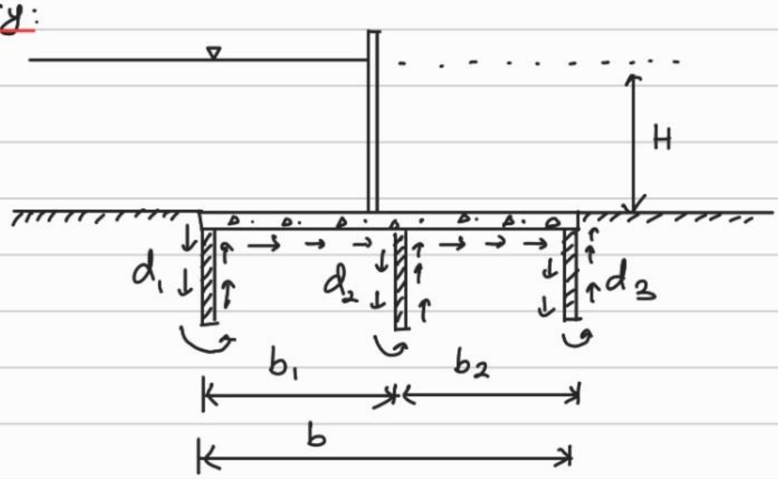


∴ for safety against uplift, the required thickness is $t = \frac{h}{G-1}$ at that point. { h = residual head at that point }

Seepage Theories:

a. Bligh's Theory:

* According to Bligh's, the seepage water follows outlines of the structure.



* Creep length,

$C_L = 2d_1 + b_1 + 2d_2 + b_2 + 2d_3$

Residual head =

$\therefore\ C_L = 2(d_1 + d_2 + d_3) + (b_1 + b_2)$

$\Rightarrow \boxed{C_L = 2(d_1 + d_2 + d_3) + b}$

* Average Hydraulic gradient (i) $= \dfrac{\text{Head loss}}{\text{Creep length}}$

or $i = \dfrac{H}{C_L}$

i) Safety Against piping

$H = 10\text{m} \rightarrow$ required floor length = 60m

$10\text{m} \rightarrow 60\text{m}. \Rightarrow i_c = \frac{10}{60} = \frac{1}{6}$

If the Available floor length = 50 m

or actual floor length = 50m

Actual Hydraulic gradient; $i = \frac{10}{50} = \frac{1}{5}$

since; $i > i_c \Rightarrow$ unsafe in piping.

For safety in piping; the actual hydraulic gradient should be less than or equal to critical hydraulic gradient.

i = average/actual hydraulic gradient $\leq i_c$

i_c = critical hydraulic gradient.

$$\Rightarrow \frac{H}{C_L} \leq i_c \Rightarrow C_L \geq \frac{1}{i_c} * H \Rightarrow C_L \geq C * H;$$

Where; $C = \frac{1}{i_c}$ = Bligh's constant

$\therefore$ For no piping; minimum creep length $= C * H$.

$i_c = \frac{1}{10}$ means, $C = 10$

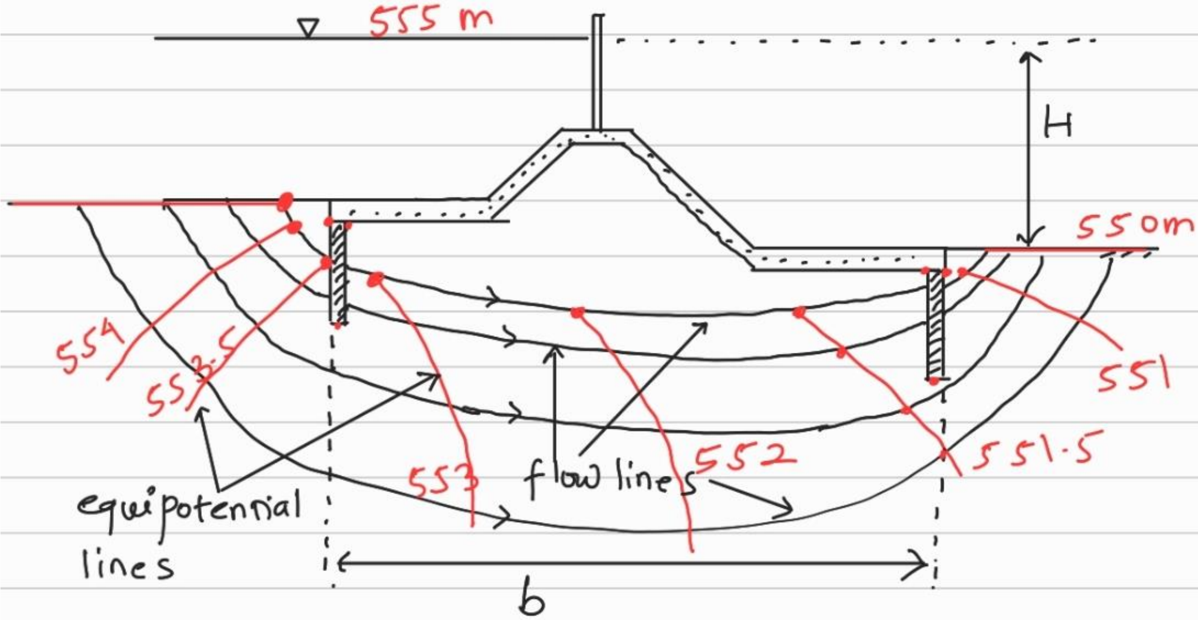
ii) Safety Against Uplift

* He has used factor of safety of 1.33 for floor thickness.
* Required minimum floor thickness $= 1.33 * \frac{h}{G-1}$

factor of safety

h = residual head; G = specific gravity of floor.

c. Khosla Theory:



i) The seepage water doesnot follow outlines of structures as suggested by Bligh's. The seepage path is obtained by solving second order laplacian equation guided by Darcy low.

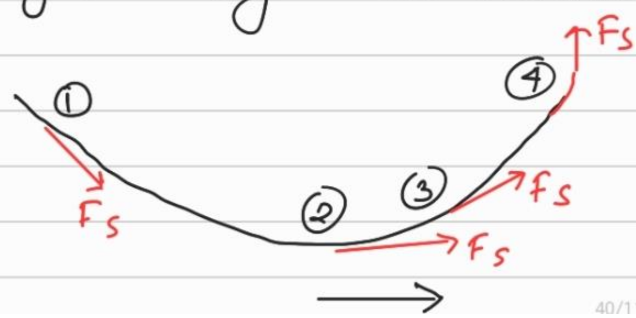
$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0 \quad \text{------ i)}$$

h = residual head at any point.

Eqn i) Gives a set of lines called streamlines or flow lines and equipotential lines.

ii) The residual head at any point is estimated by solving laplacian equation. The residual head calculated by assuming linear variation is not acceptable.

iii) The seepage force, Fs acts act every point in the flow path. The



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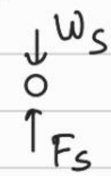
direction of F_s is tangential to the flow path and it is completely vertical at exit end.

* The seepage force per unit volume

$F_s = \gamma_w * \frac{dh}{dl}$; $\frac{dh}{dl}$ = critical hydraulic gradient

The submerged unit wt. of particles,

$W_s = \{G * \gamma_w - \gamma_w\} * (1-\eta)$



η = porosity = % of voids.

G = specific gravity of soil particles.

$G * \gamma_w$ = specific wt. of solid.

$(1-\eta)$ factor is due to porosity

or $W_s = (G-1) * (1-\eta) * \gamma_w$

At equilibrium; $F_s = W_s$

$\Rightarrow \gamma_w * \frac{dh}{dl} = (G-1) * (1-\eta) * \gamma_w$

$\Rightarrow \frac{dh}{dl} = (2.65-1) * (1-0.4)$ $\{\eta = 40\%\}$

$\Rightarrow \frac{dh}{dl} = 1$

He assumed, porosity, $\eta = 0.4$.
Also; specific Gravity, $G = 2.65$

However, he has used factor of safety of 4 to 5.

$\therefore \frac{dh}{dl} = \frac{1}{4}$ to $\frac{1}{5}$ ← Critical hydraulic gradient

iv) Since; piping starts from d/s end, the whole structure will be safe in piping if d/s pile is safe in piping.

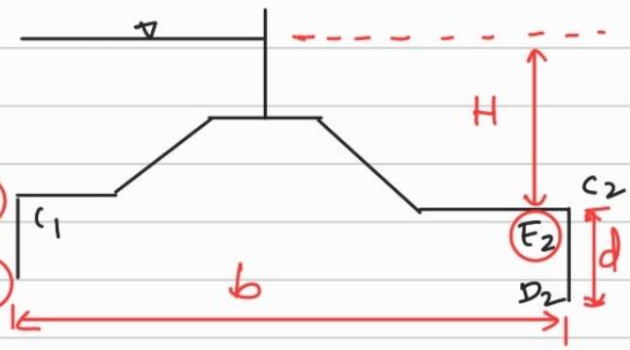
i.e; for safety in piping

$G_E \le G_C$

where; G_C = Critical exit gradient

G_E = Actual exit gradient

or $G_E = \frac{H}{d} * \frac{1}{\pi\sqrt{\lambda}}$

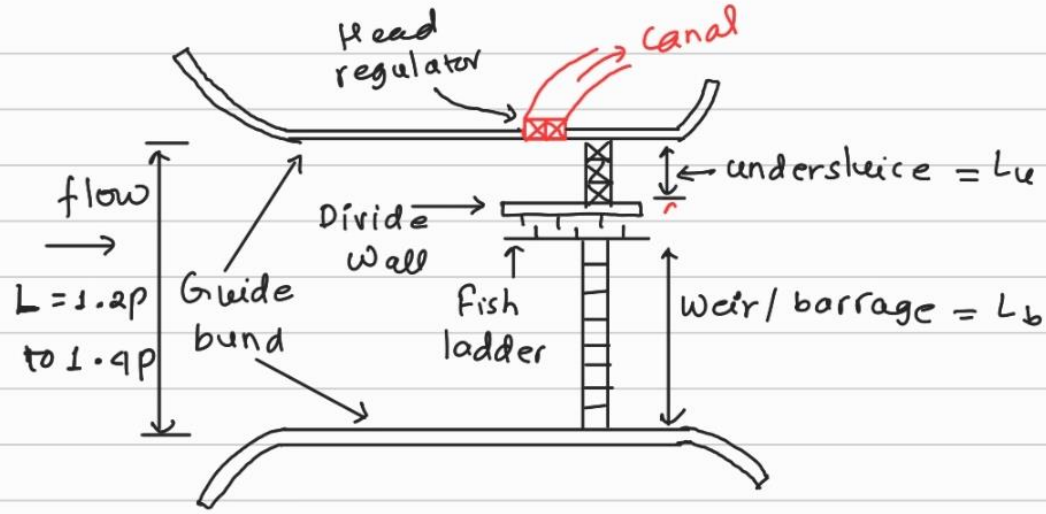


H = maximum head difference between upsream and downsream

d = depth of d/s cutoff measured from top of floor

$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$; $\alpha = \frac{b}{d}$

Design of Barrage Based on Modern Theories



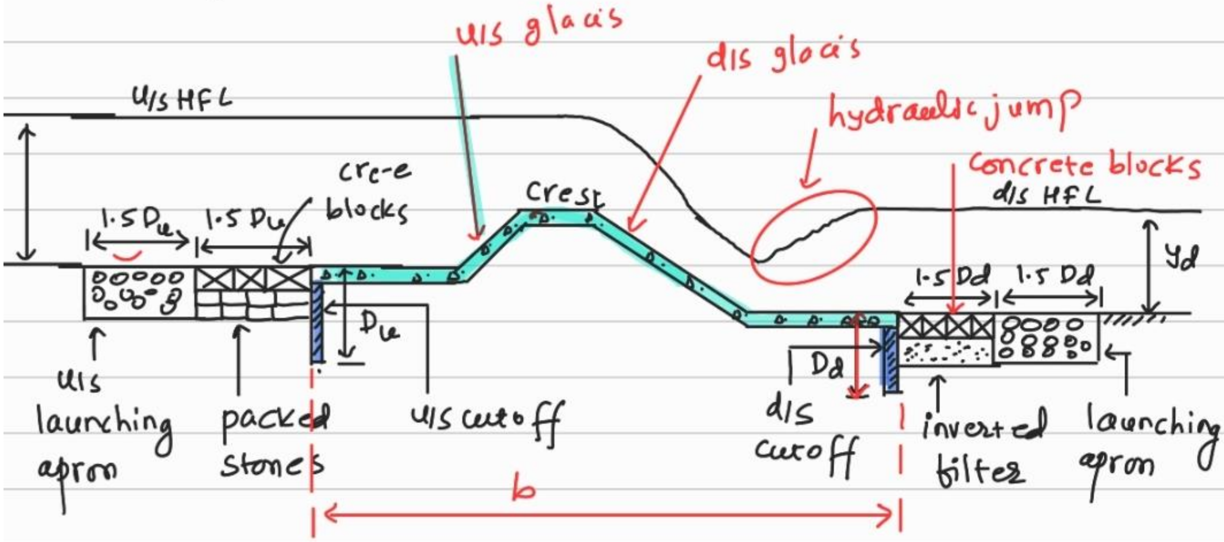
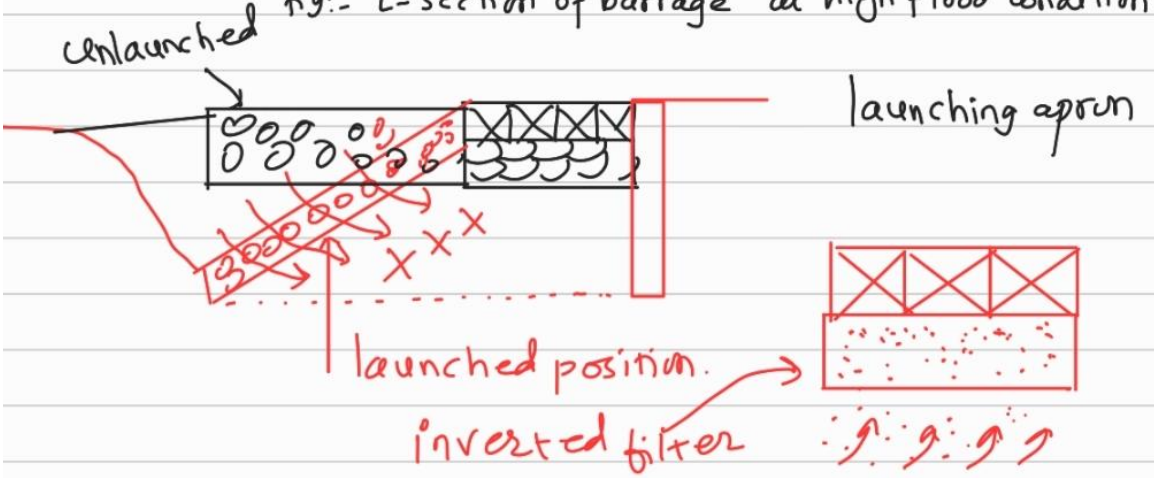


fig:- L-section of barrage at high flood condition



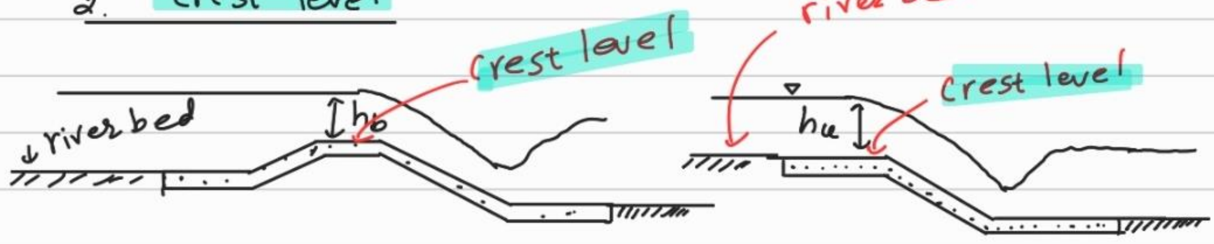
1. Waterway:

* Lacey's waterway = $P = 4.75\sqrt{Q}$
* Increase lacey's waterway by 20 to 40%.

∴ Actual waterway = $1.2P$ to $1.4P$

However, it is subjected to change according to the discharge capacity of structure.

2. Crest level



L-Section of barrage

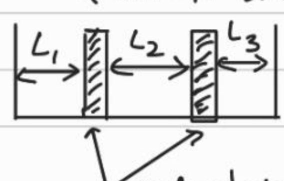
L-section of undersluice

* Crest level of undersluice = river bed level
* Crest level of barrage portion = river bed level + 1 to 1.5 m

3. check for flood discharge (design discharge)

* Design flood discharge should be removed by barrage and undersluice portion.
* $Q = Q_u + Q_b$; generally Q_u = 10 – 15% of design flood.

$$Q_u = C_d * (L - 0.1 * n * h_u) * h_u^{3/2} \qquad \{L = L_1 + L_2 + L_3\}$$



L = clear-waterway of undersluice portion

n = no of bottom corners
= no of gates * 2 (e.g; 6 in given figure)

h_u = Head over undersluice crest.

C_d = coefficient of discharge or weir coefficient
= 1.7 for undersluice portion.

$Q_b = C_d \times (L - 0.1 \times n \times h_b) \times h_b^{3/2}$

C_d = discharge coeff or weir coefficient = 1.84

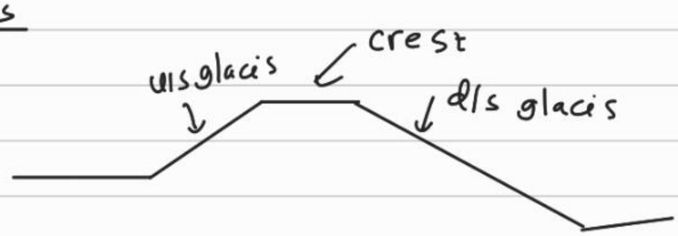
L = clear waterway of barrage

h_b = Head over barrage crest

n = no. of bottom corners.

4. Design of glacis

* Slope of u/s glacis
  = 1H:1V to 3H:1V
* Slope of d/s glacis
  = 3H:1V to 5H:1V
* crest width = 2-3 m



5. Design of floor

a. length

i> Khosla Theory

* $G_E = \frac{H}{d} * \frac{1}{\pi\sqrt{\lambda}}$

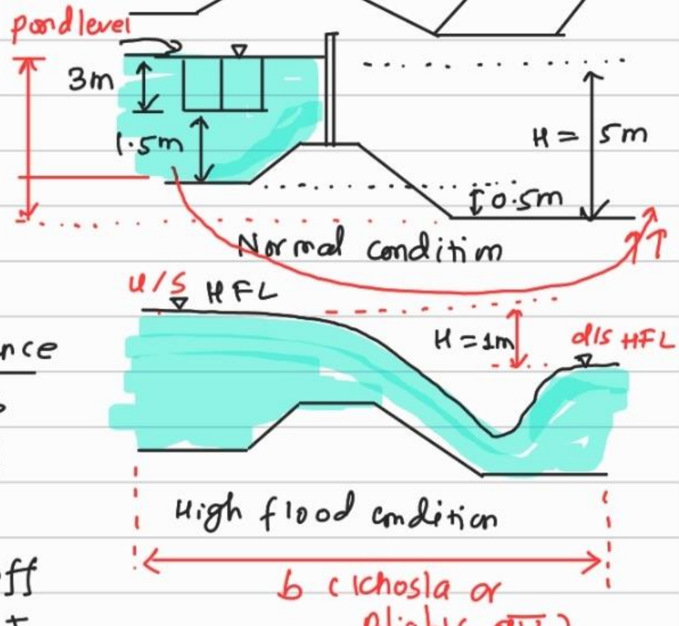
H = maximum head difference between u/s and d/s
= pond level − d/s bed level

d = depth of d/s cutoff

G_E = exit gradient

from this; we get λ.

Also; $\lambda = \frac{1 + \sqrt{1+\alpha^2}}{2}$; from this we get α.



But; $\alpha = b/d \Rightarrow \boxed{b = \alpha * d}$ → b = total floor length

ii> Bligh's Theory: Required creep length, $b = C * H$

C = Bligh's constant

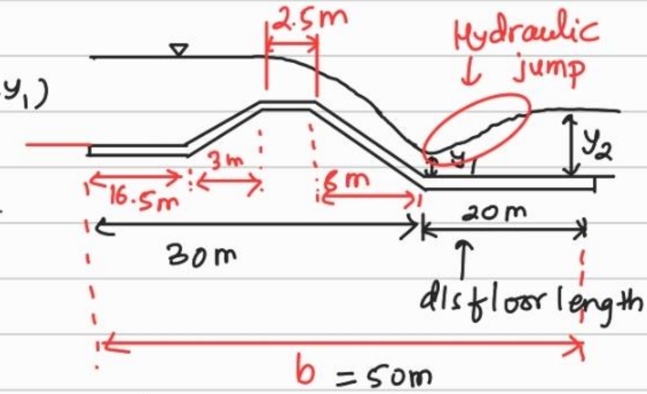
H = maximum head difference = pond level − d/s bed level

b. length of d/s floor

length of d/s floor $= 5(y_2 - y_1)$

y_2 = depth after jump

y_1 = depth before jump.



c. floor thickness:

required floor thickness;

$t = \dfrac{h}{G-1}$; h = residual head at any point

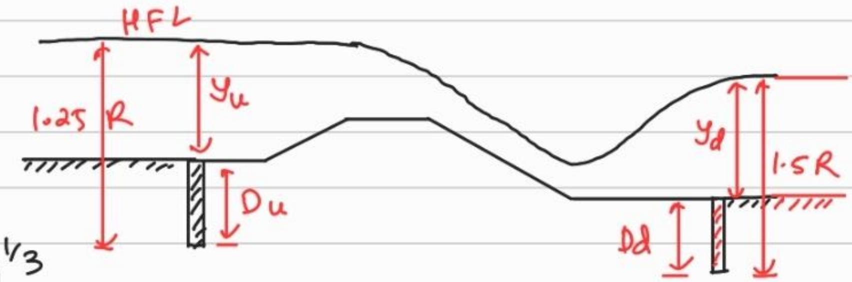
6. Depth of Cutoff

Depth of cutoff on u/s = Du

$= 1.25R - y_u$

R = lacey's scour depth

$= 1.35 \times \left(\dfrac{q^2}{f}\right)^{1/3}$; q = specific discharge = discharge per unit width

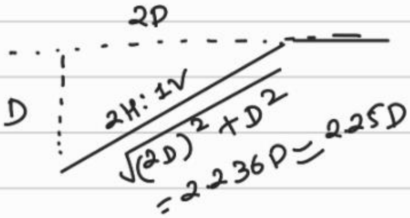


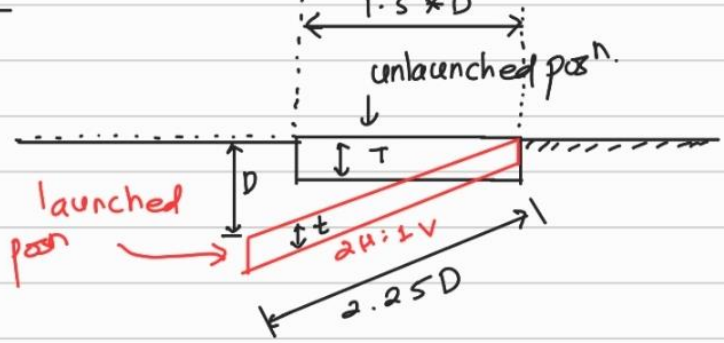
y_u = depth of water on u/s during flood.

* Depth of d/s cutoff = Dd = $1.5R - y_d$

y_d = depth of water on d/s during flood.

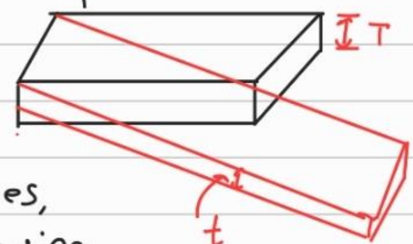
7. Protection works:





fractional Increase in length of apron due to launching $= \frac{2.25D}{1.5D}$

$= 1.5$



Since; length is increasing by 1.5 times, thickness is decreasing by same proportion

a. Protection Works on u/s

i) launching apron

* length = 1.5 * Du ← Du = depth of u/s cutoff
* Thickness = T = 1.5 * t

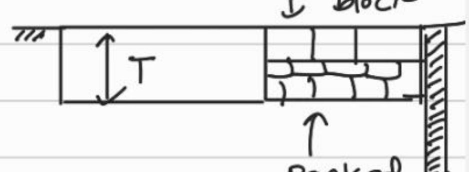
t = thickness of apron in launched position

T = " " " " unlanched "

t = 2 * diameter of stones used.

ii) concrete blocks:

* Total length = 1.5 * Du



iii) Packed stone

* Total length = 1.5 * Du

b. Protection works on d/s

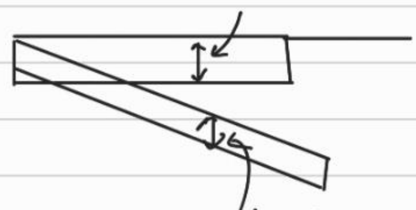
a. Launching Apron

* length = 1.5 * Dd → Dd = depth of d/s cutoff
* Thickness = T = 1.5 * t

t = thickness in launched posn = 2 * dia of stones used

b. Concrete blocks:

* Total length = 1.5 * Dd



c. Inverted filter

* Total length = 1.5 * Dd

Different Types of Energy Dissipators:

Energy Dissipator: Energy dissipator is a hydraulic structure provided at the toe of spillway/barrage/weir/ regulators/ drop structures to dissipate the energy of water flowing over that structure. It may dissipate the excess kinetic energy of water by either hydraulic jump formation or by the impact (ठोकिएर) or by roller action. The energy dissipators are required due to following reasons.

a. When water flows over spillway and reaches it's toe, the potential energy of water is converted into kinetic energy at the toe of spillway. Due to huge kinetic energy, the water starts to flow at very high velocity which has huge erosion potential.
b. The energy of the water shall be dissipated by providing suitable structure, otherwise, erosion of bed and sides will occur in downstream part of channel.
c. If energy dissipator is not provided, due to scouring of channel bed, there may be risk of failure of dam itself.

Types of Energy Dissipators

Following are the generally used type energy dissipators

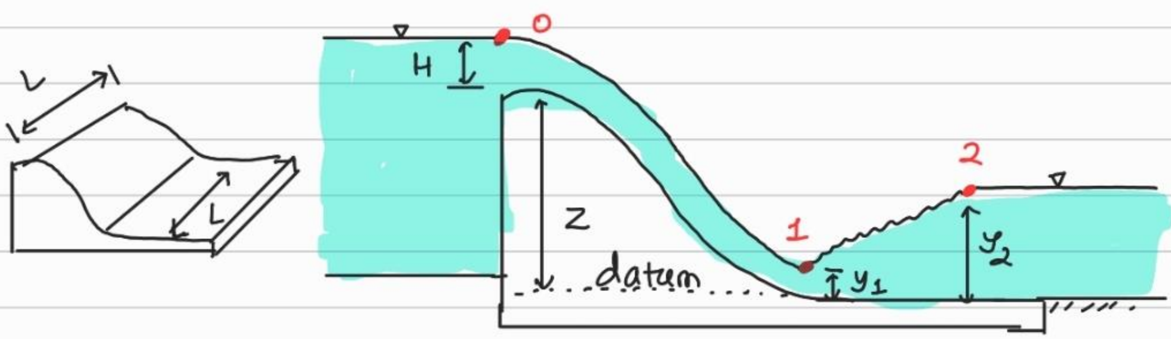
1. Hydraulic jump type energy dissipators or stilling basins

These types of energy dissipators are provided with horizontal of slopping floor at the toe of the spillway. The energy dissipation is due to formation of hydraulic jump. The floor is kept either horizontal or in mild slope so that hydraulic jump could be formed in the basin floor. As the water reaches the toe of spillway or starting of stilling basin floor, due to huge velocity, the flow condition at the start of basin (i. e; at point 1) becomes super critical.
So, a hydraulic jump will be formed and flow changes into sub – critical state at point 2.

Design of Stilling Basin

Design of simple stilling basin with horizontal floor without any additional structures is presented here. Some special structures and special provisions are made in USBR stilling basins.

1. Calculate discharge or head over soillway crest (refer the formula and hand written note below)



$Q = \boxed{\frac{2}{3} * C_d\sqrt{2g}}\, L\, H^{3/2} \rightarrow C_d < 1$

$= C_d * L * H^{3/2} \rightarrow C_d$ = weir coeff = coeff. of discharge $(C_d > 1)$

$= 1.7 * L * H^{3/2}$ → If C_d is not given

L = length of crest

H = Head over crest

2. Applying Bernoulli's eqn between 0 & 1

$$z_0 + \frac{p_0}{\gamma} + \frac{V_0^2}{2g} = z_1 + \frac{p_1}{\gamma} + \frac{V_1^2}{2g} + h_f \quad \ldots\ i)$$

Assuming d/s floor as datum; $z_0 = Z + H$

Z = Height of crest above d/s floor.

$p_0 = 0$; $V_0 = 0$; $z_1 = y_1$; $p_1 = 0$; h_f = head loss between 0 to 1.

Eqn i) becomes;

$$(Z+H) + 0 + 0 = y_1 + 0 + \frac{V_1^2}{2g} + 0$$

Neglecting y_1; $Z + H = \frac{V_1^2}{2g} \Rightarrow V_1 = \sqrt{2g(Z+H)}$

$y_1 = 0.8\,m$; $1\,0m$

③ Calculate y_1 $\qquad Q = A_1 * V_1$

$\Rightarrow Q = L * y_1 * V_1$

$$\Rightarrow \boxed{y_1 = \frac{Q}{L * V_1}} = \frac{q}{V_1}$$

$$y_1 = \frac{q}{V_1} \Rightarrow \frac{q}{y_1} = V_1 = \sqrt{2g(z+H)}$$

④ Calculate y_2 from equation of hydraulic jump.

$$y_2 = \frac{y_1}{2} * \left[-1 + \sqrt{1+8Fr_1^2}\right]$$

$$\text{or } y_1 = \frac{y_2}{2}\left[-1 + \sqrt{1+8Fr_2^2}\right]$$

y_2 = depth after jump formation → post-jump depth

$Fr_1 = \frac{V_1}{\sqrt{gy_1}}$ = froude no. at 1

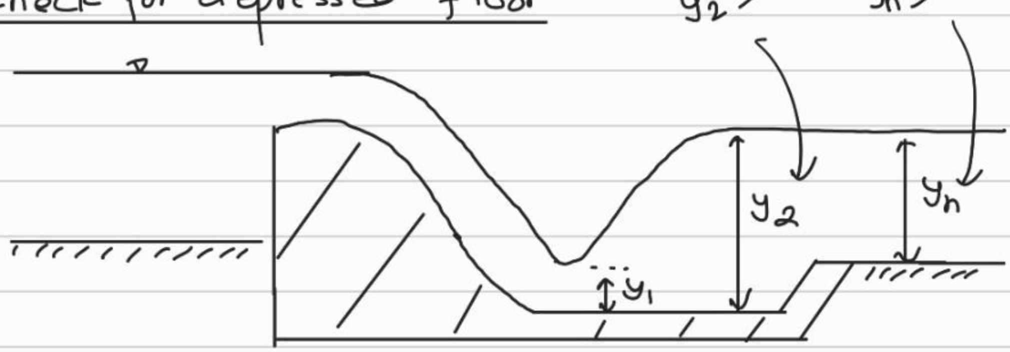
$Fr_2 = \frac{V_2}{\sqrt{gy_2}}$ = froude no. at 2

⑤ length of Basin

length of basin $= 5(y_2 - y_1)$ if $Fr_1 < 4.5$

" " " $= 6y_2$ if $Fr_1 > 4.5$

⑥ Check for depressed floor $y_2 = 8m$ $y_n = 6m$



let; y_n = normal water water depth in d/s channel.

if $y_2 > y_n$ ⇒ provide depressed floor a shown.

Calculation of y_n:

$Q = \frac{A}{n} R^{2/3} S^{1/2}$ for channel

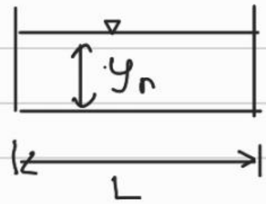
S = Channel bed slope ; R = Hydraulic mean depth

n = manning's rugosity coeff.

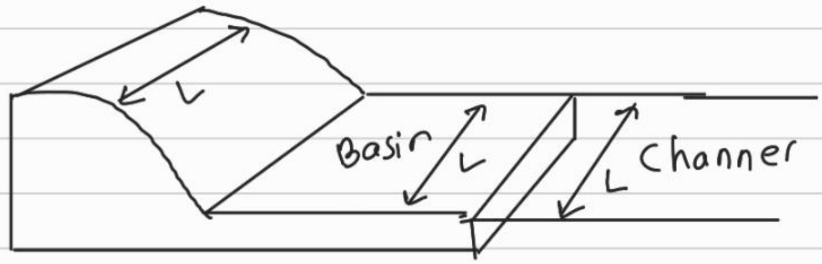
50/110

$A = L * y_n \; ; \; P = L + 2y_n$

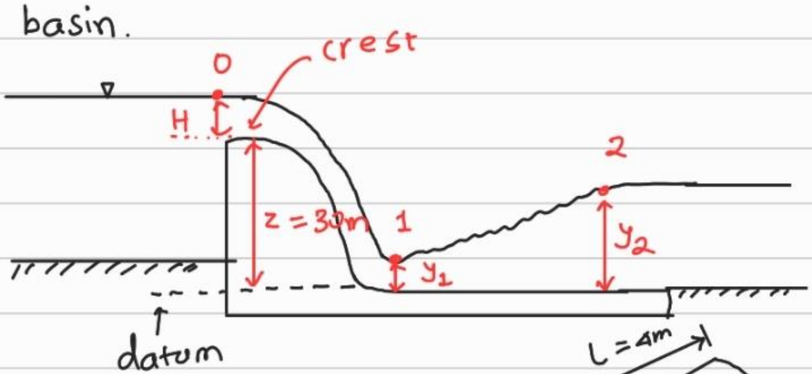
$$\therefore \quad Q = \frac{L * y_n}{n} * \left[\frac{L * y_n}{L + 2y_n}\right]^{2/3} * S^{1/2}$$

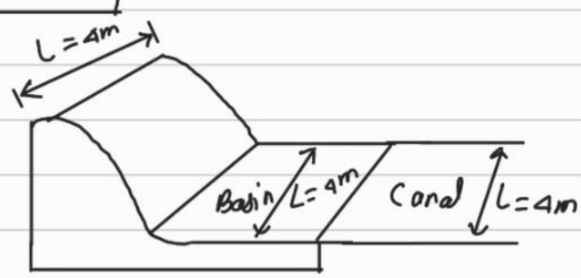


from this; get y_n.



Q. Design a stilling basin using following data, if the crest above downstream bed is 30 m. Design discharge is 80 m3/sec, width of Canal is 4m, d/s bed slope is 1/500, roughness Coefficient is 0.016 and Cd = 0.7. Recommend suitable stilling basin.





Given:

Width of Canal, L = 4m,

Discharge, Q = 80 m3/sec.

Cd = 0.7

Slope of Canal, S = 1/500

n = 0.016

Height of crest above downstream bed, z = 30m

51/110

1. Calculate head over crest

$Q = \frac{2}{3} C_d \sqrt{2g} \, L \, H^{3/2}$ ($C_d < 1$; use this formula)

L = length of crest = width of canal = 4m

$\therefore \; 80 = \frac{2}{3} * 0.7 * \sqrt{2*9.81} * 4 * H^{3/2}$

$\Rightarrow H$ = Head over crest = 4.54m

2. Calculate V_1

Applying Bernoulli's eqn between 0 and 1,

$$z_0 + \frac{P_0}{\gamma} + \frac{V_0^2}{2g} = z_1 + \frac{P_1}{\gamma} + \frac{V_1^2}{2g}$$

Taking d/s bed level as datum;

$z_0 = z + H = 30 + 4.54 = 34.54$ m

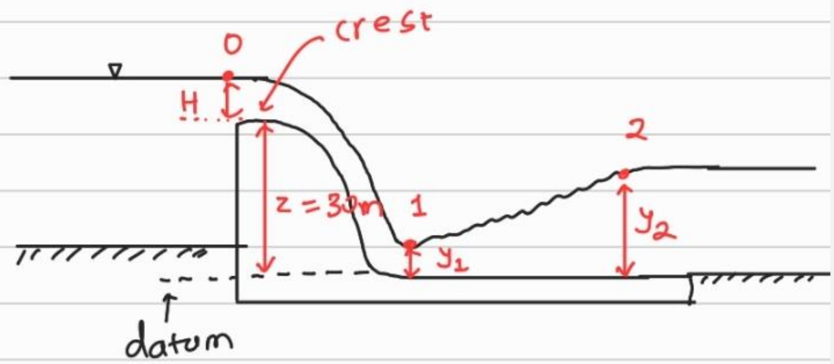
$P_0 = 0$; $V_0 = 0$; $P_1 = 0$; $z_1 = y_1$

$\therefore \; 34.54 + 0 + 0$

$= y_1 + 0 + \frac{V_1^2}{2g}$

Neglecting y_1;

$V_1 = \sqrt{2g * 34.54}$



$\Rightarrow V_1 = \sqrt{2*9.81*34.54} = 26.03 \text{ ms}^{-1}$

(3) Calculate y_1

Now; $Q = A_1 * V_1 = L * y_1 * V_1$

$\Rightarrow y_1 = \frac{Q}{L * V_1} = \frac{80}{4 * 26.03} = 0.76$m

(4) Calculate y_2 from jump eqn.

$$y_2 = \frac{y_1}{2}\left[-1 + \sqrt{1 + 8 Fr_1^2}\right]$$

$$Fr_1 = \frac{V_1}{\sqrt{g y_1}} = \frac{26.03}{\sqrt{9.81 * 0.76}} = 9.53$$

$\therefore y_2 = \frac{0.76}{2}\left[-1 + \sqrt{1+8*9.53^2}\right]$

$= 9.86m$

(5) length of Basin

For $Fr_1 < 4.5 \Rightarrow$ Length of basin $= 5(y_2-y_1)$

For $Fr_1 > 4.5 \Rightarrow$ length of basin $= 6y_2$

Since; $Fr_1 > 4.5$; length of basin $= 6y_2$

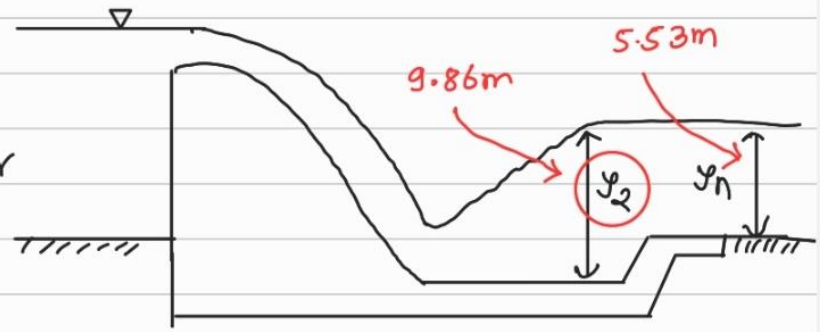
$= 6*9.86 = 59.16m \simeq 59.2m$

(6) Check for depressed floor.

let; y_n = normal depth.

Then;

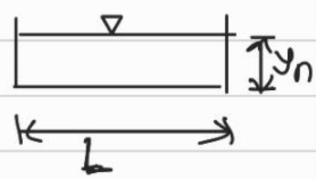
$Q = \frac{A}{n}R^{2/3}S^{1/2}$ for channel.



$A = L*y_n = 4*y_n$

$P = L+2y_n = 4+2y_n$

$n=0.016$; $S = 1/500$; $Q = 80\ m^3/sec$



$$80 = \frac{4*y_n}{0.016} * \left(\frac{4*y_n}{4+2y_n}\right)^{2/3} * \left(\frac{1}{500}\right)^{1/2}$$

$\Rightarrow y_n = 5.53m$

Since; $y_2 > y_n \Rightarrow$ provide depressed floor.

7. Suitable stilling Basin

For $Fr_1 > 4.5$; use USBR type II stilling basin.

Refer to figure below.

* In this example, use USBR type II stilling

basin.

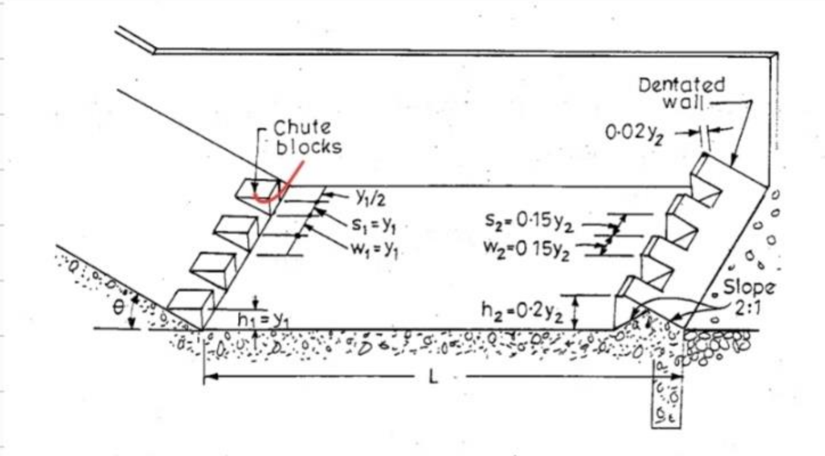


Fig. 21.32. U.S.B.R. Stilling basin II ($F_1 > 4.5$).

* For $Fr_1 = 2.5$ to 4.5; use USBR type IV stilling basin. Refer to attached figure.

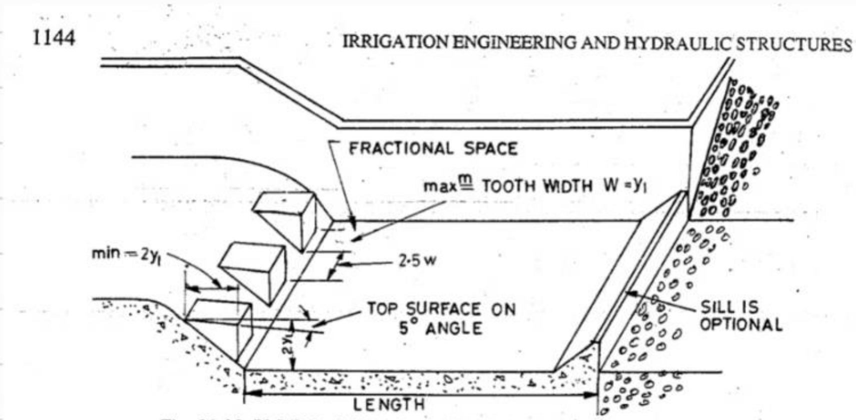
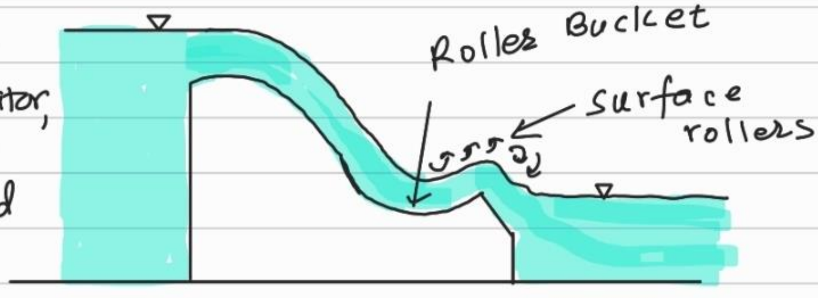


Fig. 21.33. U.S.B.R. Stilling basin IV (F_1 lies between 2.5 and 4.5).

2. Roller Bucket Type Energy Dissipator:-

* In this type energy dissipator, roller bucket is provided and due to roller action, surface



rollers are formed which dissipate energy.
* This type of energy dissipator is provided when good rock is available at downstream of spillway or when tailwater depth is low due to steeper slope and hydraulic jump can not be formed under normal conditions.

3. Deflector bucket or ski-jump type energy dissipator:

In this type energy dissipator, a jet of water shoots directly in downstream. The nappe of water turns upward as it reaches the bucket. The energy dissipation occurs mainly by impact on downstream whereas a part of energy is also dissipated due to friction in the air.

This type of energy dissipator is provided if there is strong rock at downstream channel or if the tailwater depth is low so that hydraulic jump can not be formed.

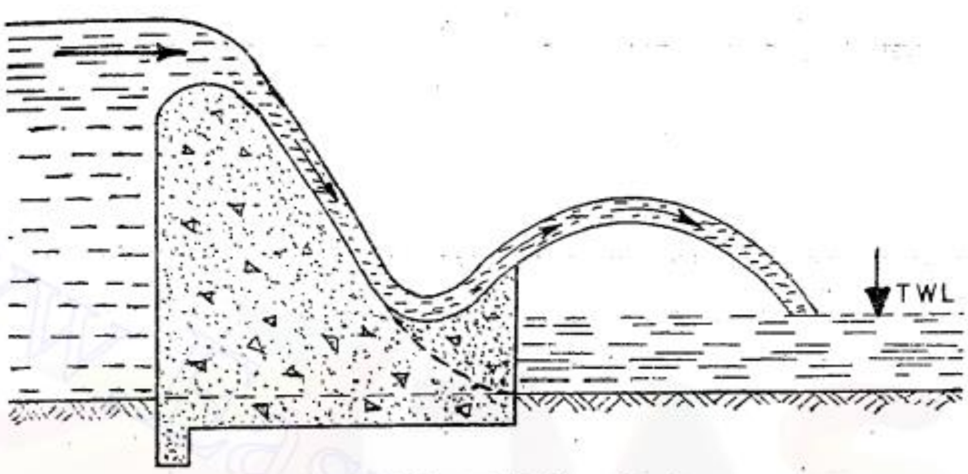


Fig. 21.31. (c_1) Ski jump bucket.

Design of Silt Excluder :-

i) Silt Excluders are designed for 15 to 20% of canal full supply discharge.

ii) flow velocity = 2 to 4.5 m/sec in order to avoid silt deposition. Generally 2 m/sec for sandy rivers, 4 to 4.5 m/sec for boulder stage rivers and 3 m/sec for ordinary straight reaches.

iii) Area of opening = area of flow = $\frac{Q}{V}$

iv) Height of tunnel, h = Crest level of head regulator − river bed level − thickness of roof slab.

v) The width of tunnels, $B = \frac{A}{h}$

vi) The total tunnel width is divided into number of tunnels with span of 1.8 to 2.4 m for each tunnel separated by dividing piers of 0.6 m.

vii) Generally 4 to 6 tunnels are provided for each excluder.

viii) The length of tunnel is different but head loss through each tunnel should be same. Head loss is given by

$$h_f = \frac{V^2 n^2}{R^{4/3}} * L$$

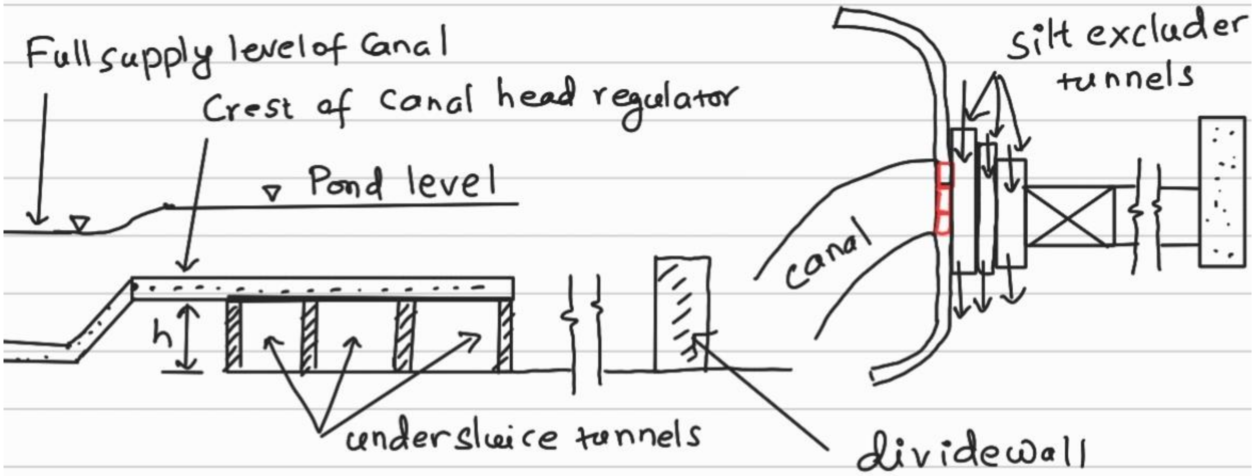
V = flow velocity ; n = manning's roughness coefficient

R = hydraulic mean radius

L = length of excluder tunnel.

ix) The height of tunnels varies from 0.5 to 0.6m for sandy rivers and 0.8 to 1.2m for boulder stage rivers.

x) At entrance, the tunnels are generally given a bell mouth shape and radius of bell mouth opening varies from 2 to 6 times the tunnel width.



Design of silt ejectors :-

i) Silt ejectors are designed for 20 to 25% of canal discharge.

ii) The bed of canal is depressed by 0.3 to 0.5m at the mouth of the ejector so that the approach velocities are reduced and the bed load may be trapped.

iii) The tunnels are low height about 0.5 to 0.6m.

iv) The ideal distance between head regulator and silt ejector is usually between 150m to 500m.

v> The section of tunnels is gradually reduced such that there is overall increase in velocity by 10 to 15 % at the exit end.

vi> The section of tunnel at the exit end is designed to attain velocity of 2.5 to 6 m/sec depending uppon the grade of sediment to be removed.

vii> The portion of the approach channel immediately upstream of the ejector should be pitched for a length of 3-4 times the depth of water in the channel so that there is no erosion of bed and sides of the channel.

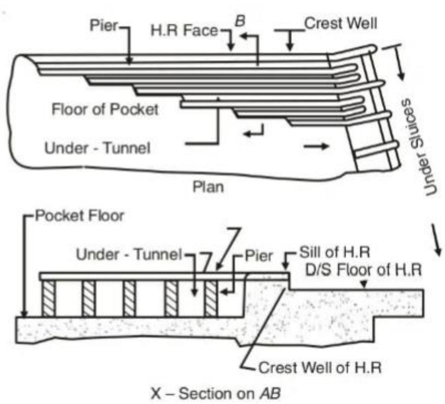


Fig. 15.14. *Silt excluder.*

fig: silt excluder diagram

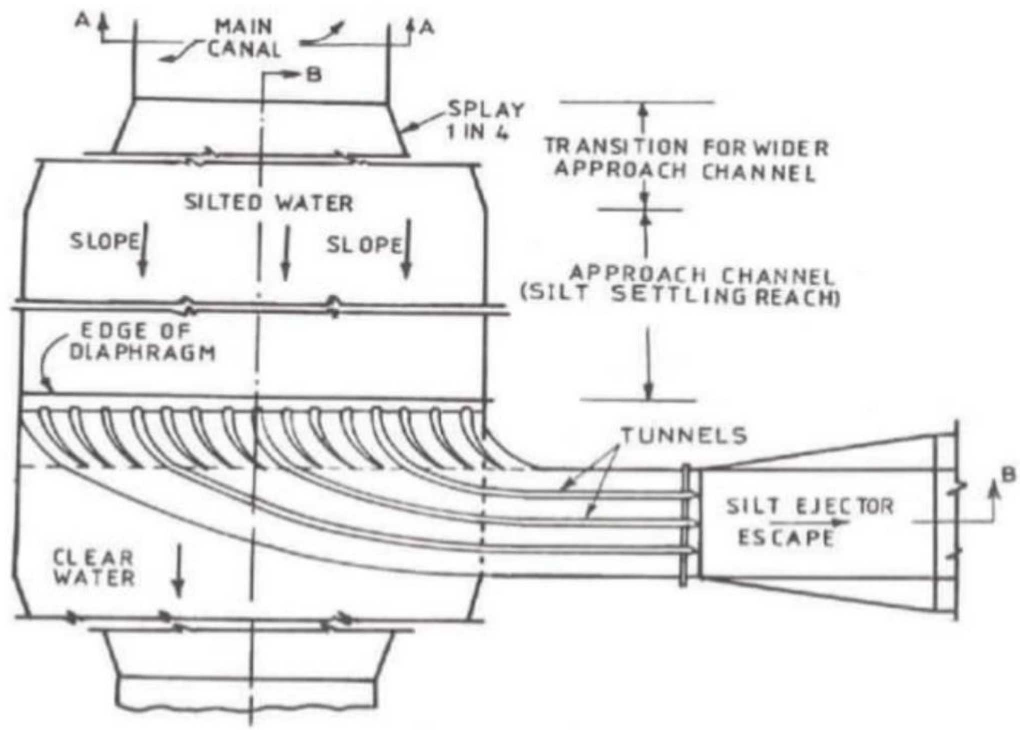


Fig: Plan of Silt Ejector

fig:- silt ejector

Design of settling Basin

Design of Settling Basin

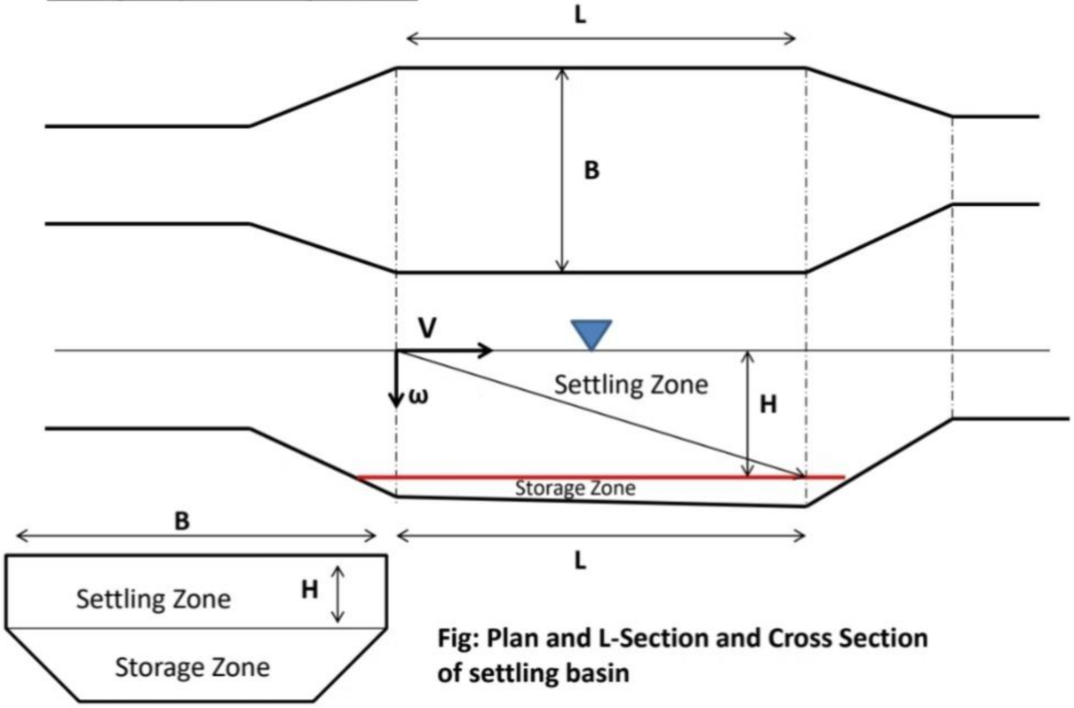


Fig: Plan and L-Section and Cross Section of settling basin

To design a Settling Basin, following steps are adopted.

i. Assume the height of settling basin, h = 3 to 5 m.

ii. Calculate the settling velocity, ω.

iii. Calculate critical horizontal velocity, v_c

$V_c = a\sqrt{d_{mm}}$

Where, V_c = velocity in m/sec,

a = a parameter which depends on size of particles

= 0.36 for d> 1 mm

= 0.44 for d = 0.1 to 1 mm.

= 0.51 for d< 0.1 mm

d_{mm}= diameter of particle in mm.

The critical horizontal velocity is the velocity above which the water will scour deposited sediments. So, the actual flow velocity should be less than critical horizontal velocity, V_c .

iv. Calculate the Surface area of Basin

Surface area, $A_s = k*\frac{Q}{\omega}$

Where, Q= design discharge of the setting basin. Add 10 % extra for flushing. That means if given discharge is design discharge of settling basin, it's ok. If given discharge is the turbine discharge, add 10 % extra to get design discharge of settling basin.

K = coefficient to account for turbulence. Value of k = 1.2 to 1.5

v. Calculate Length and Width of Settling Basin

As we have surface area, we can calculate L and B. Assume L/B = 4 to 10. We can calculate L and B.

If L/B = 8, $A_s = L*B$

Or, $A_s = 8B*B = 8B^2$. So we can calculate B.

Again, L = 8B

vi. Calculate actual velocity of settling basin

The actual velocity = V = Q/A = Q/(B*H)

The velocity of settling basin should be less than critical horizontal velocity, v_c . If the velocity comes to be more than v_c, then increase B and decrease L to reduce v. This can be achieved by reducing L/B ratio.

vii. Check Length of Settling Basin.

a. Calculate the length of settling basin using principle of settling and without considering turbulence.

i. e; Travel time = Settling Time

$$\frac{L}{V} = \frac{H}{\omega}$$

Or, L = H* v/ω

b. Calculate the length of settling basin using principle of settling and considering turbulence.

Due to turbulence, the settling velocity reduces from ω to ω'

$\omega' = \omega - \alpha v$

$= \omega - \frac{0.132}{\sqrt{H}} v$

Now the length of basin can be calculated using the same principle of settling.

Travel Time = Settling Time

$$\frac{L}{V} = \frac{H}{\omega'}$$

Or, L = H* v/ω'

c. Calculate the length of settling basin using Vellikanov's Method

$$L = \frac{\lambda^2 * V^2 (\sqrt{H} - 0.2)^2}{7.51 * \omega^2}$$

For $\eta = 90\%$, $\lambda = 0.9$ and for $\eta = 95\%$, $\lambda = 1.2$

Where, λ is the parameter which depends on the efficiency of settling.

To know the value of λ for given value of efficiency, use following curve.

If the length of settling basin calculated from a, b and c comes to be more than that that calculated in step v, adopt the maximum value. Otherwise, value calculated in iv is ok.

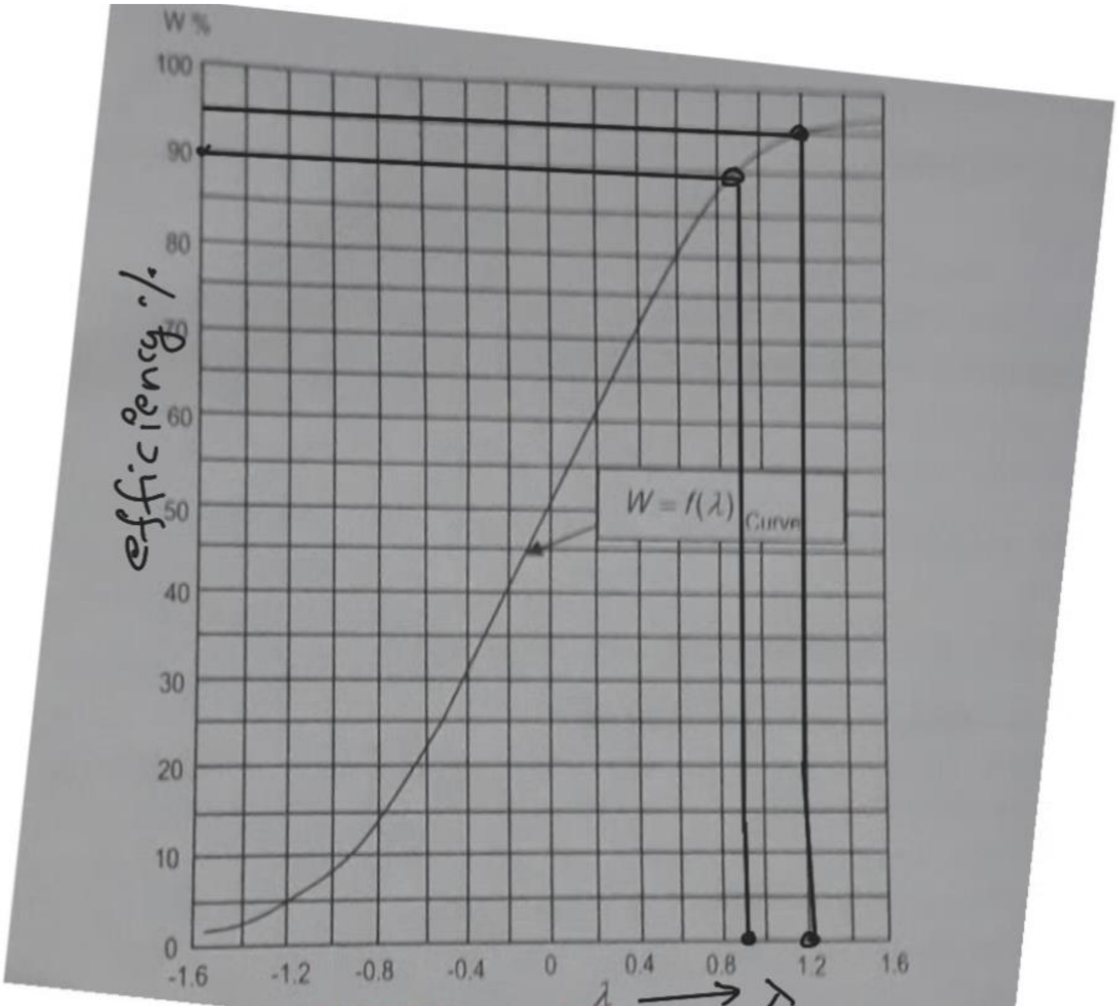


Fig: Vellikanov's Curve

viii. Find the efficiency of Settling Basin

The efficiency of settling basin may be calculated by following formula.

a. Hazen's Equation

$\eta = 1-(1+m*\frac{\omega}{v_0})^{-1/m}$

Where, v_0 = surface overloading rate = $Q/A_s = Q/(L*B)$

Here, m is hazen's constant. Generally m = 0.17 is adopted.

b. Vetter's Equation

Efficiency, $\eta = 1- e^{-\frac{\omega A_S}{Q}}$

Where, A_s = surface area of the basin = L*B

Please use adopted L

c. Camp's Method

This is graphical method and in this method, we plot ω/u* in X-axis and for corresponding value of $\frac{\omega A_S}{Q}$, we find efficiency.

Whereas, u* $=\frac{0.042v}{R^{1/6}}$

Here, R = Hydraulic radius of the basin

= A/P = Cross Sectional area/Wetted perimeter

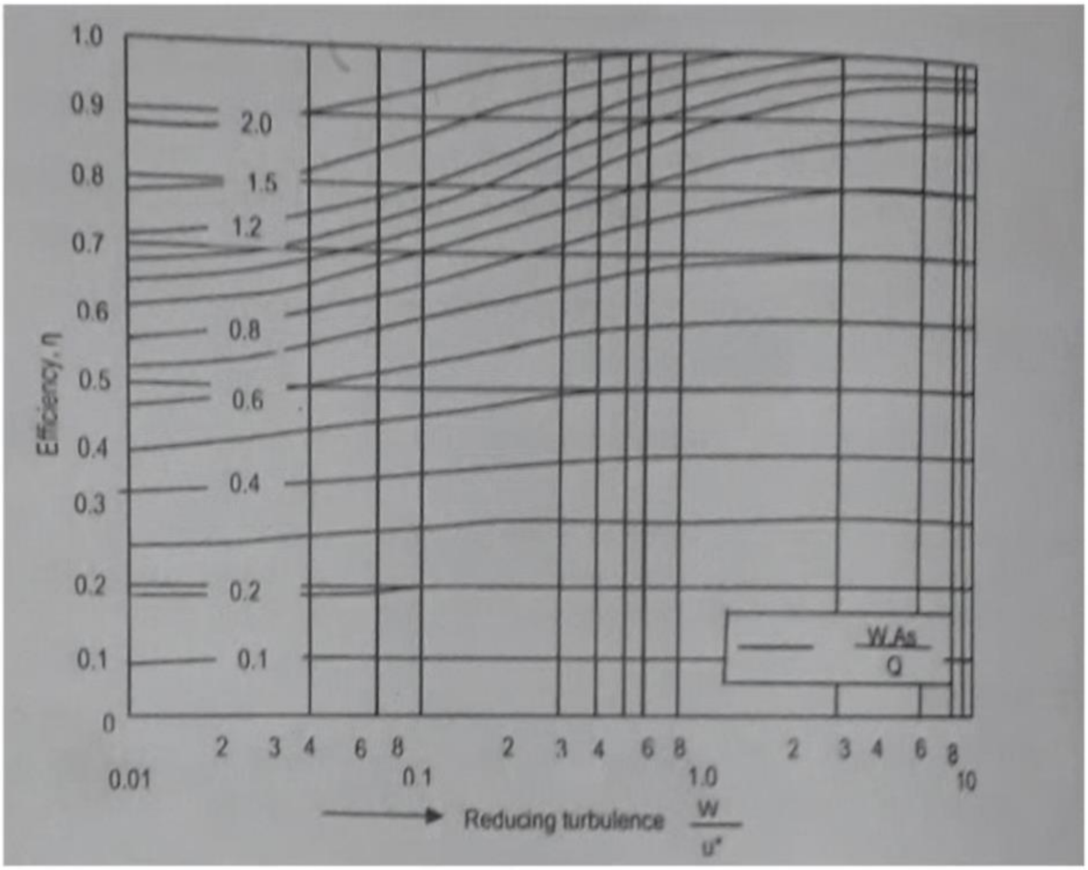


Fig: Camp's Graph

Design of Guide Bund:-

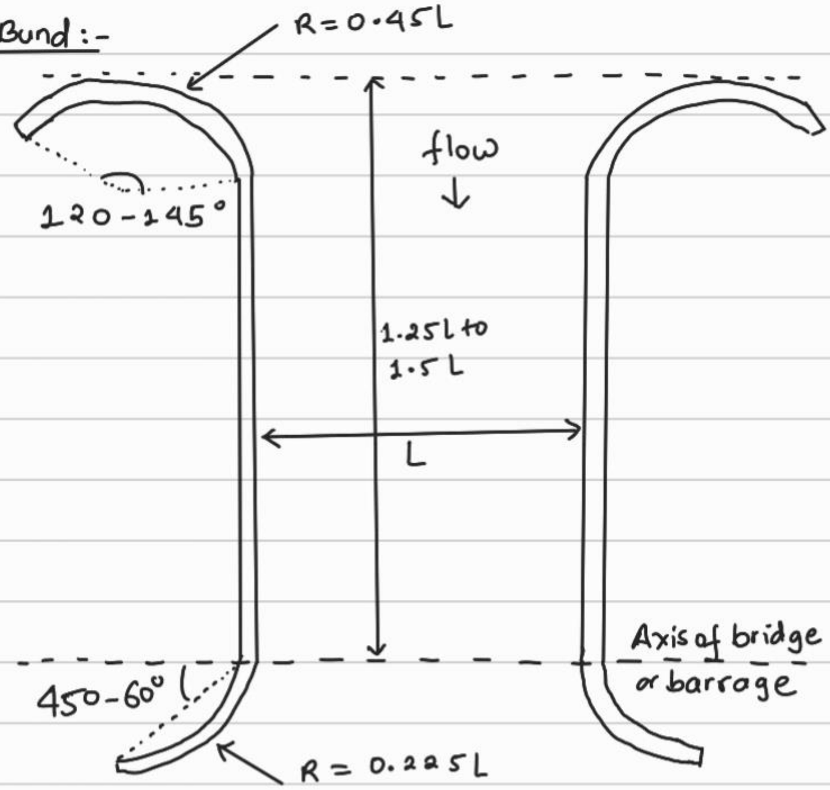


fig. plan of a guide bund

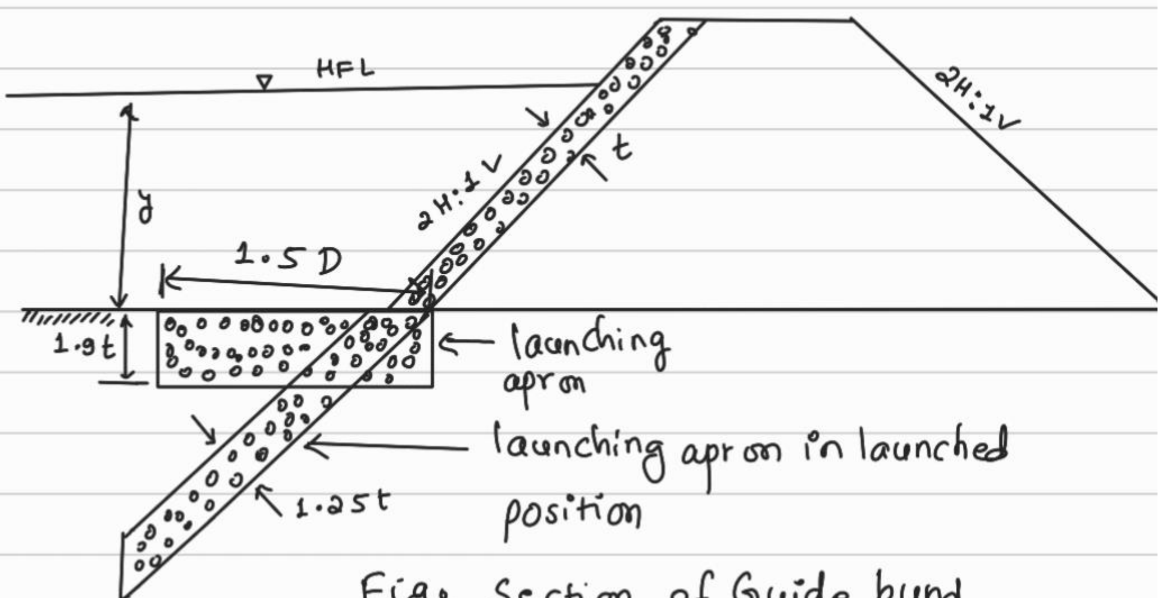


Fig. Section of Guide bund

1. Waterway

* Lacey's waterway, $P = 4.75\sqrt{Q}$
* Add 20% extra for actual waterway for bridges (Generally 10 to 20% extra is added.)

$\therefore L = P + 0.2P = 1.2P$

2. Top level :-

Top level of bank = HFL before construction + $\frac{v^2}{2g}$ + free board

Assume, afflux = 1 m

free board = 1.2 m to 1.5 m

3. length of Guide Bunds :-

Discharge	length of u/s Guide bund
< 20,000 m^3/sec	1.25 L
20,000 – 40,000 m^3/sec	1.25 L – 1.5 L
> 40,000 m^3/sec	1.5 L

4. Radius of curved end

* Radius of u/s curved end = 0.45 L
* Radius of d/s curved end = 0.225 L
* Angle of u/s curved head = 120° – 145°
* Angle of d/s curved head = 45° – 60°

5. Cross-Section of Bund

* Top width of the guide bund should not be less than 4 m.
* Side slope = 2H : 1V

$\therefore$ $H = 4 + 0 + 1 + 1.5 = 6.5m$

* Assume top width = 4m
* Adopt side slope = 2H : 1V

5. Thickness of pitching :-

$t = 0.06 * Q^{1/3} = 1.09m \simeq 1.1m$

6. Design of Launching Apron :-

* The thickness of launching apron is,

$T = 1.9t = 1.9 * 1.1 = 2.09m \simeq 2.1m$

length of launching apron = 1.5D

D = Scour depth below ground level

$= xR - y$

x = factor = 2.25 for nose

x = 1.5 for transition

x = 1.25 for straight portion

R = lacey's scour depth

$= 0.47\left(\frac{Q}{f}\right)^{1/3}$

$f = 1.76\sqrt{d_{mm}} = 1.76 * \sqrt{0.1} = 0.55$

$\therefore R = 0.44 \times \left(\frac{6000}{0.55}\right)^{1/3} = 10.36m$

$\therefore$ At nose, $D = 2.25 * 10.36 - 4$

$= 19.31m$

So, length of apron $= 1.5 * 19.31 = 28.96m$

$\simeq 29m$

At Transition :-

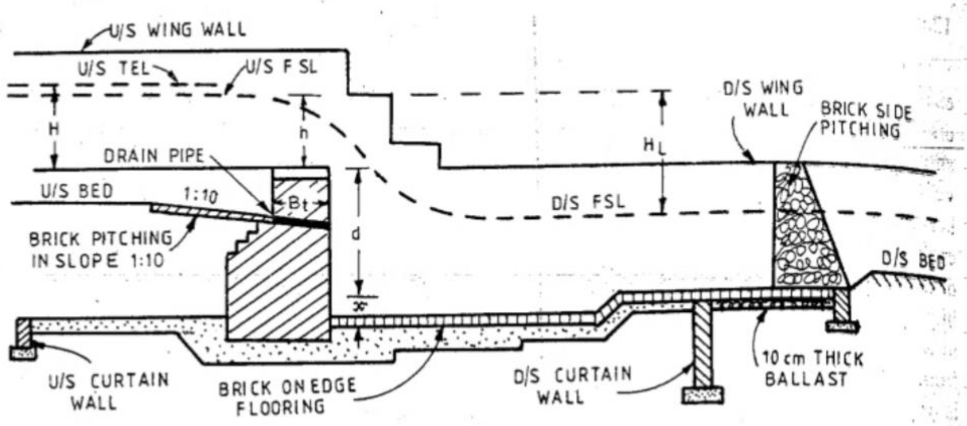
$D = 1.5 * 10.36 - 4 = 11.54m$

$\therefore$ length of apron $= 1.5 * D = 17.31m$

At straight Portion :-

* D = 8.95m (1.25 * 10.36 − 4)
* length of apron = 1.5 * 8.95
= 13.425
= 13.5m

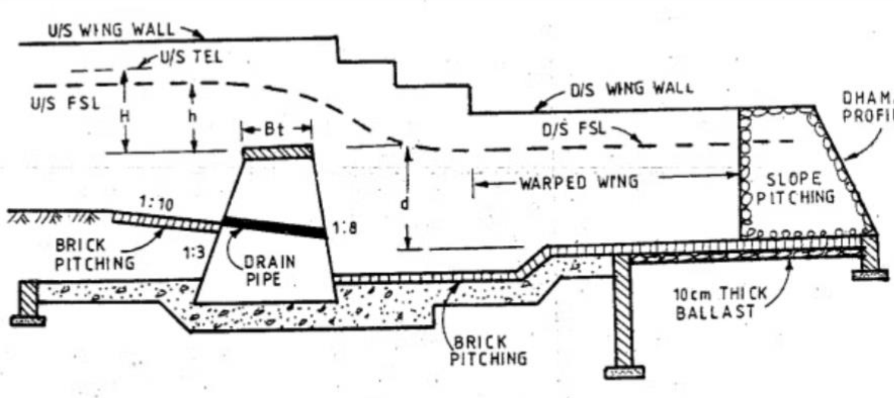
Design of Vertical Fall/Sarda Drop:-



Q = upto a maximum of 14 cumecs
B_t = Top width of crest = $0.55\sqrt{d}$

Base width = $\dfrac{h+d}{2}$

fig: sarda fall with rectangular crest



Q = for 14 cumecs and over
B_t = Top width of crest = $0.55\sqrt{H+d}$
Base width = as determined by the batters

$$Q = 1.99 \cdot LH^{3/2} \left(\frac{H}{B_t}\right)^{1/6}$$

fig:- sarda fall with trapezoidal crest

i) Length of Crest :-

Length of crest, L = Bed width of canal

ii) Shape of Crest :-

For $Q \le 14\ m^3/sec$, rectangular crest is provided.

For $Q > 14\ m^3/sec$, trapezoidal crest is provided.

iii) Discharge Capacity :-

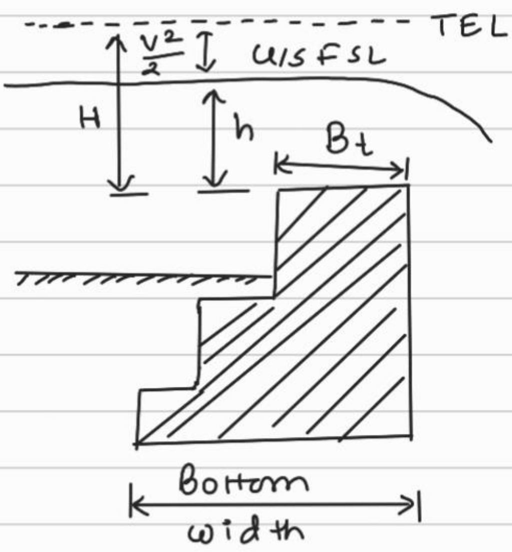
The discharge capacity of rectangular crest sarda fall, $Q = 1.84 \times L \times H^{3/2} \times \left(\frac{H}{B_t}\right)^{1/6}$

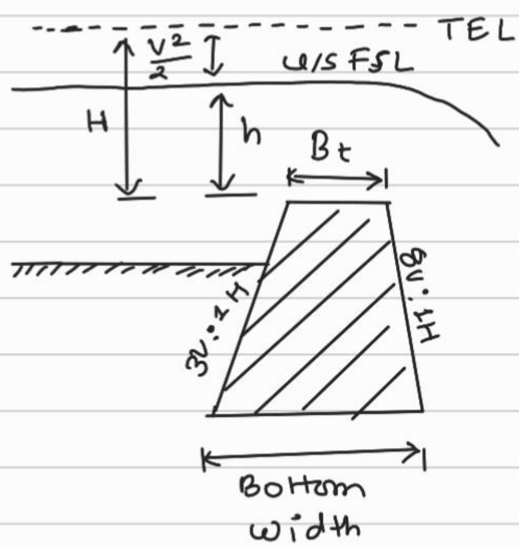
where, H = Head over crest measured from total energy line/Energy Grade line.

B_t = Top width of crest

Discharge Capacity for a trapezoidal crest Sarda fall, $Q = 1.99 \times L \times H^{3/2} \times \left(\frac{H}{B_t}\right)^{1/6}$

iv) Crest level and Crest dimensions :-





Total Energy Line, TEL = upstream FSL + $\frac{v^2}{2g}$

where, v = approach velocity.

Crest level = TEL − H

h = u/s FSL − crest level = head measured from crest

Rectangular Crest :-

Top width, $B_t = 0.55\sqrt{d}$

where, d = crest level − d/s bed level

Bottom width = $\frac{h+d}{G}$

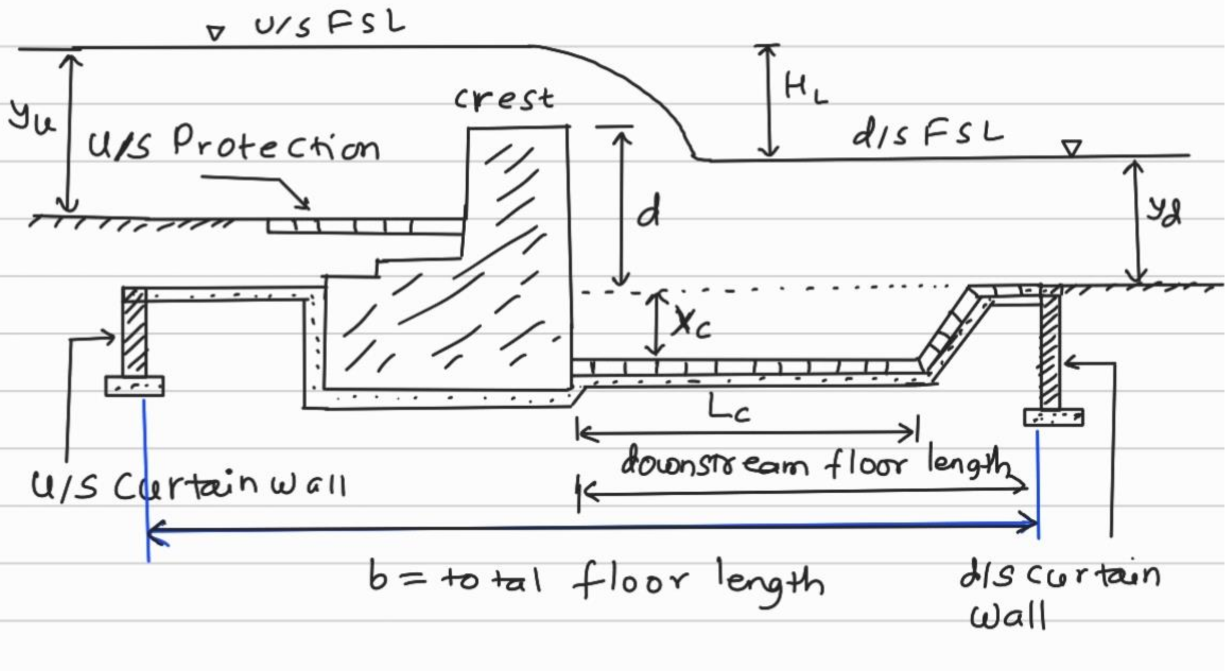
where G = specific gravity of material

= 2 (for brick masonry crest)

Trapezoidal Crest :-

* Top width, $B_t = 0.55 \times \sqrt{h+d}$
* Bottom width is calculated by assuming upstream slope of 3V : 1H and 8V : 1H.

V> Design of Concrete Floor and Water Cushion :-



* length of water cushion, $L = 5\sqrt{H * H_L}$
* Depth of water cushion, $x_c = \frac{1}{4} * \left[H * H_L\right]^{2/3}$

H_L = drop = u/s FSL − d/s FSL
H = Head over crest

* length of downstream floor
= 2 * { Water depth on d/s + 1.2 } + Drop
= $2 * (y_d + 1.2) + H_L$

* Total Floor length is calculated in following ways.

i) <u>By using Bligh's Theory :-</u>

$b = C * H_{max}$
C = Bligh's Constant
H_{max} = maximum possible water level difference between u/s and d/s
= Crest level − d/s bed level

ii) By using Khosla's theory

$$G_E = \frac{H_{max}}{d} * \frac{1}{\pi * \sqrt{\lambda}}$$

H_{max} = Crest level − d/s bed level
d = depth of downstream curtain wall
G_E = Critical / Permissible Exit Gradient

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

and $\alpha = b/d$

first, we get λ from $G_E = \frac{H_{max}}{d} * \frac{1}{\pi\sqrt{\lambda}}$

then, we get α from, $\lambda = \frac{1 + \sqrt{1+\alpha^2}}{2}$

then, we get $b = \alpha d$

vi) Design of Protection :-

* length of upstream brick pitching $= y_u$

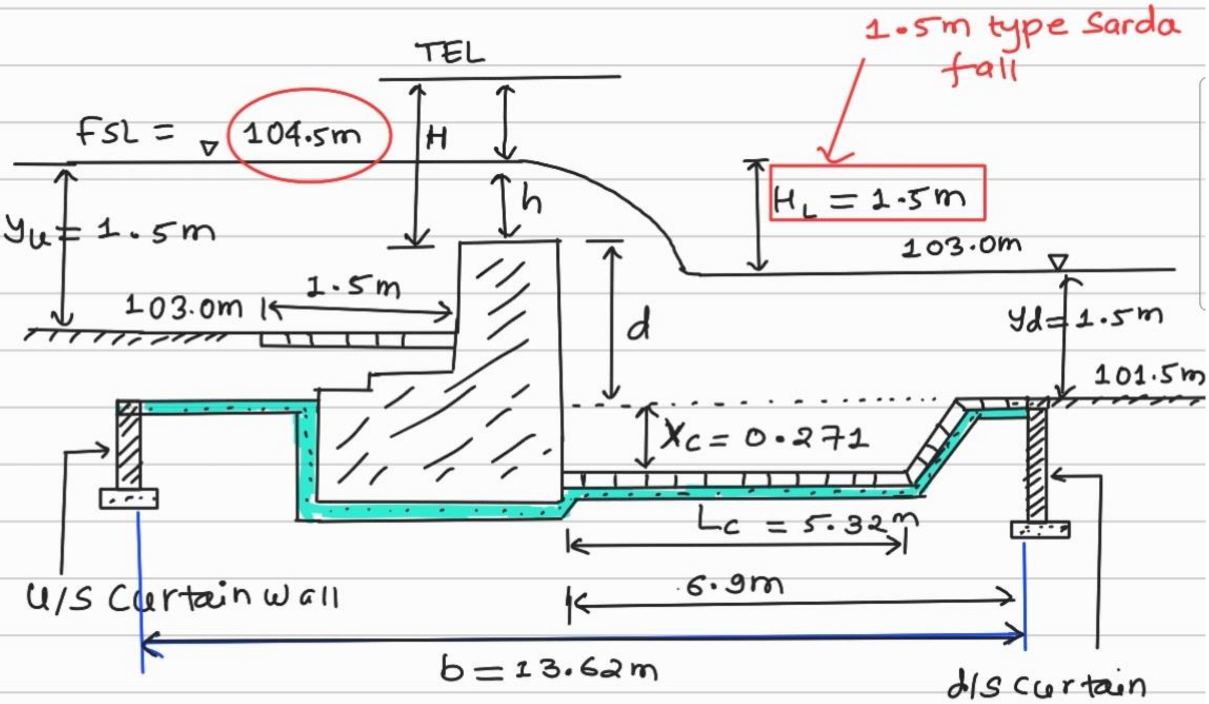
  y_u = depth of water on upstream.

* depth of u/s curtain wall $= \frac{y_u}{3}$

* depth of d/s curtain wall $= \frac{y_u}{2}$.

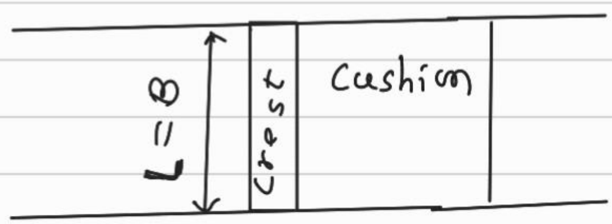
Example 12.3. *Design a 1.5 metres Sarda type fall for a canal having a discharge of 12 cumecs, with the following data :*

Bed level upstream	= 103.0 m
Side slopes of channel	= 1 : 1~~½~~ (1H : 1V)
Bed level downstream	= 101.5 m
Full supply level upstream	= 104.5 m
Bed width u/s and d/s	= 10 m
Soil	= Good loam
Assume Bligh's Coefficient	= 6



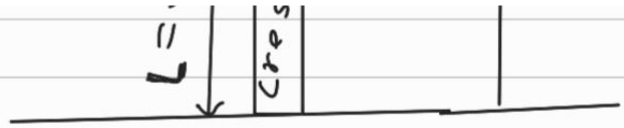
i> Crest length :-

* length of Crest = Bed width of Canal = 10.0m



ii> Type of Crest:

ii> Type of Crest:



$Q > 14\, m^3/sec \Rightarrow$ trapezoidal crest
$Q \leq 14\, m^3/sec \Rightarrow$ rectangular crest
$Q = 12\, m^3/sec \Rightarrow$ we will provide rectangular crest.

iii> Calculation of TEL:-

Approach velocity, $V = \frac{Q}{A}$

* The canal cross-section is trapezoidal on u/s of drop.

$\therefore \quad V = \frac{Q}{A} = \frac{Q}{By + y^2 z} = \frac{12}{10 * 1.5 + 1.5^2 * 1}$

(slope = 1H : 1V $\Rightarrow z = 1$)

$\therefore V = 0.696\, ms^{-1}$

Approach velocity head $= \frac{V^2}{2g} = \frac{0.696^2}{2 * 9.81}$

$= 0.025 m$

$\therefore$ Elevation of U/S TEL = U/S FSL + $\frac{V^2}{2g}$

$= 104.5 + 0.025 = 104.525 m$

iv> Crest level and Crest shape:-

Discharge capacity of rectangular crest,

$Q = 1.84 * L * H^{3/2} * \left(\frac{H}{B_t}\right)^{1/6}$

$Q = 12\, m^3/sec$

L = length of crest = 10.0 m

H = Head over crest measured from TEL

B_t = Top width of crest

Assume, top width, $B_t = 0.8$ m

$$\therefore\ 12 = 1.84 * 10 * H^{3/2} * \left(\frac{H}{0.8}\right)^{1/6}$$

$$\Rightarrow\ H = 0.755\text{ m}$$

$\therefore$ Crest level = TEL − H

= 104.525 − 0.755

= 103.77 m

Check for Crest width

Top width of crest, $B_t = 0.55\sqrt{d}$

$\Rightarrow$ d = Crest level − d/s bed level

= 103.77 − 101.5 m

= 2.27 m

$\therefore B_t = 0.55\sqrt{2.27} = 0.828$ m

Provide top width of 0.828 m.

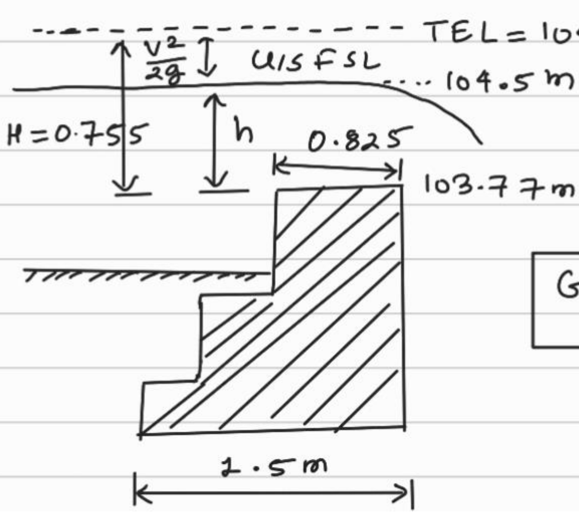
Bottom width of Crest $= \dfrac{h+d}{G}$

h = FSL − Crest level = 104.5 − 103.77

= 0.73 m

$\therefore$ Bottom width of Crest $= \dfrac{2.27 + 0.73}{2}$

= 1.5 m



G = 2 for brick masonry

v> Design of Concrete floor and cushion:-

* length of cushion, $L_c = 5 * \sqrt{H * H_L}$

H_L = drop = 1.5m

H = 0.755m

$\therefore L_c = 5 * \sqrt{0.755 * 1.5} = 5.32m$

Depth of Cushion, $X_c = \frac{1}{4} * (H * H_L)^{2/3}$

$= \frac{1}{4} * (0.755 * 1.5)^{2/3}$

$= 0.271m$

* length of floor:-

since, Bligh's constant is given,

length of floor, b = C * H_{max}

Here, C = 6

H_{max} = maximum water level difference betn u/s and d/s in worst condition.

= Crest level - d/s bed level

= 103.77 - 101.5 = 2.27m

$\therefore$ Total floor length, b = 6 * 2.27

= 13.62 m

d/s floor length = 2 [y_d + 1.2] + H_L

y_d = water depth on downstream = 1.5m

$\therefore$ d/s floor length = 2[1.5 + 1.2] + 1.5

= 6.9m

vi> Protection Works:-

a. U/s Bed pitching:

* Provide one brick pitching of length equal to y_u = 1.5m.

b. U/s Curtain wall.

* depth of Curtain wall = $\frac{y_u}{3} = \frac{1.5}{3} = 0.5m$

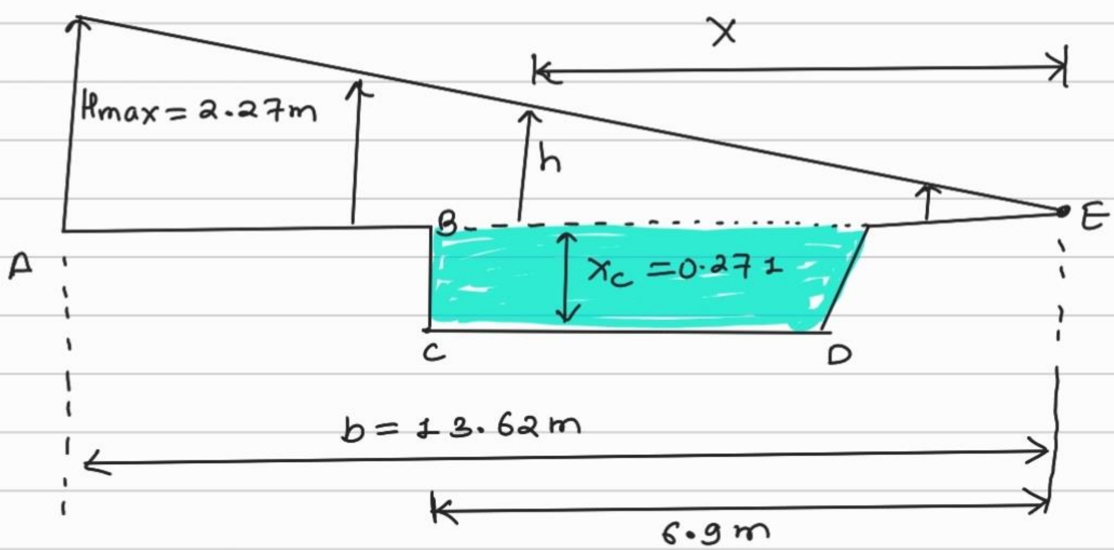
b. u/s Curtain wall.

* depth of Curtain wall = $\frac{y_u}{3} = \frac{1.5}{3} = 0.5m$

c. d/s curtain wall :-

* depth of curtain wall = $\frac{y_u}{2} = \frac{1.5}{2} = 0.75m$

viii> Thickness Calculation:



Head at any distance, x from d/s (E) is given by :- $\frac{h}{x} = \frac{H_{max}}{b}$

$\Rightarrow h = x * 2.27/13.62 = 0.167x$

for any point that lies in cushion, additional head will be 0.271 m

$\therefore h_{cushion} = h + 0.271 = 0.167x + 0.271$

Calculation of thickness at C.

for C, $x = 6.9$ m (E बाट)

$\therefore$ head at C, $h_c = 0.271 + 0.167x$

$= 0.271 + 0.167 * 6.9$

$= 1.42$ m

$\therefore$ thickness of floor required,

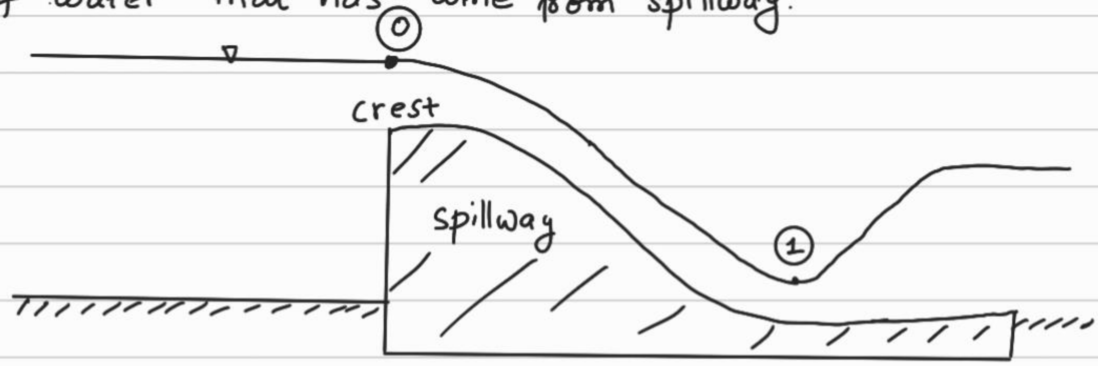
$$t = \frac{h}{G-1}$$

$G = 2.2$ (Brick and concrete average)

$$\therefore t = \frac{1.42}{2.2-1} = 1.183\text{m}$$

Energy Dissipating Structures:-

Energy dissipation :- It means killing the excess energy of water that has come from spillway.



* As the water flows from spillway to the downstream, the potential energy of water at point 0 is converted into kinetic energy as it reaches at toe of spillway.
* If the water with high velocity is released to the river without dissipating it's energy, it will cause erosion of the river channel and bank.
* Therefore energy dissipator is required at the toe of spillway to dissipate the energy of water.

Design of Surface Drains:-

There are separate assumptions for design of surface drains in Terai and hills.

a. Design of surface drains in Terai

Following assumptions should be made for design of surface drains in Terai.

i> Rainfall of 5 to 10 years return period should be considered.

ii> 3 day rainfall is considered for the design of surface drains.

iii> Initial water level in the field should be considered as 40mm.

iv> Maximum water level in the field is 300mm which persists for one day.

v> Depth in excess of 200 mm may persist for 3 days.

vi> No raifall follows the design rain for several days. That means there is no rainfall after 3-days.

let, P_3 = 3 day rainfall of 10 year return period at that area in mm

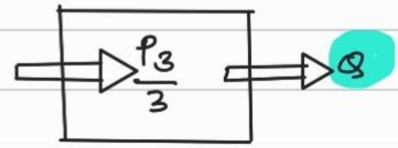
q = Rate of discharge of the drainage per day in mm.

h_0 = Water level at the field at the begining of a day

h = Water level at the field at the end of a day.

Then according to water balance eqⁿ;

$$h = h_0 + \frac{P_3}{3} - Q$$



Note:

* $h_0 = 40$ mm for day 1.
* $\frac{P_3}{3}$ is the 3-day rainfall depth which continues for first 3-days only.
* h at the end of any day will be h_0 at the begining of next day.
* We have to try to minimize Q but the value of h should be within the limits of depth of water, h. If we can minimize Q, size of drain will be small.

<u>Design of Surface Drains in Hilly Regions:-</u>

Following assumptions are made for the design of surface drains in hilly region.

i> 1-day rainfall depth of 5-10 year return period is considered.

ii> Initial water level in the field should be 40 mm for design.

iii> No rain follows the design rainfall for several days.

iv> Evaporation and transpiration losses are neglected.

v> Maximum water depth is 100 mm which persists for one day.

Using, water balance equation,

$$40 + P - Q = 100$$

Here: P = 1-day rainfall of 5-10 year return period in mm.

Q = rate of drainage from the field per day.

$\therefore \quad P - Q = 60$

$$\boxed{\Rightarrow Q = P - 60}$$

Design of Sub-Surface Drains or Tile Drains:-

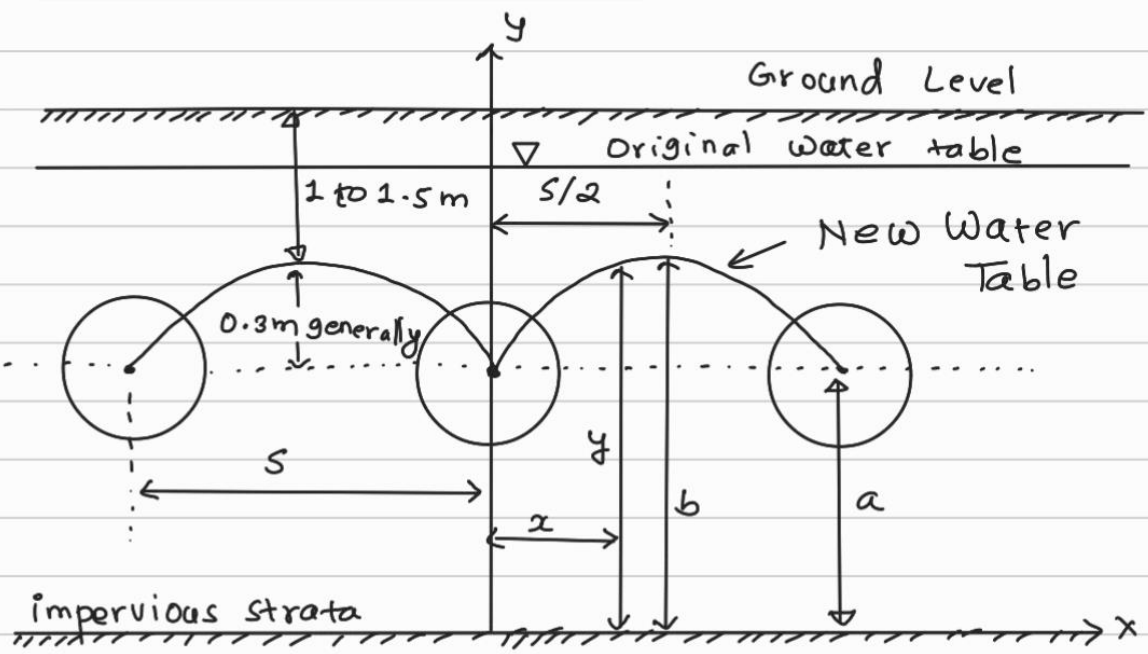
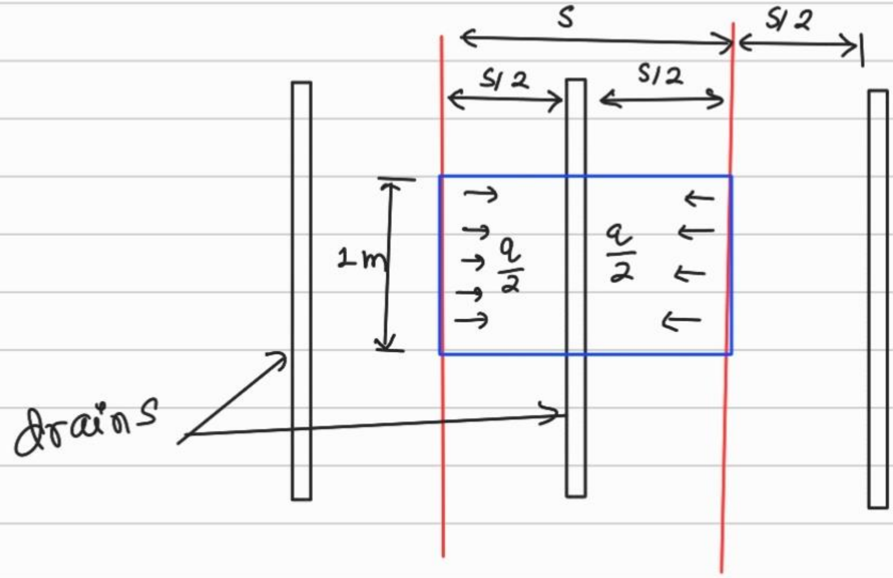


fig: Cross-section of lay out of sub-surface drains.



Depth and spacing of drain pipes:-

* The depth and spacing of sub-surface drains should be sufficient to lower the water table from root zone of plants.
* The highest point of water table is generally 1 to 1.5m for most of the plants. This distance may vary 0.7 to 2.5m depending upon soil type and type of crop.
* The tile drains may be placed at about 0.3m below the designed highest level of the water table.

let, S = spacing of the drain pipes.

a = height of drain centerline from the impervious strata.

b = height of highest point of water table from impervious strata.

b = height of highest point of water table from impervious strota.

Consider a point on the water table at horizontal distance x and vertical distance y.

$\therefore$ from darcy's law,

$$Q = KiA$$

The discharge per unit width of the drains at the point with height y is given by:

$$q_y = KiA$$

where, i = hydraulic gradient = $\frac{dy}{dx}$

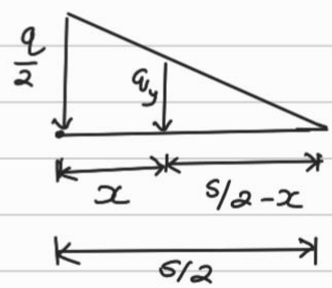
A = Area of flow = $y * 1$

$$\therefore q_y = K * \frac{dy}{dx} * y \text{ .... i>}$$

When $x = 0$, $q_y = q/2$

When $x = s/2$, $q_y = 0$

$\therefore$ By linear interpolation,



$$\frac{q_y}{\frac{s}{2} - x} = \frac{q/2}{s/2}$$

$$\Rightarrow q_y = \frac{q}{2} * \frac{\frac{s}{2} - x}{\frac{s}{2}}$$

$$\Rightarrow q_y = q * \left(\frac{\frac{s}{2} - x}{s} \right) \text{ .... ii>}$$

Substituting value of q_y from ii> in i>,

$$q * \frac{\frac{s}{2} - x}{s} = K * \frac{dy}{dx} * y$$

$$\Rightarrow q * \left(\frac{s}{2} - x \right) * dx = Ks * y\,dy$$

Integrating

$$\int_0^{S/2} q * \left(\frac{S}{2} - x\right) dx = \int_a^b KS * y\, dy$$

$$\Rightarrow q * \left[\frac{S}{2}x - \frac{x^2}{2}\right]_0^{S/2} = KS * \left[\frac{y^2}{2}\right]_a^b$$

$$\Rightarrow q * \left[\frac{S}{2} * \frac{S}{2} - \frac{1}{2} * \left(\frac{S}{2}\right)^2\right] = KS * \frac{b^2 - a^2}{2}$$

$$\Rightarrow q * \frac{S^2}{8} = KS * \frac{b^2 - a^2}{2}$$

$$\therefore \quad S = \frac{4K(b^2 - a^2)}{q} \quad \ldots\ldots\ldots \text{iii)}$$

<u>Calculation of q:</u>

* q is the discharge per unit width in a drain separated by spacing s.
* let P_{AA} is the average annual rainfall of the area in meters. Then, it is assumed that the drains will have to drain 1% of P_{AA} in 24 hours.
* So, Volume of rainfall drained = Area * depth

or $Vol = S * 1 * 0.01 * P_{AA}$

$= 0.01 * P_{AA} * S$

$$\text{discharge} = q = \frac{\text{volume}}{\text{time}} = \frac{0.01 * P_{AA} * S}{24 \text{ hours}}$$

or

$$\boxed{q = \frac{0.01 * P_{AA} * S}{24 * 60 * 60} \ \text{m}^3/\text{sec}}$$

Drainage Coefficient (D.C.):- Depth of water discharged by subsurface drain in 24 hours is called drainage coefficient.

Example 6.4. *Determine the size of a tile at the outlet of a 6 hectare drainage system, if the D.C. is 1 cm and the tile grade is 0.3%. Assume the rugosity coefficient for the tile drain material as 0.011.*

Solution. 1 cm D.C. means that 1 cm of water from an area of 6 hectares is entering the tiles per day.

$\therefore$ Volume of water passing the drain in 1 day $= \left(\frac{1}{100} \times 6 \times 10^4\right) = 600\ \text{m}^3/\text{day}$

$\therefore$ Volume of water passing the drain in 1 second $= \left(\frac{600}{24 \times 3600}\right) = \frac{1}{144}\ \text{m}^3/\text{s}$

$\therefore \quad Q = \frac{1}{144}\ \text{m}^3/\text{s} \qquad$ Now, $\quad Q = \frac{1}{n} \cdot A.R^{2/3} \cdot S^{1/2}$

For a circular drain of diameter D, we have

$$A = \frac{\pi D^2}{4}, P = \pi D, R = \frac{D}{4}$$

$$R = \frac{A}{P} = \frac{\frac{\pi D^2}{4}}{\pi D} = \frac{D}{4}$$

or $$\frac{1}{144} = \frac{1}{0.011} \times \left(\frac{\pi D^2}{4}\right) \cdot \left(\frac{D}{4}\right)^{2/3} \cdot \left(\frac{0.3}{100}\right)^{1/2}$$

or $$\frac{1}{144} \times \frac{0.011 \times 4}{\pi} = \frac{D^2 \cdot D^{2/3}}{(4)^{2/3}} \times \frac{1}{\sqrt{333.3}}$$

or $$\frac{0.011 \times 4 \times 2.52 \times 18.26}{144 \times \pi} = D^{8/3}$$

or $D = (0.00447)^{3/8} = 0.132\ \text{metre} = 13.2\ \text{cm}$ **Use 15 cm dia. pipe. Ans.**

Hill Irrigation:-

1. location of headworks:-

- Straight portion of river
- Geologically stable section.
- Should be accessible.
- should not be located on deep gorges.
- Elevation should be such that water reaches to the whole command area.