

# IRRIGATION AND DRAINAGE ENGINEERING

## Text Books:

1. Irrigation Engineering and Hydraulic Structures. S K Garg
2. Irrigation and Water Power Engineering. B C Punmia, Pande B B Lal, Ashok Kumar Jain and Arun Kumar Jain
3. Irrigation Engineering. Gurucharan Singh
4. Theory and Design of Irrigation Structures Volume I and II. R S Varshney, S C Gupta and R L Gupta

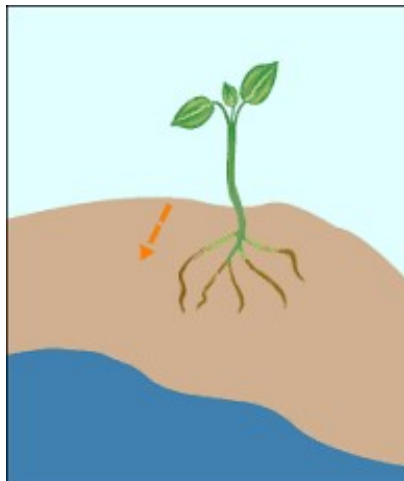
## Internal Evaluation: 20 Marks

- i) Class Attendance – 2 Marks
- ii) Assignment/Tutorial - 5 Marks
- iii) Field Visit - 3 Marks
  - a. Attendance – 1 Mark
  - b. Report Submission – 2 Marks
- iv) Written Examination – 10 Marks

# INTRODUCTION

## Introduction

- Three basic requirements of agricultural production are soil, seed, and water.
- In addition, fertilizers, insecticides, sunshine, suitable atmospheric temperature, and human labour are also needed.
- Of all these, water is most important requirement of agricultural production.
- The moisture available in the root-zone soil, either from rain or from underground water
- Deficiency of moisture may be either for the entire crop season or for only part of the crop season reduces for optimum plant growth
- Therefore, it becomes necessary to make up the deficiency by adding water to the root-zone soil.



## Irrigation:



- Artificial application of water to the soil for raising crops
- Science which pertains to the planning, designing, controlling and maintenance of irrigation works so that available water resources may be used in the best manner
- Irrigation projects are sanctioned on the basis of benefit cost ratio (BCR);  $BCR > 1.5$

## Objectives of Irrigation:

- To supply water partially or totally for crop need
- To cool both the soil and the plant
- To leach excess salts
- To improve groundwater storage
- To facilitate continuous cropping
- To enhance fertilizer application

## Purposes of Irrigation:

- Providing insurance against short duration droughts
- Reducing the hazard of frost (increase the temperature of the plant)
- Reducing the temperature during hot spells
- Washing or diluting salts in the soil softening tillage pans and clods
- Promoting the function of some micro organisms

## **Necessity of Irrigation:**

- **Insufficient rainfall:** when the seasonal rainfall is less than the minimum requirement for the satisfactory growth of crops, the irrigation system is essential.
- **Uneven distribution of rainfall:** when the rainfall is not evenly distributed during the crop period or throughout the cultivable area, the irrigation is extremely necessary.
- **Improvement of perennial crops yield:** some crops such as sugarcane etc. require water through out the major parts of the year but the rainfall fulfills the demand during the rainy season only. Therefore, for remaining part of the year irrigation is necessary.
- **Development of agriculture in the desert areas:** in the desert, area where the rainfall is very scanty, irrigation is required for the development of agriculture.
- **Insurance of drought:** irrigation may not required during the normal rainfall condition and can be necessary during drought

## **Function of Irrigation:**

- Dissolves chemicals, manures and renders them plant growth; thus water acts as a nutrient carrier.
- Supplies moisture to the soil which is essential for the life of bacteria which are beneficial to plant growth.
- Supplies moisture which is essential for the metabolism within the plant leading to plant growth.
- Reduces the concentration of the harmful salts in the soil.
- Saves plants from harmful effects of frost during intensive cold.
- Lowers the temperature of the soil and atmosphere during summer; thus creating a healthy environment for plant growth.
- Softens the tillage pans.
- Helps in bringing up ground water table (GWT).

### **Advantages or Benefits:**

- Increase of food production
- Protection against famine
- Modify soil or climate environment
- Increase income and national cash flow
- Increase labor employment
- Increase standard of living
- Increase value of land
- Inland navigation in large canals
- Improve communication
- Domestic and industrial water supply
- Improve ground water storage

- Generation of hydro-electric power with multi-purpose project or from canal fall
- Plantation of canal banks
- Mixed cropping is eliminated, which is generally not accepted

### **Disadvantages or Ill-effects:**

- High initial cost
- Water logging and water pollution problem
- Damp and cold climate
- Salinity and alkalinity of land
- Ill aeration of soil
- Loss of valuable lands

### **Status of Irrigation Development in Nepal:**

- Before 1922, operated and maintained by farmers called Farmers Managed Irrigation System (FMIS)
- From 1922 to 1957, Government made little effort (Chandra Nahar, Juddha Nahar, Jagadispur Jalasraya (Banganga), Phewa Bhadh)
- Irrigation infrastructure development has got high priority since 1957
- The minor irrigation program was introduced in the second three-year development plan 1962-1965 to provide low-cost-irrigation facilities
- The Third Plan Period (1966-1970) saw the countrywide implementation of the minor irrigation program
- The government investment in irrigation development – especially in the large-scale irrigation systems in the Terai increased tremendously from 1970 onwards

### **Status of Irrigation Development in Nepal:**

- Until the middle of 1980s, irrigation development by the government focused largely on the construction of physical infrastructure of canals and structures rather than effective management of the completed systems
- Improved management of government-operated irrigation systems from 1985 onwards; implementation of a number of management-oriented projects in 1985-1989: the United States Agency for International Development (USAID)-funded Irrigation Management Project (IMP) in 1985, the Irrigation Line of Credit (ILC) in 1988 financed by the World Bank, the irrigation Sector Project (ISP) in 1988 financed by the ADB, and the Irrigation Sector Support Project (ISSP) in 1989 under the co-financing of the UNDP, the World Bank and the Asian Development Bank (ADB)
- Introduction of the Basic Needs Program (BNP) in 1987, the working Policy on Irrigation Development for the fulfillment of 'Basic Needs' was formulated in the early 1989

### **Status of Irrigation Development in Nepal:**

- Promulgation of the Irrigation Regulations (IR) in April 1989: emphasis on the greater collaboration with water users in all phases of irrigation projects – planning, construction, operation and maintenance
- Irrigation Regulations gave water users, for the first time, a legal mandate to form water users' associations in accordance with the 1976 Association Registration Act
- In 1989, the participatory management of large irrigation systems were formulated
- Promulgation of Water Resources Act and Irrigation Policy in 1992 with the clear vision of irrigation development; this policy was amended in 1997 and now Irrigation Policy 2004 is in practice
- Irrigation Master Plan 1990, Agriculture Perspective Plan 1995, Water Resources Strategy 2002 and National Water Plan 2005 are other few documents which guide irrigation development in Nepal

### **Status of Irrigation Development in Nepal:**

- At present, DoI is involved in the development of many irrigation projects (Sikta, Ranijamara Kulariya, Mahakali III, Babai, IWRMP, CMIASP and MIP are few examples of major activities in the implementation)
- DoI is equally responsible for development of new irrigation projects and O&M of developed schemes
- For the last couple of years, DoI has been working with marginalized farmers in remote areas under the program of Non-conventional Irrigation Technology Project (NITP)
- DoI has given high priority to IWRM principles while planning and developing new projects
- DoI has realized the importance of year round irrigation, and underway to start multipurpose inter basin water transfer project (Bheri Babai diversion project is the first one to be implemented)

### **Need of Irrigation Development in Nepal:**

- Agriculture is Nepal's primary economic sector, with about 80% of the population dependent upon it
- Agricultural growth is essential for attaining broad-based growth and improving the livelihoods of most Nepalese
- Agriculture in Nepal depends largely on monsoon rains from June to September (75% of annual rainfall occurs), and regulated and controlled irrigation is therefore critical to improving agricultural productivity
- Total area of Nepal = 14.7 million hectare; 2.6 million hectare area is arable and 1.8 million hectare land is irrigable; 76% of potential irrigable area lies in the Terai Region; the remaining 0.40 million hectare is in river valleys, upland valleys, and terraces on hills and mountains

### **Need of Irrigation Development in Nepal:**

- 70% of the command areas of surface water irrigation infrastructure is actually irrigated, with only 38% of irrigated land irrigated year round
- The incidence of poverty in irrigated areas is half that in rain-fed areas and that access to irrigation water mitigates poverty
- Once irrigation is available, for low-income households, higher production will increase household food security and cash incomes through the sale of small quantities of cash crops

## **Crops, their seasons and periods (Cropping pattern & intensity)**

### **Principal crop seasons:**

1. Kharif (Monsoon/Summer) and 2. Rabi (Winter)

#### 1. Kharif

- Starts on 1<sup>st</sup> April and ends on 30<sup>th</sup> September
- The principal crops of these seasons are paddy, millet, cotton, groundnut, etc.

#### 2. Rabi

- Starts on 1<sup>st</sup> October and ends on 31<sup>st</sup> March
- The principal crops of these seasons are barley, wheat, peas, gram, oilseeds, etc.

#### Note:

- Sugarcane covers both seasons
- Cotton eight months crop
- Kharif and Rabi seasons dates are not rigid; may vary up to 1-3 months on either sides

## **Crops, their seasons and periods (Cropping pattern & intensity)**

### **Crop classifications:**

#### 1. Agricultural classification:

- i) Field crops – wheat, rice, maize, barley, gram, pulses, potato, etc.
- ii) Plantation crops – tea, coffee, rubber, coconut, etc.
- iii) Commercial crops – oilseed, mustard, groundnut, sugarcane, sesame, cotton, tobacco, hemp, etc.
- iv) Horticulture crops – fruit crops, vegetables
- v) Forage crops and Grass fodder
- vi) Miscellaneous crops – silk, medicinal crops, etc.

#### 2. Classification based on irrigation requirement:

- i) Wet crops – irrigation water requires
- ii) Dry crops – irrigation water does not require

## Crops, their seasons and periods (Cropping pattern & intensity)

### Crop classifications:

3. Classification based on crop seasons:
  - i) Kharif crops – rice, maize, cotton, millet, etc.
  - ii) Rabi crops – wheat, barley, gram, peas, potato, tobacco, mustard, etc.
  - iii) Perennial crops – sugarcane, fruits, some vegetables, etc.
4. Classification based on consumption of foods:
  - i) Food crops – rice, maize, wheat, barley, etc.
  - ii) Cash crops – sugarcane, tobacco, hemp, tea, cotton, etc.

### Cropping pattern:

The crop planting sequences practiced in an area is termed as cropping pattern. It changes over space and time.

The cropping pattern depends mainly following factors: i) availability of water, ii) type of soil, iii) climatic condition, iv) value of crops, and v) socio – economic condition.

+ Cropping Pattern of Rice																									
Crop	Jan		Feb		Mar		Apr		May		June		July		Aug		Sep		Oct		Nov		Dec		Approx. Duration (days)
	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	
Monsoon													#	#	#	#	#	#							90
Early							#	#	#	#	#	#													90
Late														#	#	#	#	#	#						90
Monsoon														#	#	#	#	#	#						105
Early					#	#	#	#	#	#	#														105
Late														#	#	#	#	#	#						105
Monsoon														#	#	#	#	#	#	#					120
Monsoon														#	#	#	#	#	#	#	#				135
Monsoon														#	#	#	#	#	#	#	#	#			150

Cropping Pattern of other Crops																									
Crop	Jan		Feb		Mar		Apr		May		June		July		Aug		Sep		Oct		Nov		Dec		Approx. Duration (days)
	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	
Maize 1					#	#	#	#	#	#	#	#													105
Maize 2							#	#	#	#	#	#	#	#											105
Pulses	#	#	#	#	#	#																	#		105
Oilseeds	#	#	#	#	#																		#		90
Wheat 1	#	#	#	#	#																		#	#	120
Wheat 2	#	#	#	#	#	#																	#	#	120
Vegetable (summer)													#	#	#	#	#	#							-
Vegetable (winter)	#	#	#	#																			#	#	-
Potatoes	#	#	#	#	#	#																	#	#	130
Potatoes	#	#	#	#	#	#	#																#	#	130

## Crops, their seasons and periods (Cropping pattern & intensity)

### Cropping intensity:

Cropping intensity is the ratio between total cultivation areas to the total command area within a year.

It is the ratio of net area sown to the total cropped area.

### Crop Rotation:

Crop rotation is the systematic planting of different crops in a particular order over several years in the same growing space.

Necessity:

- To obtain the effect of fallow land
- To help to battle against the forces of erosion
- To check the crop diseases and insect pests
- To increase nitrogen content (if leguminous crop introduced)
- To get better utilization of soil

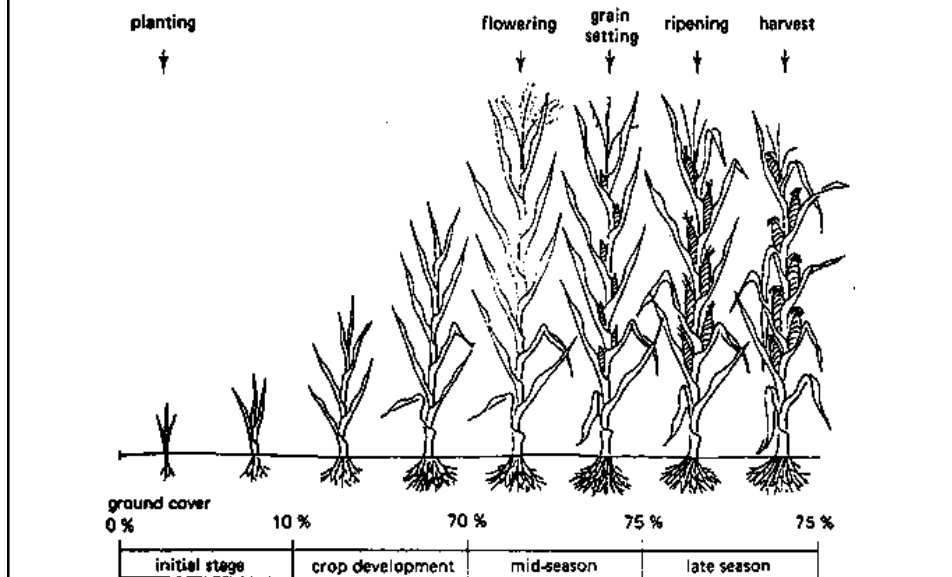
## Crops, their seasons and periods (Cropping pattern & intensity)

### Crop Development Stages:

The total growing period is divided into four growth stages.

1. **Initial stage:** This is the period from sowing or transplanting until the crop covers about 10% of the ground.
2. **Crop development stage:** This period starts at the end of the initial stage and lasts until the full ground cover has been reached (ground cover 70-80%); it does not necessarily mean that the crop is at its maximum height.
3. **Mid-season stage:** This period starts at the end of the crop development stage and lasts until maturity; it includes flowering and grain-setting.
4. **Late season stage:** This period starts at the end of the mid season stage and lasts until the last day of the harvest; it includes ripening.

## Crops, their seasons and periods (Cropping pattern & intensity)



## **Crops, their seasons and periods (Cropping pattern & intensity)**

### **Command Areas and Irrigation Intensity**

#### **Command area (CA):**

The total area which can be irrigated by a canal system.

#### **Gross command area (GCA):**

The total area lying between drainage boundaries which can be commanded or irrigated by a canal system. The boundary is usually defined by the drainage on either side across which irrigation cannot be extended economically.

#### **Culturable command area (CCA):**

The GCA less the area of unculturable land laying within the gross area. The residential area, ponds, reserve forests, etc. are excluded but pasture and undeveloped fellow lands are included.

In the formulation of projects and schemes CCA is roughly taken as 80-90% of GCA.

## **Command Areas and Irrigation Intensity**

CCA can be classified as

- i) Culturable cultivated area  
Crop is grown at a particular time or crop season.
- ii) Culturable uncultivated area  
Crop is not grown at a particular time or crop season.

The crop is kept without cultivation due to following reasons:

- To increase the fertility of the soil which has been reduced due to intense cultivation
- To provide pasture land for animals
- The crop to be sown has a different crop season
- To protect the land from possible danger of water logging

## Command Areas and Irrigation Intensity

### Net command area (NCA):

The CCA excluding also pasture and undeveloped fellow lands.

### Irrigation Intensity (II):

The ratio of irrigated area during a crop season to CCA (percentage of CCA proposed to be irrigated seasonally).

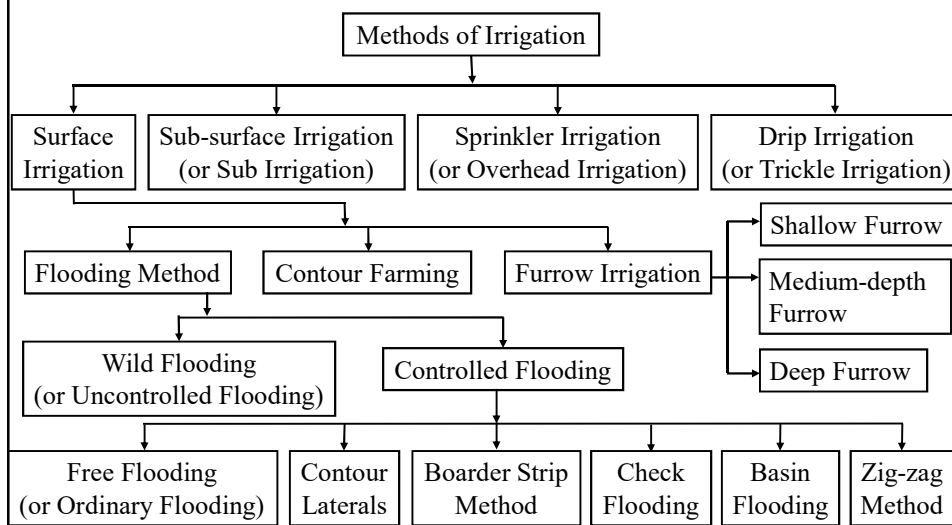
Area to be irrigated = CCA  $\times$  II

### Annual Irrigation Intensity:

The ratio of actually irrigated area during the entire year to CCA.

## Methods of Field Irrigation and Their Suitability

Depending upon availability of water, cultural practices within the community and ability of farmers to afford to the installations, there are many irrigation methods in practices.



## **Methods of Field Irrigation and Their Suitability**

### **1. Surface Irrigation:**

It is most common type of irrigation. Water is applied in the field in the varied quantities and different times with the flow over the land surfaces. Generally one-half of the water released reaches plants.

It may be classified as follows:

- I) Flooding Method
- II) Contour Farming
- III) Furrow Irrigation

#### **I) Flooding Method:**

In flooding irrigation, water is allowed to cover the surface of land in a continuous sheet. The water standing just long enough in the field for the soil to absorb the water applied to refill the root zone.

It may be classified into

- i) Wild Flooding (or Uncontrolled Flooding), and
- ii) Controlled Flooding

## **Methods of Field Irrigation and Their Suitability**

### **i) Wild Flooding (or Uncontrolled Flooding):**

This is earliest and the primitive method of application of water to the land, and the most inefficient of irrigation methods. In this method the water is applied by spreading it over the land prior to the application of water, and no land preparations is done in the form of border or field ditches. The water is allowed to flow the natural slope of the land.



## **Methods of Field Irrigation and Their Suitability**

### **i) Wild Flooding (or Uncontrolled Flooding):**

#### **Suitability**

Suitable for inundation irrigation system or pastures or forage crops where water is available in abundance at the highest elevation and is inexpensive.

#### **Advantages**

- Very low level of expertise is sufficient
- Low cost
- Does not interfere with tillage

#### **Disadvantages**

- Inefficient use of water
- Non-uniform distribution of water
- Excessive soil erosion on steeper slopes
- Require drainage arrangement to reduce ponding
- Irregular crop responses
- Over irrigation and large percolation losses

## **Methods of Field Irrigation and Their Suitability**

### **ii) Controlled Flooding:**

In controlled flooding methods irrigation water is applied by spreading it over the land to be irrigated with proper control on the flow of water as well as the quantity of water applied. All the methods of control flooding require prior preparation of the land. The land is properly graded & agricultural fields are divided into small units by levees.

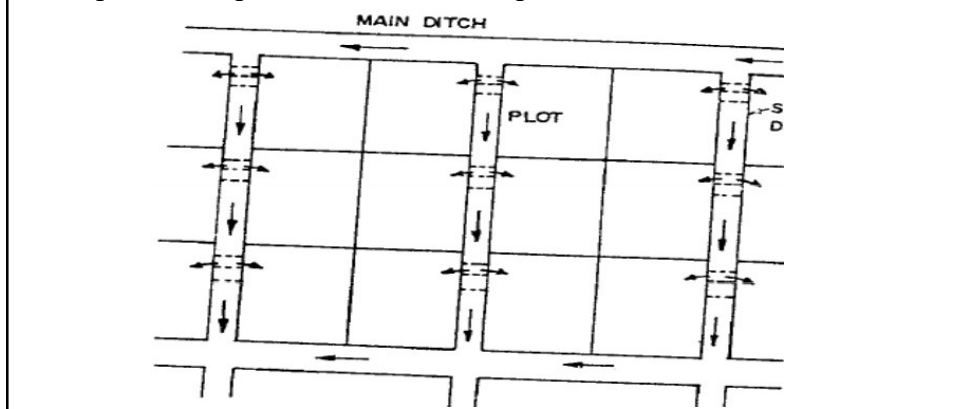
The various methods of controlled flooding are:

- a) Free Flooding
- b) Contour Laterals
- c) Border Strip Method
- d) Check Flooding
- e) Basin Flooding
- f) Zig-zag Method

## Methods of Field Irrigation and Their Suitability

### a) Free Flooding

It is the commonly adopted method where irrigation water is in abundance and cheap. The land is divided into plots of suitable size depending on porosity of soil. Water is supplied to these plots at the higher end and the supply is cut off as soon as this water reaches the lower part of the plot with sufficient depth of water.



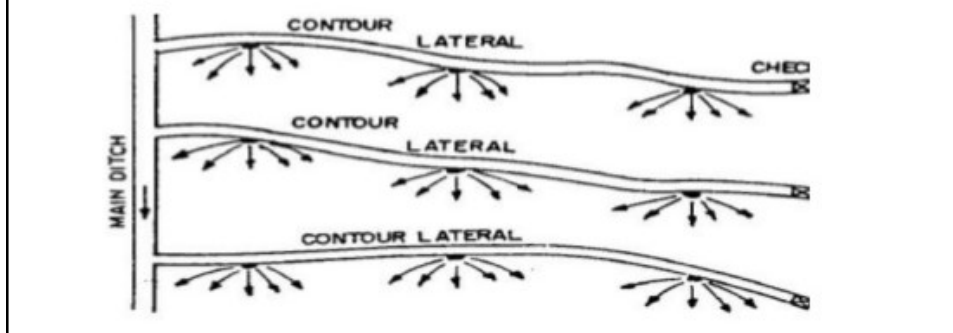
## Methods of Field Irrigation and Their Suitability

### Suitability

Most suitable for soils of medium texture and with moderate slopes.

### b) Contour Laterals

This is a special case of free flooding in which the field channels or laterals are aligned approximately along the contour lines. In this method, irrigation is possible only on side of the laterals. The spacing of laterals may vary from 15 to 50 m.



## Methods of Field Irrigation and Their Suitability

### Suitability

Suitable on close growing crops on sloping or rolling lands not subjected to any degree of levelling necessary for other methods of irrigation.

### Advantages

- Low cost
- Can be used in all types of soil

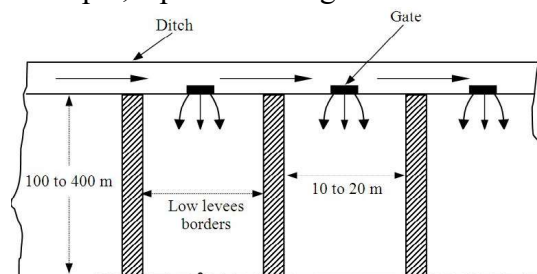
### Disadvantages

- Inefficient use of water
- Very uneven distribution of water
- Excessive soil erosion on steeper slopes
- Irregular crop responses can be seen
- Over irrigation and large percolation losses

## Methods of Field Irrigation and Their Suitability

### c) Border Strip Method

In this method a field is divided into number of strips. The width of strip varies from 10 to 20 m and length varies from 100 to 400 m. The water is diverted from the field channel into the strips. The water flows slowly towards lower end, wetting the soil as it advances. The surface between two embankments should essentially be level. It helps in covering the entire width of the strip. There is a general surface slope from opening to the lower end. The surface slope from 2 to 4 m/1000 m is best suited. When the slope is steeper, special arrangement is made to prevent erosion of soil.



## Methods of Field Irrigation and Their Suitability

### Suitability

This method is suitable for all close growing crops, some row crops and orchard where topography and soil are suitable.

### Advantages

- Utilizes large water streams safely
- Requires less labor
- Provides uniform wetting and efficient use of water

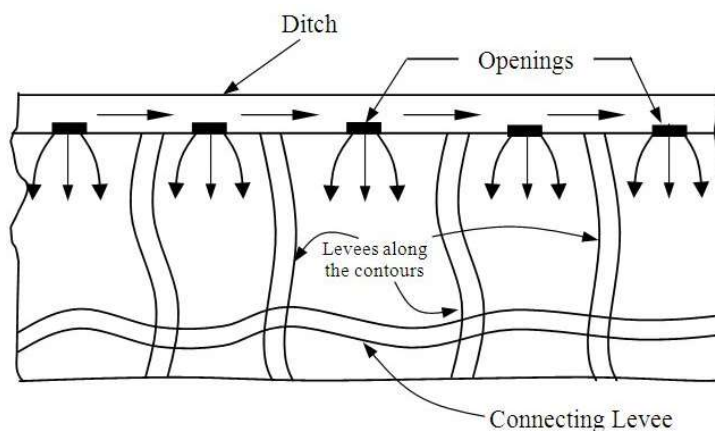
### Disadvantages

- Requires proper levelling
- High initial cost
- A large supply of water is needed

## Methods of Field Irrigation and Their Suitability

### d) Check Flooding

In this method, the farm is divided into small check areas. These are surrounded on all sides by low, flat levees. The checks are square, rectangular or irregular plots. If the ground has initial slope, levees may follow the contours.



## Methods of Field Irrigation and Their Suitability

### Suitability

This method is suitable to irrigate grain and fodder crops in heavy soils where water is absorbed very slowly.

### Advantages

- High irrigation efficiency can be achieved
- Unskilled labor can be employed as there is no danger of erosion

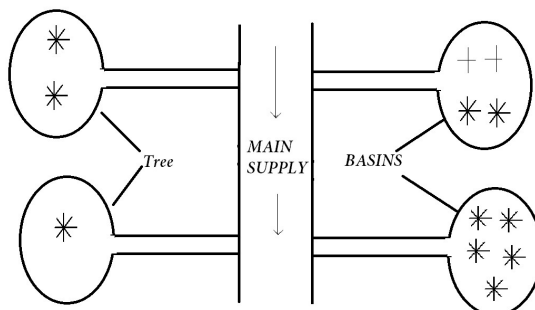
### Disadvantages

- High labor required
- Levees impose restrictions in the use of modern farm machinery
- Use is generally restricted to relatively smooth lands because of the expenditure involved in levelling the plots

## Methods of Field Irrigation and Their Suitability

### e) Basin Flooding

This method is special form 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 the check method. This method is also used extensively to irrigate rice.



## Methods of Field Irrigation and Their Suitability

### Suitability

This method is suitable for close growing crops and orchards on medium to coarse textured soils.

### Advantages

- Provides efficient use of water
- Involves less labor and low maintenance cost

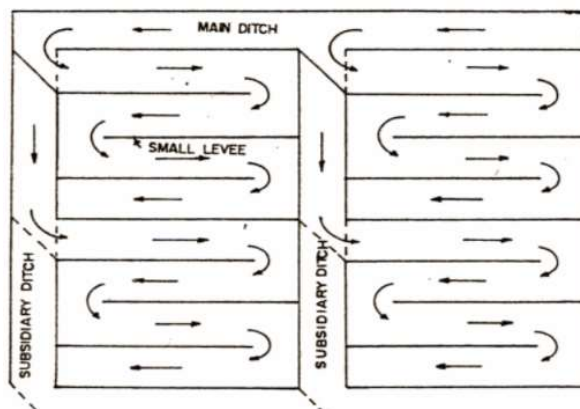
### Disadvantages

- Requires expert levelling and layout
- High initial cost
- Large quantity of water is needed

## Methods of Field Irrigation and Their Suitability

### f) Zig-zag Method

In this method, the agricultural area is sub-divided into small plots by low bunds in a zig-zag manner. The water is supplied to the plots from the field channel through the openings. The water flows in a zig-zag way to cover the entire area. When the desired depth is attained, the openings are closed.



## Methods of Field Irrigation and Their Suitability

### II) Contour Farming

Contour farming is the practice of tillage, planting, and other farming operations performed on or near the contour of the field slope. This method is most effective on slopes between 2 and 10%. Tillage and planting operations follow the contour line to promote positive row drainage and reduce ponding.

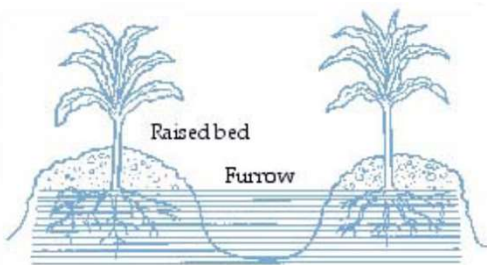
Farming on the contour reduces sheet and rill erosion and the resulting sediment deposition at the foot of the slope or off-site. It can increase water infiltration, thereby reducing the transport of nutrients and organics to surface water and increasing water storage in the soil profile.



## Methods of Field Irrigation and Their Suitability

### III) Furrow Irrigation

Furrow irrigation is a type of surface irrigation in which trenches or “furrows” are dug between crop rows in a field. The dimension of furrows depend on the crop grown, equipment used and soil type. Water in furrows contacts only one half to one fifth of the land surface. Farmers flow water down the furrows (often using only gravity) and it seeps vertically and horizontally to refill the soil reservoir. In heavy soils furrows can be used to dispose the excess water.



## **Methods of Field Irrigation and Their Suitability**

### **Suitability**

- Suitable for wide spaced row crops including vegetables that would be damaged by direct inundation by water (maize, sugarcane, cotton, tobacco, groundnut, potatoes, beans, etc.)
- Suitable to most soils except sand

### **Advantages**

- Relatively high water efficiency reducing evaporation losses and puddling requirement
- Labour requirement for land preparation and irrigation is reduced
- No wastage of lands in field ditches
- Furrows serve as drainage ways for surface runoff in areas of heavy rainfall

### **Disadvantages**

- Requires skill in developing furrow
- Silts from the furrows should be regularly removed
- Adequate drainage provision should be made at the end of each row

## **Methods of Field Irrigation and Their Suitability**

The furrow irrigation can be further subdivided into

1. Shallow furrow
2. Medium-depth furrow, and
3. Deep furrow

1. Shallow furrow

Shallow furrows are 10 to 15 cm deep and 30 to 35 cm wide on the top. These furrows are used for crops with narrow row-spacing and for band sowing.

2. Medium-depth furrow

Medium-depth furrows are 15 to 20 cm deep and 40 to 45 cm wide on the top. These furrows, which are trenched with a row-spacing of 60-70 cm.

3. Deep furrow

Deep furrows are made with a wide row-spacing (80 to 90 cm). Their depth is 20 to 25 cm and at off-season irrigation come up to 30 cm. Such furrows have a large volume of filling, but poor water yield.

## **Methods of Field Irrigation and Their Suitability**

### **2. Subsurface Irrigation:**

In subsurface irrigation, water is applied beneath the ground by creating and maintaining an artificial water table at some depth, usually 30-75 cm below the ground surface. The idea is to raise the water by capillary movement.

It may be classified into

- i) Natural subsurface irrigation, and
- ii) Artificial subsurface irrigation

#### **i) Natural subsurface irrigation**

Natural subsurface irrigation is applicable to low laying lands where an impervious layer exists below the root zone. Water is allowed in to series of ditches dug up to the impervious layer, which then moves laterally and wets root zone.

#### **Advantage**

- Offers most economical means of raising crops

#### **Disadvantage**

- May develop water logging conditions

## **Methods of Field Irrigation and Their Suitability**

### **ii) Artificial subsurface irrigation**

In artificial sub surface irrigation, perforated or porous pipes are laid out underground below the root zone and water is led into the pipes by suitable means.

#### **Advantages**

- Minimum water requirement for raising crops
- Minimum evaporation and deep percolation losses
- No wastage of land
- No interference to movement of farm machinery
- High crop yield

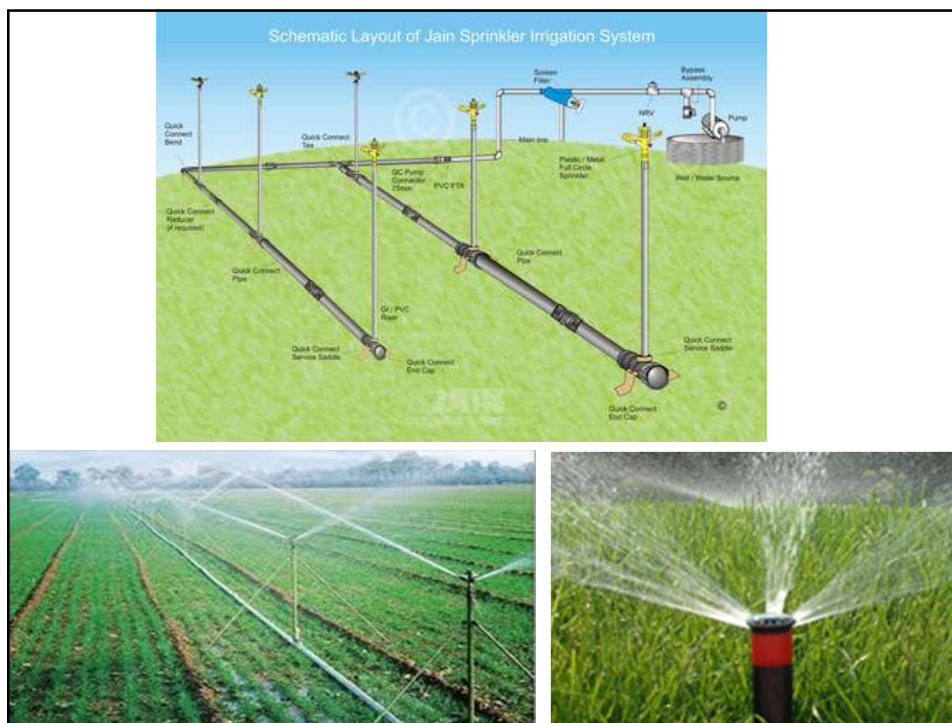
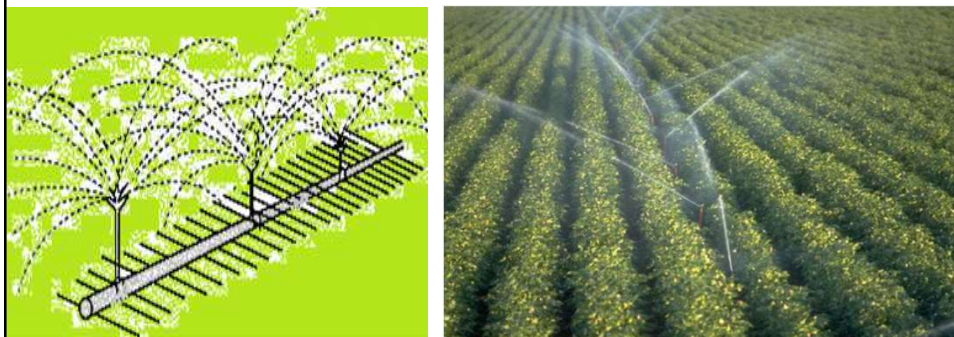
#### **Disadvantages**

- Requires a special combination of natural conditions.
- Danger of water logging
- Possibility of choking of the pipes lay underground.
- High cost

## Methods of Field Irrigation and Their Suitability

### 3. Sprinkler Irrigation:

In sprinkler irrigation, water is sprinkled into the air and allowed to fall on the ground surface just like rainfall. The spray is done by the flow of water under pressure through small orifices or nozzles. The pressure is generally obtained by pumping. Through proper selection of nozzle sizes, operating pressure and sprinkler spacing the amount of irrigation water required to refill the crop root zone can be applied almost uniform at the rate to suit the infiltration rate of soil.



## **Methods of Field Irrigation and Their Suitability**

### **Suitability**

- Suitable for areas having uneven topography and where erosion hazards are great
- Suitable for almost all crops except crops such as paddy and jute
- The dry crops, vegetables, flowering crops, orchards, plantation crops like tea, coffee are all suitable

The system comprises four main parts

- i. Power generator
- ii. Pump
- iii. Pipeline and
- iv. Sprinkler

## **Methods of Field Irrigation and Their Suitability**

### **Advantages**

- No conveyance loss.
- Suitable to all types of soil apart from heavy clay.
- Saves water
- Higher water application efficiency (about 80%)
- Increases in crop yield.
- Mobility of system
- May also be used for undulating area
- Saves land as no bunds and canal systems are required
- Areas located at a higher elevation than the source can be irrigated
- Possibility of using soluble fertilizers and chemicals.
- Less problem of clogging of sprinkler nozzles due to sediment laden water
- The overall cost of labour is generally reduced
- Erosion of soil cover can be reduced

## **Methods of Field Irrigation and Their Suitability**

### **Disadvantages**

- Initial cost of implementation is high
- High and constant energy requirement for operation
- Under high wind condition and high temperature distribution and application efficiency is poor
- Highly saline water causes leaf burning when temperature is higher than 35°C
- When lands have been already levelled and developed for surface or other irrigation methods sprinkler irrigation is not so economical.
- There is loss of water due to evaporation from the area during irrigation
- Not suitable for crops requiring frequent and larger depth of irrigation, and plantation crops as well
- Can not be used on fine textured and heavy clay soils
- Use of marginal (recycled sewage) water is restricted
- Nozzles need screened water supply, otherwise likely to be plugged

## **Methods of Field Irrigation and Their Suitability**

### **Disadvantages**

- Physical damage to crops by application of high intensity spray
- Pipe system has to be assembled and dissembled frequently

### **Problems of Sprinklers**

- Silt and debris in the source of water
- Problems in the operation of the pump
- Problems in the leakage in the pipe lines
- Sprinkling problem due to strong wind
- Pipe network assembling and dissembling

### **Classification of Sprinkler Systems**

- a) Permanent system
- b) Semi-permanent system
- c) Portable system

## Methods of Field Irrigation and Their Suitability

### Classification of Sprinkler Systems

#### a) Permanent system

In this system, pipes are permanently buried in such a way that they do not interfere with the farming operations.

#### b) Semi-permanent system

In this system, the main and sub-main lines are buried while the laterals are portable so as to move from farm to farm.

#### c) Portable system

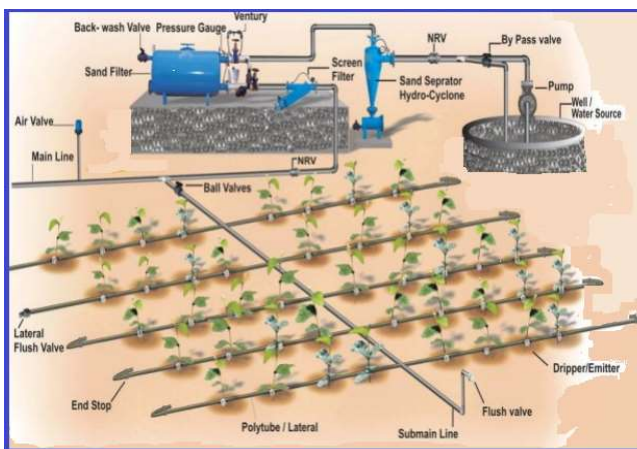
In this system, the main, sub-mains and laterals are portable so as to move from farm to farm.

## Methods of Field Irrigation and Their Suitability

### 4. Drip Irrigation:

It involves slow application of water to the root zone. The drip irrigation system consist of

- i) Head,
- ii) Main line and sub line,
- iii) Lateral lines, and
- iv) Drip nozzles



### **Methods of Field Irrigation and Their Suitability**

The head consists of a pump to lift water and produce the desired pressure (about 2.5 atmosphere) and to distribute water through nozzles. A fertilizer tank for applying fertilizer solution directly to the field along with the irrigation water and filter which cleans the suspended impurities in irrigation water to prevent the blockage of holes and passage of drip and nozzles.

Mains and sub mains are normally of flexible material such as black PVC pipes. Laterals or drip lines are small diameter flexible lines (usually 1 to 1.25 cm diameter black PVC tubes) taking off from the mains or sub mains. Laterals are normally laid parallel to each other. Lateral lines can be up to about 50 meters long and are usually 1.2 cm diameter black plastic tubing. There is usually one lateral line for each crop row. By laying the main line along the center line of the field, it is possible to irrigate either side of the field alternately by shifting the laterals. A pressure drop of 10 percent is permitted between the ends of lateral.

### **Methods of Field Irrigation and Their Suitability**

Drip nozzles are also known as emitters or valves and are fixed at regular intervals in the laterals. These PVC valves allow water to flow at the extremely slow rates, ranging from 2 to 11 liters per hour. The spacing between laterals is controlled by the row-to-row spacing of the crop to be irrigated. Drip laterals laid on soil surface or buried underground at the depth of 5 to 10 cm.

#### **Suitability**

- Suitable for any topography
- Suitable for the soil with different texture

#### **Advantages**

- Less requirement of irrigation water
- Water supply at optimum level
- Water logging avoided
- High yield
- Cultivation of cash crops
- No over irrigation

### **Methods of Field Irrigation and Their Suitability**

- Variation in application rate
- Weed control
- Increase in net irrigable area
- Nutrients preservation
- Reduced labour cost
- No soil erosion
- Suitable for saline soils
- Maintenance of high soil temperature

#### **Disadvantages**

- High initial cost
- Not suitable for close growing crops
- Danger of blockage of emitters
- Pipe laid beneath may interfere with the cultivation
- Can not adopt by ordinary farmers

### **Selection of Field Irrigation Method**

Each of the irrigation methods has some advantages and disadvantages, and the selection of the method depends on the following factors:

- (i) Size, shape, and slope of the field,
- (ii) Soil characteristics,
- (iii) Nature and availability of the water supply subsystem,
- (iv) Types of crops being grown,
- (v) Initial development costs and availability of funds, and
- (vi) Preferences and past experience of the farmer.

**Principal Criteria for the Design of a Suitable Irrigation Method**

- (i) Store the required water in the root-zone of the soil,
- (ii) Obtain reasonably uniform application of water,
- (iii) Minimise soil erosion,
- (iv) Minimise runoff of irrigation water from the field,
- (v) Provide for beneficial use of the runoff water,
- (vi) Minimise labour requirement for irrigation,
- (vii) Minimise land use for ditches and other controls to distribute water,
- (viii) Fit irrigation system to field boundaries,
- (ix) Adopt the system to soil and topographic changes, and

- (x) Facilitate use of machinery for land preparation, cultivating, furrowing, harvesting, and so on.

**Planning of Irrigation Projects**

The preparation of plans of an irrigation project is a complicated task and needs the expertise of specialists.

The process of planning of an irrigation project can be divided into the following two stages:

1. Preliminary planning, and
2. Detailed planning

**1. Preliminary planning**

- Collecting and analyzing all available data for the current study
- Securing additional data needed for preparing preliminary plans
- Determining the feasibility of the proposed development by making the preliminary study of major features in sufficient detail

## **Planning of Irrigation Projects**

### **2. Detailed planning**

- Accurate data on all aspects of the proposed irrigation project are required to work out the detailed plans and designs of various engineering works and to determine their economic site locations
- Physical data needed for detailed planning are collected by topographic and location surveys, land and soil investigations and geological explorations (surfaces as well as subsurface) at the sites of major engineering works
- Hydrological data are usually determined by extensive studies of all available records and collecting additional data, if possible
- Photographic records of pre-construction (and also during construction) condition at locations of all engineering works and aerial surveys for dams and reservoir sites must be supplemented by accurate ground surveys

### **Factors to be considered while planning:**

- Type of project and general plan of irrigation works,
- Location, extent and type of irrigable lands,
- Irrigation requirements for profitable crop production,
- Available water supplies for the project,
- Irrigable (culturable) areas which can be economically supplied with water,
- Types and locations of necessary engineering works,
- Needs for immediate and future drainage,
- Feasibility of hydroelectric power development,
- Cost of storage, irrigation, power and drainage features,
- Evaluation of probable power, income and indirect benefits,
- Method of financing the project construction,
- Desirable type of construction and development,
- Probable annual cost of water to the farmers,
- Cost of land preparations and farm distribution systems, and
- Feasible crops, costs of crop production, and probable crop returns

**Basic information for planning and design**

To adequately plan and design an irrigation system, certain basic information is needed. This data includes:

1. Field Information
2. Soil and Water Data
  - Soil profile and texture classification,
  - Soil depth,
  - Water intake rate, and
  - Soil water holding capacity or available soil moisture
3. Plant Data
  - The type of cropping system,
  - Crop rotation plans, and
  - Peak rate of water use by crops
4. Water Availability
5. System Design

# IRRIGATION WATER REQUIREMENTS

## Relation between Duty, Delta and Crop Periods

### **Crop Period and Base Period:**

The time period that start 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**.

The **crop period** is the total period during which the crop remains on the field, whereas the **base period** is the total period during which irrigation is done. Generally **crop period** is slightly greater than the **base period**.

### **Kor Watering, Kor depth and Kor Period:**

- The distribution of water during the period of crop is not uniform.
- Crops require maximum water during first watering after the crops have grown few centimeters.
- During the subsequent watering the quantity of water needed by crops gradually decreases and is least when crop gain maturity.

### **Kor Watering, Kor depth and Kor Period:**

The first watering is known as **kor watering**, the depth applied is known as **kor depth**. The portion of the base period in which kor watering is needed is known as **kor period**.

**Paleo:** It is defined as the first watering before sowing the Crop.

### **Duty and Delta of a Crop:**

**Delta:** The total quantity of water required by the crop for its full growth may be expressed in hectare-meter or simply as depth to which water would stand on the irrigated area if the total quantity supplied were to stand above the surface without percolation or evaporation.

*This total depth of water is called delta ( $\Delta$ ).*

**Duty (D):** It may be defined as the number of hectares of land irrigated for full growth of a given crop by supply of 1 m<sup>3</sup>/s of water continuously during the entire base of that crop. Duty is the capacity of water to irrigate land. Simply we can say that, the area (in hectares) of *land can be irrigated for a crop period (in days) using one cubic meter of water.*

### **Relation between Duty, Delta and Base period**

Let, base period of the crop be  $B$  days, and  
one cumec of water be applied to this crop on the field for  $B$  days.

Now, volume of water applied to this crop during  $B$  days

$$= V = (1 \times 60 \times 60 \times 24 \times B) \text{ m}^3$$

$$= 86,400 B \text{ m}^3$$

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

$\therefore$  This quantity of water ( $V$ ) matures  $D$  hectares of land or  $10^4 D$  sq. m of area.

Total depth of water applied on this land

$$= \text{Volume/area} = 86400 B / 10^4 D = 8.64 B / D \text{ metres}$$

By definition, this total depth of water is called delta ( $\Delta$ ),

$$\Delta = 8.64 B / D \text{ meter}$$

$$\Delta = 864 B / D \text{ cm}$$

where,  $\Delta$  is in cm,  $B$  is in days; and  $D$  is duty in hectares/cumec.

**Outlet factor**

The duty of water at the outlet is known as the outlet factor.

**Capacity factor**

The is the ratio of the mean supply discharge to the full supply discharge of a canal.

**Time factor**

The time factor of a canal is the ratio of the number of days the canal has actually run to the number of days of irrigation period.

For example, if the number of days of irrigation period = 12, and the canal has actually run for 5 days, the time factor will be 5/12.

(Note: A day has a period of 24 hours (i.e. it includes the night also).

**Overlap allowance**

It might happen that the crop of one season may sometimes overlap the next crop season for some period. During such a period of overlapping, irrigation water is required to be supplied simultaneously to the crops of both the seasons. Thus there is extra demand of water during this period and thus the water supply must be increased by some amount. The extra discharge that has to be supplied for this purpose is known as Overlap allowance.

**Example**

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

**Solution.**

In this question,  $B = 120$  days; and  $D = 864$  hectares/cumec

$$\begin{aligned} \text{Since, } \Delta &= 864 B / D \text{ cm} \\ &= 864 \times 120 / 864 \\ &= 120 \text{ cm} \end{aligned}$$

**Example**

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

**Solution.**

Water is required at an interval of 10 days for a period of 120 days.

Hence, No. of required waterings =  $120/10 = 12$

Therefore, Total depth of water required = No. of waterings x Depth of watering

$$= 12 \times 10 \text{ cm} = 120 \text{ cm.}$$

Hence,  $\Delta$  for rice = 120 cm. **Ans.**

**Example**

If wheat requires about 7.5 cm of water after every 28 days, and the base period for wheat is 140 days, find out the value of delta for wheat.

**Solution.**

No. of required waterings =  $140/28 = 5$

The depth of water required each time = 7.5 cm.

$\therefore$  Total depth of water reqd. in 140 days =  $5 \times 7.5 \text{ cm} = 37.5 \text{ cm}$

Hence  $\Delta$  for wheat = 37.5 cm **Ans.**

**Example**

An irrigation canal has gross commanded area of 80,000 hectares out of which 85% is culturable irrigable. The intensity of irrigation for Kharif season is 30% and for Rabi season is 60%. Find the discharge required at the head of canal if the duty at its head is 800 hectares/cumec for Kharif season and 1700 hectares/cumec for Rabi season.

**Solution:**

Gross culturable area = GCA = 80,000 hectares

Culturable commanded area = CCA =  $0.85 \times 80,000 = 68,000$  hectares

Area under Kharif season =  $68,000 \times 0.30 = 20,400$  hectares

Area under Rabi season =  $68,000 \times 0.60 = 40,800$  hectares

Water required at the head of the canal in Kharif = Area/duty

$$= 20,400/800 = 25.5 \text{ cumecs}$$

Water required at the head of the canal in Rabi = Area/duty

$$= 40,800/1700 = 24.0 \text{ cumecs}$$

Since water requirement in Kharif is more so the canal may be designed to carry a discharge of **25.5 cumecs**.

**Example**

A watercourse has a culturable commanded area of 2600 hectares, out of which the intensities of irrigation for perennial sugar-cane and rice crops are 20% and 40% respectively. The duty for these crops at the head of watercourse are 750 hectares/cumec and 1800 hectares/cumec respectively. Find the discharge required at the head of watercourse if the peak demand is 20% of the average requirement.

**Solution:**

Culturable commanded area = CCA = 2,600 hectares

Area under sugar-cane =  $2600 \times 0.2 = 520$  hectares

Area under rice =  $2600 \times 0.4 = 1040$  hectares

Water required for sugarcane =  $\text{Area/duty} = 520/750 = 0.694$  cumecs

Water required for rice =  $\text{Area/duty} = 1040/1800 = 0.577$  cumecs

Since sugar-cane is a perennial crop, it will require water throughout the year.

Hence,

Watercourse must carry a total discharge =  $0.694 + 0.577$

$$= \mathbf{1.271 \text{ cumecs}}$$

∴ The design discharge, to meet the peak demand, will be  $1.271 \times 1.20 = \mathbf{1.52 \text{ cumecs}}$ .

**Example**

The left branch canal carrying a discharge of 20 cumecs has a culturable commanded area of 20,000 hectares. The intensity of Rabi crop is 80% and the base period is 120 days. The right branch canal carrying a discharge of 8 cumecs has a culturable commanded area of 12,000 hectares, intensity of irrigation of Rabi crop is 50% and base period is 120 days. Compare the efficiencies of the two canal systems.

**Solution:****(a) For left branch canal:**

Area under Rabi crop =  $20,000 \times 0.8 = 16,000$  hectares

Discharge = 20 cumecs

Duty =  $\text{Area/Discharge} = 16,000/20 = 800$  hectares / cumec

**(b) For right branch canal:**

Area under Rabi crop =  $12,000 \times 0.5 = 6,000$  hectares

Discharge = 8 cumecs

Duty =  $\text{Area/Discharge} = 6,000/8 = 750$  hectares / cumec

Since left canal system has higher duty, it is more efficient.

**Example**

A watercourse has a culturable commanded area of 1200 hectares. The intensity of irrigation for crop A is 40% and for B is 35%, both the crops being Rabi crops. Crop A has kor period of 20 days and crop B has a kor period of 15 days. Calculate the discharge of the watercourse if the kor depth for crop A is 10 cm and for crop B is 16 cm.

**Solution:****(a) For crop A:**

Area under irrigation =  $1200 \times 0.40 = 480$  hectares

Kor period =  $b = 20$  days; Kor depth =  $\delta = 10$  cm = 0.1 m

Duty =  $(8.64 \times b) / \delta = (8.64 \times 20) / 0.1 = 1728$  hectares/cumec

Hence discharge required = Area / duty =  $480/1728 = 0.278$  cumecs

**(b) For crop B:**

Area under irrigation =  $1200 \times 0.35 = 420$  hectares

Kor period =  $b = 15$  days; Kor depth =  $\delta = 16$  cm = 0.16 m

Duty =  $(8.64 \times b) / \delta = (8.64 \times 15) / 0.16 = 810$  hectares/cumec

Hence discharge required =  $420/810 = 0.518$  cumecs

Thus the design discharge of watercourse =  $0.278 + 0.518 = 0.796$

say **0.8 cumecs**

**Example**

A watercourse commands an irrigated area of 600 hectares. The intensity of irrigation of rice in this area is 60%. The transplantation of rice takes 12 days, and total depth of water required by the crop is 50cm on the field during the transplantation period. During the transplantation period, the useful rain falling on the field is 10 cm. Find the duty of irrigation water for the crop on the field during transplantation, at the head of the field, and also at the head of the distributary, assuming losses of water to be 20% in the watercourse. Also calculate the discharge required in the watercourse.

**Solution:****Note:**

- ❖ Rice seed is initially germinated in separate seed beds.
- ❖ Afterwards, Seedlings (young plants) of rice are thrust (transplanted) by hand in another previously prepared land.
- ❖ Preparation of land for rice crop includes its thorough saturation before ploughing, so as to puddle and soften the surface soil.
- ❖ Transplantation takes about 10-15 days; requires large quantity of water, i.e. 30-60 cm on the field.

We know that  $\Delta = 8.64 B / D$

Where

$B$  = transplantation period = 12 days

$\Delta$  = Depth of irrigation water actually applied in the field  
 $= 50 - 10 = 40 \text{ cm} = 0.40 \text{ m}$

$D$  = Duty of the irrigation water on the field in hectares/cumec

$D = 8.64 B / \Delta = (8.64 \times 12) / 0.40 = 259.5 \text{ hectares/cumec}$

This duty is on the field.

Since the losses in the canal are 20%, 1 cumec of water discharge at the head of watercourse will become 0.8 cumecs at the head of field and hence will irrigate  $259.5 \times 0.8 = 207.6$  hectares only.

Hence the duty of water at the head of watercourse will be 207.6 ha/cumec.

Now total area under rice plantation =  $600 \times 0.6 = 360$  hectares

Discharge at the head of watercourse =  $360/207.6 = 1.735 \text{ cumecs}$

### Example

Table below gives the necessary data about the crop, their duty and the area under each crop commanded by a canal taking off from a storage reservoir. Taking a time factor for the canal to be 13/20. calculate the discharge required at the head of the canal. If the capacity factor is 0.8, determine the design discharge.

Crop	Base period (days)	Area (hectares)	Duty at head of canal (hectares/cumec)
Sugar-cane	320	850	580
Overlap for sugar-cane (hot weather)	90	120	580
Wheat (Rabi)	120	600	1600
Bajri (Monsoon)	120	500	2000
Vegetable (hot weather)	120	360	600

**Solution:**

Discharge required for crops:

Discharge for sugar-cane =  $850/580 = 1.465$  cumecs

Discharge for overlap sugar-cane =  $120/580 = 0.207$  cumecs

Discharge for wheat =  $600/1600 = 0.375$  cumecs

Discharge for Bajri =  $500/2000 = 0.250$  cumecs

Discharge for vegetables =  $360/600 = 0.600$  cumecs

Since sugar-cane has a base period of 320 days, it will require water in all seasons i.e. Rabi, Monsoon & Hot weather.

Discharge required in Rabi =  $1.465 + 0.375 = 1.84$  cumecs

Discharge required in Monsoon =  $1.465 + 0.25 = 1.685$  cumecs

Discharge required in hot weather =  $1.465 + 0.207 + 0.600 = 2.272$  cumecs

Thus the maximum demand of 2.272 cusecs is in the hot weather.

The time factor =  $13/20$

Therefore,

Full supply discharge at the head of the canal will be

$$= 20272 \times 20/13$$

$$= 3.32 \text{ cumecs}$$

Since, Capacity factor = 0.8

Hence,

Design discharge = full supply discharge / capacity factor

$$= 3.32 / 0.8$$

$$= \mathbf{4.15 \text{ cumecs}}$$

**Example**

The base period, intensity of irrigation and duty of various crops under a canal system are given in the table below. Find the reservoir capacity if the canal losses are 20% and the reservoir losses are 12%.

Crop	Base period (days)	Area (hectares)	Duty at the field (hectares/cumec)
Wheat	120	4800	1800
Sugar-cane	360	5600	800
Cotton	200	2400	1400
Rice	120	3200	900
Vegetables	120	1400	700

**Solution:****(i) Wheat**

Discharge required =  $4800 / 1800$  cumecs

Volume of water required =  $(4800 / 1800) \times 120 = 320$  cumec-days

**(ii) Sugar-cane**

Discharge required =  $5600 / 800$  cumecs

Volume of water required =  $(5600 / 800) \times 360 = 2520$  cumec-days

**(iii) Cotton**

Discharge required =  $2400 / 1400$  cumecs

Volume of water required =  $(2400 / 1400) \times 200 = 342$  cumec-days

**(iv) Rice**

Discharge required =  $3200 / 900$  cumecs

Volume of water required =  $(3200 / 900) \times 120 = 426$  cumec-days

**(v) Vegetables**

Discharge required =  $1400 / 700$  cumecs

Volume of water required =  $(1400 / 700) \times 120 = 240$  cumec-days

Hence, total volume of water required on the field for all crops = 320 + 2520 + 342 + 426 + 240 = 3848 cumec-days

1 cumec-day = 1 cumec flowing for a whole day

$$= 1 \times 24 \times 60 \times 60 \text{ m}^3$$

1 hectare meter =  $1 \times 10^4 \text{ m}^3$

Hence, 1 cumec-day =  $(1 \times 24 \times 60 \times 60) / (1 \times 10^4)$  hectare-meters

$$= 8.64 \text{ hectare-meters}$$

Hence, total volume of water required on the field =  $3848 \times 8.64$

$$= 33300 \text{ hectare-meters}$$

Since losses in the canal system are 20%, the volume of water required at the head of canal =  $33300 \times (100/80) = 41600 \text{ ha-m}$

Allowing 12 % reservoir losses,

The capacity of the reservoir =  $41600 \times (100/88) = \mathbf{47300 \text{ ha-m}}$

**Note:** Alternatively this problem can also be solved in a tabular form. (Next slide)

Crop	Base period B (days)	Duty at the field D (ha/cumec)	Delta $\Delta = (8.64 B)/D$	Area (ha)	Volume = $(\Delta \times A)$ (ha-m)
Wheat	120	1800	0.576	4800	2765.0
Sugar-cane	360	800	3.890	5600	21800.0
Cotton	200	1400	1.235	2400	2965.0
Rice	120	900	1.152	3200	3690.0
Vegetables	120	700	1.480	1400	2070.0
				Total	<b>33290</b>

Therefore, capacity of the reservoir =  $33290 / (0.8 \times 0.88) = 47,300 \text{ ha-m}$

## Factors affecting Duty

### 1. Soil Characteristics:

If the soil of the canal bed is porous and coarse grained, it leads to more seepage loss and consequently low duty. If it consists of alluvial soil, the percolation loss will be less and the soil retains the moisture for longer period and consequently the duty will be high.

### 2. Climatic Condition:

When the temperature of the command area is high the evaporation loss is more and the duty becomes low and vice versa.

### 3. Rainfall:

If rainfall is sufficient during the crop period, the duty will be more and vice versa.

## Factors affecting Duty

### 4. Base Period:

When the base period is longer, the water requirement will be more and the duty will be low and vice versa.

### 5. Type of Crop:

The water requirement for various crops is different. So the duty varies from crop to crop.

### 6. Topography of Agricultural Land:

If the land is uneven the duty will be low. As the ground slope increases the duty decreases because there is wastage of water.

### 7. Method of Ploughing:

Proper deep ploughing which is done by tractors requires overall less quantity of water and hence the duty is high.

## Factors affecting Duty

### 8. Methods of Irrigation:

The duty of water is high in case of perennial irrigation system as compared to that in inundation irrigation system.

### 9. Water Tax:

If some tax is imposed the farmer will use the water economically thus increasing the duty.

### Importance of Duty

It helps us in designing an efficient canal irrigation system. Knowing the total available water at the head of a main canal,

*If we know the crops area required to be irrigated and their duties, we can work out the discharge required for designing the channel.*

### METHODS OF IMPROVING DUTY

When once the various factors affecting duty are properly understood, the duty can be improved by making those factors less effective which tend to reduce the duty.

1. Suitable *method of applying water* to the crops should be used.
2. The *land should be properly ploughed and leveled* before sowing the crop. It should be *given good tith*.
3. The *land should be cultivated frequently*, since frequent cultivation *reduces loss of moisture* specially when the ground water is within capillary reach of ground surface.
4. The *canals should be lined*. This *reduces seepage and percolation losses*. Also, water can be conveyed quickly, thus reducing, *evaporation losses*.
5. *Parallel canals* should be constructed. If there are two canals running side by side, the *F.S.L. will be lowered*, and the *losses will thus be reduced*.
6. The *idle length* of the canal should be reduced.
7. The *alignment* of the canal either in sandy soil or in fissured rock should be avoided.
8. The canal should be so aligned that the areas to be cultivated are concentrated along it.

9. The source of supply should be such that it gives *good quality of water*.
10. The rotation of crops must be practiced.
11. Volumetric method of assessment should be used.
12. The farmers must be trained in the proper use of water, so that they apply correct quantity of water at correct timing.
13. The land should be redistributed to the farmers so that they get only as much land as they are capable of managing it.
14. Research stations should be established in various localities *to study the soil, the seed and conservation of moisture*. The problems concerning the *economical use of water* should be studied at research stations.
15. The canal administrative staff should be efficient, responsible and honest. The operation of the canal system should be such that the farmers both at the head of the canal as well as at the tail end get water as and when they need it.

## Crop Water Requirements

Water requirements of a crop mean the total quantity and the way in which a crop requires water from the time it is sown to the time it is harvested. It is the quantity of water utilized by the plant during its life time. This water may be supplied either entirely by rainfall, entirely by irrigation or by a combination of both.

### **Reference Crop Evapotranspiration ( $ET_o$ ):**

The rate of evapotranspiration from an extensive surface of 8 to 15 cm tall, green grass cover of uniform height, actively growing, completely shading the ground and not short of water is known as reference crop evapotranspiration ( $ET_o$ )

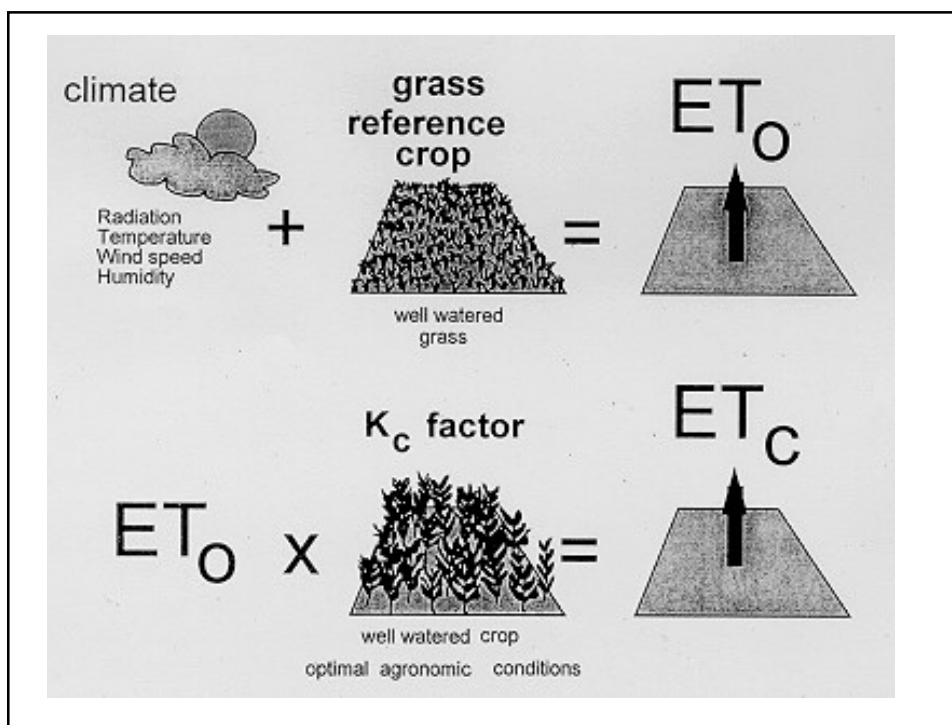
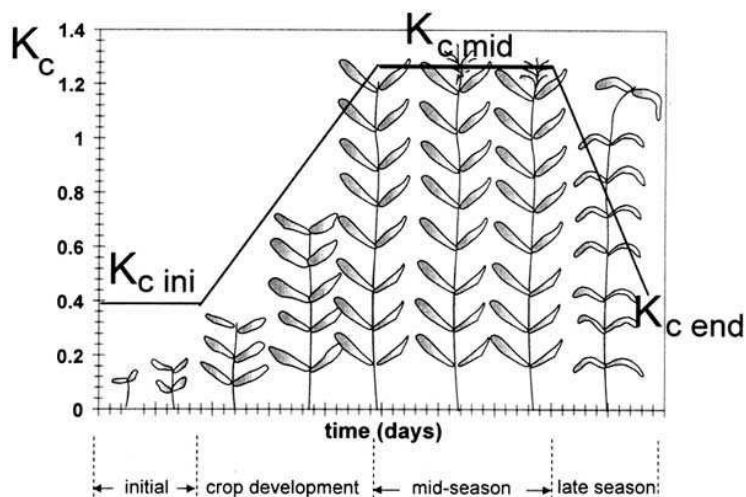
### **Crop Water Requirement (CWR) or Crop Evapotranspiration ( $ET_c$ or $ET_{crop}$ ):**

It is the depth of water need to meet the water loss through evapotranspiration of a disease free crop, growing in large field without restriction in soil conditions including water and fertility, having good production potential with particular growing environment.

### Crop Co-efficient (kc):

The ratio of crop evapotranspiration (ET<sub>c</sub>) to the reference crop evapotranspiration (ET<sub>o</sub>) is called Crop co-efficient (k<sub>c</sub>).

$$k_c = ET_c / ET_o$$



### Estimation of reference crop $ET_0$

The commonly used methods are two:

- Experimental methods, using the experimentation data from evaporation pan.
- Theoretical methods using empirical formulae, that take into account, climatic parameters

#### *i) Experimental method*

Estimation of  $ET_0$  can be made using the formula

$$ET_0 = K_{pan} \times E_{pan}$$

where,  $ET_0$  = the **reference crop evapotranspiration** in mm/day,

$K_{pan}$  = a coefficient called **pan coefficient**, and

$E_{pan}$  = the **evaporation** in mm/day from the pan.

The factor  $K_{pan}$  varies with the position of the equipment (say, whether placed in a fallow area or a cropped area), humidity and wind speed. Generally, the details are supplied by the manufacturers of the pan. For the **US Class A evaporation pan**,  $K_{pan}$  varies between 0.35 and 0.85, with an average value of 0.7.

### Estimation of reference crop $ET_0$

#### *i) Experimental method*

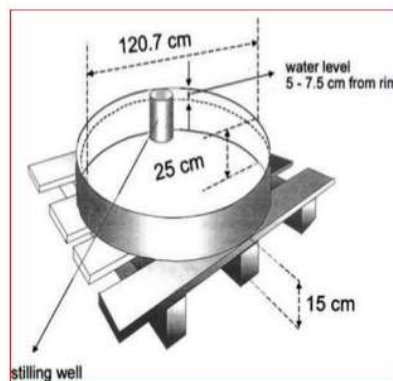
It may be noticed that finding out  $ET_c$  would involve the following expression

$$ET_c = K_{crop} \times ET_0 = K_c \times E_{pan} \times K_{pan}$$

If instead,  $K_{crop} \times K_{pan}$  is taken as a single factor, say  $K$ , then  $ET_c$  may directly be found from  $E_{pan}$  as under:

$ET_c = K \times E_{pan}$ , where  $K$  may be called the crop factor

#### ***Class A Pan***



### Estimation of reference crop $ET_0$

#### **ii) Theoretical methods**

The important methods that have been proposed over the years take into account, various climatic parameters. The most commonly used methods are as follows.

#### **a) Blanney-Criddle formula:**

This formula gives an estimate of the mean monthly values of  $ET_0$ , which is stated as

$$ET_0 = p (0.46 T_{mean} + 8.13)$$

where,  $T_{mean}$  = the mean daily temperature in degree centigrade over the month considered and may be taken as  $\frac{1}{2} (T_{max} + T_{min})$  for a particular month;

$p$  = the mean daily percentage of annual day time hours and has been estimated according to latitude

### Estimation of reference crop $ET_0$

#### **b) Radiation method:**

$$ET_0 = c(W R_s)$$

where  $ET_0$  = reference crop evapotranspiration (mm/day)

$R_s$  = solar radiation in equivalent evaporation (mm/day)

$c$  = adjustment factor which depends on the mean humidity and day-time wind conditions;

$W$  = weighting factor which depends on temperature and altitude

#### **c) Penman method:**

$ET_0$  could be calculated from Doorenbos and Pruitt (1977) version of Penman equation, known as Penman modified formula, as follows:

$$ET_0 = C[w R_n + (1-w).f(u).(e_a - e_d)]$$

where  $ET_0$  = reference crop evapotranspiration (mm/day)

$C$  = adjustment factor to compensate for the effect of day and night weather conditions

$w$  = temperature related weighting factor

$R_n$  = net radiation in equivalent evaporation (mm/day)

### Estimation of reference crop $ET_o$

**c) Penman method:**

$f(u)$  = wind related function

$e_a$  = actual vapor pressure at mean air temperature (mbar)

$e_d$  = saturation vapor pressure at mean air temperature (mbar)

**d) Penman-Monteith method:**

This method suggests that the value of  $ET_o$  may be evaluated by the following formula:

$$ET_o = \frac{0.408 \Delta(R_n - G) + \gamma \frac{900}{T + 273} u_2 (e_s - e_a)}{\Delta + \gamma(1 + 0.34u_2)}$$

where  $ET_o$  = reference evapotranspiration [ $\text{mm day}^{-1}$ ],

$R_n$  = net radiation at the crop surface [ $\text{MJ m}^{-2} \text{day}^{-1}$ ],

$G$  = soil heat flux density [ $\text{MJ m}^{-2} \text{day}^{-1}$ ],

$T$  = mean daily air temperature at 2 m height [ $^{\circ}\text{C}$ ],

### Estimation of reference crop $ET_o$

**d) Penman-Monteith method:**

$u_2$  = wind speed at 2 m height [ $\text{m s}^{-1}$ ],

$e_s$  = saturation vapour pressure [kPa],

$e_a$  = actual vapour pressure [kPa],

$e_s - e_a$  = saturation vapour pressure deficit [kPa],

$\Delta$  = slope vapour pressure curve [ $\text{kPa } ^{\circ}\text{C}^{-1}$ ],

$g$  = psychrometric constant [ $\text{kPa } ^{\circ}\text{C}^{-1}$ ]

**iii) Computer software**

CROPWAT is a decision support tool developed by the Land and Water Development Division of FAO. CROPWAT 8.0 for Windows is a computer program for the calculation of crop water requirements and irrigation requirements based on soil, climate and crop data. In addition, the program allows the development of irrigation schedules for different management conditions and the calculation of scheme water supply for varying crop patterns. CROPWAT 8.0 can also be used to evaluate farmers' irrigation practices and to estimate crop performance under both rainfed and irrigated conditions.

## Factors Affecting Crop-water Requirements

**The following are the factors which affect on the water requirements of the crops:**

### 1. Climate

In hot climate the evaporation loss is more and hence the water requirement will be more and vice versa.

### 2. Type of crop

Different crops require different amount of water for maturity

### 3. Water table

If the water table is nearer to the ground surface, the water requirement will be less & vice versa.

### 4. Ground Slope

If the slope of the ground is steep the water requirement will be more due to less absorption time for the soil.

### 5. Intensity of Irrigation

It is directly related to water requirement, the more the intensity greater will be the water required for a particular crop.

### 6. Conveyance Losses

Conveyance Losses take place from barrage/weir to the field (outlet). Major loss of water in an irrigation channel is due to absorption, seepage or percolation and evaporation. The absorption losses depend upon a) type of soil, b) subsoil water, c) age of canal, d) position of FSL with respect to NSL, e) amount of silt carried by canal, and f) wetted perimeter.

### 7. Method of Application of water

In sprinkler method less water is required as it just moist the soil like rainwater whereas in flood more water is required.

### 8. Method of Ploughing

In deep ploughing less water is required and vice versa.

### 9. Crop Period

The longer the crop period greater will be the water required for a particular crop.

### 10. Base Period:

The longer the base period greater will be the water required for a particular crop.

### 11. Delta of a crop:

The higher the delta greater will be the water required for a particular crop.

## Canal Losses

3. In the field  $\approx 27\%$

### 1. In Canal System

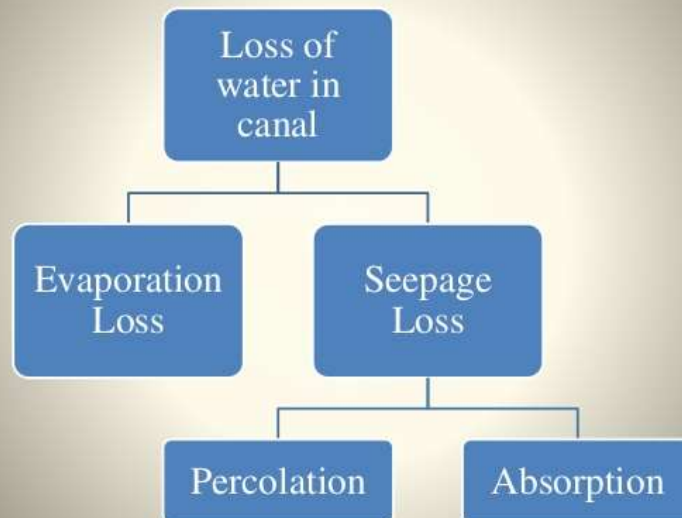
i) Canals  $\approx 17\%$  (vary between 15-20%)

ii) Distributaries  $\approx 8\%$  (vary between 6-8%)

2. In water courses  $\approx 20\%$  (vary between 17-22%)

## Water Losses due to Seepage and Evaporation

### Types of losses of water in canals



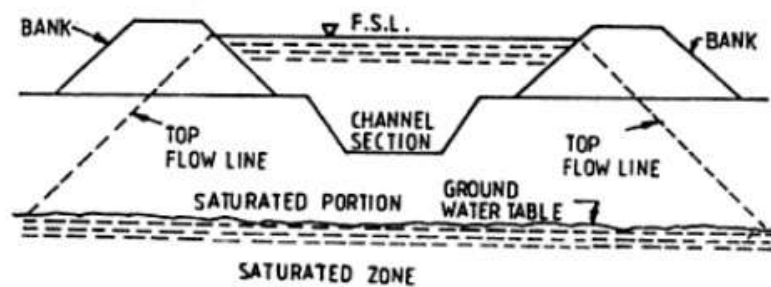
## Evaporation Loss

- The water lost by evaporation is generally very small, as compared to seepage loss.
- Evaporation Loss are generally 2-3% of total loss (max. 7% in summer)

## Seepage Loss

### Percolation:

- In percolation, there exist a zone of continuous saturation from canal to water table and direct flow is established.

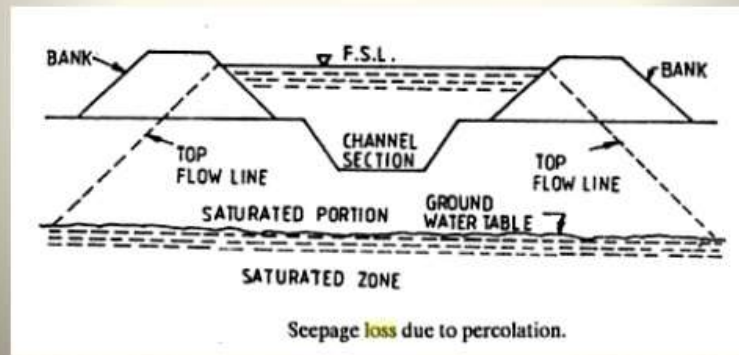


Seepage loss due to percolation.

## Seepage Loss

### Percolation:

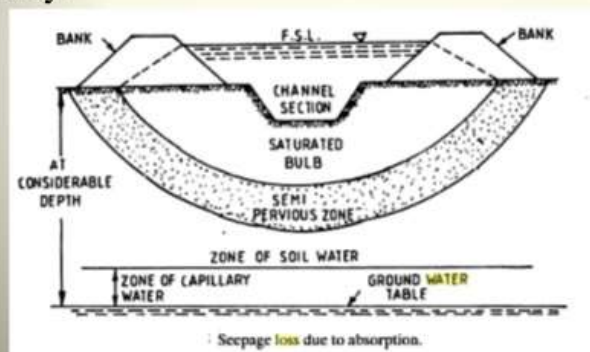
- Almost all water lost from canal reaches ground water reservoir.
- Loss of water depends on the difference of the top water surface level of channel and level of water-table.



## Seepage Loss

### Absorption:

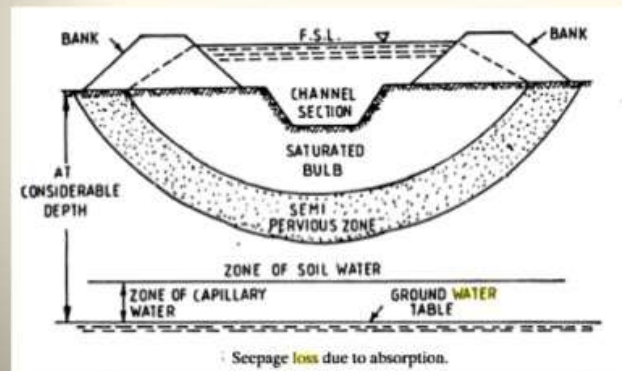
- In absorption, a small saturated zone exist round the canal section and is surrounded by zone of decreasing saturation.
- A certain zone just above water table is saturated by capillarity.



## Seepage Loss

### Absorption:

- Thus, there exists an unsaturated soil zone between two saturated zones.
- This result in seepage loss.



## Irrigation Efficiencies

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

### 1. Water Conveyance Efficiency ( $\eta_c$ ):

It is defined as the ratio of the irrigation water supplied at outlets to the field  $W_f$  and the irrigation water supplied at diversion (river or reservoir)  $W_r$ . It accounts the conveyance or transit losses into account.

$$\eta_c = W_f / W_r$$

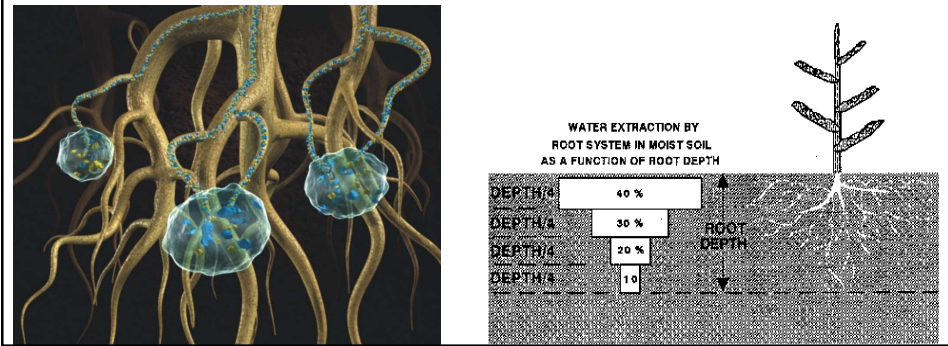
## 2. Water Application Efficiency ( $\eta_a$ ):

It is defined as the ratio of amount of irrigation water stored into the root zone of the crop  $W_s$  to the amount of water supplied to the field  $W_f$ . It may also be termed as farm efficiency, as it takes into account the water lost in the farm.

$$\eta_a = W_s / W_f = [W_f - (R_f + D_f)] / W_f$$

where,  $R_f$  is the surface runoff and

$D_f$  is the deep percolation



**4. Water Use Efficiency ( $\eta_u$ ):** It may be defined as the ratio between amount of water beneficially used including leaching  $W_u$  to the amount of water delivered to the field  $W_f$ .

$$\eta_u = W_u / W_f$$

**5. Water Storage Efficiency ( $\eta_s$ ):** It may be defined as the ratio of amount of water stored in the root zone during irrigation  $W_s$  to the amount of water needed in the root zone for irrigation (i.e., field capacity – existing moisture content)  $W_n$ .

$$\eta_s = W_s / W_n$$

**6. Consumptive Use Efficiency ( $\eta_{cu}$ ):** It may be defined as the ratio between normal consumptive use of the water  $W_{cu}$  to the net amount of water depleted from the root zone of soil  $W_d$ .

$$\eta_{cu} = W_{cu} / W_d$$

### 6. Uniformity Coefficient or Distribution Efficiency ( $\eta_d$ ):

The effectiveness of irrigation may also be measured by its water distribution efficiency ( $\eta_d$ ), which may be expressed mathematically as

$$\eta_d = (1 - \bar{y}/\bar{d})$$

where,  $\bar{y}$  = average numerical deviation in depth of water stored from average depth stored during the irrigation

$\bar{d}$  = average depth of water stored during the irrigation

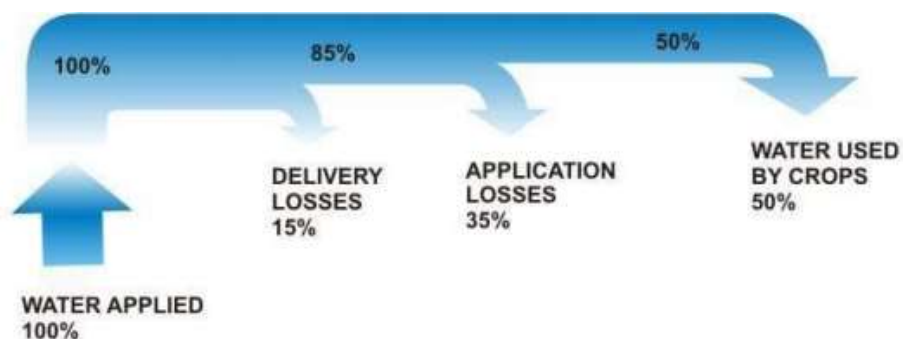
The water distribution efficiency represents the extent to which the water has penetrated to a uniform depth, throughout the field. When the water has penetrated uniformly throughout the field, the deviation from the mean depth is zero and water distribution efficiency is 1.0.

### Project Efficiency or Overall Efficiency ( $\eta_o$ ):

It may be defined as the irrigation water evapotranspired by the crop  $W_{cu}$  to the irrigation water supplied at the diversion point  $W_r$ .

$$\eta_o = W_{cu} / W_r$$

Overall efficiency is the product of all efficiencies and takes into account total amount of losses incurred.



**Example**

The depths of penetrations along the length of a border strip at points 30 m apart were probed. There observed values are: 2.0, 1.9, 1.8, 1.6 and 1.5. compute the water distribution efficiency.

**Solution:**

$$\text{Mean depth} = D = \frac{2.0+1.9+1.8+1.6+1.5}{5} = 1.76 \text{ m}$$

Penetration Depths	2	1.9	1.8	1.6	1.5
Deviation from Mean	0.24	0.14	0.04	-0.16	-0.26
Abs. Value of Dev. from Mean	0.24	0.14	0.04	0.16	0.26

$$\text{Mean of Abs. Values of Dev. from Mean} = d = \frac{0.24+0.14+0.04+0.16+0.26}{5} = 0.168 \text{ m}$$

$$\begin{aligned} \therefore \text{The Water Distribution Efficiency} &= \left(1 - \frac{d}{D}\right) \\ &= \left(1 - \frac{0.168}{1.76}\right) \\ &= 0.905 \\ &= 90.5\% \quad \text{Answer} \end{aligned}$$

**Example**

A stream of 135 litres per second was diverted from a canal and 100 litres per second were delivered to the field. An area of 1.6 hectares was irrigated in 8 hours. The effective depth of root zone was 1.8 m. the runoff loss in the field was 432 cu.m. The depth of water penetration varied linearly from 1.8 m at the head end of the field to 1.2 m at the tail end. Available moisture holding capacity of the soil is 20 cm per meter depth of soil. Determine the water conveyance efficiency, water application efficiency, water storage efficiency and water distribution efficiency. Irrigation was started at a moisture extraction level of 50 percent of the available moisture.

**Solution:**

(i) Water conveyance efficiency,

$$\eta_c = \frac{W_f}{W_d} \times 100 = \frac{100}{135} \times 100 = 74\%$$

(ii) Water application efficiency,

$$\eta_a = \frac{W_s}{W_f} \times 100$$

Water delivered to the plot

$$= \frac{100 \times 60 \times 60 \times 8}{1000} = 2880 \text{ cu.m}$$

Water stored in the root zone  
 $= 2880 - 432 = 2448 \text{ cu.m}$

Water application efficiency

$$= \frac{2448}{2880} \times 100 = 85\%$$

(iii) Water storage efficiency,

$$\eta_s = \frac{W_s}{W_n} \times 100$$

Water holding capacity of the zone

$$= 20 \times 1.8 = 36 \text{ cm}$$

Moisture required in the root zone

$$= 36 - \frac{36 \times 50}{100} = 18 \text{ cm}$$

$$= \frac{18}{100} \times 1.6 \times 10,000 = 2880 \text{ cu.m}$$

$$\text{Water storage efficiency} = \frac{2448}{2880} \times 100 = 85\%$$

(iv) Water distribution efficiency

$$\eta_d = \left(1 - \frac{\bar{y}}{d}\right) 100$$

$$\bar{d} = \frac{1.8 + 1.2}{2} = 1.5 \text{ m}$$

Numerical deviation from depth of penetration:

$$\text{At upper end} = 1.8 - 1.5 = 0.3$$

$$\text{At lower end} = 1.5 - 1.2 = 0.3$$

$$\text{Average numerical deviation, } \bar{y} = \frac{0.3 + 0.3}{2} = 0.3 \text{ m}$$

$$\eta_d = \left(1 - \frac{0.3}{1.5}\right) 100$$

$$= 80\%$$

## Effective Rainfall

### Effective Rainfall (Re):

Precipitation falling during the growing period of a crop that is available to meet the evapotranspiration needs of the crop is called effective rainfall.

It is that part of rainfall which is available to meet ET needs of the crop.

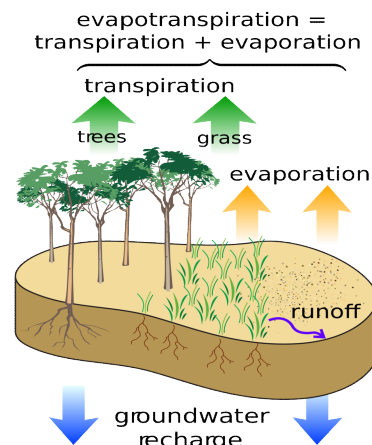
$$Re = R - Rr - Dr$$

where,  $R$  = Precipitation

$Rr$  = Surface runoff

$Dr$  = Deep percolation

Generally a percentage of total rainfall is taken as effective rainfall



## Irrigation Water Requirements

### **Operational water requirement:**

It is the water used for land preparation, percolation losses in flooded paddy fields. For irrigation water requirement, it must be supplied in addition to the crop water requirement.

### **Field water requirement:**

It is the sum of crop water requirement and the operational water requirement.

### **Consumptive Irrigation Requirement (CIR):**

It is defined as the amount of irrigation water that is required to meet the evapotranspiration needs of the crop during its full growth.

$$CIR = C_u - R_e$$

where  $C_u$  is the consumptive use of water.

### **Net Irrigation Requirement (NIR):**

It is defined as the amount of irrigation water required at the plot to meet the evapotranspiration needs of water as well as other needs such as leaching etc.

$NIR = C_u - R_e + \text{Water lost in deep percolation for the purpose of leaching etc.}$

### **Field Irrigation Requirement (FIR):**

It is the amount of water required, to meet net irrigation requirements, plus the water lost in percolation in the field water courses, field channels and in field applications of water.

$$FIR = NIR + \text{water application losses} = NIR/\eta_a$$

where  $\eta_a$  is the water application efficiency.

### **Gross Irrigation Requirement (GIR):**

It is the sum of water required to satisfy the field irrigation requirement and the water lost as conveyance losses in distributaries up to the field.

$$GIR = FIR + \text{Conveyance loss} = FIR/\eta_c$$

where  $\eta_c$  is the water conveyance efficiency.

### **Flow Chart to Estimate Crop Water Requirement:**

- Step-1: Decide future cropping pattern
- Step-2: Calculate 10/15 days  $ET_o$  values using standard method
- Step-3: Calculate 10/15 days  $K_c$  (crop coefficients) from standard table
- Step-4: Calculate  $ET_c$  for 10/15 days basis
- Step-5: Allow for land preparation (rice only)
- Step-6: Allow for deep percolation (rice only)
- Step-7: Calculate evaporation from land preparation (rice only)
- Step-8: Calculate total crop water requirements
- Step-9: Calculate effective rainfall
- Step-10: Calculate net crop water requirements

### **Flow Chart to Estimate Irrigation Requirement:**

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  - Step-7: Calculate evaporation from land preparation (rice only)
  - Step-8: Calculate total crop water requirements
  - Step-9: Calculate effective rainfall
  - Step-10: Calculate net crop water requirements
  - Step-11: Allow for field efficiency
  - Step-12: Calculate field crop water requirements
  - Step-13: Calculate canal system efficiency
  - Step-14: Calculate intake water requirements
- Irrigation Water Need =  $ET_c$  + Land Prepration + Evaporation + Deep Percolation – Effective Rainfall

## Soil-Moisture-Irrigation Relationship

### Forms of soil water (Availability of soil water)

#### 1. Gravitational or free water:

This form of water is loosely held in the soil and could be easily lost by gravitational force.

#### 2. Capillary water

This form of soil water is held in the soil by capillary action (force) that is less than atmospheric pressure. It is the water available for plant growth.

#### 3. Hygroscopic water

This is a form of soil water that is present not only in the pores but also on the surface of the soil particle. It is tightly held in the soil and cannot be removed except by oven drying at 105°C. It is the water not available for plant growth (unavailable water)

### Water-retaining properties of the soil

#### Soil Moisture Content

It is the amount of water in the soil (usually expressed in %).

#### Saturation Capacity/Level

When all pores of soil are filled with water, then it is called the saturation capacity of that particular soil. It is also called maximum moisture holding capacity.

#### Field Capacity (FC)

It is the moisture content of the soil when downward movement of water has vertically ceased. This condition usually exists in a well-drained soil about two or three days after rain or irrigation. It is closely related to soil texture and is influenced by the organic matter content, types of minerals present and soil structure.

### Wilting Point (WP)

It is the moisture content beyond which plants can no longer extract enough moisture and remain wilted unless water is added to the soil. The moisture tension at wilting is about or often equal to 15 atmosphere. It represents the lower limit of available moisture. It is also called **permanent wilting point (PWP)**.

### Two stages of wilting points are recognized

#### 1. Temporary Wilting Point (TWP)

It is the moisture content at which plants wilt during hot windy day but recover in the cooler portion of the day (say night) or when water is added to the soil.

#### 2. Ultimate Wilting Point (UWP)

It is the moisture content at which plants wilt and fail to regain life even after addition of water to the soil.

### Wilting Coefficient

It is the level of soil moisture at which water becomes unavailable to plants and permanent wilting ensues.

### Hygroscopic Coefficient

It is the percentage of water remaining in an air-dry soil.

### Available Moisture (AM)

It is the moisture holding capacity of a given undisturbed soil sample between FC and PWP (i.e., FC-PWP).

### Readily Available Moisture (RAM)

It is the portion of AM which can be easily extracted by plants. It is generally 75-80% of AM.

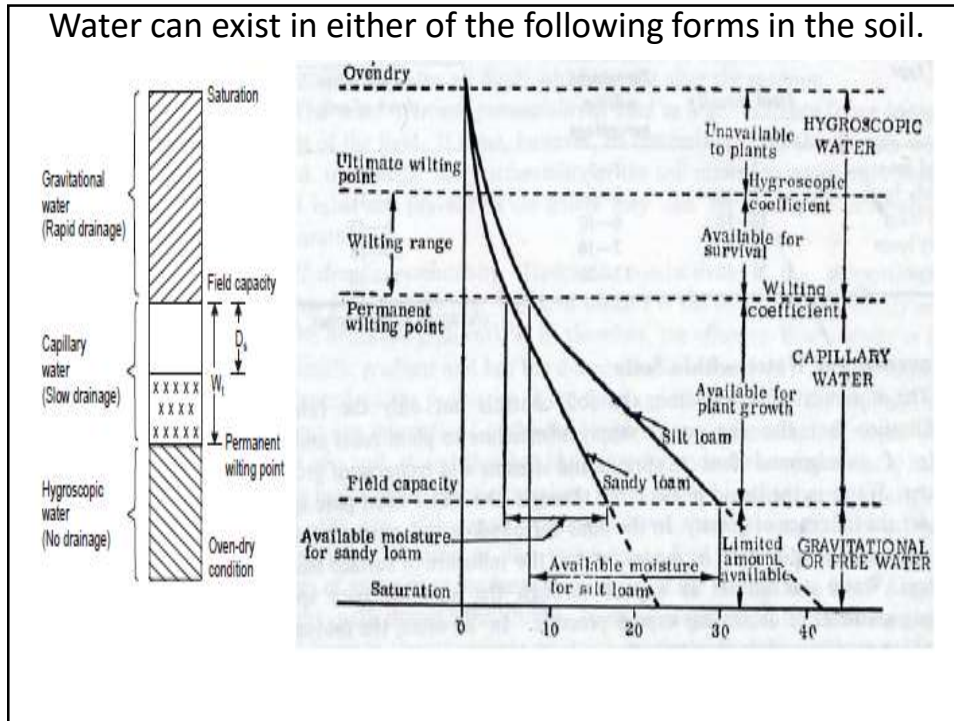
### Soil moisture deficiency (Field moisture deficiency)

It is the water required to bring the soil moisture content of a given soil to its FC.

### Optimum Moisture Content (OMC)

It is the moisture content in the soil corresponding to which optimum growth of plant takes place.

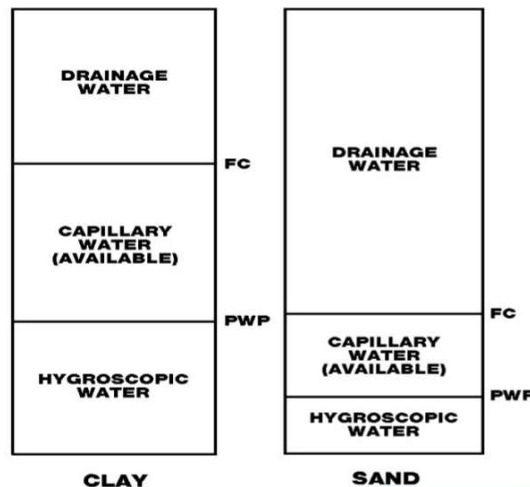
Water can exist in either of the following forms in the soil.

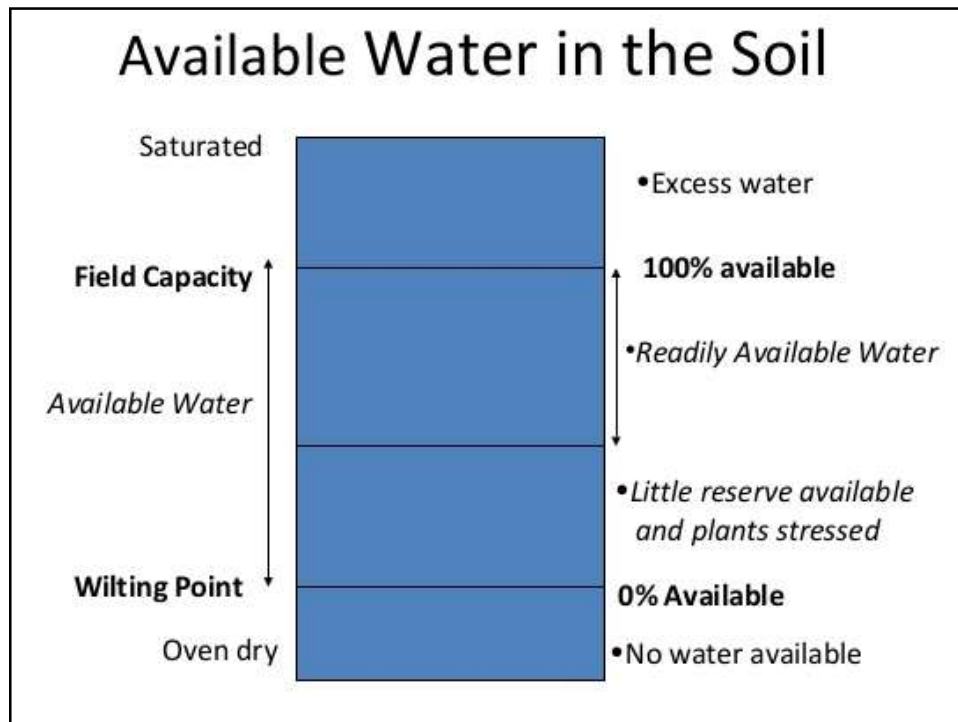


**3 Kinds of Soil Water or Soil Moisture:**

- a. drainage water;
- b. capillary water; and
- c. hygroscopic water.

**Different soil textures have different relative proportions of these 3 kinds of soil moisture.**





## Depth and Frequency of Irrigation

### Root zone depth ( $d$ )

It is the depth below the ground surface in which crops develop roots system to derive water for growth.

### Consumptive use ( $C_u$ )

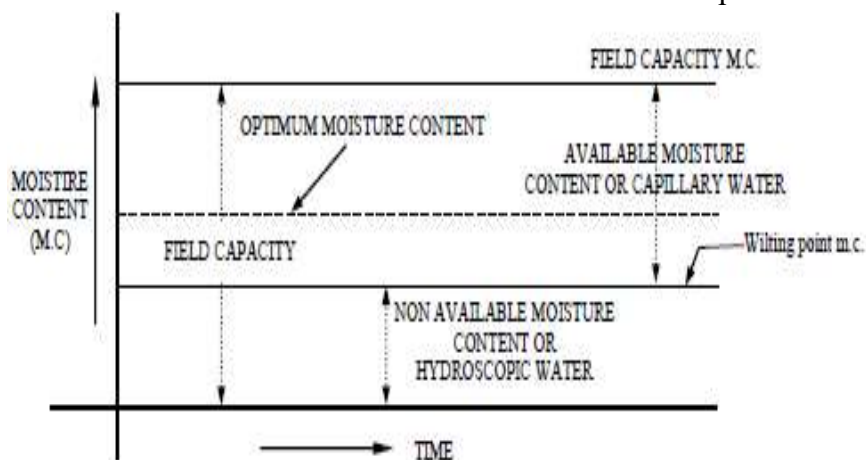
It is the total amount of water used by the plant in transpiration and evaporation from adjacent soils or levees in a specified time.  $C_u$  may be different for different crops, and may be different for same crop at different time and place.

### Frequency of Irrigation ( $f_w$ )

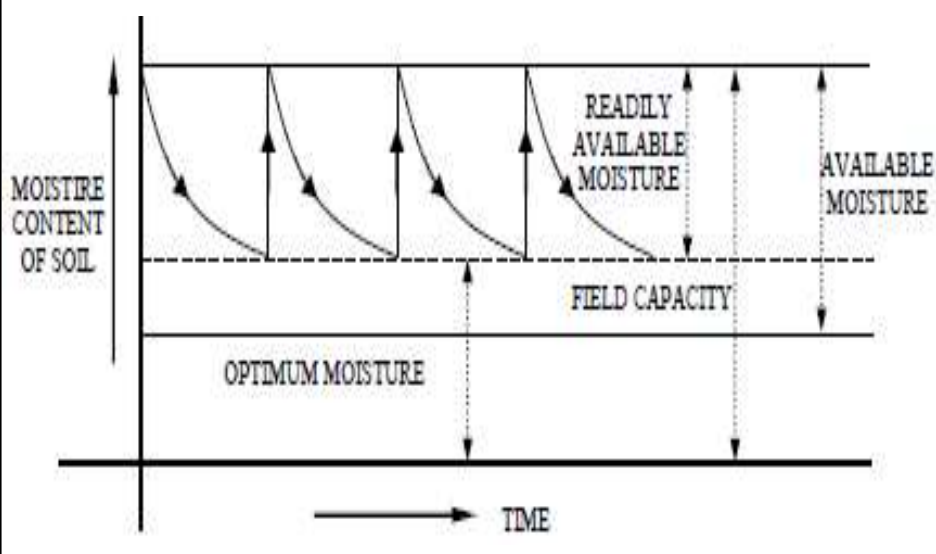
It is the time lag between the two adjacent supply of water to soil in order to ensure sufficient irrigation of the given crop

### Estimating depth and frequency of irrigation on the basis of soil moisture regime concept

Water or soil moisture is consumed by plants through their roots. It, therefore, becomes necessary that sufficient moisture remains available in the soil from the surface to the root zone depth.

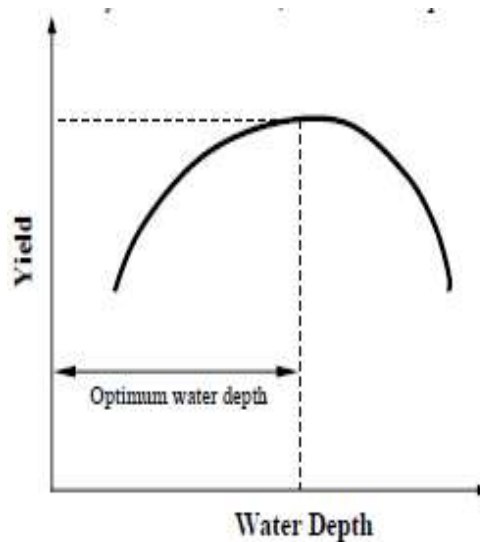


The irrigation water should be supplied as soon as the moisture falls up to this optimum level and its quantity should be just sufficient to bring the moisture content up to its field capacity,



### Optimum Utilization of Irrigation Water

- Yield is going to vary with the application of different quantities of water.
- The yield increases with water, reaches maximum value and then falls down.
- *The quantity of water at which the yield is maximum, is called the optimum water depth. Optimum utilization of irrigation generally means, getting maximum yield with any amount of water.*



### Depth of water stored in root zone.

In order to estimate the depth of water stored in the root zone of soil containing water up to field capacity, let,

$d$  = depth of root zone (in metres) ;  $F_c$  = field capacity (expressed as ratio);

$\gamma$  = unit weight of soil; and  $\gamma_w$  = unit weight of water.

- Consider 1 sq. m area of soil having 'd' meter depth.

Volume of soil = 1 x d = '**d**' cubic m

Dry unit weight of soil is =  $\gamma_d$  **KN/cub. m**

then, wt. of d cubic meter soil is  $\gamma_d$  **d KN**

**F.C. (F) = Wt. of water retained in unit area of soil**

$\gamma_d$  **d**

Wt. of water retained in unit area of soil =  $\gamma_d$  **d F KN/sq. m**

If  $\gamma_w$  = unit wt. of water per unit volume **KN/cub. m**

then ,

Volume of water stored in unit are of soil =  $\frac{\gamma_d d F}{\gamma_w}$  **KN/sq. m**  
 $\text{KN/cub. M}$

Hence the depth of water stored in the root zone in filling the soil upto field capacity

$$= \frac{\gamma_d d F}{\gamma_w} \text{ meters.}$$

### Consumptive Use ( $C_u$ )

The consumptive use of irrigation water is the quantity of actually water required by the plant. The combine process of Evaporation and transpiration (Evapotranspiration) is a consumptive use. It is use of water by a crop is the depth of water consumed by evaporation and transpiration during crop growth.

**Frequency of irrigation** is calculated by dividing the amount of soil moisture which may be depleted (*i.e.*, allowable depletion below field capacity and well above permanent wilting point) within the root-zone soil by the rate of consumptive use.

Thus, **Frequency of irrigation** = Allowable soil moisture depletion / Rate of consumptive use

### Irrigation Schedule:

It is a decision making process involving:

When to irrigate?

How much water to apply each time?

How to apply (method of irrigation)?

Find the field capacity of a soil for the following data:

Root zone depth	=	2 m
Existing water content	=	5%
Dry density of soil	=	1.5 g/cm <sup>3</sup>
Water applied to the soil	=	500 m <sup>3</sup>
Water loss due to evaporation etc	=	10%
Area of plot	=	1000 m <sup>2</sup>

**Solution:**

Total water applied = 500 m<sup>3</sup>

Loss of water = 10%

Hence, Water used in the soil = 90% x 500 = 450 m<sup>3</sup> = 450 x 10<sup>6</sup> cm<sup>3</sup>  
= 450 x 10<sup>6</sup> gm

Total dry weight of soil = (1000 m<sup>2</sup> x 2 m) x 1.5 x 10<sup>6</sup> = 3 x 10<sup>9</sup> gm

$$\% \text{ of water added} = \frac{450 \times 10^6}{3 \times 10^9} \times 100$$

Hence, New water content = 5% + 15% = 20%

After how many days will you supply water to soil (clay loam) in order to ensure efficient irrigation of the given crop, if:

Field capacity of the soil	=	27%
Permanent wilting point	=	14%
Density of soil	=	1.5 g/ cm <sup>3</sup>
Effective depth of root zone	=	75 cm
Daily consumptive use of water for the given crop	=	11 mm

**Solution:**

Available moisture = Field capacity – Permanent wilting point  
= 27 – 14 = 13%

Let, the Readily available moisture be 80% of the Available moisture.

Therefore, Readily available moisture = 0.8 x 13% = 10.4%

$$\begin{aligned}\text{Optimum moisture} &= \text{Field capacity} - \text{Readily available moisture} \\ &= 27 - 10.4 = 16.6\%\end{aligned}$$

Hence, when irrigation water is applied, moisture content is raised from 16.6 to 27%.

$$\begin{aligned}\therefore \text{Depth of water stored in root zone, during each watering} \\ &= \frac{\gamma_d \cdot d}{\gamma_w} [F.C. - OMC] = \frac{1.5 \times 0.75}{1} [0.27 - 0.166] = 0.117 \text{ m} \\ &= 11.7 \text{ cm}\end{aligned}$$

Thus, depth of water available for Evapo-transpiration = 11.7 cm

Since, daily consumptive use = 1.1 cm

$$\text{Hence, Watering frequency} = \frac{11.7}{1.1} \approx 10 \text{ days}$$

Therefore, water should be applied after every 10 days.

A loam soil has a field capacity of 22% and wilting coefficient of 10%. The dry unit weight of soil is 1.5 g/cm<sup>3</sup>. If the root zone depth is 70 cm, determine the storage capacity of the soil. Irrigation water applied when moisture content falls to 14%. If water application efficiency is 75%, determine the water depth required to be applied in the field.

**Solution:**

$$\begin{aligned}\text{Maximum storage capacity} &= \text{Available moisture} \\ &= \frac{\gamma_d \cdot d}{\gamma_d} [F.C. - \text{Wilting Coef.}] = \frac{1.5 \times 0.70}{1} [0.22 - 0.10] \\ &= 0.126 \text{ m} = 12.6 \text{ cm}\end{aligned}$$

$$\begin{aligned}\text{Depth of irrigation water} &= \frac{\gamma_d \cdot d}{\gamma_d} [F.C. - OMC] \\ &= \frac{1.5 \times 0.70}{1} [0.22 - 0.14] = 0.084 \text{ m} = 8.4 \text{ cm}\end{aligned}$$

Therefore, Field irrigation requirement = 8.4/0.75 = 11.2 cm

# CANAL IRRIGATION SYSTEM

## Classification of Canals

### Canal:

A canal is defined as an artificial channel constructed on the ground to carry water from a river or another canal or a reservoir to the fields.

Canals are usually open channels through earth or rock formations, and have a trapezoidal, rectangular, triangular cross-section.

### A. Classification of canals based on the function of canal

Classification of canals on the basis of their functions are given below:

- |                       |                   |
|-----------------------|-------------------|
| 1. Irrigation canal   | 5. Sanitary canal |
| 2. Navigation canal   | 6. Carrier canal  |
| 3. Power canal        | 7. Link canal     |
| 4. Water supply canal | 8. Feeder canal   |

## **Classification of Canals**

1. Irrigation canals:

These are the canals which carry water to the fields.

2. Navigation canal:

These are the canals which are used to provide transport and voyage facility.

3. Power canal:

These are the canals which are constructed to supply water for the purpose of generating electric power.

4. Water supply canal:

These are the canals which are constructed to supply water for public demand such as drinking purpose.

## **Classification of Canals**

5. Sanitary canal:

These are the canals which are constructed to supply water for sanitation purpose.

6. Carrier canal:

These are the canals which besides doing irrigation carry water for another canal.

7. Link canal:

These are the canals which are constructed to transfer water to the other conveyance structure which contains in-sufficient quantity of water.

8. Feeder canal:

These are the canals which are constructed with the idea of feeding two or more canals.

## **Classification of Canals**

### **B. Classification of canals based on financial output**

1. Productive canal
2. Protective canal

#### 1. Productive canal:

The canal which yields net revenue to the nation after full development of the irrigation in the area is known as productive canal.

#### 2. Protective canal:

The canal which is constructed with idea of protecting a particular area from famine is termed as protective canal.

### **C. Classification of canals based on nature of source**

1. Permanent canal
2. Inundation canal

## **Classification of Canals**

### **C. Classification of canals based on nature of source**

#### 1. Permanent canal:

A canal is said to be permanent when provision of a regular graded channel and masonry works for regulation and distribution is justified by a well assured source of supply. The canal which get water throughout the year are called permanent canal. It is taken from the source which is dry for the part of the year is called seasonal permanent canal.

#### 2. Inundation canal:

They draw their supply form the river when there is high stage in the river. They are not provided with any head works for diversion of river water to the canal.

## **Classification of Canals**

### **D. Classification of canals based on discharge and its relative importance**

1. Main canal (MC)
2. Branch canal (BC)
3. Distributary canal (DC)
4. Water courses (WC) or Field channels (FC)

#### 1. Main Canal (MC):

A main canal carries discharge directly from river. It carries large amount of water and cannot be used for direct irrigation. Main canal supplies water to the branch canals.

#### 2. Branch Canal (BC):

Canals having discharge not less than 5 cumecs are called as branch canals. These are the branches of main canal in either direction at regular intervals.

## **Classification of Canals**

### **D. Classification of canals based on discharge and its relative importance**

Branch canals also do not carry out direct irrigation but sometimes direct outlets are provided. Branch canals are actually the feeders for major and minor distributaries.

#### 3. Distributary canal:

These are the canals taking off from main or branch canal or from another distributary. These are feeders for another distributary or water courses for irrigation.

They are further divided into two types:

- i) Major Distributary or Distributary
- ii) Minor Distributary or Minor

## Classification of Canals

### D. Classification of canals based on discharge and its relative importance

#### 3. Distributary canal:

##### i) Major Distributary

These canals discharge varies from 5 - 0.25 cumecs.

##### ii) Minor Distributary

These canals have discharge less than 0.25 cumecs.

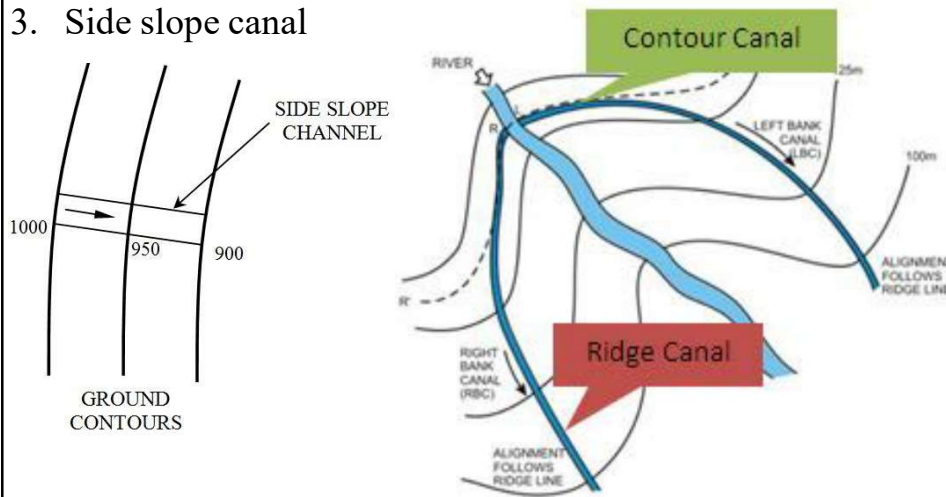
#### 4. Water courses or Field channels:

These are the small channels which ultimately feed water to the irrigation fields. The discharge in water courses is less than 0.25 cumecs. Depending upon the extent of irrigation, a field channel may take off from a major distributary or minor. Sometimes, it may even take off water from the branch canal for the field situated very near to the branch canal.

## Classification of Canals

### E. Classification of canals based on alignment

1. Contour canal
2. Watershed (Ridge) canal
3. Side slope canal



## **Classification of Canals**

### **E. Classification of canals based on alignment**

#### 1. Contour canal:

A contour canal is aligned almost parallel to the contours of the terrain. It can irrigate land only on one side because the land on the other side is higher. There are large number of cross drainage works because all the drainage are at right angles to the contour.

#### 2. Watershed (Ridge) canal:

The canal which is aligned along a watershed or ridge is called watershed canal. It can irrigate on both sides of the ridge by gravity, and, therefore, it has a large commanded area. Most of the drainages originate from the ridge and do not cross the canal, therefore number of cross drainage works is minimum.

## **Classification of Canals**

### **E. Classification of canals based on alignment**

#### 3. Side slope canal:

A side slop canal is a canal aligned at right angle to the contour line along the side slopes of the terrain. It does not normally intercept drainages, and therefore, no cross drainage work is required

### **F. Classification of canals based on canal surface**

#### 1. Lined canal

#### 2. Unlined canal

##### 1. Lined canal:

If the bed and sides of a canal are lined with impervious or fairly impervious material to cut down seepage losses, then it is called lined canal.

## **Classification of Canals**

### **F. Classification of canals based on canal surface**

#### 2. Unlined canal:

If the bed and sides of a canal are made up of existing natural material through which it traverses, then it is called unlined canal. The unlined canals may be further subdivided into the following type

- i) Alluvial canal
- ii) Non-alluvial canal

#### i) Alluvial canal

The soil which is formed by transportation and deposition of silt over a course of time is called the alluvial soil. The canals when excavated through such soils are called alluvial canals.

## **Classification of Canals**

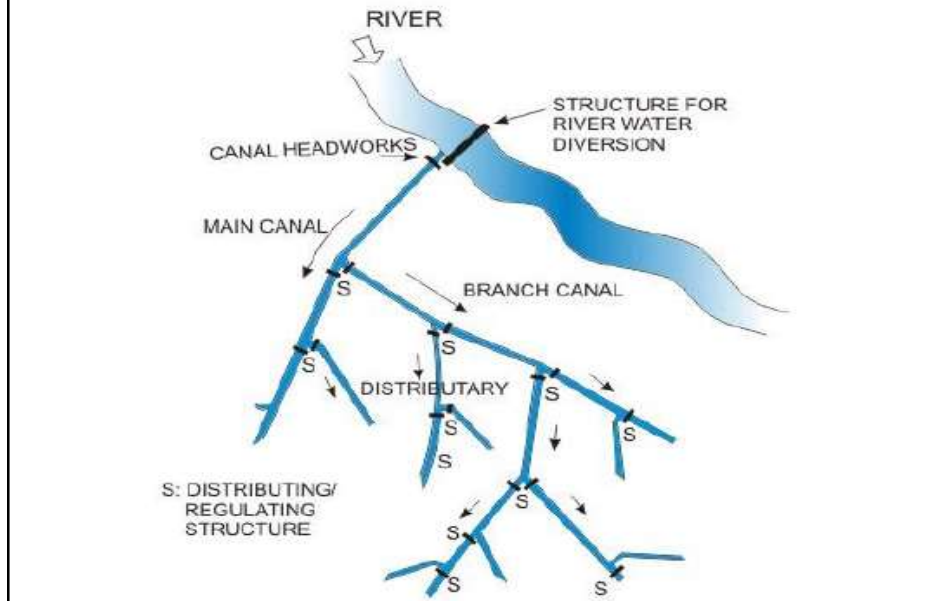
### **F. Classification of canals based on canal surface**

#### 2. Unlined canal:

#### ii) Non alluvial canal

Mountainous regions may go disintegrating over a period of time, resulting in the formation of a rocky plain area, called non-alluvial area. It has an uneven topography, and hard foundations are generally available. The rivers, passing through such areas, have no tendency to shift their courses, and they don't pose much problems for designing irrigation structures on them. Canals passing through such areas are called non-alluvial canals.

## Components of the Canal Irrigation System



## Factors to be considered for the alignment of a canal

1. An irrigation canal should be aligned in such a way that maximum area is irrigated with least length of canal.
2. Cross drainage works should be avoided as far as possible, such that the cost is reduced.
3. The off taking point of the canal from the source should be on a ridge, such that the canal must run as a ridge canal and irrigate lands on both sides.
4. Sharp curves in canals must be avoided.
5. In hilly areas, when it is not possible to construct ridge canals, the canal must be made to run as a contour canal.
6. The canal should be aligned such that the idle (blind) length of the canal is minimum.

### **Factors to be considered for the alignment of a canal**

7. The alignment should be such that heavy cutting or heavy filling are avoided. If possible balanced depth of cutting and filling is achieved.
8. It should not be aligned in rocky and cracked strata.
9. The alignment should avoid villages, roads, places of worship and other obligatory points.

### **Assessment of Irrigation Water**

The fixation of some nominal charges as levy on the farmers for using irrigation water is called assessment of irrigation water.

**The charges must be levied for the following reasons:**

- To recover the cost of construction of the irrigation project
- To recover the maintenance cost
- To collect some revenue for the nation
- To check the cultivators against uneconomical and careless use of water
- Planning for new canal

## Assessment of Irrigation Water

Assessment of irrigation water charges can be done in one of the following ways:

- (i) Assessment on area basis
- (ii) Volumetric assessment
- (iii) Assessment based on outlet capacity
- (iv) Permanent assessment
- (v) Consolidated assessment

### (i) Assessment on area basis

In the area basis method of assessment, water charges are fixed per unit area of land irrigated for each of the crops grown. The rates of water charges depend on the cash value of crop, water requirement of crop and the time of water demand with respect to the available supplies in the source.

## Assessment of Irrigation Water

Since the water charges are not related to the actual quantity of water used, the farmers (particularly those whose holdings are in the head reaches of the canal) tend to over irrigate their land. This results in uneconomical use of available irrigation water beside depriving the cultivators in the tail reaches of the canal of their due share of irrigation water.

### (ii) Volumetric assessment

In the volumetric assessment, the charges are in proportion to the actual amount of water received by the cultivator. This method, therefore, requires installation of water meters at all the outlets of the irrigation system.

## **Assessment of Irrigation Water**

This method results in economical use of irrigation water and is, therefore, an ideal method of assessment. But it has several drawbacks. This method requires installation and maintenance of suitable devices for water measurement. Further, there is a possibility of water theft by cutting of banks or siphoning over the bank through a flexible hose pipe. Also, the distribution of charges among the farmers, whose holdings are served by a common outlet, may be difficult.

### **(iii) Assessment based on outlet capacity**

Assessment of canal water charges based on outlet capacity is a simple method and is workable if the outlets are rigid or semi-modular and the channel may run within outlet's designed range of parameters.

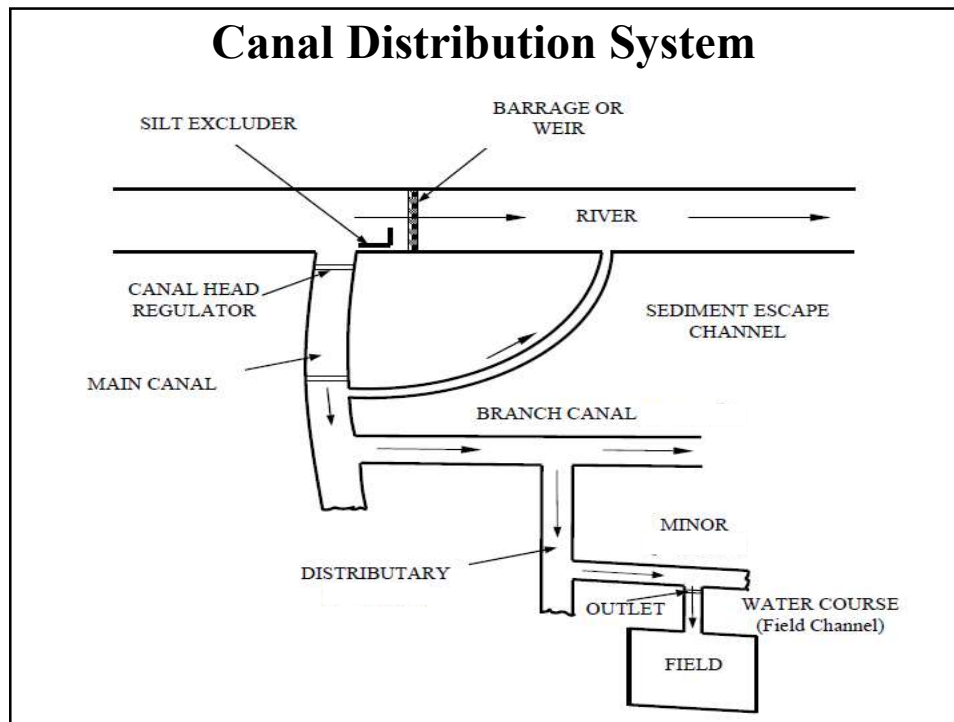
## **Assessment of Irrigation Water**

### **(iv) Permanent assessment**

In some regions, artificial irrigation, though not essential, has been provided to meet the water demand only in drought years. Every farmer of such a region has to pay a fixed amount. The farmers have to pay these charges even for the years in which they do not take any water.

### **(v) Consolidated assessment**

In the consolidated assessment method, both the land revenue and the water charges are combined, and the cultivators are accordingly charged.



## Canal Standards and Balancing Canal Depth

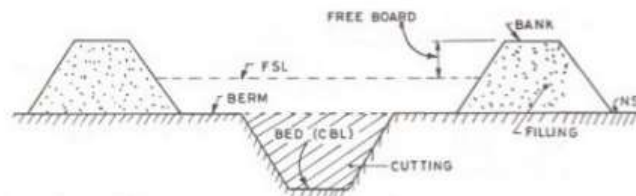
### *Cross-section of an Irrigation Canal*

A typical and most desired section of a canal, i.e. partly in cutting and partly in filling, is shown in the following figure.

NSL = Natural Surface Level

CBL = Canal Bed Level

FSL = Full Supply Level



When NSL is above the top of the bank, the canal section will have to be cut, and it shall be called “*canal in cutting*”.

When NSL is lower than the CBL, the canal section will have to be built in filling, and it is called “*canal in filling*”.

### Side Slopes

Side slopes (H:V) are fixed according to stability requirements, depending upon the type of the soil (angle of repose of soil).

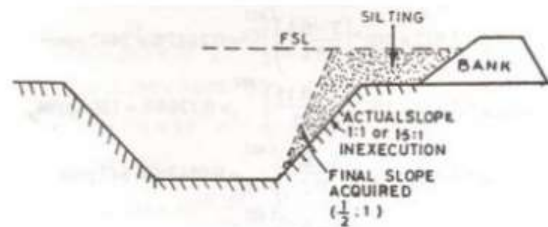
A comparatively steeper slope can be provided in cutting because soil is naturally consolidated, and hence, more stable.

Generally adopted slopes:

In cutting: 1:1 to 1½:1

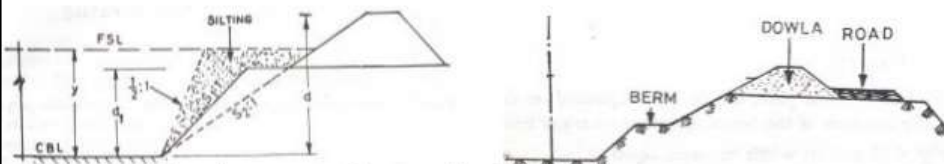
In filling: 1½:1 to 2:1

For channels with silt laden water, the actual capacity of the channel is worked out with ½ :1 side slopes.



### Berms

➤ Berm is the horizontal distance left at ground level between the toe of the bank and the top edge of cutting.



➤ If  $s_1:1$  is the slope in cutting and  $s_2:1$  in filling, then the initial berm width =  $(s_2 - s_1) d_1$

➤ Since NSL fluctuates considerably, while canal bed level (CBL) varies very slightly,  $d_1$  shall vary; and, therefore, the berm width shall vary.

➤ After the water flows in the channel for some time, the silt gets deposited on the sides giving them a slope of ½:1.

➤ The position of the berm, therefore, changes from ground level to FSL, as shown in Fig. and its width becomes equal to  $(s_2 - \frac{1}{2}) \cdot y$ . If  $s_2 = 1 \frac{1}{2}$  then the final berm width =  $y$ , i.e. equal to the depth of the canal.

The berms when fully formed, serve the following purposes :

- (i) The silt deposited on the sides is very fine and impervious. It, therefore, serves as a good lining for reducing losses, leakage and consequent breaches, etc.
- (ii) They help the channel to attain regime conditions, as they help in providing a wider waterway, if required. Even fluctuations of discharge do not produce much fluctuations in depths because of wider waterway.
- (iii) They give additional strength to the banks and provide protection against erosion and breaches.
- (iv) The possibility of breaches gets reduced because the saturation line comes more in the body of the embankment.
- (v) They protect the banks from erosion due to wave action.
- (vi) They provide a scope for future widening of the canal.
- (vii) Berms can be used as borrow pits for excavating soil to be used for filling.

### ***Freeboard***

The margin between FSL and bank level is known as freeboard.

The amount of freeboard depends upon the size of the channel. The generally provided values of freeboard are given in following table.

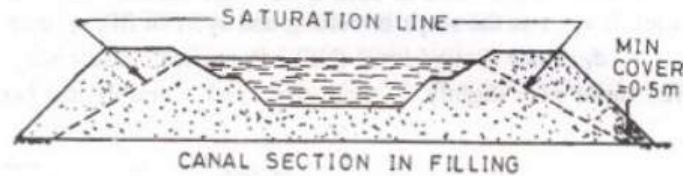
Values of Freeboard in Canals	
Discharge (cumec)	Extent of Free board (m)
1 to 5	0.5
5 to 10	0.6
10 to 30	0.75
30 to 150	0.90

### ***Banks***

The primary purpose of banks is to retain water. They can be used as means of communication and as inspection paths.

They should be wide enough, so that a minimum cover of 0.5 m is available above the saturation line, as shown in below Fig.

High banks will have to be designed as earth dams.



### ***Service Roads***

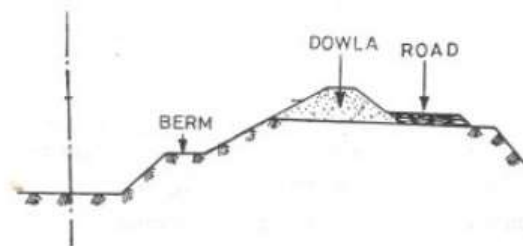
Service roads are provided on canals for inspection purposes, and may simultaneously serve as the means of communication in remote areas.

They are provided 0.4 m to 1.0 m above FSL, depending upon the size of the channel.

### ***Dowlas***

As a measure of safety in driving, dowlas 0.3 m high and 0.3 to 0.6 m wide at top, with side slopes of 1:1 to 2:1, are provided along the banks, as shown in below figure.

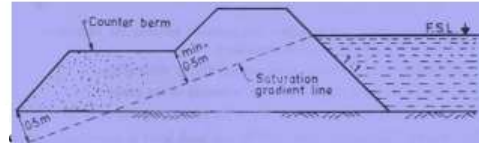
They also help in preventing slope erosion due to rains, etc.



### ***Back Berm or Counter Berms.***

Even after providing sufficient section for bank embankment, the saturation gradient line may cut the downstream end of the bank.

In such a case, the saturation line can be kept covered at least by 0.5 m with the help of counter berms, as shown in following figure.



The straight saturation gradient line may be drawn with the following slopes.

Assumed Values of Saturation Gradients in Different Soils	
Type of soil	Slope (H:V)
Clay	1 in 4
Clayey loam	1 in 6
Loam	1 in 8
Loamy sand	1 in 10
sand	1 in 15

### ***Spoil Banks.***

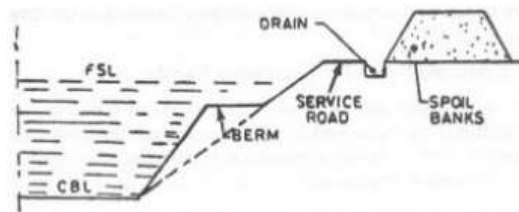
When the earthwork in excavation exceeds earthwork in filling, even after providing maximum width of bank embankments, the extra earth has to be disposed off economically.

To dispose off this earth by mechanical transport, etc. may become very costly, and an economical mode of its disposal may be found in the form of collecting this soil on the edge of the bank embankment itself.

The soil is, therefore, deposited in such a case, in the form of heaps on both banks or only on one bank, as shown in following figure.

These heaps of soil are discontinued at suitable intervals and longitudinal drains running by their sides are excavated for the disposal of rain water.

Cross drains through the spoil banks may also be excavated, if needed.



### ***Borrow Pits***

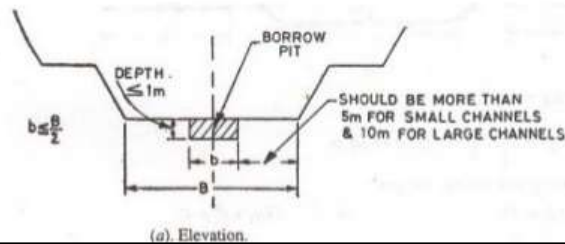
When earthwork in filling exceeds the earthwork in excavation, the earth has to be brought from somewhere.

The pits, which are dug for bringing earth, are known as borrow pits.

If such pits are excavated outside the channel, they are known as *external borrow pits*, and if they are excavated somewhere within the channel, they are known as *internal borrow pits*.

It is a very costly affair to bring soil from distances. Even in the nearby areas, these pits may cause mosquito nuisance due to collection of rain water in these pits, and hence, external borrow pits are not preferred.

When needed, internal borrow pits are excavated on the bed of the canal, as shown in figure.

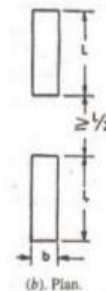


The borrow pits should start from a point at a distance more than 5 m from the toe for small channels, and 10 m for large channels.

The width of these pits (b) should be less than half the width of the canal (B), and should be dug in the centre.

The depth of these pits should be equal to or less than 1 m.

Longitudinally, these pits should not run continuous but a minimum space of  $0.5 L$  should be left between two consecutive pits. (where L is the length of one pit) as shown in following figure.



## Balancing Depth:-

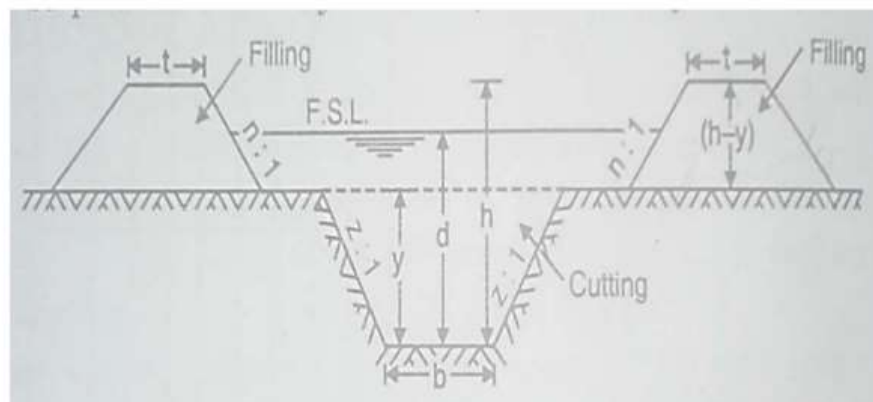
A canal section will be economical when earth work involved at a particular section has an equal amount of cut and fill. Usually a canal section has a part in cutting and part in filling as shown in fig.

If the amount of cut is equal to the amount of fill, it has to be paid for once only.

**Definition:-**

For a given C/S there is always only one depth of cutting for which the cutting and filling will be equal. The depth is known as **balancing depth**.

## Balancing Depth:-



## Balancing Depth:-

If :-

$h$  = vertical height of top of bank from the bed of canal.

$b$  = bed width of the channel.

$t$  = top width of the canal bank.

$n:1$  = side slope of bank in filling.

$z:1$  = side slope of canal in cutting.

$d$  = full supply depth of canal.

$y$  = depth of cutting.

## Balancing Depth:-

$$\begin{aligned} \text{Area of the cut} &= by + zy^2 \\ &= y(b + zy) \end{aligned}$$

$$\text{Area of fill} = 2[(h - y)t + n(h - y)^2]$$

Equating the area of cut and fill:

$$y(b + zy) = 2[(h - y)t + n(h - y)^2]$$

$$by + zy^2 = 2th + 2nh^2 - 2nhy - 2ty - 2nhy + 2ny^2$$

$$y^2(2n - z) - (b + 4nh + 2t)y + 2h(t + nh) = 0$$

From this equation the balancing depth of the canal may be determined.

## Balancing Depth:-

A canal is usually constructed with side slope of 1:1 in cutting and a slope 1.5:1 in filling.

Putting  $n = 1.5$  and  $z = 1$  in above equation.

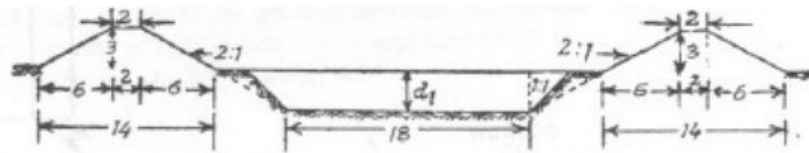
We get;

$$y^2 - (b/2 + 3h + t)y + h(t + 3/2 h) = 0$$

### Example

Calculate the balancing depth for a channel section having a bed width equal to 18 m and side slopes of 1:1 in cutting and 2:1 in filling. The bank embankments are kept 3.0 m higher than the ground level (berm level) and crest width of banks is kept as 2.0 m.

### Solution



The channel section is shown in Fig. Let  $d_f$  be the balancing depth, i.e. the depth for which excavation and filling becomes equal.

$$\text{Area of cutting} = (18 + d_f) \cdot d_f \text{ m}^2$$

$$\text{Area of filling} = 2 \left[ \frac{2 + 14}{2} \times 3 \right] = 48 \text{ m}^2$$

$$\text{Equating cutting and filling, we get} \quad (18 + d_f) \cdot d_f = 48$$

Solving for  $d_f$ , we get

$$\text{Balancing depth} = 2.35 \text{ m}$$

## Design of Canals

### Design Capacity of Canals

Many procedures have been developed for the hydraulic design of open channel sections.

The development of Chezy equation was based on the dimensional analysis of the friction equation under the assumption that the condition of flow is uniform.

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

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

The Manning equation has proved to be very reliable in practice. The Manning equation is determination of flow velocity based on the slope of channel bed, surface roughness of the channel, cross-sectional area of flow, and wetted perimeter of flow.

$$Q = \frac{A^{5/3} S^{1/2}}{nP^{2/3}}$$

$$Q = \frac{1}{n} AR^{2/3} S^{1/2}$$

### Design Capacity of Canals

For the calculation of the capacity of canals by above equation, flow of canal is assumed uniform. The two types of canals are considered

- (1) lined or non-erodible;
- (2) unlined, earthen, or erodible.

Basic data required for design

1. Shape of the cross section of the canal.
2. Side slope of the canal.
3. Longitudinal bed slope.
4. Permissible velocities - Maximum and Minimum.
5. Roughness coefficient.
6. Free board.

From the above data we can easily calculate the capacity of canal by Manning's and Chezy's equation by assuming neither scouring nor silting velocity (permissible velocity)

### Sediment Transport in Canals

- ❖ When the average shear stress  $\tau_o$  on the bed of an alluvial channel exceeds the critical shear  $\tau_c$ , the sediment particles start moving in different ways depending on the flow condition, sediment size, fluid and sediment densities, and the channel condition.
- ❖ At relatively low shear stresses, the particles roll or slide along the bed.
- ❖ The particles remain in continuous contact with the bed and the movement is generally discontinuous.
- ❖ Sediment material transported in this manner is termed *contact load*.
- ❖ On increasing the shear stress, some sediment particles lose contact with the bed for some time, and ‘hop’ or ‘bounce’.
- ❖ The sediment particles moving in this manner fall into the category of *saltation load*.

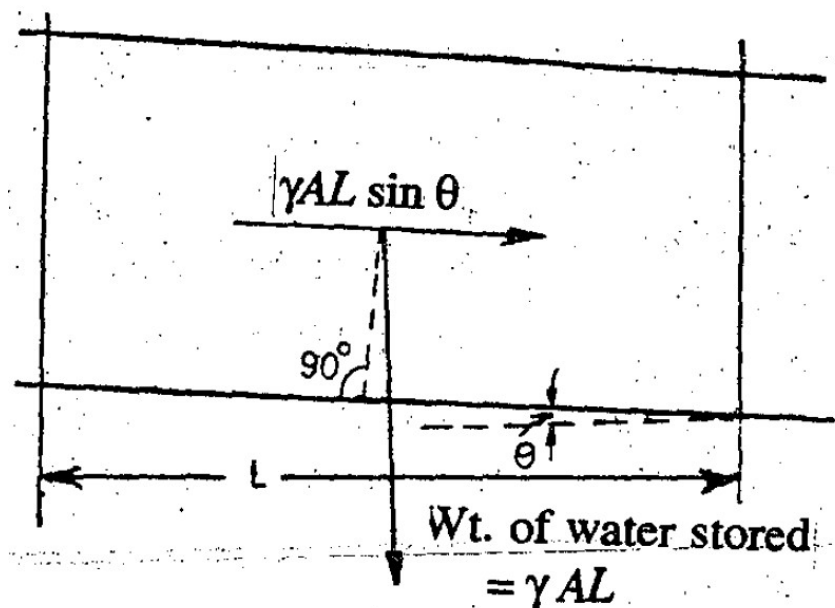
### Sediment Transport in Canals

- ❖ This mode of transport is significant only in case of non-cohesive materials of relatively high fall velocities
- ❖ It is difficult to distinguish between saltation load and contact load, the two are grouped together and termed *bed load*,
- ❖ which is transported on or near the bed.
- ❖ With further increase in the shear stress, the particles may go in suspension and remain so due to the turbulent fluctuations.
- ❖ The particles in suspension move downstream. Such sediment material is included in the *suspended load*.
- ❖ The material for bed load as well as a part of the suspended load originates from the bed of the channel and, hence, both are grouped together and termed *bed-material load*.
- ❖ Irrigation channels carrying silt-laden water and flowing through alluvial bed are designed to carry certain amounts of water and sediment discharges.

### Sediment Transport in Canals

- ❖ This means that the total sediment load transport will affect the design of an alluvial channel.
- ❖ The product of erosion in the catchment is appropriately called *wash load*. The transport rate of wash load is related to the availability of fine material in the catchment and its erodibility and is, normally, independent of the hydraulic characteristics of the stream.
- ❖ As such, it is not easy to make an estimate of wash load.
- ❖ Method of estimation of total load is to determine bed load, suspended load, and wash load individually and then add these together.
- ❖ The wash load is usually carried without being deposited and is also not easy to estimate. This load is, therefore, ignored while analysing channel stability.

### Mechanics of Sediment Transport



**Force or Drag Force.**

Let us consider a channel of length  $L$  and cross-sectional area  $A$ .

The volume of water stored in this channel reach =  $AL$

Wt. of water stored =  $\gamma_w AL$

where  $\gamma_w$  = unit wt. of water =  $\rho_w g$ , where  $\rho_w$  is the density of water.

Horizontal component of this wt. =  $\gamma_w AL \sin \theta = \gamma_w ALS$

where  $S$  = channel bed slope.

This horizontal force exerted by water is nothing but Tractive force.

Average Tractive force per unit of wetted area

$$\begin{aligned} &= \text{Unit Tractive Force } (\tau_0) = \frac{\gamma_w ALS}{\text{Wetted area}} \\ &= \frac{\gamma_w ALS}{\text{Wetted perimeter} \times \text{Length}} = \frac{\gamma_w ALS}{P \cdot L} = \gamma_w \left( \frac{A}{P} \right) S = \gamma_w R \cdot S \quad \left( \because \frac{A}{P} = R \right) \end{aligned}$$

where  $R$  = is the hydraulic mean depth of channel.

$S$  = channel bed slope

$\gamma_w$  = unit wt. of water

$P$  = wetted perimeter

Hence, **Average Unit Tractive force**, also called Shear stress

$$= \tau_0 = \gamma_w RS$$

It may be noted that the unit tractive force in channels, except for wide open channels, is not uniformly distributed along the wetted perimeter. A typical distribution of shear stress (unit-tractive force) on a trapezoidal channel section is shown in Fig.

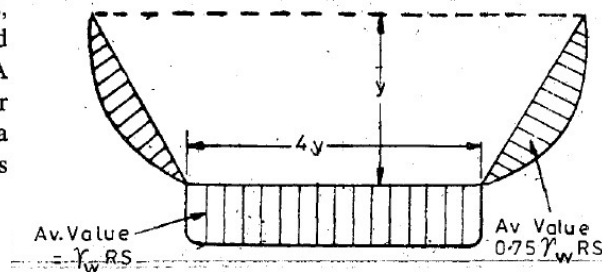


Fig. Distribution of tractive force generated in a trapezoidal channel section.

The approximate maximum unit tractive force on the bottom is

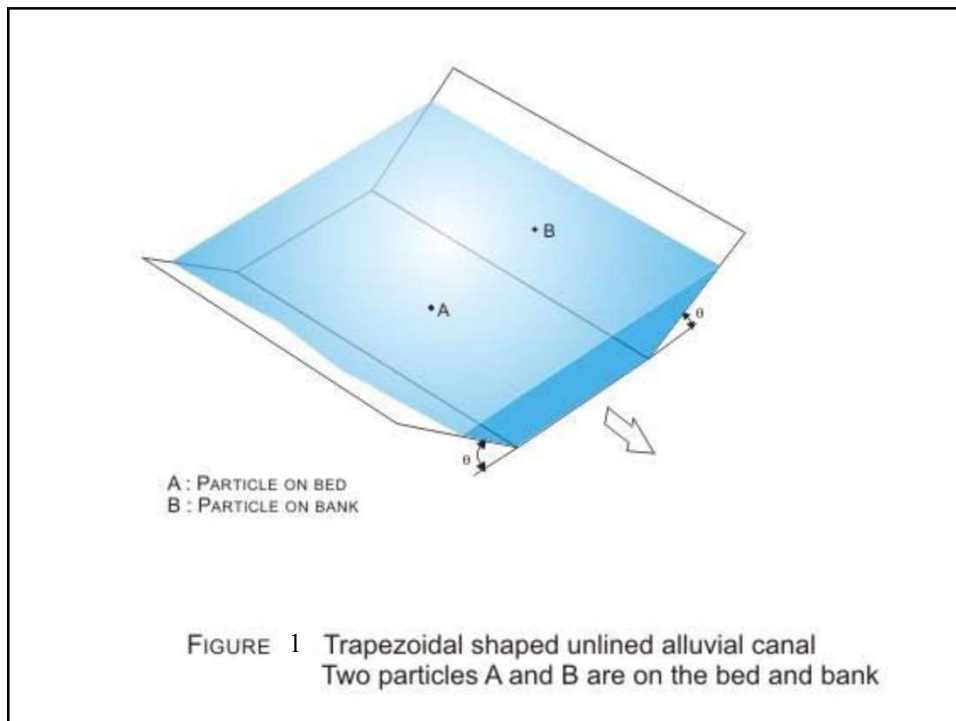
$$\tau_{bm} = 0.97 \gamma_w RS \text{ and on the sides is } \tau_{sm} = 0.76 \gamma_w RS.$$

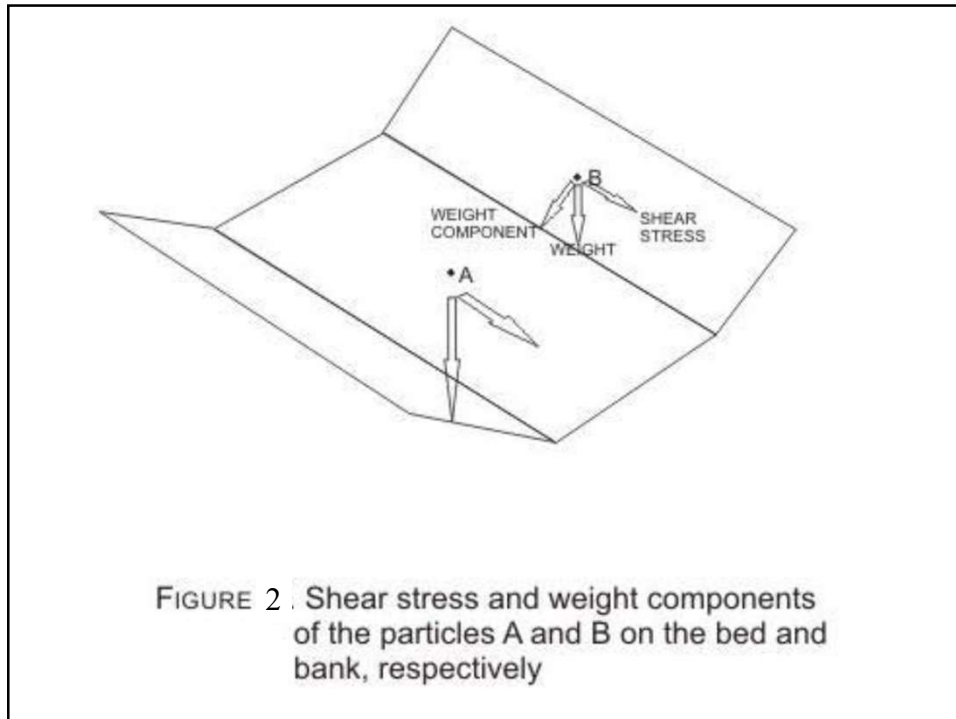
## Tractive force Approach of Canal Design

### Unlined alluvial canals in clear water

A method of design of stable channels in coarse non-cohesive material carrying clear water has been developed by the United States Bureau of Reclamation as reported by Lane (1955), which is commonly known as the **Tractive Force Method**. **Figure 1** below shows schematically such a situation where the banks are inclined to the horizontal at a given angle  $\theta$ .

It is also assumed that the particles **A** and **B** both have the same physical properties, like size, density, etc. and also possess the same internal friction angle  $\Phi$ . Naturally, the bank inclination  $\theta$  should be less than  $\Phi$ , for the particle **B** to remain stable, even under a dry canal condition. When there is a flow of water, there is a tendency for the particle **A** to be dragged along the direction of canal bed slope, whereas the particle **B** tries to get dislodged in an inclined direction due to the shear stress of the flowing water as shown in **Figure 2**.





The particle **A** would get dislodged when the shear stress ' $\tau$ ' is just able to overcome the frictional resistance. This critical value of shear stress is designated as ' $\tau_c$ ' may be related to the weight of the particle ' $W$ ' as

$$\tau_{cb} = W \tan\theta \quad (1)$$

For the particle **B**, a smaller shear stress is likely to get it dislodged, since it is an inclined plane. In fact, the resultant of its weight component down the plane ' $W \sin\theta$ ' and the shear stress (designated as ' $\tau_c$ ') would together cause the particle to move. Hence, in this case,

$$(\tau_{cs})^2 + (W \sin\theta)^2 = W \cos\theta \tan\theta \quad (2)$$

In the above expression it must be noted, that the normal reaction on the plane for the particle **B** is  $W \cos\theta$ .

Eliminating the weight of the particles  $W$  from equations (1) and (2), one obtains,

$$\tau_{cs}^2 + \left[ \frac{\tau_{cb}}{\tan \phi} \sin \theta \right]^2 = \left[ \frac{\tau_{cb}}{\tan \phi} \cos \theta \tan \phi \right]^2$$

This simplifies to

$$\tau_{cs}^2 = \tau_{cb}^2 \left[ \cos^2 \theta - \frac{\sin^2 \theta}{\tan^2 \phi} \right]$$

Or

$$\frac{\tau_{cs}}{\tau_{cb}} = \cos \theta \sqrt{1 - \frac{\tan^2 \theta}{\tan^2 \phi}} \quad (3)$$

The ratio  $\frac{\tau_{cs}}{\tau_{cb}}$  is known as *tractive force ratio or reduction factor*, denoted by  $k$ .

$$\text{i.e., } k = \cos \theta \sqrt{1 - \frac{\tan^2 \theta}{\tan^2 \phi}} \quad (4)$$

With simplification,

$$k = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}} \quad (5)$$

The equation (3) shows that  $\tau_{cs} < \tau_{cb}$ , which means that the shear stress required moving a grain on the side slope is less than that required to move on the bed.

The values for the critical stresses at bed and at sides are the limiting values. One does not wish to design the canal velocity and water depth in such a way that the actual shear stress reaches these values exactly since a slight variation may cause scouring of the bed and banks. Hence, adopt a slightly lower value for each.

Hence, in order to avoid motion of particles;

allowable critical shear stress (or limiting shear stress) for bed,

$$\tau_{bl} \approx 0.9 \tau_{cb} \quad \text{and}$$

allowable critical shear stress (or limiting shear stress) for sides,

$$\tau_{sl} \approx 0.9 \tau_{cs} \approx 0.9 k \tau_{cb}$$

Non scouring condition:

$$\tau_{bm} \leq \tau_{bl} \quad \text{and}$$

$$\tau_{sm} \leq \tau_{sl}$$

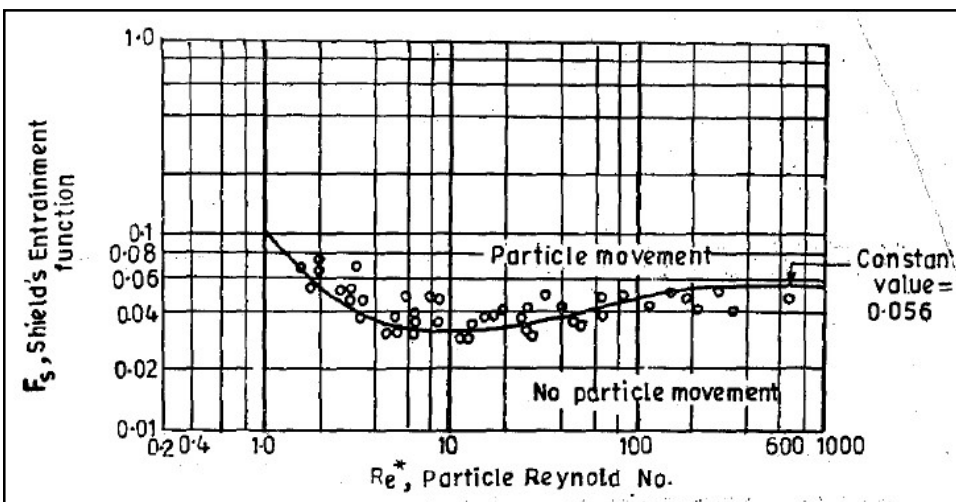


Fig. 4.4. Shield's curve for incipient motion condition.

For  $Re^* > 400$ ,  $F_s = 0.056$  (Constant)

$Re^* > 400$  is found when  $d > 6 \text{ mm}$

$$F_s = \frac{\tau_c}{\gamma_w d (S_s - 1)} = 0.056 \text{ (for } d > 6 \text{ mm)*}$$

where  $\gamma_w$  = unit wt. of water =  $9.81 \text{ kN/m}^3$   
or  $1 \text{ t/m}^3$  or  $1000 \text{ kgf/m}^3$ .

The average shear stress caused on the bed of a channel by the flowing water is given by:

$$\tau_0 = \gamma_w R S$$

where,  $R$  = Hydraulic mean radius of the channel, i.e.  $A/P$ .

$S$  = Bed slope.

Moreover,  $\tau_0 \leq \tau_c$ ;

$$\therefore \tau_0 \leq \gamma_w d (S_s - 1) \text{ (0.056)}$$

or  $\gamma_w R S \leq \gamma_w d (S_s - 1) \text{ (0.056)}$

or  $R S \leq d (S_s - 1) \text{ (0.056)}$

or  $R S \leq d (2.65 - 1) \text{ (0.056)}$

or  $R S \leq \frac{d}{11}$

or  $d \geq 11 R S$

Equation gives the minimum size of the bed material or lining stone that will remain at rest in a channel of given  $R$  and  $S$ .

Since with the passage of time, the channel bed becomes Armored (i.e. the smaller stones are flushed out of the surface lining of the coarser stones), actual size of bed or lining to be used should be somewhat more than what is calculated from the equation  $d = 11 R S$ .

\*Mittal and Swamee has worked out a general relation between  $\tau_c$  and  $d$  which gives results within +5% of the values given by Shield's curve, for all values of  $d$ . The relation for water and soil of  $S_s = 2.65$ , is given by equation

$$\tau_c \text{ (N/m}^2\text{)} = 0.155 + \frac{0.409d^2_{mm}}{\sqrt{1 + 0.177d^2_{mm}}}$$

By Strickler's formula\*, we know that Manning's rugosity coefficient ( $n$ ) is given as:

$$n = \frac{1}{24} d_{eff}^{1/6} \quad \text{for } d_{eff} > 6 \text{ mm} \quad \text{where } d_{eff} \text{ is in meters}$$

\* An irrigation channel is to be constructed in coarse alluvium gravel with size of 5 cm. The channel has to carry 5 cumec of discharge and the longitudinal slope is 1 in 100. The banks of the channel will be protected by grass against scouring. Find the minimum width of the channel.

Soln:  $Q = 5 \text{ m}^3/\text{s}$ ,  $S = 1/100$

$d = 5 \text{ cm} = 0.05 \text{ m} \text{ (} > 6 \text{ mm)}$   
 Using Strickler's formula,  

$$n = \frac{1}{24} d^{1/6} = \frac{1}{24} * (0.05)^{1/6} = 0.025$$

Now,  $d \geq 11 R S$   
 i.e.,  $R \leq \frac{d}{11 S} = \frac{0.05}{11 * 1/100} = 0.455 \text{ m}$

i.e.,  $R_{\max} = 0.455 \text{ m}$

Using Manning's formula,  

$$V_{\max} = \frac{1}{n} R_{\max}^{2/3} S^{1/2} = \frac{1}{0.025} * 0.455^{2/3} * \left(\frac{1}{100}\right)^{1/2}$$

$$= 2.37 \text{ m/s}$$

we know,  $Q = AV = BDV$   
 i.e.,  $B_{\min} = \frac{Q}{V_{\min} D} \quad [R \approx D]$ 

$$= \frac{5}{2.37 * 0.455}$$

$$= 4.64 \text{ m} //$$

\* Water flows at a depth of 1 m in a wide stream having a bed slope of 1 in 3000. The median diameter of sand bed is 2 mm. Determine whether the soil grains are stationary or moving, and comment as to whether the stream bed is scouring or non-scouring.

Soln:  
 $D = 1 \text{ m}, S = 1/3000, d = 2 \text{ mm}$

Now, 
$$\tau_c = 0.155 + \frac{0.409 d_{\text{mm}}^2}{\sqrt{1 + 0.177 d_{\text{mm}}^2}}$$

$$= 0.155 + \frac{0.409 * 2^2}{\sqrt{1 + 0.177 * 2^2}} = 1.407 \text{ N/m}^2$$

Also, 
$$\tau_b = \gamma_w R S = 9810 * 1 * \frac{1}{3000} \quad [R \approx D]$$

$$= 3.27 \text{ N/m}^2 > \tau_c$$

Hence the soil grains are moving and the stream bed is scouring and sediment transport will occur.

\* Design a straight trapezoidal channel for a design discharge of  $10 \text{ m}^3/\text{s}$ . The bottom slope is  $0.00025$  and the channel is excavated through fine gravel having particle size of  $8 \text{ mm}$ . Take Manning's  $n = 0.024$  and  $\gamma_{\text{perm}} = 7.18 \text{ N/m}^2$ ,  $\phi = 24^\circ$ .  
side slope =  $3H:1V$ .

Here,  $Q = 10 \text{ m}^3/\text{s}$ ,  $S = 0.00025$ ,  $\phi = 24^\circ$   
 $n = 0.024$ ,  $\gamma_{\text{perm}} = 7.18 \text{ N/m}^2$ ,  $\theta = \tan^{-1}(1/3) = 18.43^\circ$

Now,  $K = \sqrt{1 - \frac{\sin 2\theta}{\sin^2 \phi}} = \sqrt{1 - \frac{\sin^2 18.43^\circ}{\sin^2 24^\circ}} = 0.629$

Taking,  $\gamma_{\text{bl}} = \gamma_{\text{perm}} = 7.18 \text{ N/m}^2$

$\gamma_{\text{bl}} = K \gamma_{\text{perm}} = 0.629 * 7.18 = 4.52 \text{ N/m}^2$

Unit tractive force on the sides of channel,  
 $\tau_{\text{bl}} = 0.75 \gamma_{\text{bl}} R S = 0.75 * 3810 * R * 0.00025 = 1.84 R$

Also,  $1.84 R \leq 4.52$

$\therefore R \leq 2.46$

Take,  $R = 1 \text{ m}$

$\therefore V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.024} * (1)^{2/3} * (0.00025)^{1/2} = 0.659 \text{ m/s}$

$\therefore A = Q/V = \frac{10}{0.659} = 15.174 \text{ m}^2$

$P = \frac{A}{R} = \frac{15.174}{1} = 15.174 \text{ m}$

we have, equations

$P = b + 2D\sqrt{1+z^2}$  and  $A = (b + zD)D$

$15.174 = b + 6.32D \quad \dots (1)$   
 $15.174 = bD + 3D^2 \quad \dots (2)$

$\therefore 15.174 = (15.174 - 6.32D)D + 3D^2$   
 $\therefore 15.174 = 15.174D - 6.32D^2 + 3D^2$   
 $\therefore 15.174 = 15.174D - 3.32D^2$   
 $\therefore D^2 - 4.57D + 4.57 = 0$

$D = \frac{4.57 \pm \sqrt{(4.57)^2 - 4 * 1 * 4.57}}{2}$   
 $= 1.5 \text{ m} \text{ or } 3.1 \text{ m}$

$\therefore B = 5.69 \text{ m}$   
i.e.  $B = 5.69$  and  $D = 1.5 \text{ m} //$

\* Design a channel with light load of fine pediment without causing scour for bed width, water depth and side slope 5.0 m, 1.0 m and 1.5:1 respectively. Take  $\tau_{perm}(bed) = 0.29 \text{ kg/m}^2$  and  $\tan\phi = 0.75$ .

Soln:  $B = 5.0 \text{ m}$ ,  $D = 1.0 \text{ m}$ ,  $Z = 1.5$ ,  $\tau_{perm}(bed) = 0.29 \text{ kg/m}^2$   
 $\tan\phi = 0.75$   
 i.e,  $\phi = 36.87^\circ$   
 $\theta = \tan^{-1}\left(\frac{1}{1.5}\right) = 33.69^\circ$

Now,  $K = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} = \sqrt{1 - \frac{\sin^2 33.69^\circ}{\sin^2 36.87^\circ}} = 0.381$   $\therefore R = A/P = 1.756 \text{ m}$

Also,  $0.75 \rho_w R S \leq 0.11$   
 $\text{or } 0.75 \times 1000 \times 0.756 S \leq 0.11$   
 $\text{or } S \leq \frac{0.11}{567}$   
 $\therefore S \leq \frac{1}{5153}$

Taking,  $\tau_{bed} = \tau_{perm}(bed) = 0.29 \text{ kg/m}^2$   
 $\tau_{bed} = K \tau_{bed} = 0.381 \times 0.29 = 0.11 \text{ kg/m}^2$   
 unittractive force on sides of the channel =  $0.75 \rho_w R S$

\* A canal is to be designed to carry a discharge of 50 cumecp. The slope of the canal is 1 in 1000. The soil is coarse alluvium having a grain size of 5 cm. Assuming the canal to be unlimited and of a trapezoidal section, determine a suitable section for the canal, Take  $\phi = 37^\circ$ .

Soln:

$Q = 50 \text{ cumecp}$ ,  $S = 1/1000$ ,  $d = 5 \text{ cm} = 0.05 \text{ m}$  (~~6 mm~~)  
 $\phi = 37^\circ$

Take  $\theta = 30^\circ < \phi$

$\therefore K = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} = \sqrt{1 - \frac{\sin^2 30^\circ}{\sin^2 37^\circ}} = 0.557$

we have,

$\tau_{se} = 0.75 \rho_w R S$

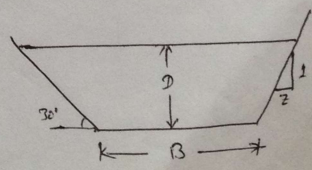
$\tau_{bed} = \rho_w R S \leq \rho_w \frac{d}{11}$  [ $\therefore$  For  $d > 6 \text{ mm}$ ,  $d \geq 11 RS$ ]

Also,  $V_{sd} = K V_{bl}$   
 $\therefore 0.75 R S \leq 0.557 \frac{d}{11}$   
 $\therefore R S \leq 0.0675 d$   
 $\therefore R \leq 0.0675 \frac{d}{S} = 0.0675 * 0.05 * 1000$   
 $\therefore R \leq 3.38 \text{ m}$   
 $\therefore D \leq 3.38 \text{ m} \quad [R \approx D]$

Take,  $D = 2.5 \text{ m}$

For Trapezoidal section:

$A = B D + z D^2 \dots \textcircled{1}$   
 $\text{and } P = B + 2 D \sqrt{1+z^2} \dots \textcircled{2}$   
 $z = \frac{1}{\tan \theta} = \frac{1}{\tan 30^\circ} = 1.73$   
 $n = \frac{1}{24} d^{1/6} = \frac{1}{24} * (0.05)^{1/6} = 0.025$



$Q = \frac{1}{n} R^{2/3} S^{1/2} \dots \textcircled{3}$

A number of trial values of B for  $D = 2.5 \text{ m}$  is tabulated below till a discharge of 50 cumecs is reached

B (m)	A (m <sup>2</sup> )	P (m)	R = A/P (m)	Q (m <sup>3</sup> /s)	Q = A Q (Cumecs)
5	23.31	14.99	1.56	2.61	39.63
4	20.81	13.99	1.49	1.65	34.33
6	25.81	15.99	1.61	1.74	44.91
7	28.31	16.99	1.67	1.78	50.39 $\approx$ 50
6.8	27.81	16.79	1.66	1.77	49.22
6.9	28.06	16.89	1.66	1.77	49.67

Hence the suitable canal section as follows:

depth = 2.5 m  
 Base width = 7.0 m  
 side slopes,  $\theta = 30^\circ$

## Design of Non-alluvial Channels

Usually design of non-alluvial channels is done by Manning's or Chezy's equation.

The mean velocity for a uniform flow as given by **Manning's** is given by

$$v = (1/n) R^{2/3} S^{1/2}$$

where  $v$  = velocity of flow in m/s

$n$  = Manning's coefficient, the value depends upon the bed and side material of the channel

$R$  = hydraulic mean depth of the channel in m

$S$  = bed slope of the channel

## Design of Non-alluvial Channels

The mean velocity for a uniform flow as given by **Chezy's** is given by

$$v = C \sqrt{RS}$$

where  $v$  = velocity of flow in m/s

$C$  = Chezy's coefficient

$R$  = hydraulic mean depth of the channel in m

$S$  = bed slope of the channel

From Bazin's equation

$$C = 87 / [1 + (K / \sqrt{R})]$$

where,  $K$  = Bazin's coefficient, depends upon the type of surface

## Design of Non-alluvial Channels

From Kutter's equation

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

where,  $n$  = Kutter's roughness factor (coefficient of rugosity)

## Design of Alluvial Channels

**Kennedy's theory of designing unlined Canals:** Kennedy selected a number of canal sections in the upper Bari-Doab region which did not required any silt clearance for more than 35 years and were supposed to be flowing with non-silting and non-scouring velocity. Kennedy put forward the following facts out of his study.

- The bed of the canal offers frictional resistance to the flow of water, as a result critical eddies (Turbulences) arise from the bottom of the bed. These eddies keep the sediments carried by water in suspension. Some eddies also arise from the sides of the canal, but do not support the sediments. Hence, the sediment supporting capacity is proportional to the bed width of the canal.
- The critical velocity or non-silting and non scouring velocity ( $V_o$ ) is a function of the depth of the flowing water ( $D$ ). It is given by the relationship  $V_o = c m D^n$

Where, 'c' is and 'n' are coefficients suggested by Kennedy for canals of Bari-Doad region. The values of 'c' differs for different materials are

Light Sandy silt	$c = 0.53$
Coarse light sandy silt	$c = 0.59$
Sandy loam	$c = 0.65$
Coarse silt	$c = 0.70$

Note: Unless otherwise specified, values of  $c$  and  $n$  can be taken as  $c = 0.55$  and  $n = 0.64$

Thus, the equation for critical velocity becomes  $V_o = 0.55 m D^{0.64}$  Where,  $V$  represents mean velocity of flow. The value of  $m$  also varies with the silt

Types of Silt	Value of $m$
Silt of Indus rivers	0.7
Light Sandy silt of north India	1.0
Coarse-sandy silt	1.1

Sandy, Loamy silt	1.2
Coarse silt of hard rock	1.3

Note: Unless, otherwise specified  $m = 1.0$

The mean velocity of flow is given by  $V = C\sqrt{RS}$

Where  $C$  represents Chezy's constant and is given by

$$C = \frac{23 + (1 + N) + (0.0015 \div S)}{1 + (23 + (0.0015 \div S) \times (N \div \sqrt{R}))}$$

Where,  $N$  represents Kutter's Rugosity coefficient,

$S$  represents Bed slope of the canal

$R$  represents Hydraulic mean radius and is given by  $R = A/P$

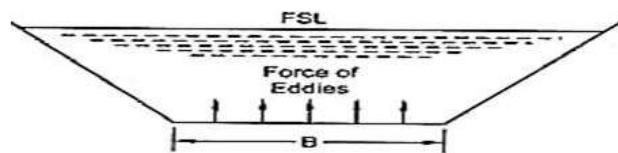
Where  $A$  is cross sectional area of canal and  $P$  is wetted perimeter.

When a canal is designed by Kennedy's method it is required that  $V_o$  is equal to  $V$ .

i.e., Critical velocity ratio  $m = 1$

#### Kennedy's theory

1. The flowing water has to counter act some amount of friction against the bed of the channel. By the result of that eddies are formed. These eddies are responsible to keep the silt in suspension without scouring.



(a) According to Kennedy

2. Kennedy define the critical velocity which is the mean velocity which will just keep the channel without silting and scouring.

3. Kennedy gave the equation for the determination of critical velocity

$$V_o = C.D^n$$

Where  $V_o$  = Critical velocity

D = Depth of channel

C & n = Constants

He found the values of C & n are 0.55 and 0.64

$$\text{Therefore } V_o = 0.55.D^{0.64}$$

4. Initially Kennedy assumed that the type of silt as Sandy silt. He realized that different types and grades of silt are available and finally he introduced the factor "m" which is the **critical velocity ratio** and is defined as the ratio of mean velocity to critical velocity.

$$\text{Therefore } m = V/V_o$$

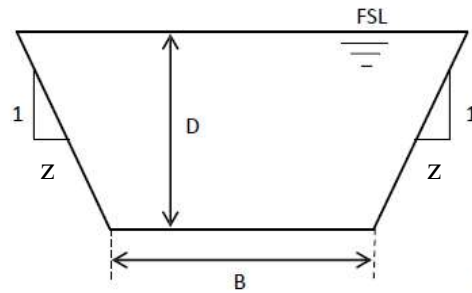
$$\text{So } V = m.V_o$$

$$V = m.0.55.D^{0.64}$$

5. He also used the kutter's equation for the determination of mean velocity.

$$V = \frac{1 + (23 + \frac{0.00155}{s})}{1 + (23 + \frac{0.00155}{s}) \frac{N}{\sqrt{R}}} \cdot \sqrt{RS}$$

Note: The cross section for an irrigation canal is assumed as a trapezoidal channel as follows.



$$\text{Cross Sectional Area of flow } A = B + z D^2$$

$$\text{Wetted Perimeter } P = B + 2D\sqrt{1 + Z^2}$$

**For the Design of Channel Kennedy's theory can be used in two different cases:**

**Case 1: When bed slope 'S' is given**

1. For the given discharge (Q) assume a trial value of the depth of flow (D)

For different values of discharge (Q) the trial values of depth of flow (D) are given as follows.

Q(m <sup>3</sup> /s)	0.283	0.708	1.416	2.832	7.079	14.158	28.317	56.634
D(m)	0.49	0.66	0.84	1.04	1.43	1.73	1.98	2.26

2. Calculate  $V_o$  from  $V_o = 0.55 m D^{0.64}$
3. Determine A from  $Q = A V_o$
4. Knowing D and A calculate the Bed width B
5. Knowing B and D calculate the wetted perimeter P
6. Knowing A and P calculate hydraulic mean radius R
7. Calculate mean velocity of flow from the equation  $V = C\sqrt{RS}$ .
8. If critical velocity ratio is equal to 1 ( $m=1$ ), then the assumed value of D is correct.
9. If not revise the depth 'D'.

**Case-2: When B/D ratio is given**

1. Let  $B/D = x$        $B = D x$
2. Calculate cross sectional area in terms of  $D$   $A = B D + Z D^2$
3. Calculate critical velocity  $V_o$  in terms of  $D$  by substituting in  $V_o = 0.55 m D^{0.64}$
4. Substituting for  $A$  and  $V_o$  in  $Q = A V_o$   $D$  can be determined.
5. Knowing  $D, A$  and  $B$  calculate  $P$  and  $R$
6. Calculate  $V_o$  from equation  $V_o = 0.55 m D^{0.64}$
7. Assuming a trial value for  $S$ , Calculate Chezzy's constant from equation

$$C = \frac{23 + \frac{1}{N} + \frac{0.0015}{S}}{1 + \left(23 + \frac{0.0015}{S}\right) \frac{N}{\sqrt{R}}}$$

8. Calculate mean velocity of flow from equation  $V = C\sqrt{RS}$
9. Calculate critical velocity ratio  $m$ . If  $m = 1$  the bed slope provided is adequate.
10. If not, revise the bed slope  $S$ .

Note: The trial values of bed slope  $S$  are assumed depending upon the discharge ( $Q$ ) as follows.

Q (m <sup>3</sup> /s)	0.283	0.708	1.416	2.832	7.079	14.158	28.317	56.634
S (1 in___)	3333	3636	4000	4444	4444	5000	5000	5714

**Problems**

1. Design and sketch an irrigation channel to carry 5 cumecs. The channel is to be laid on a slope of 0.2m per kilometer. Assume  $N=0.025$  and  $m=1$

Solution:

1. Assume a trial depth  $D$  equal to 1.0m

$$2. \quad V = V_o = 0.55 \text{ m D}^{0.64}$$

$$= 0.55 \times 1.0 \times 1.0^{0.64}$$

$$3. \quad \text{Area} = Q = A V_o$$

$$A = Q / V = 5 / 0.55 = 9.09 \text{ m}^2$$

$$4. \quad A = B D + Z D^2$$

$$= 9.09 = B \times 1.0 + 1.0^2 / 2$$

$$B = 8.59 \text{ m}$$

$$5. \quad \text{Perimeter} = P = B + D \sqrt{5}$$

$$8.59 + 1.0 \sqrt{5} = 10.83 \text{ m}$$

$$R = A / P = 9.09 / 10.83$$

$$= 0.84 \text{ m}$$

6. Mean velocity flow

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

$$C = \frac{23 + \frac{1}{N} + \frac{0.0015}{S}}{1 + \left( 23 + \frac{0.0015}{S} \right) \frac{N}{\sqrt{R}}}$$

$$R = 0.84 \text{ m}, S = 0.2 / 1000, N = 0.0225$$

$$C = 42.85$$

$$V = 42.85 \sqrt{0.84 (0.2 / 1000)} = 0.555 \text{ m/s}$$

7. Ratio of velocities found in step 6 and step 2  
 $= 0.555 / 0.55 = 1.009 = 1.0$

Hence assumed  $d$  is satisfactory.

**2. Determine the dimensions of the irrigation canal for the following data B/D ratio = 3.7,  $N = 0.0225$ ,  $m = 1.0$  and  $S = 1/4000$  side slopes of the channel is  $\frac{1}{2} H : 1V$ . Also determine the discharge which will be flowing in the channel.**

Solution:  $B/D = 3.7$

$$B = 3.7D$$

For the channel with side slopes of  $1/2H : 1V$

$$R = \frac{\frac{B + \sqrt{B^2 + 4m^2D^2}}{2} + mD}{\sqrt{1 + m^2}} = 0.708D$$

From Kennedy's equation,

$$V_o = 0.55 m D^{0.64}$$

$$V_o = 0.55 D^{0.64}$$

$$C = \frac{23 + \frac{1}{N} + \frac{0.0015}{S}}{1 + \left(23 + \frac{0.0015}{S}\right) \frac{N}{\sqrt{R}}}$$

Equating the two values of V, we get

$$0.55 D^{0.64} = 0.975 D^{1/2} / (1 + 0.781 D^{-1/2})$$

$$0.55 D^{0.64} + 0.4296 D^{0.14} = 0.9795 D^{1/2}$$

Solving the above equation by trial and error , we get

$$D = 1.0\text{m}$$

$$B = 3.7\text{m}$$

$$V = 0.55\text{m/s}$$

$$A = B D + D^2/2$$

$$A = 4.2\text{m}$$

$$Q = A \times V = 4.2 \times 0.55$$

$$= 2.31 \text{ cumes}$$

Draw backs in Kennedy's theory

1. Kutters equation is used for determining the mean velocity of flow and hence the limitations of kutter's equation are incorporated in Kennedy's theory.
2. The significance of B/D ratio is not considered in the theory
3. No equation for the bed slope has been given which may lead to varied designs of the channel with slight variation in the bed slope.
4. Silt charge and silt grade are not considered. The complex phenomenon of silt transportation is incorporated in a single factor are called critical velocity ratio.
5. The value of m is decided arbitrarily since there is no method given for determining its value.
6. This theory is aimed to design only an average regime channel.
7. The design of channel by the method based on this theory involves trial and error which is quite cumbersome.

## Lacey's Regime Theory

According to Lacey:

"Silt is kept in suspension by the vertical component of eddies generated at all points of forces normal to the wetted perimeter".

**Regime Channel**

"A channel is said to in regime, if there is neither silting nor scouring".

Lacey's theory is based on the concept of regime condition of the channel. The regime condition will be satisfied if,

- The channel flows uniformly in unlimited incoherent alluvium of the same character which is transported by the channel.
- The silt grade and silt charge remains constant.
- The discharge remains constant.

According to Lacey there may be three regime conditions:

- (i) True regime;
- (ii) Initial regime; and
- (iii) Final regime.

*(1) True regime*

A channel shall be in 'true regime' if the following conditions are satisfied:

- (i) Discharge is constant;
- (ii) Flow is uniform;
- (iii) Silt charge is constant; i.e. the amount of silt is constant;
- (iv) Silt grade is constant; i.e., the type and size of silt is always the same; and
- (v) Channel is flowing through a material which can be scoured as easily as it can be deposited (such soil is known as *incoherent alluvium*), and is of the same grade as is transported.

*(ii) Initial regime*

- ❑ bed slope of a channel varies
- ❑ cross-section or wetted perimeter remains unaffected

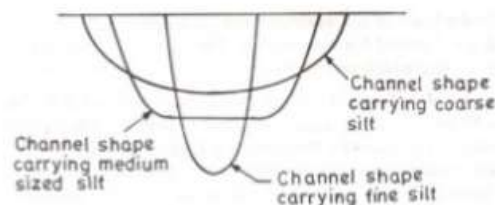
*(iii) Final regime*

- ❑ all the variables such as perimeter, depth, slope, etc. are equally free to vary and achieve permanent stability, called *Final Regime*.

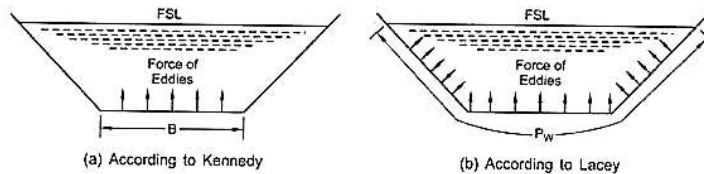
In such a channel,

The coarser the silt, the flatter is the semi-ellipse.

The finer the silt, the more nearly the section attains a semi-circle.



Lacey also states that the silt is kept in suspension solely by force of eddies. But Lacey adds that eddies are not generated on the bed only but at all points on the wetted perimeter. The force of eddies may be taken normal to the sides



In his theory, he states that the silt carried by the flowing water is kept in suspension by the vertical component of eddies. The eddies are generated at all the points on the wetted perimeter of the channel section. Again, he assumed the hydraulic mean radius  $R$ , as the variable factor and he recognized the importance of silt grade for which he introduced a factor which is known as silt factor ' $f$ '.

Thus, he deduced the velocity as;  $V = \sqrt{(2/5f R)}$   
 Where,  $V$  = mean velocity in m/sec,  $f$  = silt factor,  
 $R$  = hydraulic mean radius in meter

### Lacey's Regime Equations :

On the basis of arguments mentioned above, Lacey plotted a large mass of data to obtain a relationship between (i)  $V$  vs.  $R$  and (ii)  $A$  vs.  $V$ . Lacey recognised the importance of silt grade in the problem and introduced a function  $f$  known as silt factor in the regime relationship. The first two equations, originally suggested by Lacey in F.P.S. units, converted in M.K.S., units, are given below :

$$\boxed{V = \sqrt{\frac{2}{3} f R}} \quad \text{--- ①}$$

$$\boxed{A f^2 = 140.0 V^5} \quad \text{--- ②}$$

where  $A$  = area of the channel section and  $V$  = velocity of flow in it.

### Regime Flow Equation

By plotting a large mass of data from a number of different sources, Lacey obtained the relationship

$$\boxed{V = 10.8 R^{2/3} S^{1/3}} \quad \text{--- ③}$$

$S$  = slope of water surface

### 1. Perimeter Discharge (P-Q) Relation

From eqn (1)

$$V^4 = \frac{4}{25} f^2 R^2 \quad \text{--- (4)}$$

From eqns (2) and (4)

$$A \left[ \frac{25V^4}{4R^2} \right] = 140.0 V^5$$

or

$$A \left[ \frac{25}{4R^2} \right] = 140 V$$

Multiplying both sides of this equation by  $A$ , we get

$$\frac{25A^2}{4R^2} = 140VA = 140Q$$

But

$$\frac{A^2}{R^2} = P^2$$

or

$$P^2 = \frac{4 \times 140}{25} Q$$

or

$$P = 4.75 \sqrt{Q} \quad \text{--- (5)}$$

### 2. V-Q-f Relation

Multiplying eqn (2) by  $V$  on both sides, we get

$$AVf^2 = 140.0 V^6$$

$$Qf^2 = 140.0 V^6$$

$$V = \left[ \frac{Qf^2}{140.0} \right]^{1/6} \quad \text{--- (6)}$$

### 3. Regime-Slope Equations

(S-Q-f), (R-S-f) and (S-f-q) relationships.

From Eq. 14.7, cubing both sides of the equation

$$V^3 = 1260 R^2 S \quad \text{--- (7)}$$

From Eq. 14.7, cubing both sides of the equation

$$V^3 = (2/5)^{3/2} f^{3/2} R^{3/2}$$

Hence,  $(2/5)^{3/2} f^{3/2} R^{3/2} = 1260 R^2 S$

or

$$S = \frac{f^{3/2}}{4980 R^{1/2}} \quad \text{--- (8)}$$

Equation 14.12 can also be written as

$$S = \left(\frac{V^2}{R}\right)^{5/3} \frac{1}{1260 (RV)^{1/3}} \quad \text{--- (9)}$$

From Eq. 14.7,  $V^2/R = \frac{2}{5}f$

$$RV = q$$

where  $q$  = discharge per unit width.

or 
$$S = \left(\frac{2}{5}\right)^{5/3} \frac{f^{5/3}}{1260 q^{1/3}}$$

$$= 0.000179 \frac{f^{5/3}}{q^{1/3}} \quad \text{--- (10)}$$

The above equation can also be represented in terms of total discharge.

Rewriting equation 14.14,  $(10)$

$$S = \left(\frac{V^2}{R}\right)^{5/3} \frac{1}{1260 (RV)^{1/3}}$$

Putting

$R = A/P$ , we get

$$S = \left(\frac{V^2}{R}\right)^{5/3} \frac{1}{1260 \left[\frac{A}{P} V\right]^{1/3}}$$

$$= \left(\frac{V^2}{R}\right)^{5/3} \frac{P^{1/3}}{1260 (Q)^{1/3}}$$

But from eqn (8)  $P = 4.75 \sqrt{Q}$

$$\therefore S = \frac{(2/5)^{5/3} f^{5/3} (4.75)^{1/3} Q^{1/6}}{1260 Q^{1/3}} \quad \text{--- (11)}$$

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

#### 4. Regime Scour Depth Relation

From Eq. 14.7 as already derived

$$R^2 = \frac{25 V^2}{4 f^2}$$

$$R = \frac{5 V^2}{2 f}$$

From Eq. 14.11, we get

$$V^2 = \left[ \frac{Qf^2}{140} \right]^{1/3} \quad (6)$$

Hence

$$R = \frac{5}{2} \left[ \frac{Qf^2}{140} \right]^{1/3} \frac{1}{f} \quad (12)$$

$$= 0.47 (q/f)^{1/3} =$$

Since  $q = RV$  (1)

Substituting for  $R$  from Eq. 14.7,

$$q = \frac{5}{2} \frac{V^3}{f} \quad (6)$$

Substituting  $V$  from Eq. 14.11, we get

$$q = \frac{5}{2} \cdot \frac{1}{f} \left[ \frac{Qf^2}{140} \right]^{3/6}$$

$$= 0.21 Q^{1/2} \quad (13)$$

Substituting for  $Q = \left( \frac{q}{0.21} \right)^2$  in equation 14.11, we get

$$R = 0.47 \frac{1}{f^{1/3}} \left[ \frac{q}{0.21} \right]^{2/3}$$

$$\boxed{R = 1.35 \left[ \frac{q^2}{f} \right]^{1/3}} \quad (14)$$

Then he deduced the relationship between A, V, Q, P, S and f are as follows:

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

$$\square Af^2 = 140 \times V^5$$

$$\square V = \left( \frac{Q \times f^2}{140} \right)^{1/6}$$

$$\square P = 4.75 \times \sqrt{Q}$$

$$\square \text{Regime flow equation, } V = 10.8 \times R^{2/3} S^{1/3}$$

$\square$  Regime slope equation,

$$(a) S = \frac{f^{3/2}}{4980 \times R^{1/3}}$$

$$(b) S = \frac{f^{5/2}}{3340 \times Q^{1/6}} \Rightarrow Q = \left[ \frac{f^{5/3}}{3340 \times S} \right]^6$$

$$\square \text{Regime scour depth, } R = 0.47 \times \left( \frac{Q}{f} \right)^{1/3}$$

### Problem

**Design an irrigation channel with the following data:**

Full supply discharge = 10 cumec

Mean diameter of silt particles = 0.33 mm

Side slope = 1/2:1

Find also the bed slope of the channel

**Solution:**

$$f = 1.76 \times \sqrt{0.33} = 1.0 \text{ and } V = \left( \frac{Q \times 1^2}{140} \right)^{1/6} = 0.64 \text{ m/sec}$$

$$A = 10/0.64 = 15.62 \text{ m}^2$$

$$P = 4.75 \times \sqrt{10} = 15.02 \text{ m}$$

$$R = 0.47 \times (10/1)^{1/3} = 1.02 \text{ m}$$

$$S = \frac{1^{5/2}}{3340 \times 10^{1/6}} = 1/4902$$

But,  $A = BD + 0.5 D^2$

$$\Rightarrow 15.62 = BD + 0.5 D^2 \text{ ----- (i)}$$

$$P = B + \sqrt{5}D$$

$$15.02 = B + 2.24 D \text{ ----- (ii)}$$

Solving equation (i) & (ii)  $D = 1.21 \text{ m}$  and  $B = 12.30 \text{ m}$

### Drawbacks of Lacey's Theory

- The concept of true regime is theoretical and can not be achieved practically.
- The various equations are derived by considering the silt factor  $f$  which is not at all constant.
- The concentration of silt is not taken into account.
- Silt grade and silt charge is not taken into account.
- The equations are empirical and based on the available data from a particular type of channel. So, it may not be true for a different type of channel.
- The characteristics of regime channel may not be same for all cases.

## Comparison between Kennedy's and Lacey's theory

Kennedy's theory	Lacey's theory
It states that the silt carried by the flowing water is kept in suspension by the vertical component of eddies which are generated from the bed of the channel.	It states that the silt carried by the flowing water is kept in suspension by the vertical component of eddies which are generated from the entire wetted perimeter of the channel.
It gives relation between 'V' and 'D'.	It gives relation between 'V' and 'R'.
In this theory, a factor known as critical velocity ratio 'm' is introduced to make the equation applicable to different channels with different silt grades.	In this theory, a factor known as silt factor 'f' is introduced to make the equation applicable to different channels with different silt grades.
In this theory, Kutter's equation is used for finding the mean velocity.	This theory gives an equation for finding the mean velocity.
This theory gives no equation for bed slope.	This theory gives an equation for bed slope.
In this theory, the design is based on trial and error method.	This theory does not involve trial and error method.

### Canal Lining

An impermeable layer is provided at the bed and sides of canal to improve the life and discharge capacity of canal known as canal lining. Generally seepage can result in losses of 30 – 40 % of irrigation water in canals, so we can reduce the effect of seepage by providing lining to the canal.

#### **Necessity of Canal Lining:**

- To minimize seepage losses
- To increase the discharge in canal section by increasing the velocity
- To prevent erosion of bed and side due to high velocities
- To reduce maintenance of canal.

#### **Advantages of Canal Lining:**

- Prevents seepage losses
- Reduces the problem of water logging
- Provide smooth surface and increase velocity of flow

**Advantages of Canal Lining:**

- Higher velocity minimize loss due to evaporation
- Higher velocity helps to provide narrow cross section
- Higher velocity helps to provide flatter hydraulic gradient and flatter bed slope
- Higher velocity prevents silting of channel
- Makes the banks more stable, Prevents weed growth
- Reduces maintenance costs, Reduces breaching, Provides stability
- Assures economical water distribution
- Prevents water to come in contact with harmful salts

**Disadvantages of Canal Lining:**

- Requires heavy initial investment
- Difficult to shift outlets very often
- Difficult to repair the damaged lining
- Berms are not provided in lined sections therefore additional safety for pedestrians and vehicles is absent

**Requirements of good Lining****Suitability of lining**

- The lining material should provide complete water tightness.
- The lining material should have a low coefficient of rugosity.
- The lining material should be strong and durable.
- The lining should not have a very high initial cost.
- The lining material should be able to resist growth of weeds and attack of burrowing animals
- The lining material should withstand high velocity
- The lining material should permit construction of required slope easily.
- The lining material should be unaffected by tramping of cattles

## Types of Lining

The following are the important types of concrete lining.

- a) Hard surface type lining
  - i. Cement concrete lining
  - ii. Shotcrete lining
  - iii. Precast concrete lining
  - iv. Cement mortar lining
  - v. Brick lining
  - vi. Stone blocks or undressed stone block lining
  - vii. Asphaltic lining
- b) Earth type lining
  - i. Soil cement lining
  - ii. Clay puddle lining
  - iii. Sodium carbonate lining
- c) Buried and protected membrane type lining
  - i. Prefabricated light membrane lining
  - ii. Bentonite soil and clay membrane lining
  - iii. Road oil lining

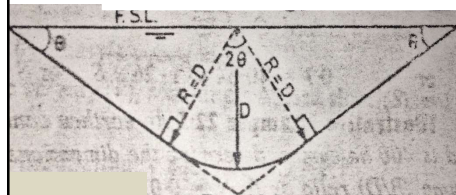
## Design of Lined Canals

A lined canal is a rigid boundary channel. It can withstand with much higher velocity as compared to an unlined, non-alluvial or alluvial channel. The design is similar to the design of non-alluvial channels. However, the maximum permissible velocity is relatively high. Generally Manning's equation is used in the design.

For the most economic section, the hydraulic radius  $R$  should be a maximum. Theoretically, a semi-circular section is the best section for an open channel.

From the practical considerations, a channel of trapezoidal section or triangular section is usually selected.

Figure below shows two types of lined canal sections : (i) triangular shaped or curved channel and (ii) trapezoidal shaped or flat bottom shaped channel. The side slopes are so selected that they are nearly equal to the angle of repose of the soil so that no earth pressure is imposed on the lining. The corners are rounded off to improve the hydraulic efficiency. Triangular shaped section is adopted for small discharges, as it is the best discharging section. For higher discharges, trapezoidal section may be adopted.

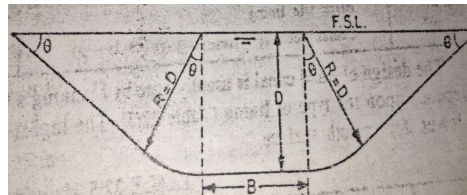


$$A = (\pi D^2) \left( \frac{2\theta}{2\pi} \right) + 2 \left( \frac{1}{2} D^2 \cot \theta \right)$$

$$A = D^2 (\theta + \cot \theta)$$

$$P = 2\pi D \left( \frac{2\theta}{2\pi} \right) + 2D \cot \theta$$

$$P = 2D (\theta + \cot \theta)$$



$$A = BD + (\pi D^2) \left( \frac{2\theta}{2\pi} \right) + 2 \left( \frac{1}{2} D^2 \cot \theta \right)$$

$$A = BD + D^2 (\theta + \cot \theta)$$

$$P = B + 2D (\theta + \cot \theta)$$

#### Design procedure for a trapezoidal lined channel

Given The following data should be collected or assumed.

- |                                  |   |
|----------------------------------|---|
| (i) Bed slope ( $S$ ),           | (ii) Side slopes, or the angle $\theta$ |
| (iii) Rugosity coefficient $N$ , | (iv) Limiting velocity $V$ .            |

Steps 1. Determine the area of flow,  $A = Q/V$

2. Determine the hydraulic radius,  $R = \left( \frac{VN}{S^{1/2}} \right)^{3/2}$

3. Determine the wetted perimeter,  $P = A/R$

4. Determine the values of  $B$  and  $D$  from the computed values of  $A$  and  $P$  by utilising the geometrical properties

**Alternative method** If instead of the limiting velocity  $V$ , the  $B/D$  ratio is given, the following procedure is used. Let  $B/D = x$

Steps 1. Determine  $V = \frac{Q}{A} = \frac{Q}{BD + D^2(\theta + \cot \theta)}$

or  $V = \frac{Q}{D^2[B/D + (\theta + \cot \theta)]} = \frac{Q}{D^2(x + \theta + \cot \theta)} \dots (a)$

2. Determine the hydraulic radius  $R = \frac{A}{P} = \frac{BD + D^2(\theta + \cot \theta)}{B + 2D(\theta + \cot \theta)}$

or  $R = \frac{D^2[B/D + (\theta + \cot \theta)]}{D[B/D + 2\theta + 2\cot \theta]} \dots (b)$

or  $R = \frac{D(x + \theta + \cot \theta)}{(x + 2\theta + 2\cot \theta)}$

3. Write down Manning's formula in terms of Eqs. (a) and (b),

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

or  $\frac{Q}{D^2(x + \theta + \cot \theta)} = \frac{1}{N} \left[ \frac{D(x + \theta + \cot \theta)}{x + 2\theta + 2\cot \theta} \right]^{2/3} \times S^{1/2} \dots (c)$

Determine  $D$  from Eq. (c).

4. Determine  $B = xD$ .

**Illustrative example** Design a lined channel to carry a discharge of 50 cumecs. Assume bed slope as 1 in 8100,  $N$  as 0.015 and side slope as  $45^\circ$ .

**Solution** Let us adopt a triangular section.

For  $\theta = \pi/4$ ,  $A = 1.785 D^2$  and  $P = 3.570 D$

From Manning's formula,  $V = \left(\frac{1}{N}\right) R^{2/3} S^{1/2}$  or  $Q = AV = \left(\frac{A}{N}\right) R^{2/3} S^{1/2}$

or  $50 = \frac{1.785 D^2}{0.015} \left(\frac{1.785 D^2}{3.570 D}\right)^{2/3} \left(\frac{1}{8100}\right)^{1/2}$

or  $50 = \frac{1.785 D^2}{0.015} \times 0.63 D^{2/3} \times \frac{1}{90} = 0.833 D^{8/3}$

or  $D = 4.64 \text{ m.}$

**Illustrative Example** Design a lined canal to carry a discharge of 120 cumecs. The velocity of flow may be taken as 2 m/s. Take the side slope as 1:1. Assume  $N = 0.018$  and bed slope = 1 in 3000.

$$A = 120/2 = 60 \text{ m}^2$$

$$R = \left( \frac{VN}{S^{1/2}} \right)^{3/2} = \left[ \frac{2 \times 0.018}{(1/3000)^{1/2}} \right]^{3/2} = 2.77 \text{ m}$$

$$P = 60/2.77 = 21.66 \text{ m}$$

Now  $BD + D^2 (\pi/4 + \cot 45^\circ) = A$

or  $BD + 1.785 D^2 = 60$

Now  $B + 2D (\pi/4 + 1) = P = 21.66$

or  $B + 3.57 D = 21.66$

From Eqs. (a) and (b),  $(21.66 - 3.57D) D + 1.785 D^2 = 60$

Solving,  $D = 4.28 \text{ m}$ ,  $B = 21.66 - 3.57 D = 6.38 \text{ m}$

**Illustrative Example** Design a lined channel to carry a discharge of 100 cumecs. Take  $B/D$  as 6:0,  $N = 0.016$ , Side slope = 1.5:1 and bed slope = 1 in 5000.

**Solution** In this case,  $\cot \theta = 1.50$ ,  $\theta = 33.69^\circ = 0.588$  radians

$$V = \frac{Q}{BD + D^2(\theta + \cot \theta)} = \frac{100}{BD + 2.088 D^2} = \frac{100}{D^2(B/D + 2.088)}$$

or  $V = \frac{100}{D^2(8.088)} = \frac{12.36}{D^2}$  ... (a)

Also  $R = \frac{A}{P} = \frac{BD + D^2(\theta + \cot \theta)}{B + 2D(\theta + \cot \theta)} = \frac{D^2(x + 2.088)}{D(x + 4.176)} = \frac{D(8.088)}{10.176}$

or  $R = 0.795 D$  ... (b)

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

Substituting the values from Eqs. (a) and (b),

$$\frac{12.36}{D^2} = \frac{1}{0.016} \times (0.795 D)^{2/3} \times \left( \frac{1}{5000} \right)^{1/2}$$

$$D^{8/3} = \frac{12.36 \times 0.016 \times 70.71}{0.858} = 16.30$$

or  $D = 2.85 \text{ m}$ ,  $B = 2.85 \times 6 = 17.1 \text{ m}$ .

## Economics of Canal 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.

## Financial Justification & Economics of Canal Lining

### Annual benefits:

(a) Saved seepage water by lining:

Let, the rate of water is sold to the cultivators = Rs.  $R_1$ /cumec

If  $m$  cumecs of water is saved by lining the canal annually, then the money saved by lining = Rs.  $m R_1$

(b) Saving in maintenance cost:

Let, the average cost of annual upkeep of unlined channel = Rs.  $R_2$

If  $p$  is the percentage fraction of the saving achieved in maintenance cost by lining the canal, then the amount saved = Rs.  $pR_2$

$\therefore$  The total annual benefits =  $mR_1 + p R_2$

**Annual costs:**

Let, the capital expenditure is Rs. C & the lining has a life of Y years

∴ Annual depreciation charges = Rs. C/Y

∴ Interest of the capital C = C(r/100) [r = percent of the rate of annual interest]

∴ Average annual interest = Rs. C/2(r/100)

[Since the capital value of the asset decreases from C to zero in Y years]

∴ The total annual costs of lining = C/Y + C/2(r/100)

$$\therefore \text{Benefit cost ratio} = \frac{\text{Annual Benefits}}{\text{Annual Costs}} = \frac{mR_1 + pR_2}{\frac{C}{Y} + \frac{C}{2} \times \frac{r}{100}}$$

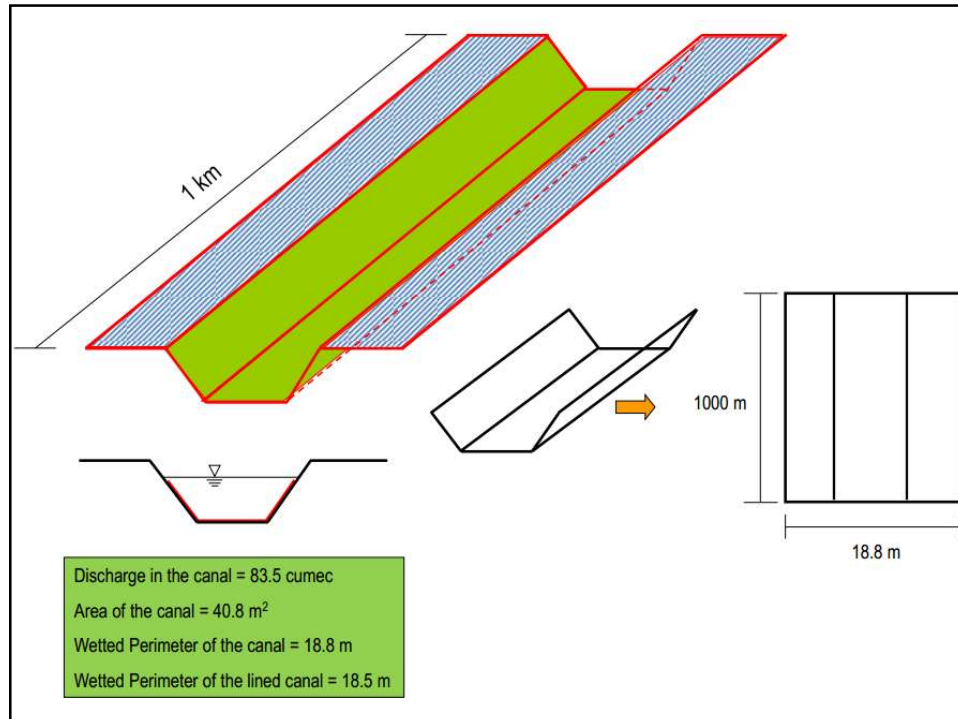
If p is taken as 0.4, then

$$\therefore \text{Benefit cost ratio} = \frac{mR_1 + 0.4R_2}{\frac{C}{Y} + \frac{C}{2} \times \frac{r}{100}}$$

### Problem

An unlined canal giving a seepage loss of 3.3 cumec per million square meters of wetted area is proposed to be lined with 10 cm thick cement concrete lining, which costs Rs. 180 per 10 square meters. Given the following data, work out the economics of lining and benefit cost ratio.

Annual revenue per cumec of water from all crops	Rs. 3.5 lakhs
Discharge in the channel	83.5 cumecs
Area of the channel	40.8 m <sup>2</sup>
Wetted perimeter of the channel	18.8 m
Wetted perimeter of the lining	18.5 m
Annual maintenance cost of unlined channel per 10 square meter	Rs. 1.0

**Solution:**

Let us consider 1 km (= 1000 m) reach of canal. Therefore,  
 the wetted surface per km =  $18.8 \times 1000 = 18,800 \text{ m}^2$

**(i) Annual Benefits****(a) Seepage loss**

Seepage loss in unlined canal @ 3.3 cumec per million sq. m

$$= (3.3/10^6) \times 18,800 \text{ cumec/km} = 62,040 \times 10^{-6} \text{ cumec/km}$$

Assume, seepage loss in lined channel at 0.01 cumec per million square meter of wetted perimeter

$$\therefore \text{Seepage loss in unlined canal} = (0.01/10^6) \times 18,800 = 188 \times 10^{-6} \text{ cumecs/km}$$

$$\text{Net saving} = (62,040 \times 10^{-6} - 188 \times 10^{-6}) \text{ cumec/km} = 0.06185 \text{ cumec/km}$$

$$\text{Annual revenue saved per km of channel} = (0.06185 \times 3.5) \text{ lakhs}$$

$$= 0.21648 \text{ lakhs}$$

**(b) Saving in maintenance**

Annual maintenance cost of unlined channel for  $10 \text{ m}^2 = \text{Rs. } 1$

Total wetted perimeter per 1 km length =  $18,800 \text{ m}^2$

$\therefore$  Annual maintenance charge for unlined channel/ km = **Rs. 1,880**

Assume that 40% of this is saved in lined channel

Annual saving in maintenance charges =  $\text{Rs. } (0.4 \times 1880) = \text{Rs. } 752$

$\therefore$  Total annual benefits per km =  $\text{Rs. } (21,648 + 752) = \text{Rs. } 22,400$

**(ii) Annual Costs**

Area of lining per km of channel =  $18.5 \times 1000 = 18500 \text{ m}^2$

Cost of lining per km of channel @  $\text{Rs. } 180$  per  $10 \text{ m}^2$

=  $\text{Rs. } (18500 \times 180 / 10) = \text{Rs. } 3,33,000$

Assume, life of lining as 40 years

Depreciation cost per year =  $\text{Rs. } (3,33,000) / 40 = \text{Rs. } 8,325$

Assume 5% rate of interest

Average annual interest =  $C/2 (r/100) = 3,33,000 / 2 \times (5/100) = \text{Rs. } 8,325$

$\therefore$  Total annual cost =  $\text{Rs. } (8,325 + 8,325) = \text{Rs. } 16,650$

Benefit cost ratio = Annual benefits/Annual costs =  $22,400 / 16,650 = 1.35$

Benefit cost ratio is **more than unity**, and hence, the lining is justified.

## DIVERSION HEADWORKS

### **Diversion Headworks:**

The works, which are constructed at the head of the canal, in order to divert the river water towards the canal, so as to ensure a regulated continuous supply of silt-free water with a certain minimum head into the canal, are known as diversion heads works.

A diversion headworks must be differentiated from storage work or dam.

On the other hand, diversion headworks are constructed on perennial rivers, which have adequate flow. Therefore, there is no necessity of creating a storage reservoir or very little storage, if any.

**Objectives/Purposes of Diversion Headworks**

- To rise the water level at the head of the canal
- To form a storage by constructing dykes (embankments) on both the banks of the river so that water is available throughout the year
- To control the entry of silt into the canal and to control the deposition of silt at the head of the canal
- To control the fluctuation of water level in the river during different seasons

**Classification of Diversion Headworks**

- (i) Temporary
- (ii) Permanent

**Temporary Diversion Head Works**

For temporary diversion headworks, spurs or bunds are constructed across the source river. Since floods in the source river can damage such bunds, it may be necessary to repair such bunds rather frequently, and construct them after every flood that occurs in the source river.

**Permanent Diversion Head Works**

Permanent headworks have a permanent structure in the form of a weir (or a barrage) constructed across the river. Most of the headworks of important canal systems are of permanent type.

Sometimes temporary headworks are constructed in the beginning and they are replaced by permanent headworks when the demand for irrigation has developed sufficiently.

### **Ideal Site for diversion Head works**

- The river section at the site should be narrow and well-defined
- The river should have high; well-defined, ineradicable and non-submersible banks so that the cost of river training works are minimum
- The site should be such that the canal commands maximum irrigable areas, with moderate earth works
- There should be suitable arrangement for the diversion of river during construction
- The site should be such that the weir (or barrage) can be aligned at right angles to the direction of the river flow
- There should be suitable locations for the under sluices, head regulator and other components of the diversion head works

### **Ideal Site for diversion Head works**

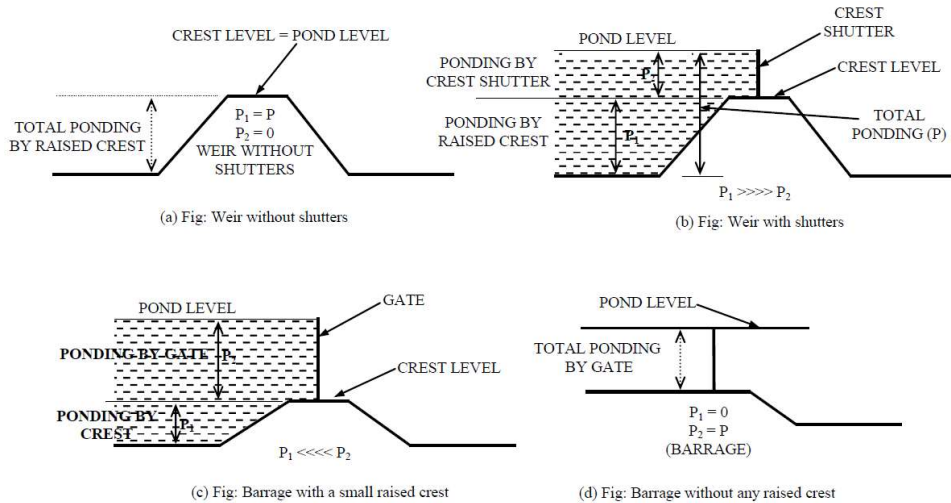
- Good foundation should be available at the site
- The required materials of construction should be available near the site
- The site should be easily accessible
- The overall cost of the project should be minimum one

### **Weir and Barrage**

It is a barrier constructed across the river to raise the water level on the upstream side of the obstruction in order to feed the main canal.

The ponding of water can be achieved either only by a raised crest across the river or by a raised crest supplemented by gates or shutters, working over the crest.

## Weir and Barrage



## Weir

If the major part or the entire ponding of water is achieved by a raised crest and a smaller part or nil part of it is achieved by the shutters, then this barrier is known as a weir.

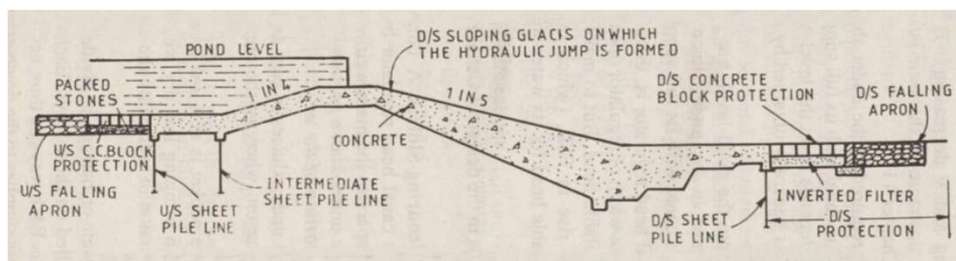


Fig: A typical cross-section of a modern concrete weir

## Classification of Weir

### A) Classification Based on Functions

- |                 |                    |
|-----------------|--------------------|
| i) Storage weir | iii) Pickup weir   |
| ii) Waste weir  | iv) Diversion weir |

## **Classification of Weir**

### **B) Classification Based on Design Aspects**

- i) Gravity weirs
- ii) Non- Gravity weirs

### **C) Classification Based on Construction Material**

- i) Masonry weir with vertical drop
- ii) Rock-fill weir with sloping aprons
- iii) Concrete weir with sloping glacis

### **A) Classification Based on Functions**

- i) Storage weir

It is a weir permanently constructed for storage of water. It is also termed as low dam.

- ii) Waste weir

It serves as spillway to escape floods which cannot be stored in the pond. It is constructed to ensure safety of the weir proper. Waste weir comes into operation when water level in the pond rises above the designed pond level.

- iii) Pickup weir

A weir constructed at the offtake of a canal across the river to raise the water level for feeding the canal. The term is generally applied to a weir across the river on which there is a storage reservoir or a dam upstream. It is constructed as an adjunct to a reservoir.

- iii) Diversion weir

It is constructed as a constituent part of the headworks to raise water level in the river and divert supplies into the offtaking channel.

### **A) Classification Based on Design Aspects**

- i) Gravity weir

When the weight of the weir (i.e. its body and floor) balances the uplift pressure caused by the head of the water seeping below the weir, it is called a gravity weir.

- ii) Non- Gravity weir

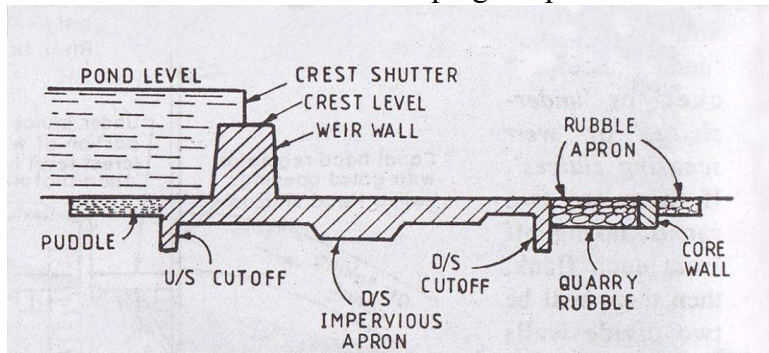
If the weir floor is designed continuous with the divide piers as reinforced structure, such that the weight of concrete slab together with the weight of divide piers keep the structure safe against the uplift then the structure may be called as a non-gravity weir.

## Classification of Weir

### B) Classification Based on Construction Material

#### i) Masonry weir with vertical drop

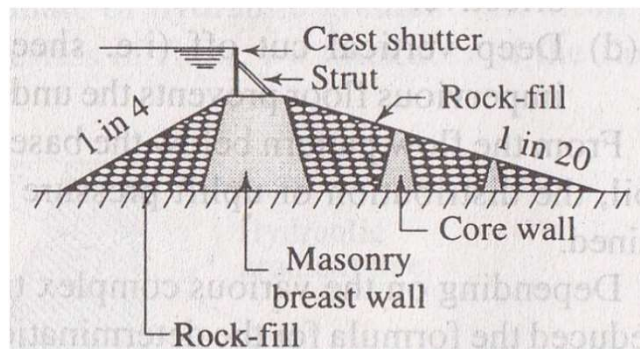
Masonry weir wall is constructed over the impervious floor. Cut-off walls are provided at both ends of the floor. Sheet piles are provided below the cut off walls. The crest shutters are provided to raise the water level, if required. The shutters are dropped down during flood. The masonry weir wall may be vertical on both face or sloping on both face or vertical on downstream face and sloping in upstream face.



## Classification of Weir

#### ii) Rock-fill weir with sloping aprons

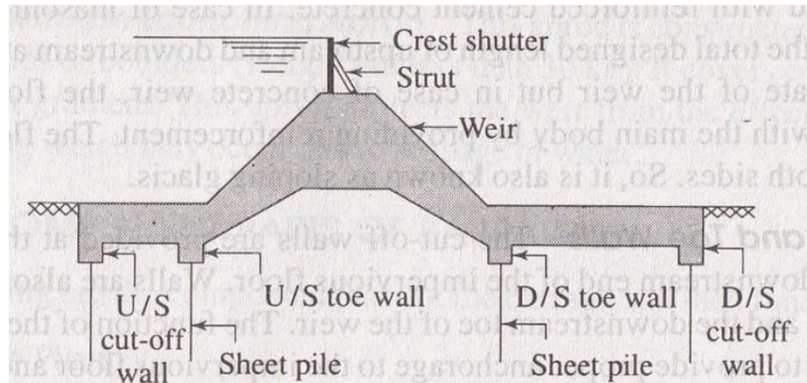
It consists of masonry breast wall which is provided with adjustable crest shutter. The upstream rock-fill portion is constructed with boulders forming a slope of 1 in 4. The boulders are grouted with cement mortar. The downstream sloping apron consists of core walls. The intermediate spaces between the core walls are filled up with boulders maintaining a slope of 1 in 20. The boulders are grouted properly with cement mortar.



### Classification of Weir

iii) Concrete weir with sloping glacis

Now-a-days, the weir is constructed with reinforced cement concrete. The impervious floor and the weir are made monolithic. The cut off walls are provided at the upstream and downstream end of the floor and at the toe of the weir. Sheet piles are provided below the cut-off walls. The crest shutters are also provided which are dropped down during the flood.



### Barrage

If most of the ponding is done by gates and a smaller or nil part of it is done by the raised crest, then the barrier is known as a barrage or a river regulator.

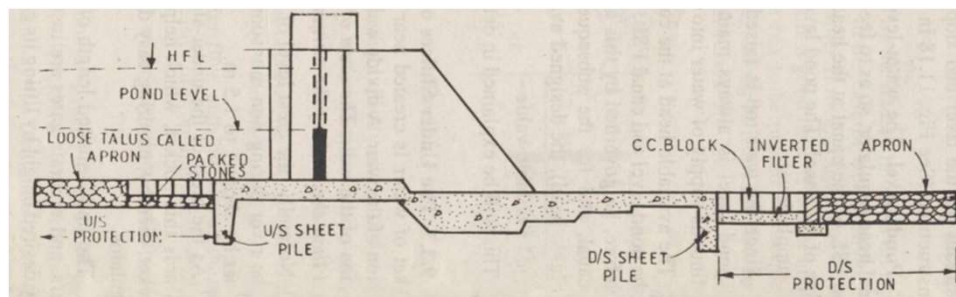


Fig: A typical cross-section of a barrage

### **Afflux**

The rise in the highest flood level (HFL) upstream of the weir due to construction of the weir across the river is called afflux. In case of weir, the afflux caused during high floods is quite high. But in case of a barrage, the gates can be opened during high floods and the afflux will be nil or minimum.

### **Choice between a weir and a barrage**

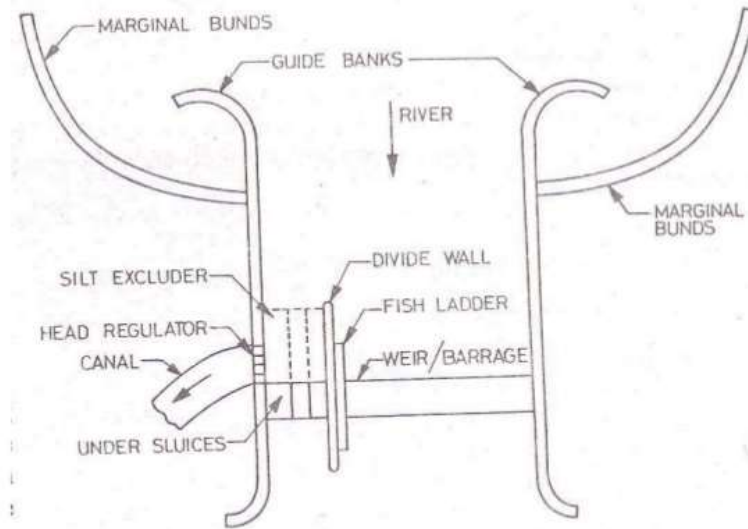
The choice between a weir and a barrage is largely governed by cost and convenience in working.

- A shuttered weir will be relatively cheaper but will lack the effective control possible in the case of a barrage.
- A barrage type construction can be easily supplemented with a roadway across the river at a small additional cost. Barrages are almost invariably constructed now-a-days on all important rivers.

#### **Difference between Barrage and Weir**

SL	Barrage	Weir
(a)	Low set crest	High set crest
(b)	Ponding is done by means of gates	Ponding is done against the raised crest or partly against crest and partly by shutters
(c)	Gated over entire length	Shutters in part length
(d)	Gates are of greater height	Shutters are of smaller height, 2 m
(e)	Gates are raised clear off the high floods to pass floods	Shutters are dropped to pass floods
(f)	Perfect control on river flow	No control of river in low floods
(g)	Gates convenient to operate	Operation of shutters is slow, involve labour and time
(h)	High floods can be passed with minimum afflux	Excessive afflux in high floods
(i)	Less silting upstream due to low set crest	Raised crest causes silting upstream
(j)	Longer construction period	Shorter construction period
(k)	Silt removal is done through under sluices	No means for silt disposal
(l)	Road and/or rail bridge can be constructed at low cost	Not possible to provide road-rail bridge
(m)	Costly structure	Relatively cheaper structure

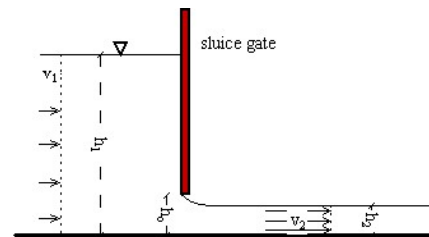
## Layout of a Diversion Headworks and its Components



Component parts of Weir/Barrage

### Under-sluices:

The under sluices are the openings provided at the base of the weir or barrage. These openings are provided with adjustable gates. Normally, the gates are kept closed. The crest of the under-sluice portion of the weir is kept at a lower level (1 to 1.5 m) than the crest of the normal portion of the weir. The suspended silt goes on depositing in front of the canal head regulator. When the silt deposition becomes appreciable the gates are opened and the deposited silt is loosened with an agitator mounting on a boat. The muddy water flows towards the downstream through the scouring sluices. The gates are then closed. But, at the period of flood, the gates are kept opened.



**Under-sluices:**

The main functions of under-sluices are:

- To maintain a well defined deep channel approaching the canal head regulator
- To ensure easy diversion of water into the canal through the canal head regulator even during low flow
- To control the entry of silt into the canal
- To help scouring and of the silt deposited over the under-sluice floor and removing towards the downstream side
- To help passing the low floods without dropping the shutters of the weir

**The divide wall:**

The divide wall is a masonry or concrete wall constructed at right angle to the axis of the weir. It extends on the upstream side beyond the beginning of the canal head regulator; and on the downstream side, it extends up to the end of the loose protection of the under-sluices. The divide wall is a long wall constructed at right angles in the weir or barrage, it may be constructed with stone masonry or cement concrete.

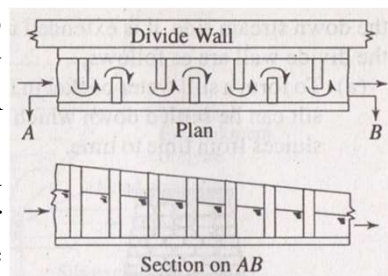
**The divide wall:**

The main functions of the divide wall:

- It separates the 'under-sluices' with lower crest level from the 'weir proper' with higher crest level.
- It helps in providing a comparatively less turbulent pocket near the canal head regulator, resulting in deposition of silt in this pocket and, thus, to help in the entry of silt-free water into the canal.
- It helps to keep cross-current, if any, away from the weir.

**Fish Ladder:**

It is device by which the flow energy can be dissipated in such a manner as to provide smooth flow at sufficiently low velocity, not exceeding 3 to 3.5 m/s. A narrow opening including suitable baffles or staggering devices in it is provided adjacent to the divide wall. The fish ladder is provided just by the side of the divide wall for the free movement of fishes.



**Fish Ladder:**

In general, the tendency of fish is to move from upstream to downstream in winters and from downstream to upstream in monsoons. This movement is essential for their survival. Due to construction of weir or barrage, this movement gets obstructed, and is detrimental to the fishes. The width, length and height of the fish ladder depend on the nature of the river and the type of the weir or barrage.

**Canal Head Regulator or Head sluices:**

A structure which is constructed at the head of the canal to regulate flow of water is known as canal head regulator. It consists of a number of piers which divide the total width of the canal into a number of spans which are known as bays. The piers consist of number tiers on which the adjustable gates are placed. The gates are operated from the top by suitable mechanical device. A platform is provided on the top of the piers for the facility of operating the gates. Again some piers are constructed on the down stream side of the canal head to support the roadway.

**Canal Head Regulator or Head sluices:**

Functions of Canal Head Regulator:

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

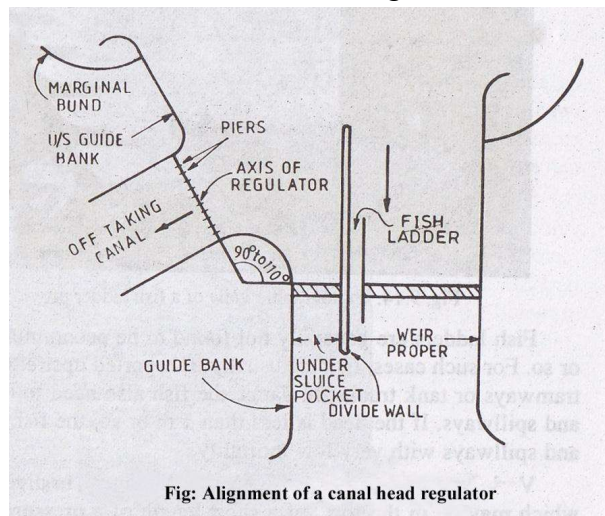


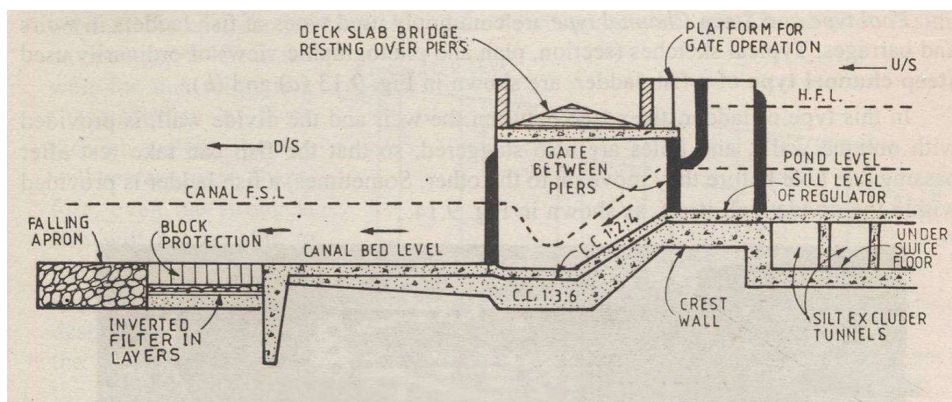
Fig: Alignment of a canal head regulator

### Canal Head Regulator or Head sluices:

The water from the under-sluice pocket is made to enter the regulator bays, so as to pass the full supply discharge into the canal. The maximum height of these gated openings, called head sluices will be equal to the difference of Pond Level and Crest Level of the regulator.

- The entry of silt into the canal is controlled by keeping the crest of the head regulator by about 1.2 to 1.5 meters higher than the crest of the under-sluices.
- If a silt-excluder is provided, the regulator crest is further raised by about 0.6 to 0.7 meter.
- Silt gets deposited in the pocket, and only the clear water enters the regulator bays.
- The deposited silt can be easily scoured out periodically, and removed through the under-sluice openings.

### Canal Head Regulator or Head sluices:



A typical section through a Canal Head Regulator (CHR)

**River Training Works:**

River training works are required near the weir site in order to ensure a smooth and an axial flow of water, and thus, to prevent the river from outflanking the works due to a change in its course.

The river training works required on a canal headwork are:

- (a) Guide banks
- (b) Marginal bunds
- (c) Spurs or Groyne

**(a) Guide Bank**

When a barrage is constructed across a river which flows through the alluvial soil, the guide banks must be constructed to help to secure a suitable river approach to flow axially through the barrage.

Guide bank serves the following purposes:

- It protects the barrage from the effect of scouring and erosion
- It provides a straight approach towards the barrage
- It controls the tendency of changing the course of the river
- It controls the velocity of flow near the structure

**(b) Marginal Bunds**

The marginal bunds are earthen embankments which are constructed parallel to the river bank on one or both the banks according to the condition. The top width is generally 3 m to 4 m. The side slope on the river side is generally 1.5: 1 and that on the country side is 2:1.

The marginal bunds serve the following purposes:

- It prevents the flood water or storage water from entering the surrounding area which may be submerged or may be water logged
- It retains the flood water or storage water within a specified section
- It protects the towns and villages from devastation during the heavy flood
- It protects valuable agricultural lands

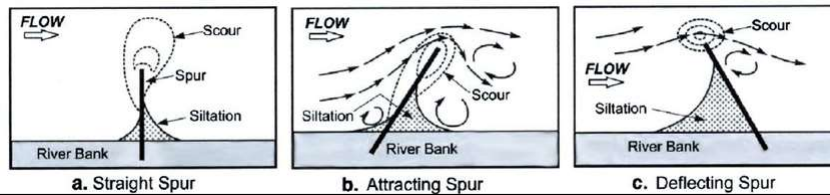
**(c) Spurs or Groyne**

They are the structures constructed transverse to the river flow. They extend from the bank into the river.

### (c) Spurs or Groynes

They serve the following purposes:

- Protect the river bank by keeping the flow away from it
- Create still pond along a particular bank with the aim of silting up the area in the vicinity
- Train the river to flow along a desired course by attracting, deflecting or repelling the flow
- Contract the wide river channel for improving the navigation depth



### Causes of Failure of Weir or Barrage on Permeable Foundation

1. Failure due to Subsurface Flow
  - (a) Failure by piping or undermining
  - (b) Failure by direct uplift
2. Failure due to Surface Flow
  - (a) By hydraulic jump
  - (b) By scouring during floods

#### 1. Failure due to Subsurface Flow

##### (a) Failure by piping or undermining

The water from the upstream side continuously percolates through the bottom of the foundation and emerges at the downstream end of the weir or barrage floor. The force of percolating water removes the soil particles by scouring at the point of emergence. As the process of removal of soil particles goes on continuously, a depression is formed which extends backwards towards the upstream through the bottom of the foundation. A hollow pipe like formation thus develops under the foundation due to which the weir or barrage may fail by subsiding. This phenomenon is known as failure by piping or undermining.

### **Causes of Failure of Weir or Barrage on Permeable Foundation**

Remedies:

- Decrease Hydraulic gradient i.e. increase path of percolation by providing sufficient length of impervious floor
- Providing curtains or piles at both upstream and downstream

#### **1. Failure due to Subsurface Flow**

(b) Failure by direct uplift

The percolating water exerts an upward pressure on the foundation of the weir or barrage. If this uplift pressure is not counterbalanced by the self weight of the structure, it may fail by rapture.

Remedies:

- Providing sufficient length of the impervious floor
- Providing impervious floor of appropriate thickness at various points
- Providing pile at upstream end to reduce uplift pressure downstream

### **Causes of Failure of Weir or Barrage on Permeable Foundation**

#### **2. Failure due to Surface Flow**

(a) By hydraulic jump

When the water flows with a very high velocity over the crest of the weir or over the gates of the barrage, then hydraulic jump develops. This hydraulic jump causes a suction pressure or negative pressure on the downstream side which acts in the direction uplift pressure. If the thickness of the impervious floor is insufficient, then the structure fails by rapture.

Remedies:

- Providing additional thickness of floor to counterbalance the extra pressure due to hydraulic jump
- Constructing the floor thickness in one concrete mass instead of in masonry layers

## Causes of Failure of Weir or Barrage on Permeable Foundation

### 2. Failure due to Surface Flow

(b) By scouring during floods

By scouring during floods, the gates of the barrage are kept open and the water flows with high velocity. The water may also flow with very high velocity over the crest of the weir. Both the cases can result in scouring effect on the downstream and on the upstream side of the structure. Due to scouring of the soil on both sides of the structure, its stability gets endangered by shearing.

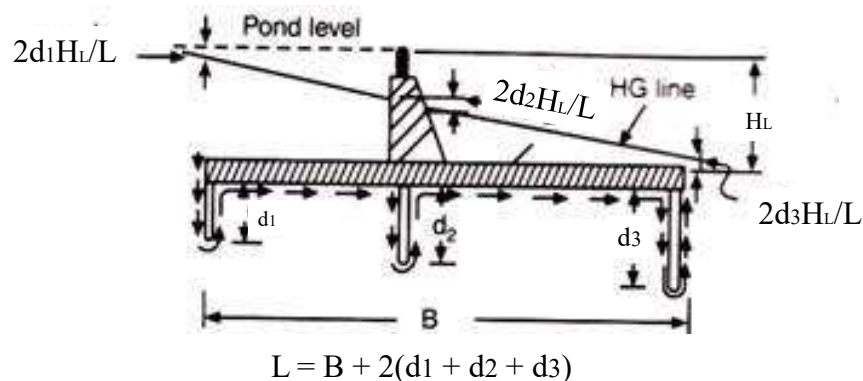
Remedies:

- Taking the piles at upstream and downstream ends of the impervious floor, much below the calculated scoured level
- Providing suitable length and thickness of launching apron at upstream and downstream sides, so that stones of the aprons may settle in the scour holes

## Bligh's, Lane's and Khosla's Seepage Theory

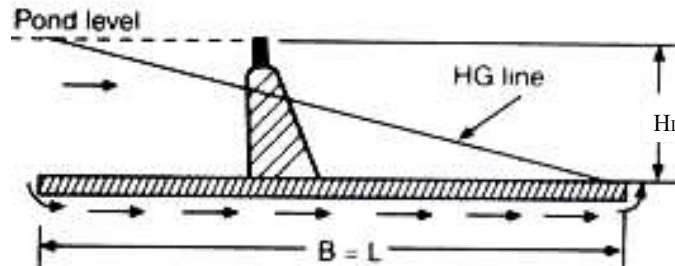
### 1. Bligh's Creep Theory

According to Bligh's Theory, the percolating water follows the outline of the base of the foundation of the hydraulic structure. In other words, water creeps along the bottom contour of the structure. The length of the path thus traversed by water is called the length of the creep. Further, it is assumed in this theory, that the loss of head is proportional to the length of the creep.



## Bligh's, Lane's and Khosla's Seepage Theory

### 1. Bligh's Creep Theory



If  $H_L$  is the total head loss between the upstream and the downstream, and  $L$  is the length of creep, then the loss of head per unit of creep length (i.e.  $H_L/L$ ) is called the hydraulic gradient. Further, Bligh makes no distinction between horizontal and vertical creep.

Bligh called the loss of head per unit length of creep ( $H_L/L$ ) as percolation coefficient. The reciprocal, ( $L/H_L$ ) of the percolation coefficient is known as the coefficient of creep  $C$ .

## Bligh's, Lane's and Khosla's Seepage Theory

### 1. Bligh's Creep Theory

#### (i) Safety against piping or undermining:

According to Bligh, the safety against piping can be ensured by providing sufficient creep length, given by

$$L = C.H_L,$$

where  $C$  is the Bligh's Coefficient for the soil.

Different values of  $C$  for different types of soils:

SL No.	Type of Soil	Value of C	Safe Hydraulic gradient should be less than
1	Fine micaceous sand	15	1/15
2	Coarse grained sand	12	1/12
3	Sand mixed with boulder and gravel, and for loam soil	5 to 9	1/5 to 1/9
4	Light sand and mud	8	1/8

Note: The hydraulic gradient i.e.  $H_L/L$  is then equal to  $1/C$ . Hence, it may be stated that the hydraulic gradient must be kept under a safe limit in order to ensure safety against piping.

## Bligh's, Lane's and Khosla's Seepage Theory

### 1. Bligh's Creep Theory

#### (ii) Safety against uplift pressure

The ordinates of the HGL above the bottom of the floor represent the residual uplift water head at each point. Say for example, if at any point, the ordinate of HGL above the bottom of the floor is 1 m, then 1 m head of water will act as uplift at that point. If  $h'$  meters is this ordinate, then water pressure equal to  $h'$  meters will act at this point, and has to be counterbalanced by the weight of the floor of thickness say  $t$ .

Uplift pressure =  $\gamma_w \times h'$  [where  $\gamma_w$  is the unit weight of water]

Downward pressure =  $(\gamma_w \times G).t$  [Where  $G$  is the specific gravity of the floor material]

For equilibrium,

$$\gamma_w \times h' = \gamma_w \times G. t$$

$$h' = G \times t$$

## Bligh's, Lane's and Khosla's Seepage Theory

### 1. Bligh's Creep Theory

#### (ii) Safety against uplift pressure

Subtracting  $t$  on both sides, we get

$$(h' - t) = (G \times t - t) = t(G - 1)$$

$$t = (h' - t) / (G - 1) = h' / (G - 1)$$

where,  $h' - t = h =$  Ordinate of the HGL above the top of the floor

$G - 1 =$  Submerged specific gravity of the floor material

#### Assumptions:

- Hydraulic slope or gradient is constant throughout the impervious length of the apron.
- The percolating water creep along the contact of the base profile of the apron with the sub soil losing head enroute, proportional to length of its travel. The length is called creep length. It is the sum of horizontal and vertical creep
- Stoppage of percolation by cut off (pile) possible only if it extends up to impermeable soil strata.

## Bligh's, Lane's and Khosla's Seepage Theory

### 1. Bligh's Creep Theory

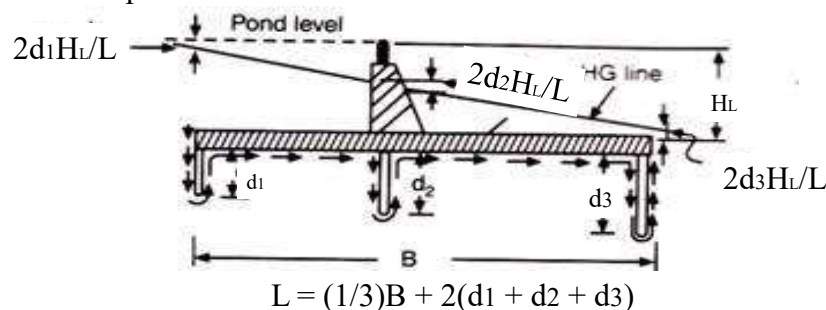
#### Limitations of Bligh's theory:

1. This theory made no distinction between horizontal and vertical creep.
2. Did not explain the idea of exit gradient - safety against undermining cannot simply be obtained by considering a flat average gradient but by keeping this gradient will be low critical.
3. No distinction between outer and inner faces of sheet piles or the intermediate sheet piles, whereas from investigation it is clear, that the outer faces of the end sheet piles are much more effective than inner ones.
4. Losses of head does not take place in the same proportions as the creep length. Also the uplift pressure distribution is not linear but follow a sine curve.
5. In case of two piles the spacing greater than twice the head or the piles depth are not effective.

## Bligh's, Lane's and Khosla's Seepage Theory

### 2. Lane's Weighted Creep Theory

Bligh, in his theory, had calculated the length of the creep, by simply adding the horizontal creep length and the vertical creep length, thereby making no distinction between the two creeps. However, Lane, on the basis of his analysis carried out on about 200 dams all over the world, stipulated that the horizontal creep is less effective in reducing uplift (or in causing loss of head) than the vertical creep. He, therefore, suggested a weightage factor of 1/3 for the horizontal creep, as against 1.0 for the vertical creep.



## Bligh's, Lane's and Khosla's Seepage Theory

### 2. Lane's Weighted Creep Theory

To ensure safety against piping, according to this theory, the creep length  $L_1$  must not be less than  $C_1 H_L$ , where  $H_L$  is the head causing flow, and  $C_1$  is Lane's creep coefficient given in table below.

Values of Lane's Safe Hydraulic Gradient for different types of Soils

SL No.	Type of Soil	Value of Lane's Coefficient $C_1$	Safe Lane's Hydraulic gradient should be less than
1	Very fine sand or silt	8.5	1/8.5
2	Fine sand	7.0	1/7
3	Coarse sand	5.0	1/5
4	Gravel and sand	3.5 to 3.0	1/3.5 to 1/3
5	Boulders, gravels and sand	2.5 to 3.0	1/2.5 to 1/3
6	Clayey soils	3.0 to 1.6	1/3 to 1/1.6

## Bligh's, Lane's and Khosla's Seepage Theory

### 3. Khosla's Theory and Concept of Flow Nets

The main principles of this theory are summarized below:

- The seepage water does not creep along the bottom contour of pucca floor as started by Bligh, but on the other hand, this water moves along a set of stream-lines. This steady seepage in a vertical plane for a homogeneous soil can be expressed by Laplacian equation:

$$\frac{d^2 \phi}{dx^2} + \frac{d^2 \phi}{dz^2} = 0$$

where,  $\phi$  = Flow potential =  $Kh$ ;

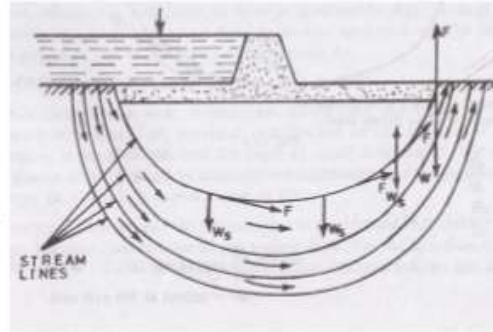
$K$  = the co-efficient of permeability of soil as defined by Darcy's law, and  $h$  is the residual head at any point within the soil.

The above equation represents two sets of curves intersecting each other orthogonally. The resultant flow diagram showing both of the curves is called a Flow Net.



### Exit Gradient

The seepage water exerts a force at each point in the direction of flow and tangential to the streamlines as shown in figure above. This force (F) has an upward component from the point where the streamlines turns upward.



This force has the maximum disturbing tendency at the exit end, because the direction of this force at the exit point is vertically upward, and hence full force acts as its upward component. For the soil grain to remain stable, the submerged weight of soil grain should be more than this upward disturbing force. The disturbing force at any point is proportional to the gradient of pressure of water at that point (i.e.  $dp/dl$ ). This gradient of pressure of water at the exit end is called the exit gradient. In order that the soil particles at exit remain stable, the upward pressure at exit should be safe. In other words, the exit gradient should be safe.

### Critical Exit Gradient

This exit gradient is said to be critical, when the upward disturbing force on the grain is just equal to the submerged weight of the grain at the exit. When a factor of safety equal to 4 to 5 is used, the exit gradient can then be taken as safe. In other words, an exit gradient equal to  $1/4$  to  $1/5$  of the critical exit gradient is ensured, so as to keep the structure safe against piping.

The submerged weight ( $W_s$ ) of a unit volume of soil is given as:

$$\gamma_w (1 - n) (S_s - 1)$$

Where,  $\gamma_w$  = unit weight of water.

$S_s$  = Specific gravity of soil particles

$n$  = Porosity of the soil material

For critical conditions to occur at the exit point

$$F = W_s$$

Where F is the upward disturbing force on the grain

Force F = pressure gradient at that point =  $dp/dl = \gamma_w \times dh/dl$

$$\therefore \gamma_w \cdot \frac{dh}{dl} = \gamma_w (1 - n) (S_s - 1)$$

$$\frac{dh}{dl} = (1 - n) (S_s - 1)$$

**Khosla's Method of independent variables for determination of pressures and exit gradient for seepage below a weir or a barrage**

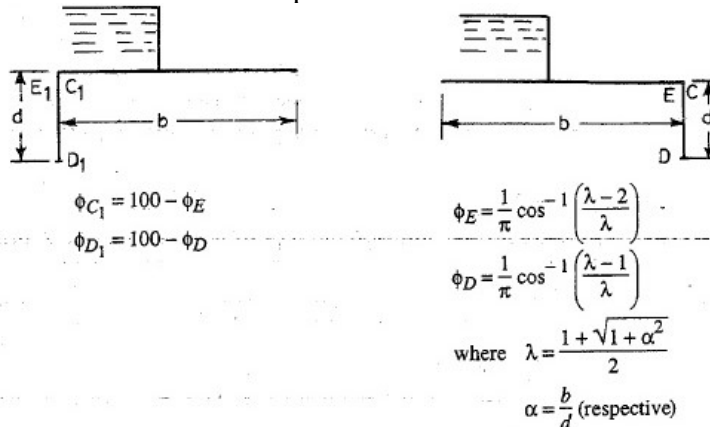
In order to know as to how the seepage below the foundation of a hydraulic structure is taking place, it is necessary to plot the flow net. In other words, we must solve the *Laplacian equations*.

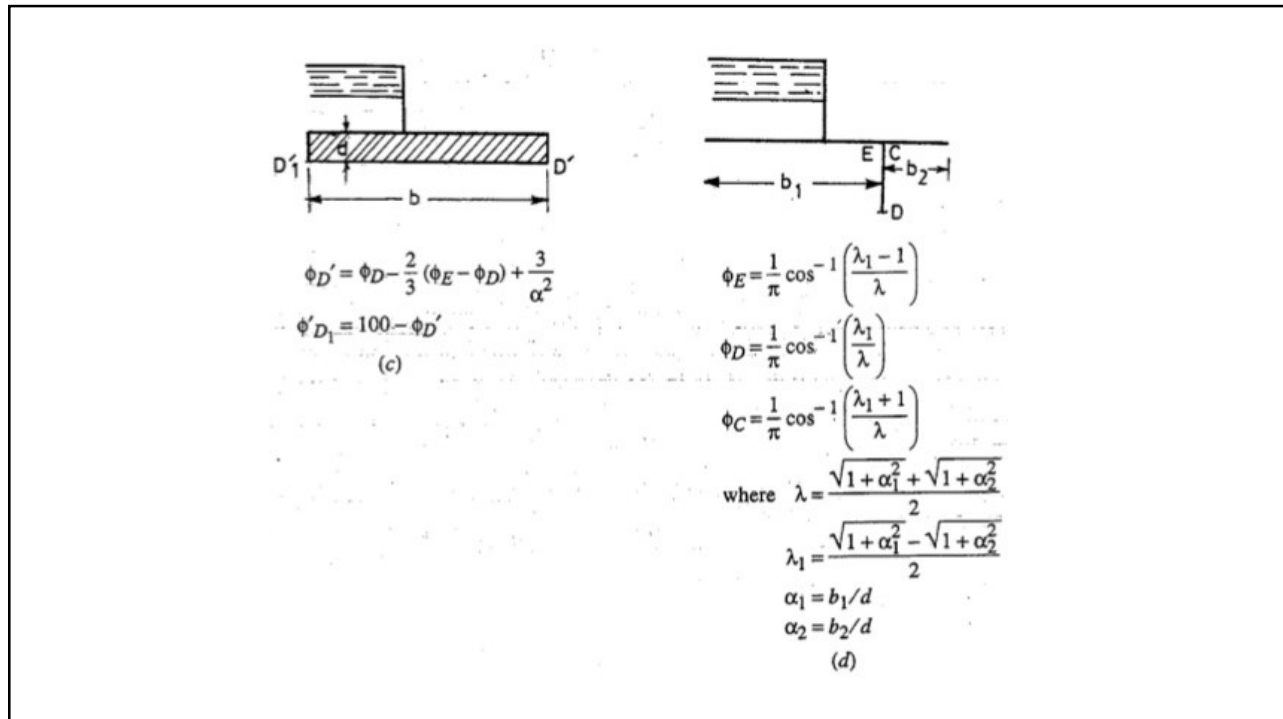
*This can be accomplished either by mathematical solution of the Laplacian equations, or by Electrical analogy method, or by graphical sketching by adjusting the streamlines and equipotential lines with respect to the boundary conditions. These are complicated methods and are time consuming.*

*Therefore, for designing hydraulic structures such as weirs or barrage or pervious foundations, Khosla has evolved a simple, quick and an accurate approach, called **Method of Independent Variables**.*

**The simple profiles which are most useful in analysis are:**

- A straight horizontal floor of negligible thickness with a sheet pile line on the upstream end and downstream end.
- A straight horizontal floor depressed below the bed but without any vertical cutoffs.
- A straight horizontal floor of negligible thickness with a sheet pile line at some intermediate point.





The key points are the junctions of the floor and the pole lines on either side, and the bottom point of the pile line, and the bottom corners in the case of a depressed floor.

The percentage pressures at these key points for the simple forms into which the complex profile has been broken is valid for the complex profile itself, if corrected for

- (a) Correction for the mutual interference of piles
- (b) Correction for the thickness of floor
- (c) Correction for the slope of the floor

(a) Correction for the Mutual interference of Piles

The correction,  $C$ , to be applied as percentage of head due to this effect, is given by

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

Where,

$b'$  = The distance between two pile lines.

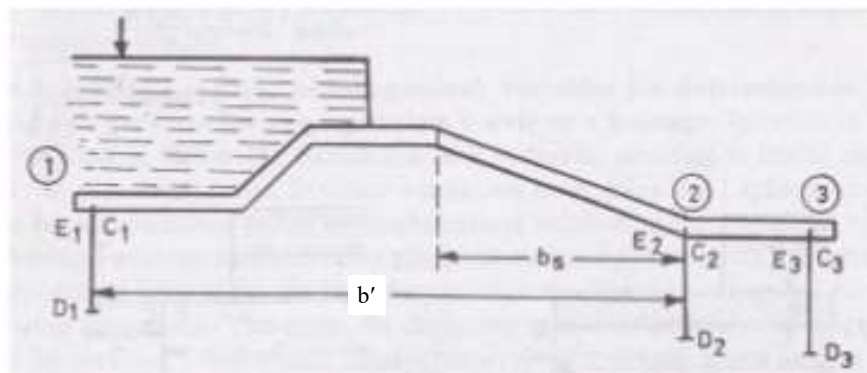
$D$  = The depth of the pile line, the influence of which has to be determined on the neighboring pile of depth,  $d$ .  $D$  is to be measured below the level at which interference is desired.

$d$  = The depth of the pile on which the effect is considered

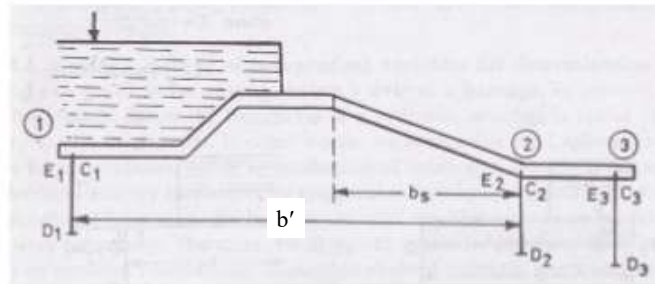
$b$  = Total floor length

(a) Correction for the Mutual interference of Piles

The correction is positive for the points in the rear of back water, and subtractive for the points forward in the direction of flow.



### (a) Correction for the Mutual interference of Piles

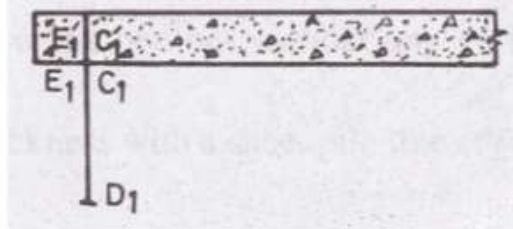


Suppose in the above figure, we are considering the influence of the pile no (2) on pile no (1) for correcting the pressure at C1. Since the point C1 is in the rear, this correction shall be positive.

While the correction to be applied to E2 due to pile no (1) shall be negative, since the point E2 is in the forward direction of flow. Similarly, the correction at C2 due to pile no (3) is positive and the correction at E3 due to pile no (2) is negative.

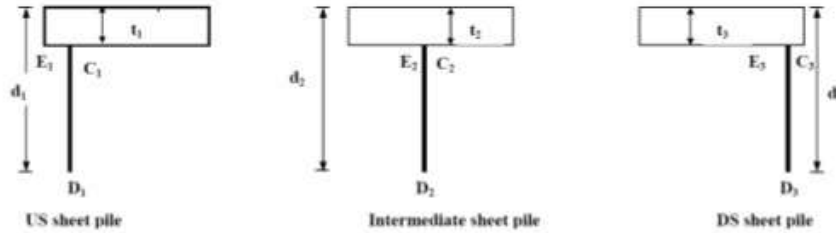
### (b) Correction for the thickness of floor

In the standard form profiles, the floor is assumed to have negligible thickness. Hence, the percentage pressures calculated by Khosla's equations or graphs shall pertain to the top levels of the floor. While the actual junction points *E* and *C* are at the bottom of the floor. Hence, the pressures at the actual points are calculated by assuming a straight line pressure variation.



Since the corrected pressure at E1 should be less than the calculated pressure at E1', the correction to be applied for the joint E1 shall be negative. Similarly, the pressure calculated C1' is less than the corrected pressure at C1, and hence, the correction to be applied at point C1 is positive.

## (b) Correction for the thickness of floor



$\varphi_{E1} = 1$  don't need any correction

$$(C_t)_{C1} = \frac{t_1 \cdot (\varphi_{D1} - \varphi_{C1})}{d_1} \quad (+ve)$$

$$(C_t)_{E2} = \frac{t_2 \cdot (\varphi_{E2} - \varphi_{D2})}{d_2} \quad (-ve)$$

$$(C_t)_{C2} = \frac{t_2 \cdot (\varphi_{D2} - \varphi_{C2})}{d_2} \quad (+ve)$$

$C_t$  represent correction

$$(C_t)_{E3} = \frac{t_3 \cdot (\varphi_{E3} - \varphi_{D3})}{d_3} \quad (-ve)$$

$\varphi_{C3} = 0$  don't need any correction

## (c) Correction for the slope of the floor

A correction is applied for a slopping floor, and is taken as **positive for the downward slopes, and negative for the upward slopes following the direction of flow. Values of correction of standard slopes such as 1 : 1, 2 : 1, 3 : 1, etc. are tabulated below**

Slope (H : V)	Correction Factor
1 : 1	11.2
2 : 1	6.5
3 : 1	4.5
4 : 1	3.3
5 : 1	2.8
6 : 1	2.5
7 : 1	2.3
8 : 1	2.0

\*\*The correction factor given above is to be multiplied by the horizontal length of the slope and divided by the distance between the two pile lines between which the sloping floor is located. This correction is applicable only to the key points of the pile line fixed at the start or the end of the slope.

### Exit gradient (GE)

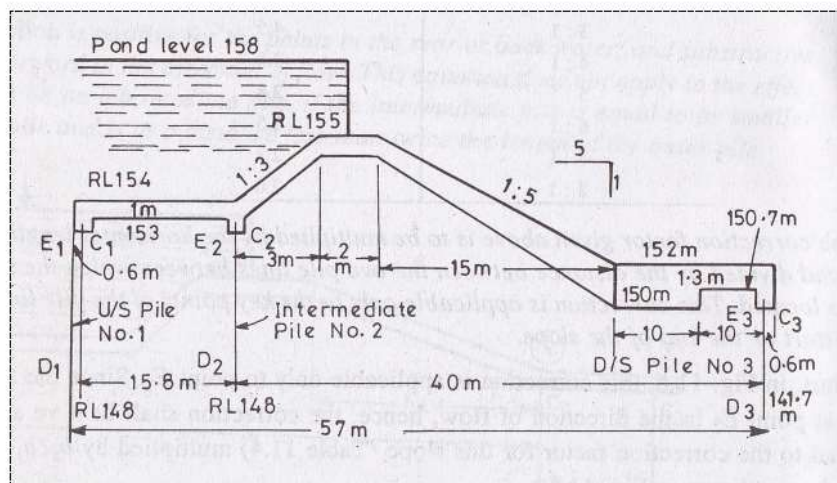
It has been determined that for a standard form consisting of a floor length ( $b$ ) with a vertical cutoff of depth ( $d$ ), the exit gradient at its downstream end is given by

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$$

Where,  $\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2}$   
 $\alpha = b/d$   
 $H = \text{Maximum Seepage Head}$

Type of Soil	Safe exit gradient
Shingle	1/4 to 1/5 (0.25 to 0.20)
Coarse Sand	1/5 to 1/6 (0.20 to 0.17)
Fine Sand	1/6 to 1/7 (0.17 to 0.14)

**Problem:** Determine the percentage pressures at various key points in figure below. Also determine the exit gradient and plot the hydraulic gradient line for pond level on upstream and no flow on downstream



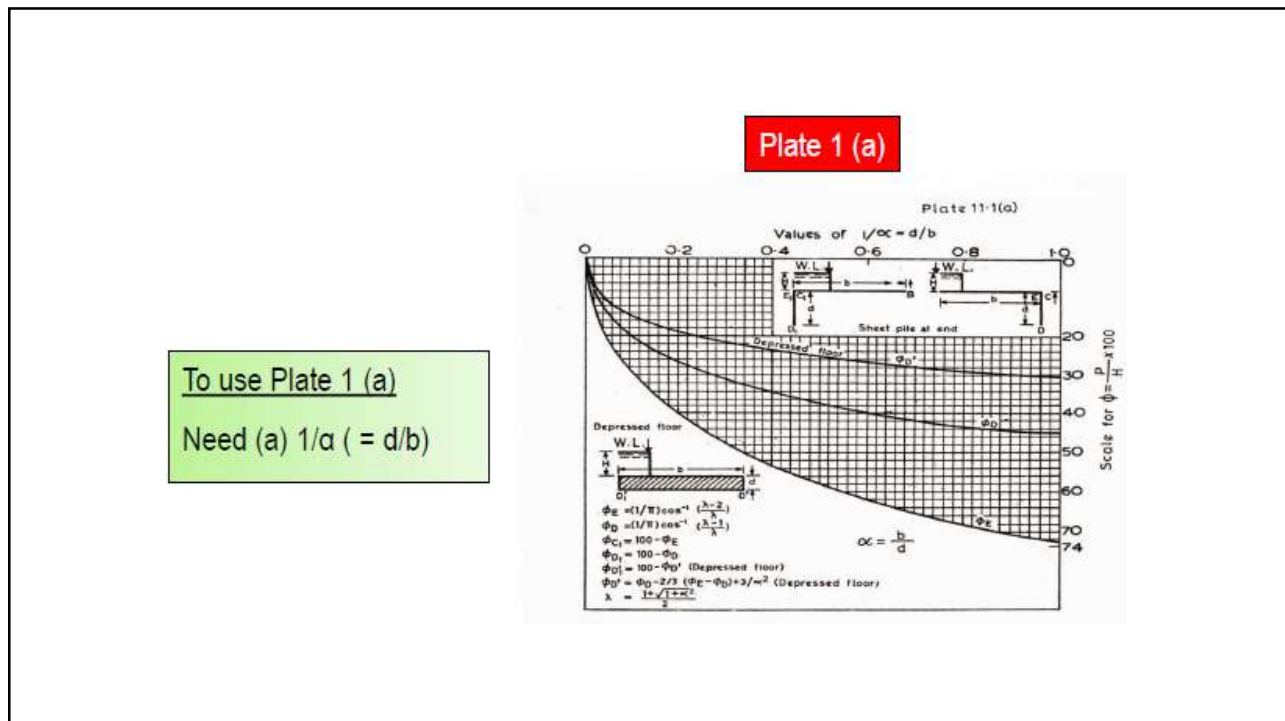
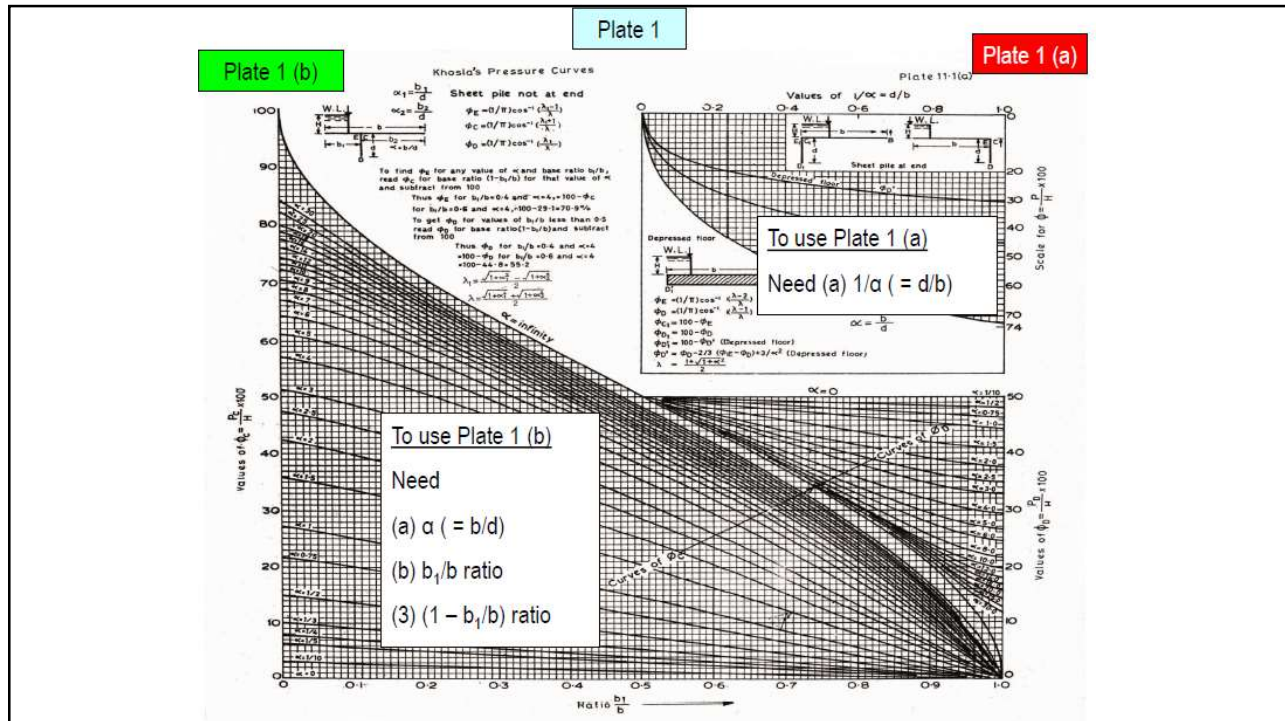
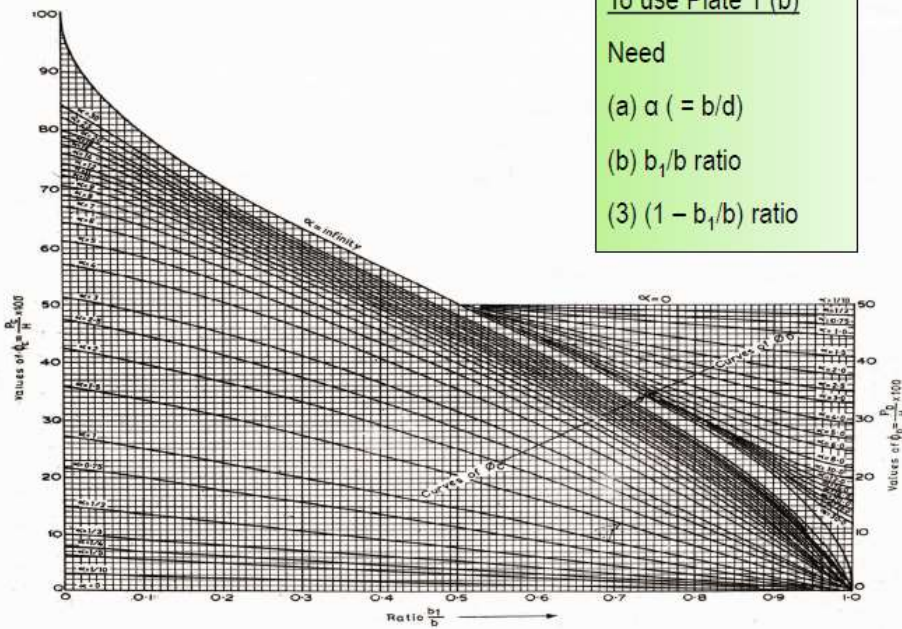


Plate 1 (b)



To use Plate 1 (b)

Need

(a)  $\alpha (= b/d)$

(b)  $b_1/b$  ratio

(3)  $(1 - b_1/b)$  ratio

Plate 2

To use Plate 2

Need

$\alpha (= b/d)$  value and read curve 1 ( $1/\pi\sqrt{\lambda}$ )

Exit Gradient:

$$G_E = H/d \times 1/\pi\sqrt{\lambda}$$

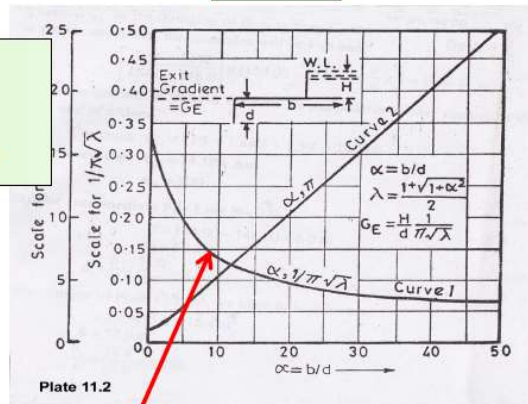
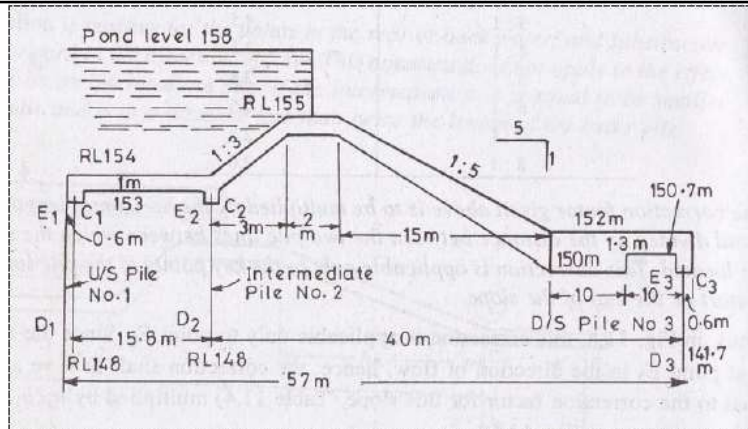


Plate 11.2

Curve 1

Plate 2



**Solution:**

**(1) For Pile No. 1**

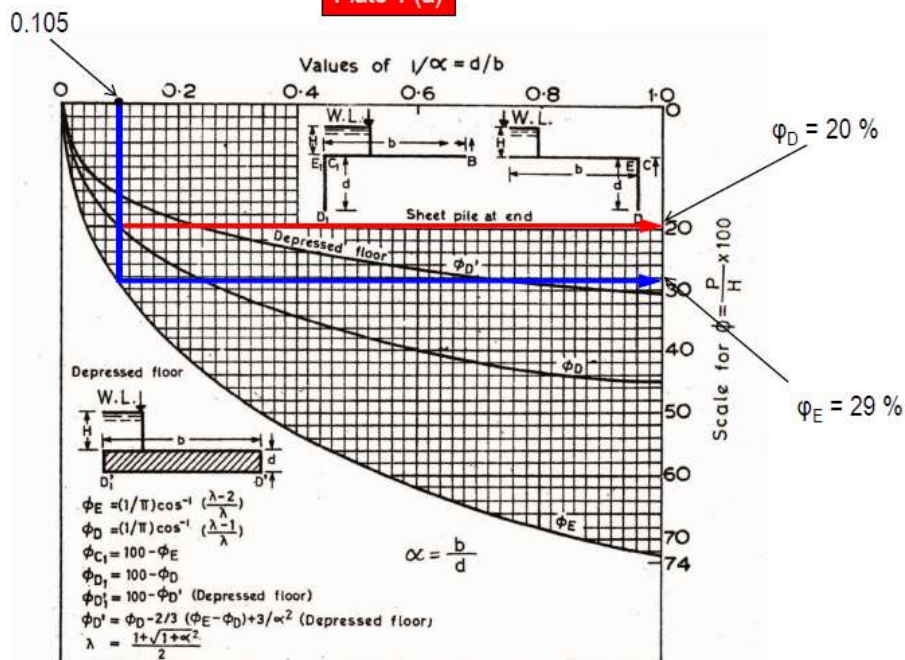
Total length of the floor,  $b = 57.0 \text{ m}$

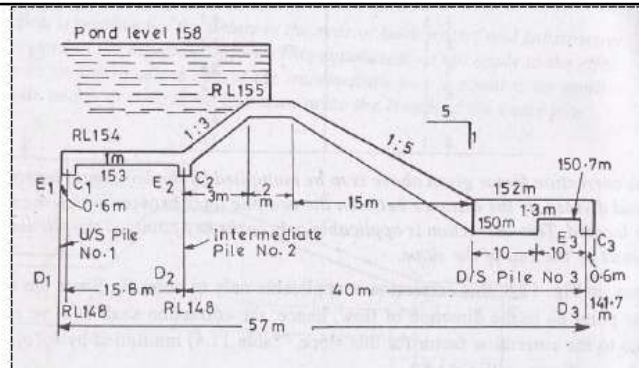
Depth of u/s pile line,  $d = 154 - 148 = 6 \text{ m}$

$$\alpha = b/d = 57/6 = 9.5$$

$$1/\alpha = 1/9.5 = 0.105$$

**Plate 1 (a)**





From plate **1 (a)**

$$\phi_E = 29\%$$

$$\phi_D = 20\%$$

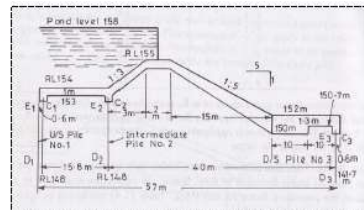
Now,

$$\phi_{C1} = 100 - \phi_E = 100 - 29 = 71\%$$

$$\phi_{D1} = 100 - \phi_D = 100 - 20 = 80\%$$

### Corrections for $\phi_{C1}$

#### (a) Mutual Interference of Piles



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

Where,  $D$  = Depth of pile No.2

$$= 153 - 148 = 5 \text{ m}$$

$$= 19 \sqrt{\frac{5}{15.8} \times \left( \frac{5+5}{57} \right)}$$

$d$  = Depth of pile No. 1 =  $153 - 148 = 5 \text{ m}$

$b'$  = Distance between two piles =  $15.8 \text{ m}$

$$= 1.88\%$$

$b$  = Total floor length =  $57 \text{ m}$

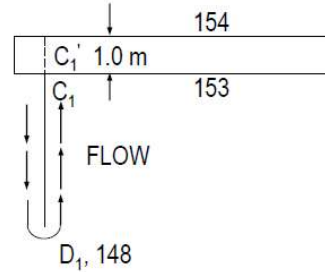
$\therefore$  Correction due to pile interference on  $C_1 = 1.88\%$  (+ve)

(b) Correction at  $C_1$  due to thickness of floor:

$$= \left[ \frac{80\% - 71\%}{154 - 148} \right] (154 - 153)$$

$$= \left[ \frac{9}{6} \right] 1$$

$$= 1.5\% (+ve)$$



(c) Correction due to slope at  $C_1$  is nil

$$\therefore \text{Corrected } (\varphi_{C1}) = 71\% + 1.88\% + 1.5\%$$

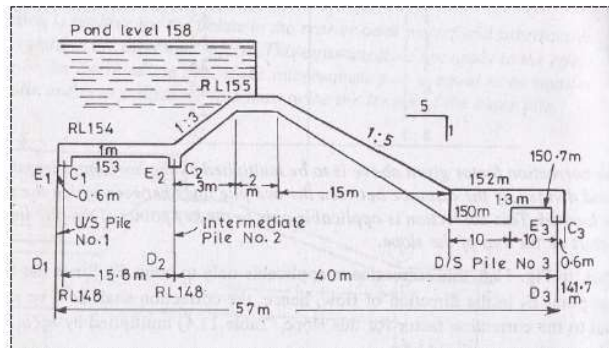
$$= 74.38\%$$

After Corrections (For Pile No.1)

$$\therefore \varphi_{E1} = 100\%$$

$$\therefore \varphi_{D1} = 80\%$$

$$\therefore \varphi_{C1} = 74.38\%$$



(2) For Pile No. 2

$$b = 57.0 \text{ m}$$

$$d = 154 - 148 = 6 \text{ m}$$

$$\alpha = b/d = 57/6 = 9.5$$

$$b_1 = 0.6 + 15.8 = 16.4$$

$$b = 57 \text{ m}$$

$$\therefore b_1/b = 16.4/57 = 0.298 \quad (b_1/b = \text{base ratio})$$

$$1 - b_1/b = 1 - 0.298 = 0.702$$

### Formula for determining key points pressure at Pile 2:

$$\varphi_{E2} = 100 - \varphi_C (1 - b_1/b \text{ value} \ \& \ \alpha)$$

$$\varphi_{C2} = \text{Direct value from chart (} b_1/b \text{ value} \ \& \ \alpha)$$

$$\varphi_{D2} = 100 - \varphi_D (1 - b_1/b \text{ value} \ \& \ \alpha)$$

### From plate 1 (b)

$$\varphi_C = 30 \%$$

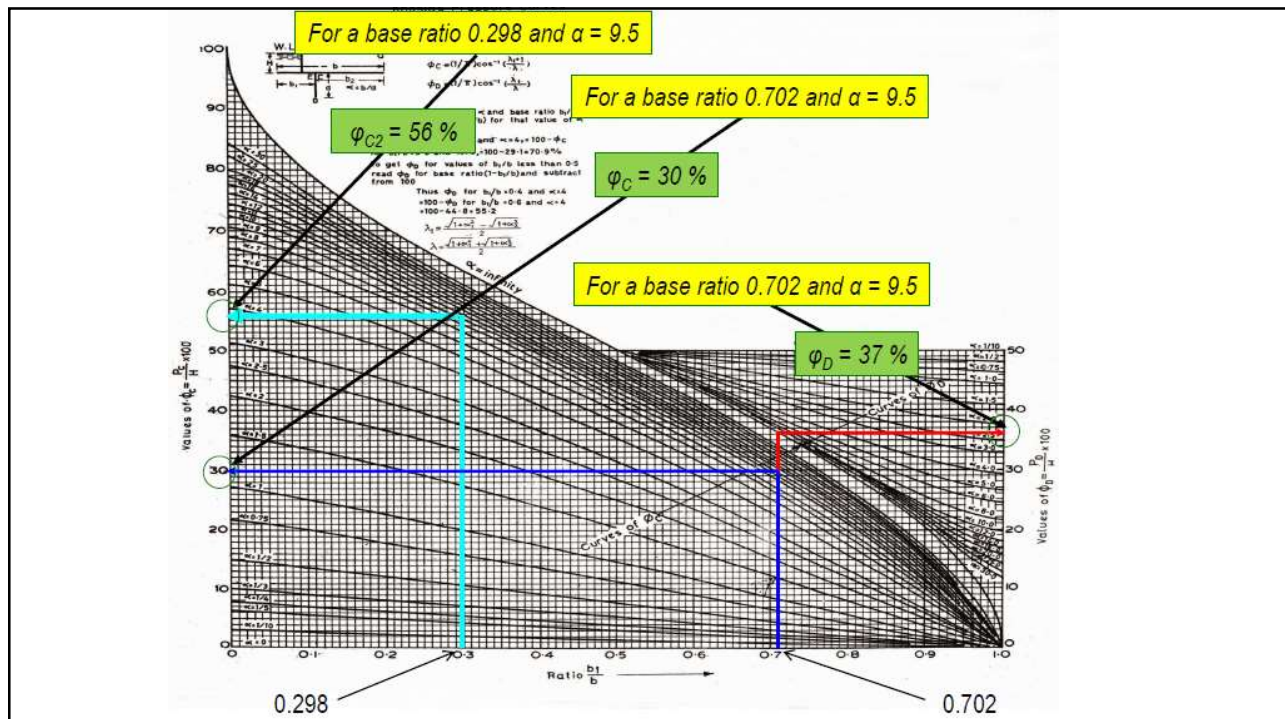
$$\varphi_D = 37 \%$$

Now,

$$\varphi_{E2} = 100 - \varphi_C = 100 - 30 = 70 \%$$

$$\varphi_{C2} = 56\%$$

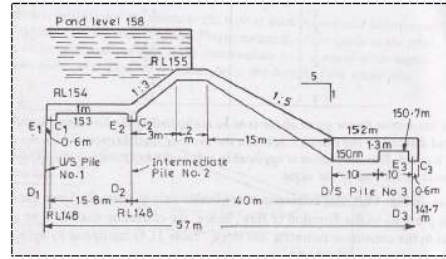
$$\varphi_{D2} = 100 - \varphi_D = 100 - 37 = 63 \%$$



### Corrections for $\varphi_{E2}$

#### (a) Mutual Interference of Piles

$$\begin{aligned}
 C &= 19 \sqrt{\frac{D}{b'}} \left( \frac{d+D}{b} \right) \\
 &= 19 \sqrt{\frac{5}{15.7}} \times \left( \frac{5+5}{57} \right) \\
 &= 1.88 \% \text{ (-) ve}
 \end{aligned}$$



Where,  $D$  = Depth of pile No.1, the effect of which is considered

$$= 153 - 148 = 5 \text{ m}$$

$d$  = Depth of pile No. 2, the effect on which is considered

$$= 153 - 148 = 5 \text{ m}$$

$b'$  = Distance between two piles

$$= 15.8 \text{ m}$$

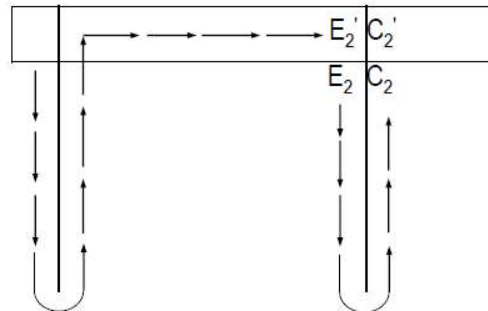
$b$  = Total floor length = 57 m

#### (b) Thickness correction ( $\varphi_{E2}$ )

$$= \frac{\text{Obs } \varphi_{E2} - \text{Obs } \varphi_{D2}}{\text{Distance between } E_2 D_2} \text{ Thickness of floor}$$

$$= \left[ \frac{70\% - 63\%}{154 - 148} \right] 1.0 = (7/6) 1.0 = 1.17 \%$$

This correction is negative,



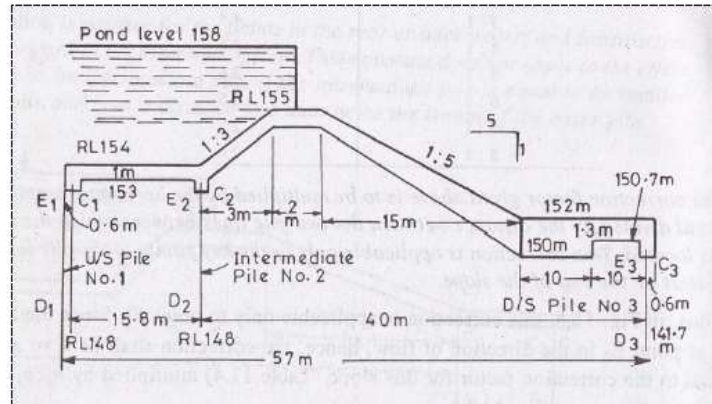
**(c) Correction due to slope**

Slope correction at  $E_2$  due to slope is nil

Hence, corrected percentage pressure at  $E_2$

$$= \text{Corrected } \varphi_{E2}$$

$$= (70 - 1.88 - 1.17) \% = 66.95 \%$$



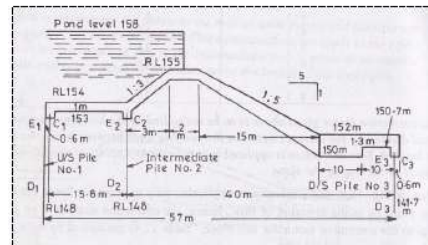
Corrections for  $\varphi_{C2}$

**(a) Mutual Interference of Piles**

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

$$= 19 \sqrt{\frac{11.3}{40}} \times \left( \frac{5+11.3}{57} \right)$$

$$= 2.89 \% (+) \text{ ve}$$



Where,  $D$  = Depth of pile No.3, the effect of which is considered

$$= 153 - 141.7 = 11.3 \text{ m}$$

$d$  = Depth of pile No. 2, the effect on which is considered

$$= 153 - 148 = 5 \text{ m}$$

$b'$  = Distance between two piles (2 & 3)

$$= 40 \text{ m}$$

$b$  = Total floor length = 57 m

**(b) Correction at  $C_2$  due to floor thickness.**

Correction at  $C_2$  due to floor thickness = 1.17 % (+ ve)

**(c) Correction at  $C_2$  due to slope.**

Correction factor for 3:1 slope from Table 5.3 = 4.5

Horizontal length of the slope = 3 m

Distance between two pile lines between which the sloping floor is located = 40 m

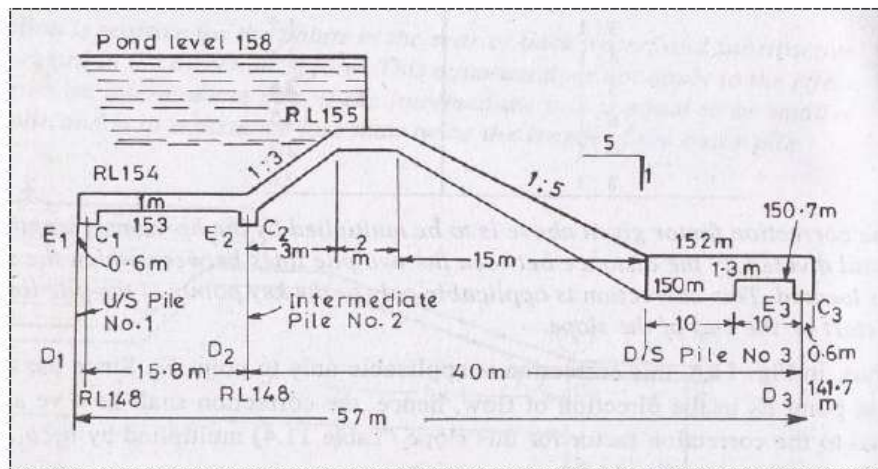
$$\therefore \text{Actual correction} = 4.5 \quad (3/40) = 0.34 \% (- \text{ve})$$

Hence, corrected  $\phi_{C2} = (56 + 2.89 + 1.17 - 0.34) \%$

$$= 59.72 \%$$

**After Corrections (For Pile No.2)**  $\therefore \phi_{D2} = 56 \%$

$$\therefore \phi_{E2} = 66.95 \% \quad \therefore \phi_{C2} = 59.72 \%$$

**(3) For Pile Line No. 3**

$$b = 57 \text{ m}$$

$$d = 152 - 141.7 = 10.3 \text{ m}$$

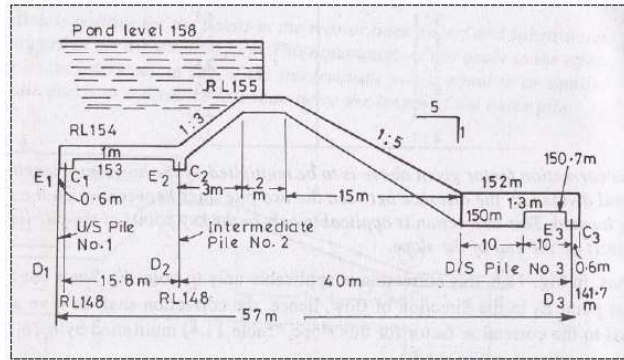
$$1/\alpha = d/b = 10.3/57 = 0.181$$

**Formula for determining key points pressure at Pile 3:**

$\phi_{E3}$  = Direct value from chart (1/  $\alpha$  value)

$\phi_{C3}$  = 0

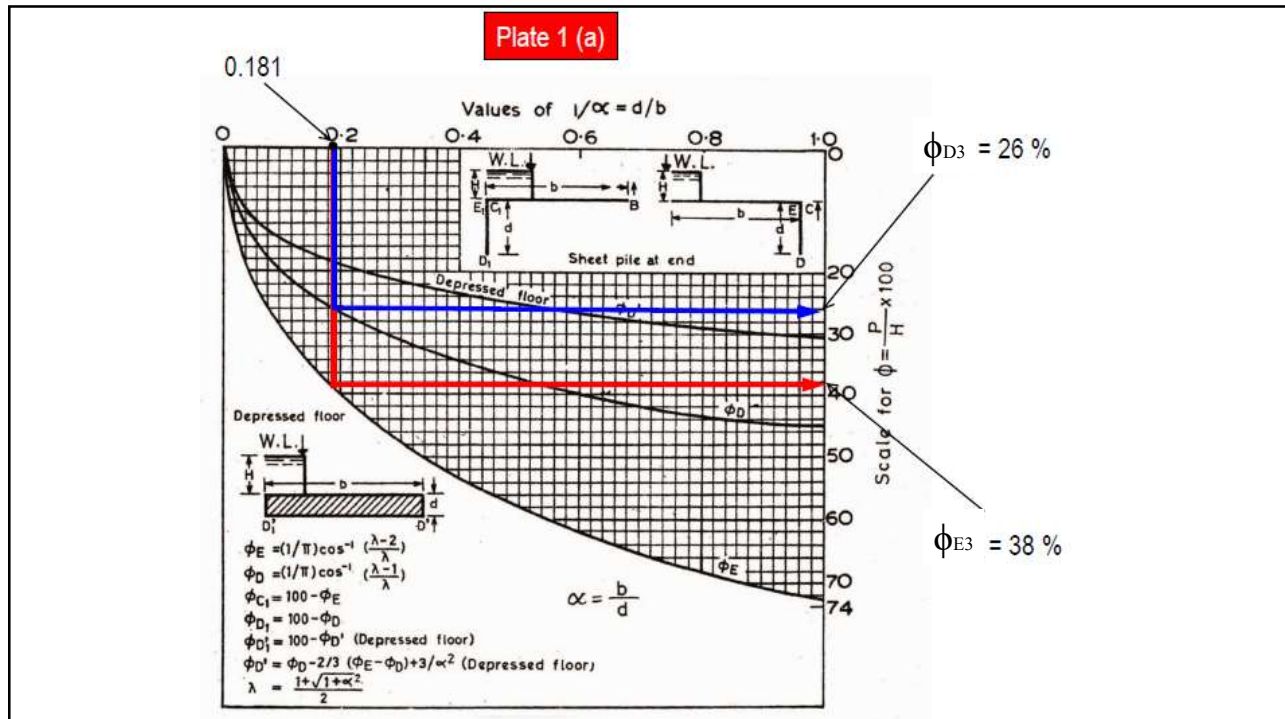
$\phi_{D3}$  = Direct value from chart (1/  $\alpha$  value)



From plate 1 (a)

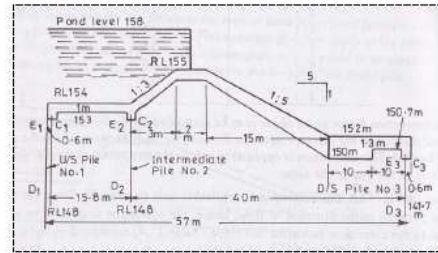
$\phi_{E3}$  = 38 %

$\phi_{D3}$  = 26%



### Corrections for $\varphi_{E3}$

#### (a) Mutual Interference of Piles



$$\begin{aligned}
 C &= 19 \sqrt{\frac{D}{b'} \left( \frac{d+D}{b} \right)} \\
 &= 19 \sqrt{\frac{2.7}{40} \times \left( \frac{9+2.7}{57} \right)} \\
 &= 1.02 \% \text{ (-) ve}
 \end{aligned}$$

Where,

$D$  = Depth of pile No.2, the effect of which is considered =  $150.7 - 148 = 2.7$  m

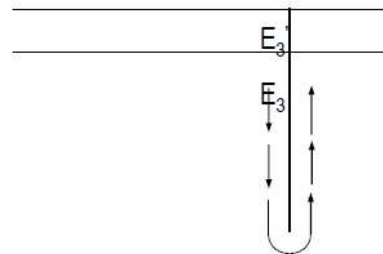
$d$  = Depth of pile No. 3, the effect on which is considered =  $150 - 141.7 = 9$  m

$b'$  = Distance between two piles = 40 m

$b$  = Total floor length = 57 m

#### (b) Correction due to floor thickness:

$$\begin{aligned}
 &= \left[ \frac{38\% - 32\%}{152 - 141.7} \right] 1.3 \\
 &= \left[ \frac{16}{10.3} \right] 1.3 \\
 &= 0.76 \% \text{ (-ve)}
 \end{aligned}$$



#### (c) Correction due to slope at $E_3$ is nil,

Hence, corrected  $\varphi_{E3} = (38 - 1.02 - 0.76) \% = 36.22 \%$

After Corrections (For Pile No.3)

$$\therefore \varphi_{E3} = 36.22 \%$$

$$\therefore \varphi_{D3} = 26 \%$$

$$\therefore \varphi_{C3} = 0 \%$$

The corrected pressures at various key points are tabulated below in Table below

Upstream Pile No. 1	Intermediate Pile No.2	Downstream Pile No. 3
$\varphi_{E1} = 100 \%$	$\varphi_{E2} = 66.95 \%$	$\varphi_{E3} = 36.22 \%$
$\varphi_{D1} = 80 \%$	$\varphi_{D2} = 63 \%$	$\varphi_{D3} = 26 \%$
$\varphi_{C1} = 74.38 \%$	$\varphi_{C2} = 59.72 \%$	$\varphi_{C3} = 0 \%$

### Exit gradient

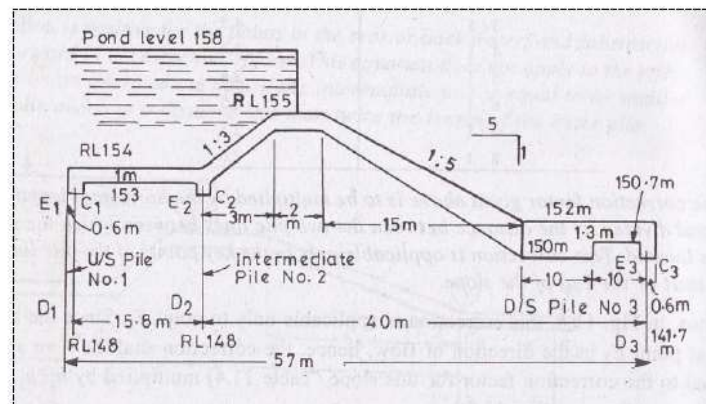
The maximum seepage head,  $H = 158 - 152 = 6 \text{ m}$

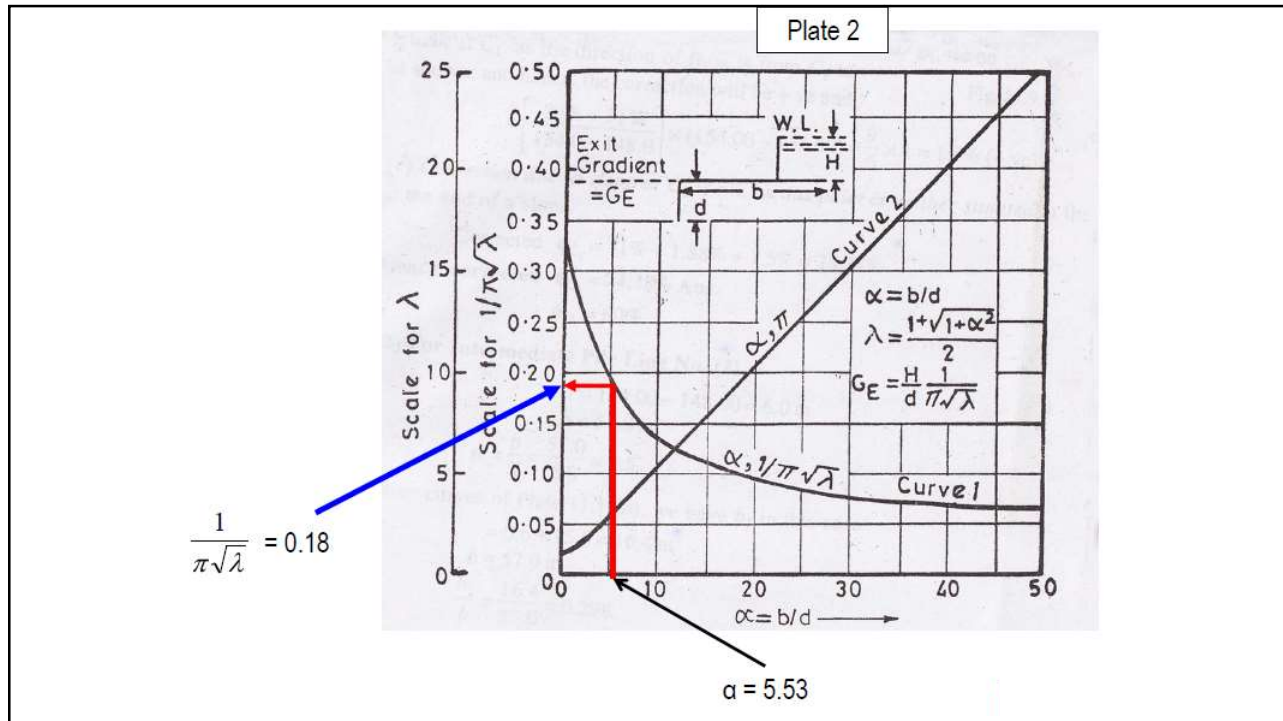
The depth of downstream cur-off,  $d = 152 - 141.7 = 10.3 \text{ m}$

Total floor length,  $b = 57 \text{ m}$

$$\alpha = b/d = 57/10.3 = 5.53$$

For a value of  $\alpha = 5.53$ ,  $\frac{1}{\pi\sqrt{\lambda}}$  from curves of Plate 2 is equal to 0.18





$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}} = \frac{6}{10.3} \quad 0.18 = 0.105$$

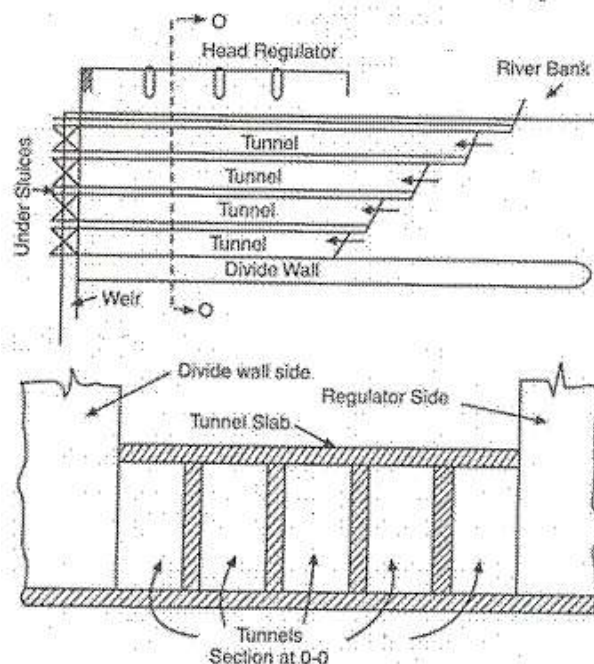
Hence, the exit gradient shall be equal to 0.105, i.e. 1 in 9.53, which is very much safe.

Type of Soil	Safe exit gradient
Shingle	1/4 to 1/5 (0.25 to 0.20)
Coarse Sand	1/5 to 1/6 (0.20 to 0.17)
Fine Sand	1/6 to 1/7 (0.17 to 0.14)

### Silt Excluder

- If silt is not controlled from entering the canal, then it reduces the capacity of the structures
- Concentration of silt particles is greater in lower layer as compared to upper layer.
- Hence device is designed to separate upper and lower layer without any disturbance.
- Silt Excluder is a device by which silt is excluded from water entering the canal
- It is constructed in the bed in front of head regulator
- The top water is then led towards the canal while the bottom water containing high silt charge is washed.

### Silt Excluder



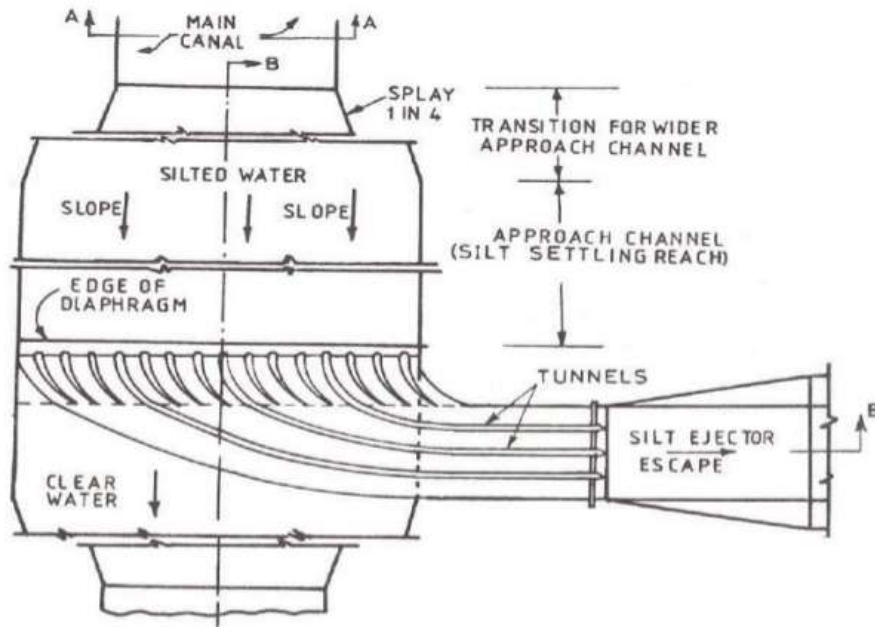
### Design Criteria for Silt Excluders:

- (i) The approach to the excluder should be straight in which the silt can settle into lower layers of flow. Artificial curves to the river flow should be avoided because it may cause turbulence and disturb bed concentration. The slope of approach channel bed should be flattest enough to carry the heaviest grade of silt likely to approach the work.
- (ii) If the flow is concentrated near head regulator better efficiency can be achieved in silt exclusion. The excluder tunnels may cover appropriate number of scouring sluice bays of the barrage.
- (iii) Usually 4 to 6 tunnels may be provided for excluders. The number of tunnels is, however, governed by conditions of approach, length of canal regulator, discharge of the canal and the discharge to be escaped through the tunnels.
- (iv) Sizes of the tunnels may be fixed to escape 25 to 30% of the canal discharge at not less than 3 m/sec velocity of flow. Height of the tunnel is fixed considering depth of flow available in the pocket. A common size of tunnel could be 2 m wide and 3 m high if other conditions are fulfilled.

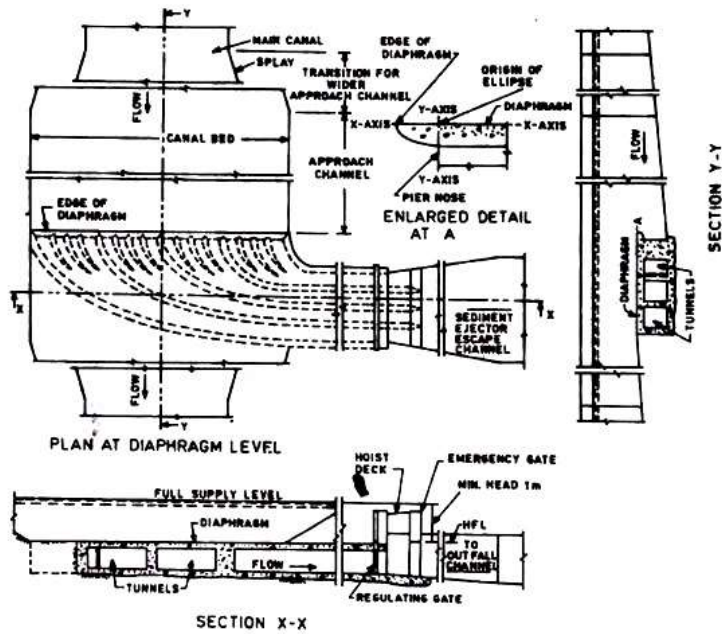
### Silt Ejectors (Silt Extractor)

- These are the devices which extract the silt from the canal-water after the silted water has travelled a certain distance in the off take canal
- **These structures are constructed on the bed of the canal & a little distance d/s from the head regulator**

Silt Ejector (Silt Extractor)



Silt Ejector (Silt Extractor)



### Design Criteria for Silt Ejectors or Silt Extractors:

- (i) Approach to the ejector should be straight and the bed should be flattest enough to carry the heaviest grade of silt to the tunnel entrance.
- (ii) Approach channel should be lined for a length of 3 to 4 times depth of flow in the channel upstream of the tunnel entrance.
- (iii) Where possible the bed of the channel may be depressed by about 30 cm gradually.
- (iv) The tunnel entrance should be so designed that there is no disturbance at entry point.
- (v) To prevent silting in the tunnels velocity of flow in the tunnel can be increased by subdividing the tunnel into convenient compartments.
- (vi) Velocity of flow in the tunnel to evacuate the silt laden flow has to be higher and should be around say 3 m/sec.

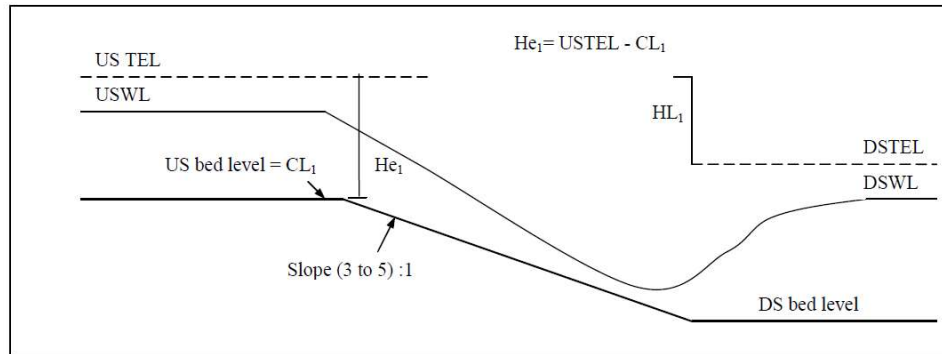
### Design criteria of the Barrage:



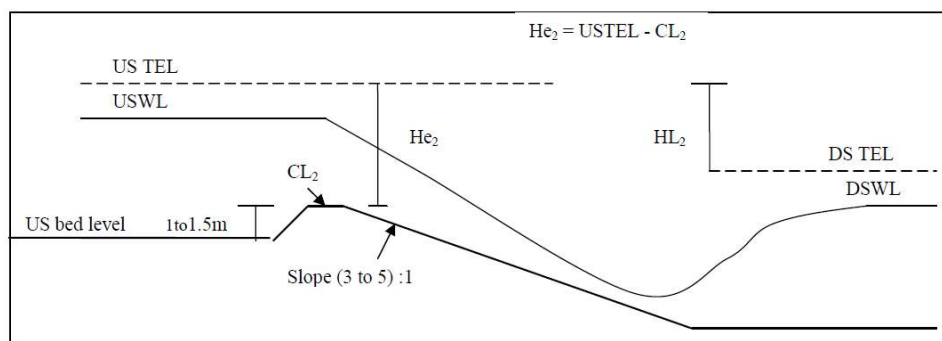
**I- Crest level and downstream floor length for both under sluices and other bays:**

**1- Crest level:**

**a- Crest level of under sluices ( $CL_1$ ):** is usually kept as near as the bed level in the deepest channel as is practically possible. For design purposes it can be provided at the average bed level of the river.



**b- Crest level of other bays ( $CL_2$ ):** is kept 1m to 1.5m higher of crest level of under sluices.



**2- Waterway and design formula:**

**a- Total waterway (L):** According to Lacey stable wetted perimeter:

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

\* For boulder reach:  $L = (0.8 \text{ to } 1.0) P$

\* For alluvial and silty reaches:  $L = (1.2 \text{ to } 1.4) P$ , where;

**P:** Lacey wetted perimeter in m

**Q:** High flood flow of the river in (m<sup>3</sup>/s)

Total waterway consist of total waterway of under sluices (L<sub>1</sub>) and total waterway of other bays (L<sub>2</sub>):

$$L = L_1 + L_2 + \text{width of fish ladder} + \text{width of divide wall}$$

**b- Discharge:**

Total discharge (Q) = discharge of under sluices (Q<sub>1</sub>) + discharge of other bays (Q<sub>2</sub>)

**Discharge of under sluices (Q<sub>1</sub>):** is the highest value of;

\* Two times the maximum discharge in the off take canal.

\* 20% of maximum possible (predicted) discharge in the river.

\* Maximum winter discharge through the recorded period.

**Other bays discharge (Q<sub>2</sub>) = total discharge (Q) – discharge of under sluices (Q<sub>1</sub>)**

- Discharge Equations: In general equation of weir is used for the design.

$$\text{For under sluices part: } Q_1 = C * (L_{C1} - 0.1 * n * H_{e1}) * H_{e1}^{3/2}$$

$$\text{For other bays part: } Q_2 = C * (L_{C2} - 0.1 * n * H_{e2}) * H_{e2}^{3/2}$$

Where: C= 1.705 for under sluice part because it is broad crest weir.  $b_t > 2.5 * H_e$

C= 1.84 for other bays part because it is sharp crest weir.  $b_t \leq \frac{2}{3} * H_e$

$b_t$  is crest width , n : is No. of contractions = 2\* number of piers

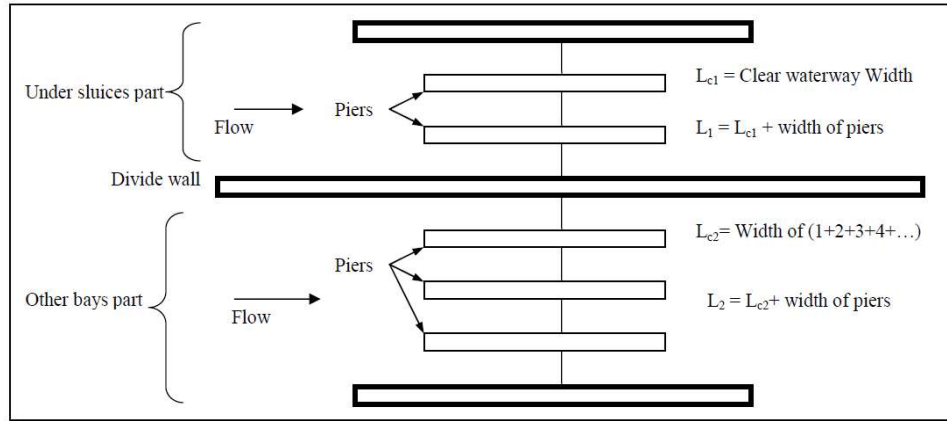
$H_{e1}$ = Head above crest level of under sluices part (m).

$H_{e2}$ = Head above crest level of other bays part (m).

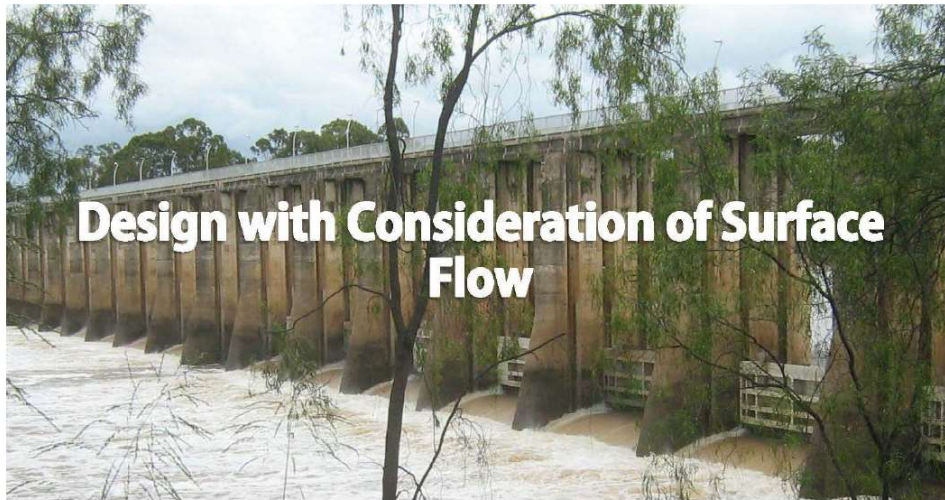
$L_{C1}$  = Clear width of waterway for under sluices part (m).

$L_{C2}$  = Clear width of waterway for other bays part (m).

n : is number of contractions = 2 \* No. of piers



### 3- Flow conditions:



A- High flood level (HFL) flow without (concentration and retrogression)

B- High flood level (HFL) flow with (20% concentration and 0.5m retrogression)

C- Pond level flow with all gates is opened but without (concentration and bed retrogression)

D- Pond level flow with all gates is opened with (20% concentration and bed 0.5m retrogression):

- Length and level of downstream floor: this must be found from hydraulic jump calculations;

a- Downstream floor length =  $5 (D_2 - D_1)$

b- Downstream floor level is provided little lower than level of the jump.

Note: the above calculation must be done for all flow conditions over the structure the largest floor length are selected.

4- Static condition (No flow): Pond level condition with gates is closed no flow:

USWL = Pond level

DSWL = DS bed level

Max static head ( $\Delta H$ ) = Pond level - DSFL

**Notes:**

a- Static condition is taken in consideration during calculating total length of impervious floor.

b- Afflux: is the rise of water level due to construction of a structure across a stream, the permissible afflux value can be taken as 1m. The head loss is taken equal to afflux.

c- Retrogression: Due to change of river regime after construction barrages, weirs, retrogression of downstream bed level of the river is necessary. Retrogression is lowering the bed level. Lowering of bed may reach 1.2m to 2.2m, but in general 0.5m retrogression of the bed is taken during the design.

5- US and DS sheet pile depth: this should be worked out for scour depth (R)

$$R = 1.35 * \left(\frac{q^2}{f}\right)^{\frac{1}{3}}$$

Upstream depth of sheet pile 1 to 1.25 times R

Downstream depth of sheet pile = 1.25 to 1.5 times R

6- Total length of impervious floor: this should be worked out for static head and safe exit

gradient:  $G_E = \frac{\Delta H}{d} \left\{ \frac{1}{\pi\sqrt{\lambda}} \right\}$ , and Total length of the floor  $b = \alpha * d$

$\Delta H$  = is the difference between pond level and downstream floor level (DS bed level).

$d$  = depth of downstream sheet pile.

Length of glacis = 3 to 5 \* (CL - DS bed level), and in general 3 is used

Upstream floor length = total length - (DS length + length of glacis + crest width + US glacis length).

7- Pressure calculation and design of floor thickness: this should be worked out using Khosla theory:

a- Coefficients of pressure at key points should be found at the upstream, downstream and intermediate (if provided) sheet piles.

b- Provide corrections for thickness, slope and mutual interference of adjacent piles.

c- Uplift pressure and unbalanced head for different points should be found.

d- 2/3 of unbalanced dynamic head should be compared with unbalanced static head and the greater should be selected for the thickness design

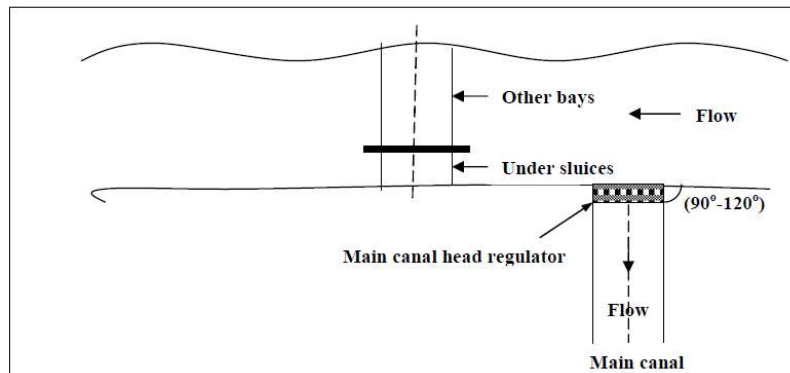
e- Thickness of floor should be found for different points.

8- Upstream and downstream protection works: this should be worked out for scour depth(R).

### 1- Main Canal Head Regulator Design

Main canal head regulator is a structure constructed at the head of a canal taking off from the upstream of a weir or barrage serves the following Functions of:

- 1- It regulates the supply of water into the canal.
- 2- It controls the entry of silt into the canal.
- 3- It completely excludes the high flood from entering into the canal.



The main canal head regulator consists of a number of spans separated by piers which support the gates. The spans of 6m to 12m are commonly used.

**Design Criteria:**

1- **Alignment:** The axis of the regulator must form with the axis of the barrage an angle between  $90^\circ$  to  $120^\circ$

2- **Crest level ( $C_L$ ):**

To control silt entering the canal,  $C_L$ , of head regulator is kept (1 to 1.5m) higher than crest of under sluices ( $C_{L1}$ ):  $C_L = C_{L1} + (1 \text{ to } 1.5) \text{ m}$

3- **Height of the gates = pond level -  $C_L$**

To check the flood water entering the canal, a breast wall between the HFL and pond level is provided.

4- **Design formula:**

Main canal head regulators are generally provided with a very wide and shallow waterway, so the drowned weir formula is used to calculate discharge:

$$Q = \frac{2}{3} * C_1 * L_c * \sqrt{2g} * \{(H + h_a)^{3/2} - h_a^{3/2}\} + C_2 * L_c * d * \sqrt{2g(H + h_a)} \dots (1)$$

$C_1 = 0.577$  and  $C_2 = 0.8$

$H = \text{USWL} - \text{DSWL} = \text{US pond level} - \text{FSL in the main canal}$

$$h_a = \text{Velocity head} = \frac{V^2}{2g}$$

$L_c$  = clear width of waterway.

$d = \text{DSWL} - C_L$ , DSWL is equal to full water supply level in the canal (FSL)

Substituting,  $g = 9.81$ ,  $C_1 = 0.577$  and  $C_2 = 0.8$  and neglecting velocity head ( $h_a$ ) because velocity in the canal is small, equation (1) can be written as follows:

$$Q = 1.7 * L_c * H^{3/2} + 3.54 * L_c * d * \sqrt{H} \dots (2)$$

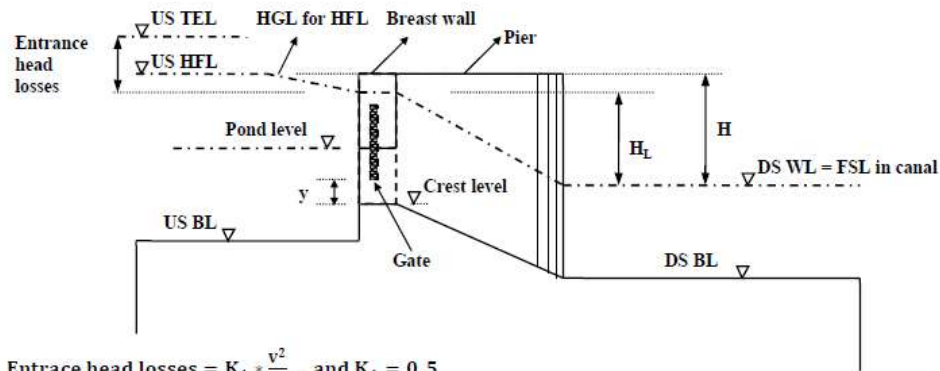
4- **Waterway:**

In general waterway is determined when full supply discharge passing the regulator during pond level in the river.

5- The principle of the design are similar to that of barrage...etc, for determining downstream floor level and length, total floor length, thickness of the floor and protection works.

6- Flow conditions considered are:

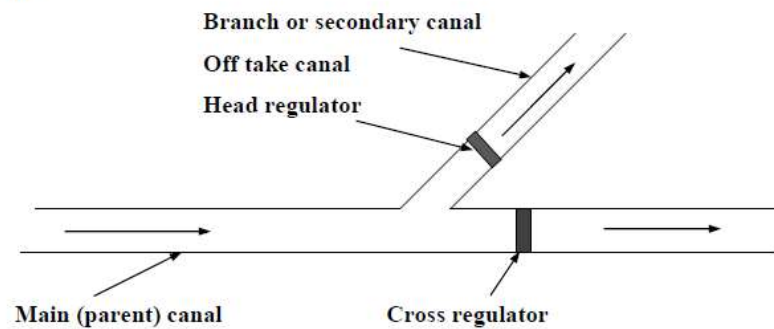
- a- Full supply discharge passing through the regulator at pond level in the river.
- b- Full supply discharge passing through the regulator at HFL in the river.
- c- Usually the most critical condition of uplift pressure occurs when high flood passing the weir or the barrage and there is no flow in the canal, and the floor thickness will be designed on maximum static head



$y$  : is partially opening of gates during HFL flow and FSL in the canal at DS

## 2- Head and Cross Regulators Design

1- Introduction:



A- Main functions of cross regulator:

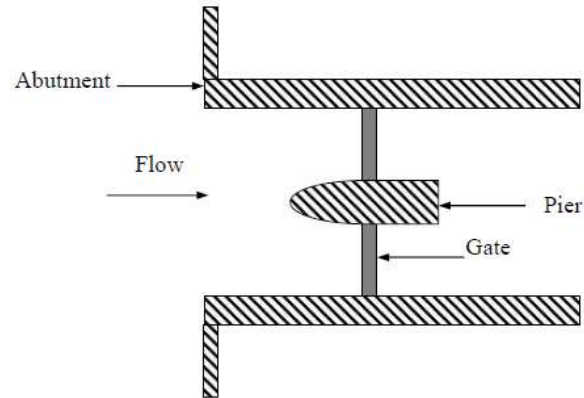
- 1- To rise water level in the canal during low water level.
- 2- Close flow to downstream sections when required especially at maintenance period.
- 3- Can be used as a bridge.
- 4- Regulation of flow in the project.

**B- Main functions of head regulator:**

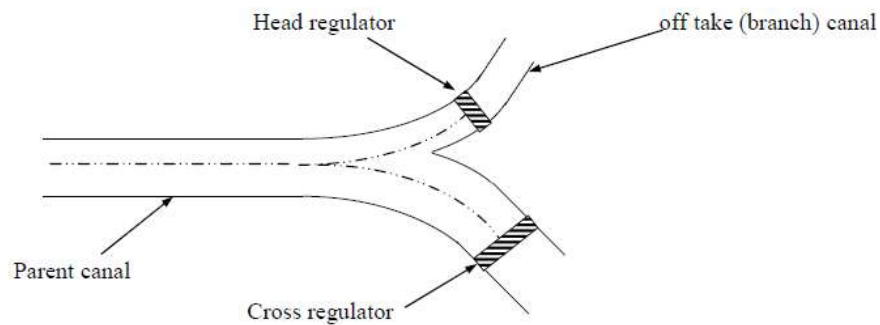
- 1- Control and regulate flow entering off take canal.
- 2- Can be used for discharge measurement.
- 3- Control silt entering in off take canal in unlined canals.

**2- Components of head and cross regulator:**

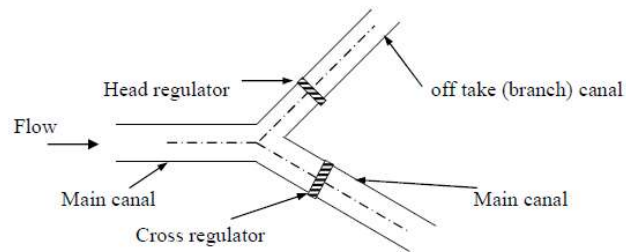
Following sketch shows the main component of the structure:

**3- Alignment of branch (off take) canal with main canal:**

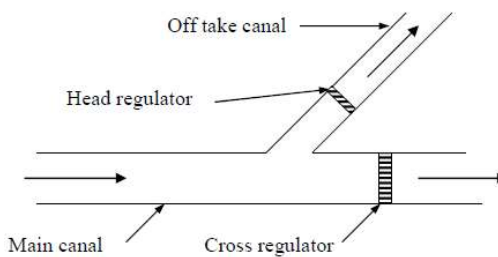
- a- Condition of the site must be considered for selection of suitable alignment.
- b- The best alignment is when the off take canal makes zero angle with the main parent canal



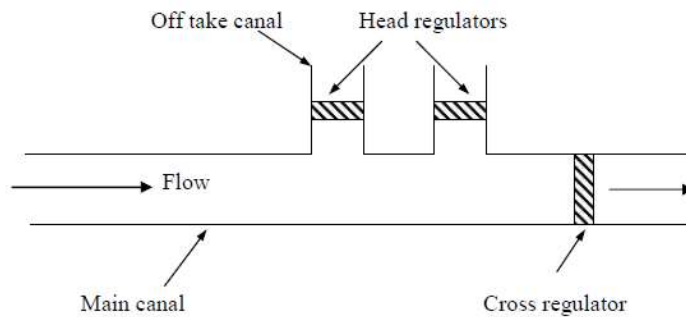
Or



c- In case the above situations are not possible:



d- One cross regulator may serve several branch canals.

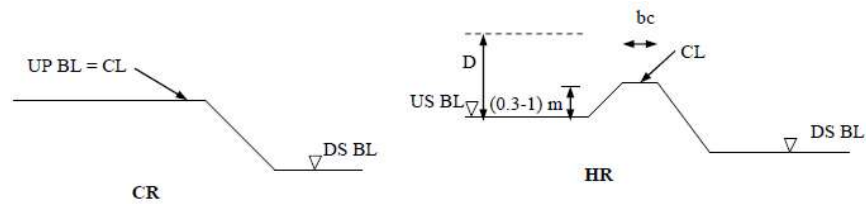


4-Design criteria for cross and head regulator:

4-1- Crest level:

A- Cross regulator: the crest of cross regulator is provided at the level of the parent canal.

B- Head regulator: For head regulator generally kept 0.3m to 1m higher than the crest level of cross regulator.



#### 4-2- Waterway

##### A- Head regulator

- \* The effective waterway width of the head regulator should be not less than 60% of the bed width of the off take canal.
- \* During full opening of the regulator, the velocity should not be more than 2.5 m/s.
- \* Overall waterway width should not be less than 70% of normal width of the branch canal (width of the canal at mid depth).

B- Cross regulator: the waterway is fixed with the consideration of maximum allowable afflux of about 150mm.

Equation of flow (discharge):

$$Q = C_s B_e H_e^{3/2} \dots (1)$$

$$CL = US\ TEL - H_e$$

$$H_e \approx US\ FSL - CL \quad (\text{velocity head is neglected})$$

$C_s$ : Submerged coefficient of the discharge, from fig. (6-5) P.209 by knowing  $(H_d/H_e)$

$$h_d = US\ FSL - DS\ FSL$$

$$C = 1.84 \text{ for sharp crested weir } bc \leq 2/3 H_e, \text{ (for HR)}$$

$$C = 1.705 \text{ for broad crested weir } bc > 2.5 H_e, \text{ (for CR)}$$

$$B_e = B_t - 2(N \cdot K_p + K_n) H_e$$

$B_e$ : effective waterway width of the crest.

$B_t$ : Net clear waterway less width of the piers.

$N$ : number of piers.

$K_p$ : pier coefficient, depends on pier form:

Square pier with rounded noses



$$K_p = 0.02$$

Rounded nose pier



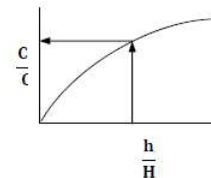
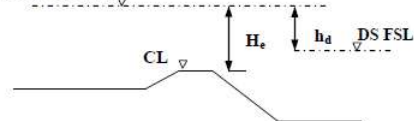
$$K_p = 0.01$$

pointed nose pier

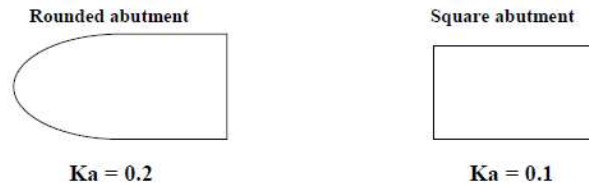


$$K_p = 0.005$$

US TEL = US FSL



Recommended values for  $K_a$  (abutment coefficient):



4-3- Level and length of downstream floor:

This must work for following flow condition;

1- Full supply discharge is passing down into main and off take canals with all gates of cross and head regulators are open.

First level of jump formation must be fixed then downstream floor level is fixed lower than level of jump formation, but not higher than bed level of the canal.

Length of DS floor =  $5 (D_2 - D_1)$

Note: If calculated DS length  $< (\frac{1}{2} - \frac{2}{3})$  of the total length (b), then

Take Length of DS floor =  $(\frac{1}{2} - \frac{2}{3}) * b$

4-4- Cutoffs:

- Upstream and downstream cutoffs are provided at US and DS of the floor for the safety against scour and exit gradient.

The following table shows the minimum depth of cutoffs depth required for upstream and downstream.

Discharge $m^3/s$ Q	Minimum depth required below floor level (m)
$Q \leq 3$	1
$3 \leq Q \leq 30$	1.2
$30 \leq Q \leq 150$	1.5
$Q > 150$	1.8

4-5- Total floor length and exit gradient:

- When cutoffs are fixed, floor length can be found for maximum static head when the gates are closed and full water level is exist at the main canal.

$$\Delta H_{\text{static}} = \text{US FSL} - \text{DS BL}$$

- Total length of the structure can be found from the equation of exit gradient:

$$G_E = \frac{\Delta H}{d_2} \left\{ \frac{1}{\pi \sqrt{\lambda}} \right\}, \text{ and } \lambda = \frac{(1 + \sqrt{1 + \alpha^2})}{2}$$

Total length of the floor (b) =  $\alpha d_2$

**4-6- Pressure calculation and design of floor thickness: find uplift pressure coefficient ( $\phi$ ) at key points**

Assuming US & DS end thicknesses as follows:

For canal  $Q > 1.5 \text{ m}^3/\text{sec}$  assume 0.6 m

For canal  $Q \leq 1.5 \text{ m}^3/\text{sec}$  assume 0.3 m

The structure should be designed for:

a- **Static condition: The max static head condition will governs the DS thickness design.**

$$\Delta H_{\text{static}} = \text{US FSL} - \text{DS BL}$$

b- **At the end of DS sloped glacis, the floor should be checked for static and dynamic condition where the unbalanced dynamic head may exceed unbalanced static head**

$$\text{Unbalanced dynamic head at point of jump} = 0.5 * (D_2 - D_1) + \phi_j * \Delta H_{\text{dynamic}}$$

$$\Delta H_{\text{dynamic}} = \text{US FSL} - \text{DS FSL}$$

**4-7- Free board:**

Discharge (Q) ( $\text{m}^3/\text{s}$ )	Minimum free board (m)
$Q \leq 1$	0.3
$1 < Q \leq 10$	0.4
$10 < Q \leq 30$	0.6
$30 < Q \leq 150$	0.8
$Q > 150$	1

**4-8-Transition:**

Functions of transitions:

- i- To minimize canal erosion.
- ii- To increase seepage path and thereby provide additional safety against piping.
- iii- To retain earth fill at the ends of the structure.

All transitions may be classified as either inlet (contraction) or outlet (expansion) transitions.

**A- Upstream transition:**

The angle between the channel centerline and line joining the channel sides at the water line between the beginning and the end of the transition should not exceed  $27.5^\circ$ . For upstream transition may be designed using the following equation:

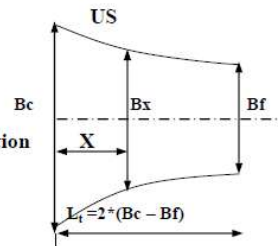
$$X = \frac{L_t \cdot B_c^{1.5}}{(B_c^{1.5} - B_f^{1.5})} \cdot \left\{ 1 - \left( \frac{B_c}{B_x} \right)^{1.5} \right\}$$

$B_c$ : total width of waterway at expanded section

$B_f$ : width of waterway at flumed section

$B_x$ : width of waterway at distance  $x$  from flumed section

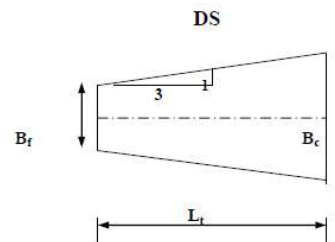
$L_t$ : length of transition



**B- Downstream transition:** The slope between channel centerline at the water surface should be about 3:1 for downstream transition.

The splay should not be steeper than 3:1

$$L_t = 3 \cdot \left\{ \frac{B_c - B_f}{2} \right\}$$

**4-9- Protection works:**

- No need for protection works for lined canals.

- For unlined canals:

**A- Downstream protection works:** the length of inverted filter may be taken as  $1.5D$ , where  $D$  is depth of downstream cutoff. The detail of block protection can be summarized in following table:

Discharge (Q) $m^3/s$	Details of protection
Up to 1	0.6*0.6*0.2 concrete blocks or stone over 150mm graded filter
$1 < Q \leq 5$	0.6*0.6*0.25 concrete blocks or stone over 250mm graded filter
$5 < Q \leq 30$	0.6*0.6*0.4 concrete blocks or stone over 400mm graded filter
$30 < Q \leq 150$	0.8*0.8*0.6 concrete blocks or stone over 600mm graded filter
$Q > 150$	1.25*1.25*0.6 concrete blocks or stone over 600mm graded filter

**B- for upstream protection:** the length of block protection should be taken as  $1D$ .

## Canal Escapes:

An escape is a side channel constructed to remove surplus water from an irrigation channel into a natural drain.

### Function of Escapes:

- Helps to overflow the extra surplus water from the canal safely
- Prevents the damage and deflection of irrigation canal
- Prevents the damage of the farming land

### Types of canal escapes:

1. Weir type escape
2. Regulator/Sluice type escape

## Canal Escapes:

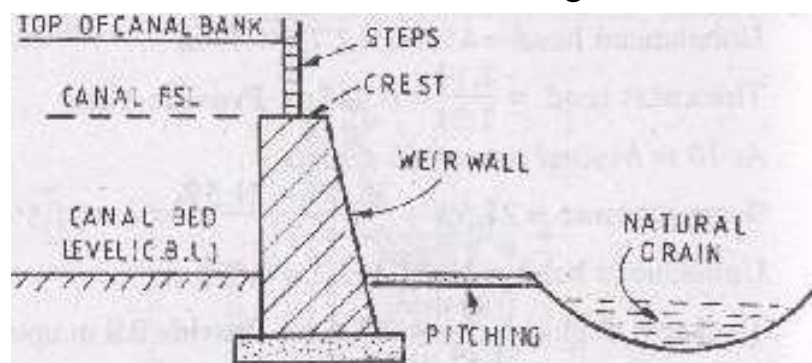
### 1. Weir type Escape:

Crest level = FSL of the canal

Water escapes if  $WL > FSL$

The crest of the weir wall is kept at R.L equal to canal FSL.

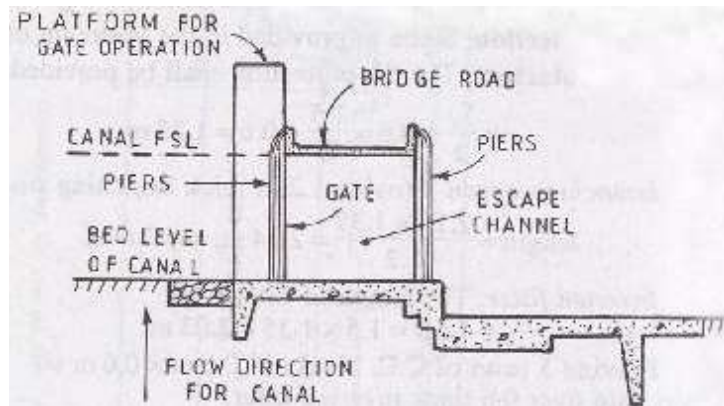
When the water level rises above FSL, it gets



## Canal Escapes:

### 2. Regulator/Sluice type Escape:

The sill of the escape is kept at canal bed level and the flow can be used for completely emptying the canal. They may be constructed for the purpose of scouring off excess bed silt deposited in the head reaches from time to time.



## Outlets/Modules:

A canal outlet or module is a small structure built at the head of the water course so as to connect it with a minor or a distributary channel.

### Function of Outlets:

- To take water from minor or distributary channel
- To distribute water in required proportions in fields
- To draw water safely from distributary channel

### Types of outlets/modules:

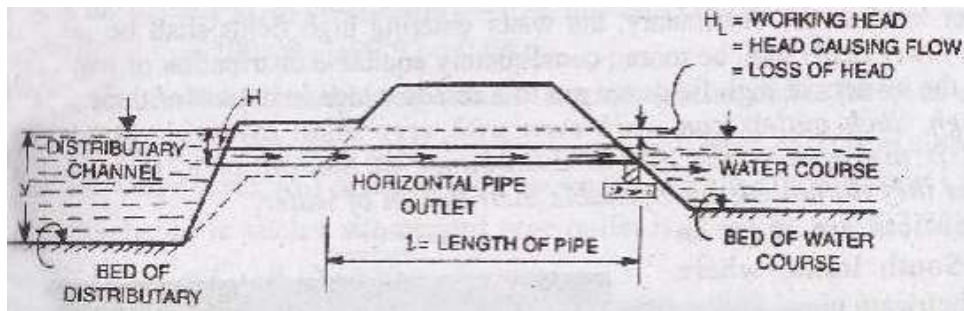
1. Non-modular module
2. Semi-modular module or Semi-module or Flexible module
3. Modular outlet or Rigid module

## Outlets/Modules:

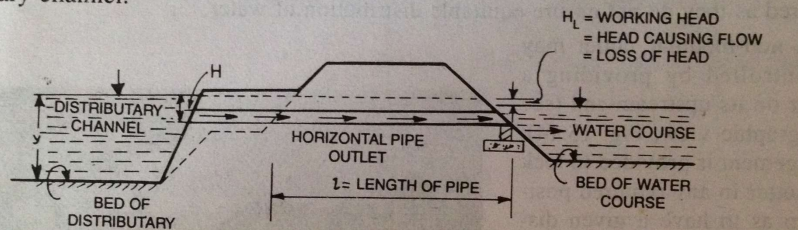
### 1. Non-modular module:

Non-modular modules are those through which the discharge depends upon the head difference between the distributary and the water course. Lowering of the bed of the water course will draw extra discharge. Thus equitable distribution of discharge may not be possible.

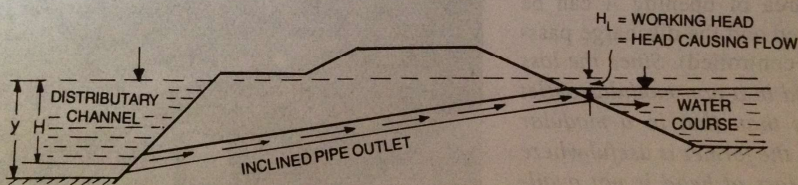
Examples are Open sluice, Drowned pipe outlet, etc.



**Submerged pipe Outlet.** Two types of pipe outlets are shown in Figs. 13.13 (a) and (b). The pipe diameter varies from 10 to 30 cm. Pipes are generally embedded in concrete and are generally fixed horizontally at right angles to the direction of flow. They may also be laid sloping upwards by depressing the upstream end of the pipe [as shown in Fig. 13.13 (b)] so as to increase silt conductivity. In U.P., the pipes are generally laid horizontally at about 21 cm below the water surface level of the distributary channel.



(a) Horizontal pipe outlet (Submerged)



(b) Inclined pipe outlet (Submerged)

The velocity through the pipe can be precisely computed by using the relation :

$$H_L = \text{Total loss of head} = \text{Entry loss} + \text{Frictional loss} + \text{Velocity head at exit}$$

$$= 0.5 \frac{V^2}{2g} + \frac{f' l V^2}{2gd} + \frac{V^2}{2g}$$

or

$$H_L = \frac{V^2}{2g} \left[ 1.5 + \frac{f' l}{d} \right]$$

where  $H_L$  is the difference in the water level of the distributary and the water course,  $l$  is the length of the pipe,  $d$  is the diameter of the pipe and  $f'$  is the coefficient of friction of the material of the pipe.

The discharge, however, for all practical purposes, may be easily computed by using

$$q = C_d \cdot A \cdot \sqrt{2gH_L} \quad \dots(13.18)$$

where  $q$  = Discharge through the outlet

$C_d$  = Coefficient of discharge, and is as high as 0.8 for submerged pipe discharges

$A$  = Area of the pipe

$H_L$  = Difference of head between the FSL of distributary and FSL of water course, usually called working head of irrigation outlet.

## Outlets/Modules:

### 2. Semi-modular module:

Semi-modules or flexible modules are those through which the discharge is independent of the water level of the water course but depends only upon the water level of the distributary so long as a minimum working head is available. The common examples are pipe outlet, venture flume, open flume and orifice semi-module.

Due to construction, a super-critical velocity is ensured in the throat and thereby allowing the formation of a jump in the expanding flume. The formation of hydraulic jump makes the outlet discharge independent of the water level in water course, thus making it a semi-module.

**13.13.1. Free Pipe Outlet.** Pipe outlet discharging freely into the atmosphere is the simplest and the oldest type of a flexible outlet. The discharge through such an outlet will depend only upon the water level of the distributary, and will be independent of the water level of the water-course so long as the pipe is discharging freely. Silt conduction for such an outlet is quite good and efficiency is high. But a freely falling jet outlet can be provided only at a few places where sufficient level difference between the distributary and water-course is available. The discharge can be easily computed by using the equation.

$$Q = C_d \cdot A \cdot \sqrt{2g H_0} \quad \dots(13.19)$$

where  $C_d$  is coefficient of discharge = 0.62 for average condition of free over fall.

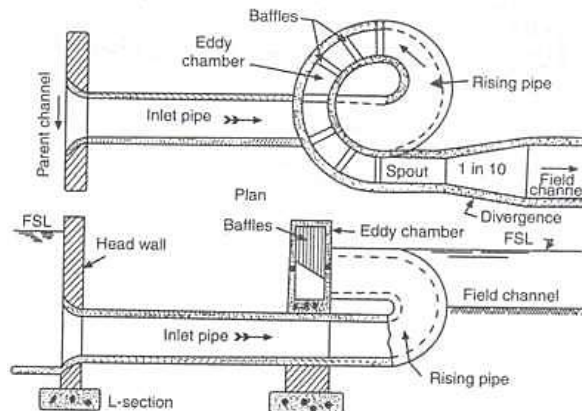
$H_0$  = Head on u/s side measured from FSL of distributary up to the centre of pipe outlet.

$A$  = Area of cross-section of pipe

## Outlets/Modules:

### 3. Rigid module:

Rigid modules or modular outlets are those through which discharge is constant and fixed within limits, irrespective of the fluctuations of the water levels of either the distributary or of the water course or both. An example is Gibb's module.



**Example 13.1.** Design an irrigation outlet for the following data :

FSQ of outlet = 50 lit/sec.

FSL in distributary on u/s side of outlet = 200.00 m.

FSL in water course on d/s side of outlet = 199.92 m.

FSD in distributary on u/s side of outlet = 1.05 m.

**Solution.**

Available head across the outlet

$$= \text{FSL of Distributary} - \text{FSL of water course}$$

$$= 200.00 - 199.92 = 0.08 \text{ m.}$$

Since the available head is very small, a non-modular outlet (such as a submerged pipe outlet) will have to be provided [ ∵ The very small head of 0.08 m clearly indicates that the d/s water level will definitely be above the opening on that side.]

The discharge in such a submerged pipe outlet is given by Eq. (13.18) as :

$$Q = C_d \cdot A \cdot \sqrt{2g H_L}$$

where  $Q = 50 \text{ lit/sec} = 0.05 \text{ cumec.}$

$A =$  Area of pipe

$H_L =$  working head or Loss of head between u/s and d/s.

$$= 0.08 \text{ m}$$

$$C_d = 0.8.$$

Putting these values in the above Eqn. we get

$$0.05 = 0.8 A \cdot \sqrt{2 \times 9.81 \times 0.08}$$

or  $A = 0.05 \text{ m}^2$

If a pipe of dia  $d$  is used, then

$$\frac{\pi}{4} d^2 = 0.05 \text{ m}^2$$

or  $d = 0.252 \text{ m.}$

Hence, use a pipe of dia, say 30 cm.

The R.L. of the bed of the distributary

$$= 200.00 - 1.05 = 198.95 \text{ m.}$$

The pipe top can be fixed at about 22 cm below the FSL of the distributary. In other words, the pipe can be laid horizontally with its invert level (or sill level)

$$= 200.00 - 0.22 - 0.30 = 199.48 \text{ m}$$

i.e. at  $199.48 - 198.95 = 0.53 \text{ m}$  above the bed of the distributary.

**Example 13.2.** Design a pipe outlet for the following data :

Full supply discharge at the head of water course	= 90 lit/sec
FSL in distributary	= 205.00 m.
FSL in water course	= 204.00 m.

**Solution**

Available head across the outlet

$$= \text{FSL of distributary} - \text{FSL of water course}$$

$$= 205.00 - 204.00 = 1 \text{ m.}$$

This available head of 1 m is sufficient enough to make the pipe outlet discharge freely into the water course, as the d/s end of the pipe can be fixed below the water level of the water course, thus making it a semi-module.

The discharge through such an outlet is given as :

$$Q = C_d \cdot A \sqrt{2g H_0}$$

where  $C_d = 0.62$

$H_0$  = Head on upstream side above the centre line of pipe.

$$Q = 9.0 \text{ lit/sec} = 0.09 \text{ cumec.}$$

Assuming the dia of the pipe as 25 cm, we have

$$0.09 = 0.62 \left[ \frac{\pi}{4} \times (0.25)^2 \right] \sqrt{2 \times 9.81 H_0}$$

$$0.09 = 0.62 \left[ \frac{\pi}{4} \times (0.25)^2 \right] \sqrt{2 \times 9.81 H_0}$$

$$= 0.62 \times 0.049 \times 4.43 \sqrt{H}$$

$$\sqrt{H_0} = 0.657 \text{ m.}$$

$$\therefore H_0 = 0.44 \text{ m}$$

$\therefore$  R.L. of the centre of outlet pipe

$$= 205.00 - 0.44 = 204.56 \text{ m.}$$

R.L. of invert of outlet pipe (i.e. sill level)

$$= 204.56 - \frac{0.25}{2}$$

$$= 204.43 \text{ m} > \text{FSL of water course i.e. } 204.00 \text{ m.}$$

Hence, a pipe of 25 cm dia can be laid horizontally with its bottom or sill level at RL 204.43 m, and it will be discharging freely as a semi-module. **Ans.**

## Outlets/Modules:

### Performance Criteria:

#### (a) Flexibility, F:

Flexibility is defined as the ratio of the rate of change of discharge of the outlet to the rate of change of discharge of the distributary channel.

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

Where, F = Flexibility of the outlet

q = Discharge passing through the outlet

Q = Discharge in the distributary channel

If H = the head acting on the outlet,

$$q = CH^m$$

Where, C and m are constants depending upon the type of outlet

If y = the depth of water in the distributary,

$$Q = Ky^n$$

Where, k and n are constants

## Outlets/Modules:

### Performance Criteria:

$$\frac{dq}{dH} = CmH^{m-1} = (CH^m) \times (m/H) = q \times \frac{m}{H}$$

$$\Rightarrow \frac{dq}{q} = \frac{m}{H} \times dH$$

Again,

$$\frac{dQ}{dy} = Kny^{n-1} = (Ky^n) \times (n/y) = Q \times \frac{n}{y}$$

$$\Rightarrow \frac{dQ}{Q} = \frac{n}{y} \times dy$$

## Outlets/Modules:

### Performance Criteria:

Thus,

$$F = \frac{\frac{m}{H} \times dH}{\frac{n}{y} \times dy} = \frac{m}{n} \times \frac{y}{H} \times \frac{dH}{dy}$$

A change in water depth of the distributary (dy) would result in an equal change in the head working on the outlet (dH), so that

$$dy = dH$$

$$\text{So, } F = \frac{m}{n} \times \frac{y}{H}$$

## Outlets/Modules:

### Performance Criteria:

#### (b) Proportionality:

The outlet is said to be proportional when the rate of change of outlet discharge equals the rate of change of channel discharge.

$$\text{Thus } \frac{dq}{q} = \frac{dQ}{Q}$$

$$\text{So, } F = 1, \text{ i.e. } \frac{m}{n} \times \frac{y}{H} = 1$$

$$\Rightarrow \frac{H}{y} = \frac{m}{n} = \frac{\text{Outlet index}}{\text{Channel index}}$$

## Outlets/Modules:

### Performance Criteria:

The outlet is said to be sub-proportional, if  $F < 1$ ,

$$\text{Or, } \frac{H}{y} > \frac{m}{n}$$

The outlet is said to be hyper-proportional, if  $F > 1$ ,

$$\text{Or, } \frac{H}{y} < \frac{m}{n}$$

### (c) Setting:

It is the ratio of the depth of the sill level of the outlet below the FSL of the distributary, to the full supply depth of the distributary.

$$\text{Setting} = H/y$$

## Outlets/Modules:

### Performance Criteria:

For proportional outlet, setting =  $y/H = n/m$

For a wide trapezoidal channel, the discharge is proportional to  $y^{5/3}$

So  $n = 5/3$

Discharge through an orifice type outlet is proportional to  $H^{1/2}$

So  $m = 1/2$

Thus, setting =  $H/y = m/n = (1/2)/(5/3) = 3/10 = 0.3$

For a weir type outlet, the discharge is proportional to  $H^{3/2}$

Hence, the setting for a combination of a weir type outlet and a trapezoidal channel =  $m/n = (3/2)/(5/3) = 9/10 = 0.9$

Thus an orifice or a weir type outlet shall be proportional, if the outlet is set at 0.3 and 0.9 times depth below the water surface respectively.

## Outlets/Modules:

### Performance Criteria:

#### (d) Sensitivity, S:

It is defined as the ratio of the rate of change of discharge through the outlet to the ratio of change of water level of the distributary.

$$S = (dq/q)/(dG/y)$$

where  $dG$  = Gauge reading at the outlet

## Outlets/Modules:

### Performance Criteria:

#### Relation between Sensitivity and Flexibility:

$$F = (dq/q)/(dQ/Q)$$

$$\text{But, } dQ/Q = (n/y) \times dy$$

$$\text{Therefore, } F = (dq/q)/((n/y) \times dy) = (1/n) \times (dq/q)/(dy/y)$$

$$\text{Since, } dG = dy,$$

$$\text{So, } F = (1/n) \times S$$

$$\text{Thus, } S = n \times F$$

For rigid modules, the discharge is fixed, and hence sensitivity is zero.

The greater the variation of discharge through an outlet for a given rise or fall in water level of the distributary, the larger is the sensitivity of the outlet.

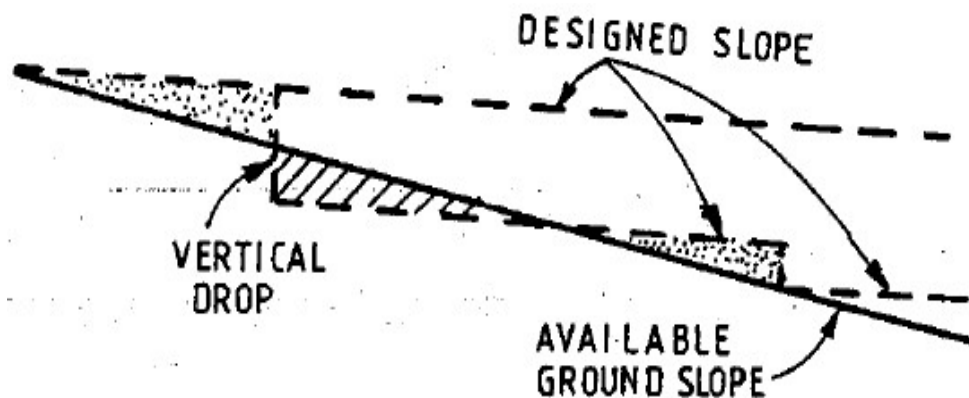
## Canal Fall/Drop:

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 will not be stable and, therefore, in order to retain this drop, a masonry structure is constructed. Such a pucca structure is called a *canal fall* or a *canal drop*.

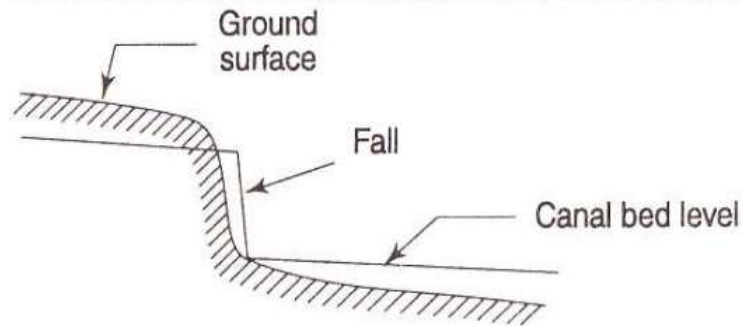
**Canal fall/drop** is a structure designed to secure lowering of the water surface in a canal and to dissipate safely the surplus energy so liberated, which otherwise scour the bed and banks of the canal

## Canal Fall/Drop:



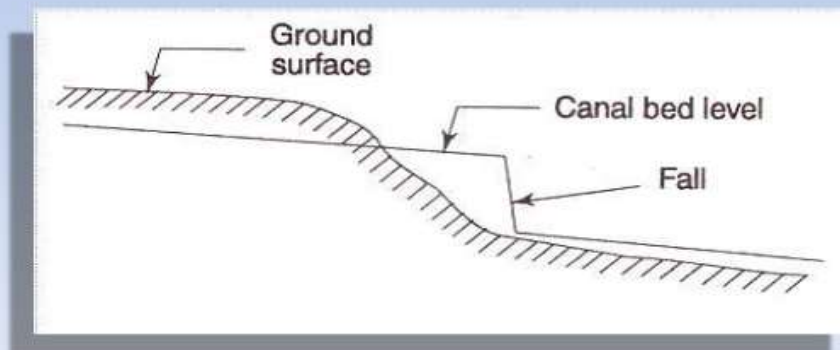
### Necessity of Canal Falls:

- When the slope of the ground suddenly changes to steeper slope, the permissible bed slope can not be maintained. It requires excessive earthwork in filling to maintain the slope. In such a case falls are provided to avoid excessive earth work in filling



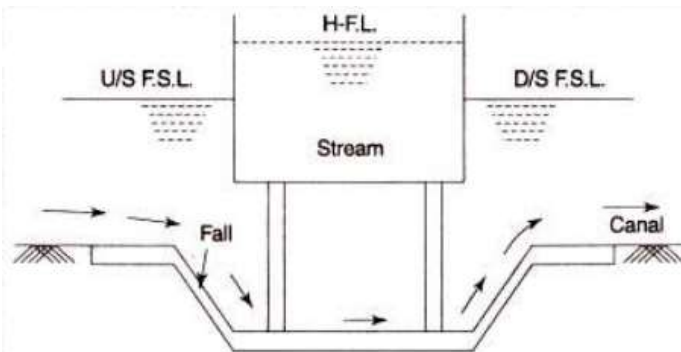
### Necessity of Canal Falls:

- When the slope of the ground is more or less uniform and the slope is greater than the permissible bed slope of canal.



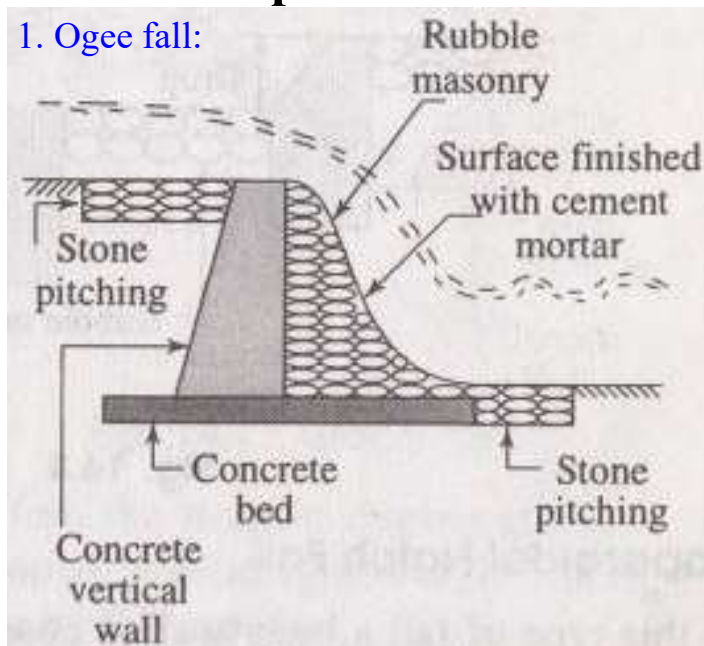
## Necessity of Canal Falls:

- In cross-drainage works, when the difference between bed level of canal and that of drainage is small or when the F.S.L. of the canal is above the bed level of drainage then the canal fall is necessary to carry the canal water below the stream or drainage.



## Canal Fall/Drop:

### 1. Ogee fall:



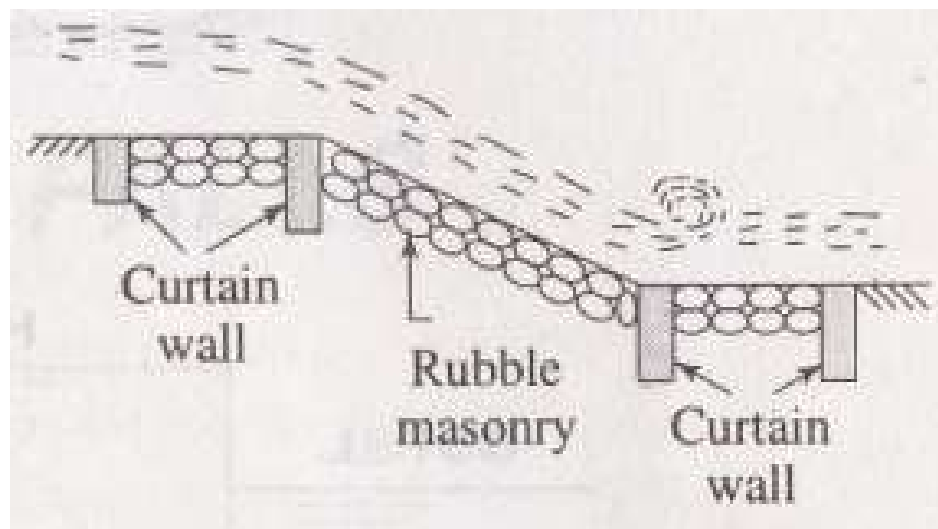
## Canal Fall/Drop:

### 1. 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. Upstream and downstream of the fall is provided by Stone Pitching.

## Canal Fall/Drop:

### 2. Rapid fall:



## Canal Fall/Drop:

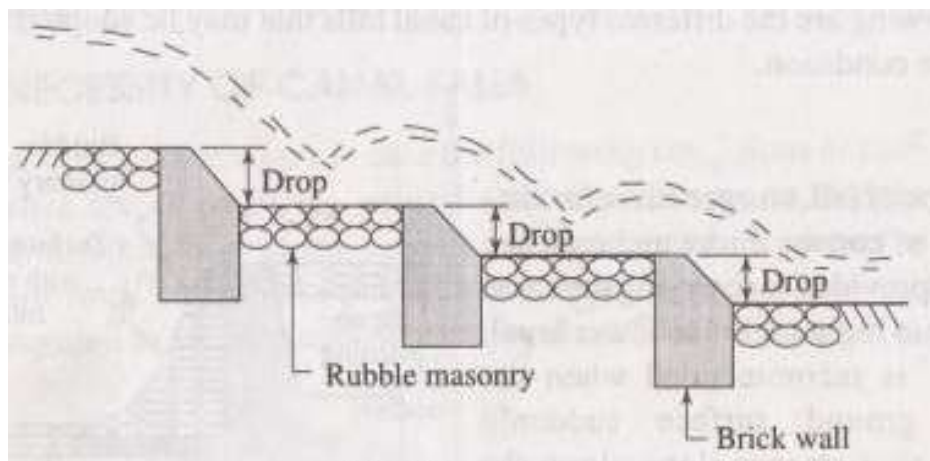
### 2. Rapid fall:

It is provided with long sloping floor having gentle slope in the range of 1 in 10 to 1 in 20.

When the natural ground level is even and rapid, this rapid fall is suitable. It consists of long sloping glacis. Curtain walls are provided on both u/s and d/s sides. Rubble masonry with cement grouting is provided from u/s curtain wall to d/s curtain wall. Masonry surface is finished with a rich cement mortar.

## Canal Fall/Drop:

### 3. Stepped fall:



## Canal Fall/Drop:

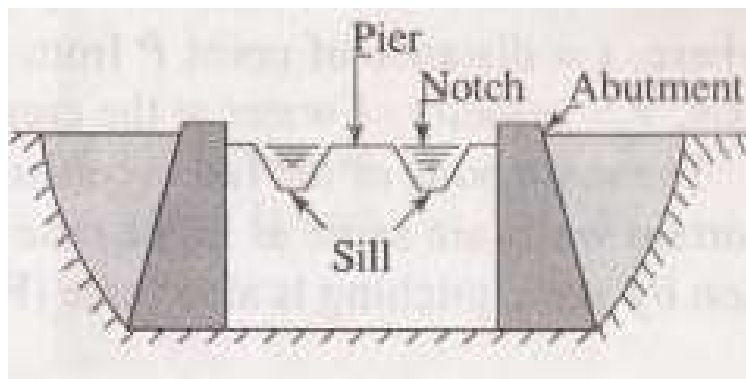
### 3. Stepped fall:

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 sloping glacis is divided into a number drops to bring down the canal bed step by step to protect the canal bed and sides from damage by erosion. Brick walls are provided at each drop. The bed of the canal within the fall is protected by rubble masonry with surface finishing by rich cement mortar.

## Canal Fall/Drop:

### 4. Trapezoidal notch fall:

Numbers of notches are constructed in a high crested wall across the channel with a smooth entrance and a flat circular tip projecting downstream from each notch to spread out the falling jet.



## Canal Fall/Drop:

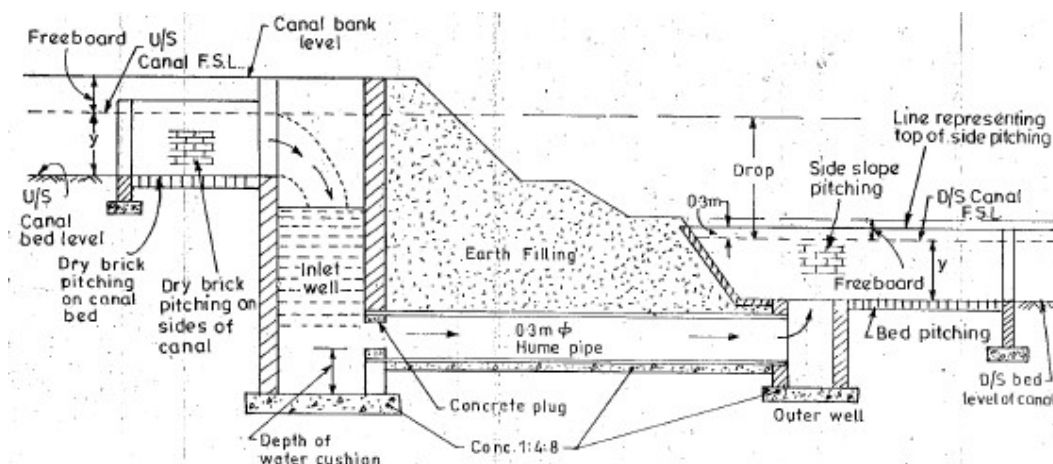
### 4. Trapezoidal notch fall:

It was designed by Reid in 1894. In this type a body or foundation wall across the channel consisting of several trapezoidal notches between side pier and intermediate pier is constructed. The sill of the notches are kept at upstream bed level of the canal. The body wall is made of concrete. An impervious floor is provided to resist the scouring effect of falling water. Upstream and downstream side of the fall is protected by stone pitching finished with cement grouting

There would neither be drawdown nor heading of water, as the channel approaches the fall. These falls remained quite popular, till simpler, economical, and better modern falls were developed.

## Canal Fall/Drop:

### 5. Well type falls or Cylinder falls or syphon well drops:



## Canal Fall/Drop:

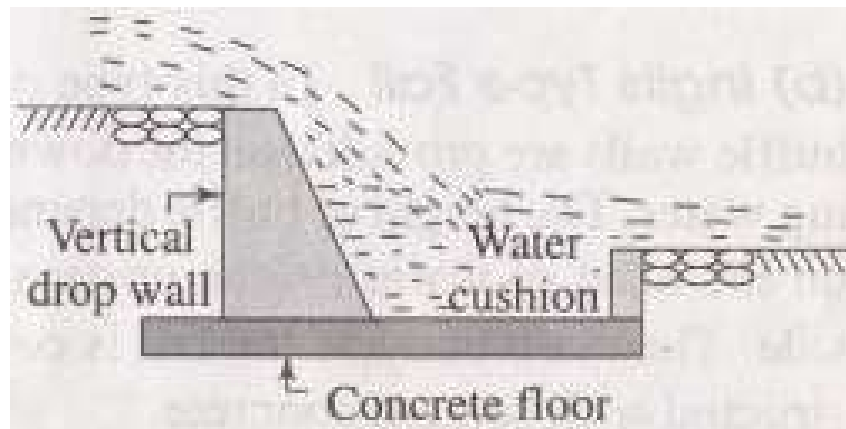
### 5. Well type falls or Cylinder falls or syphon well drops:

In this type, water of canal from higher level is thrown in a well or a cylinder from where it escapes from bottom. Energy is dissipated in the well in turbulence. They are suitable for low discharges and are economical also.

## Canal Fall/Drop:

### 6. Vertical fall (Sarada type fall):

A crest wall is constructed to create a vertical drop a cistern is provided to dissipate the surplus energy of water leaving the crest.



## Canal Fall/Drop:

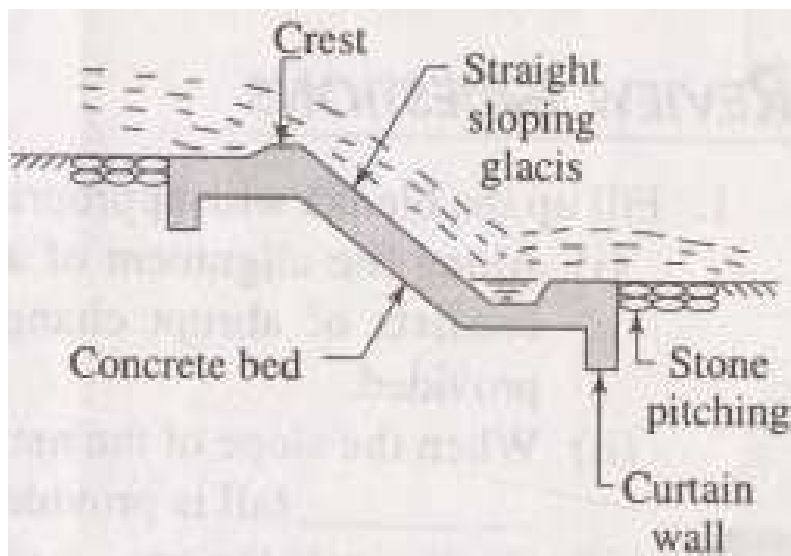
### 7. Straight glacis fall:

This is modern type fall where, slope of 2:1; is provided after a raised crest. The hydraulic jump is made to occur on the glacis causing sufficient energy dissipation.

It consists of a straight glacis provided with a crest wall. For dissipation of energy of flowing water, a water cushion is provided. Curtain walls are provided at toe and heel. Stone pitching is required at upstream and downstream of the fall.

## Canal Fall/Drop:

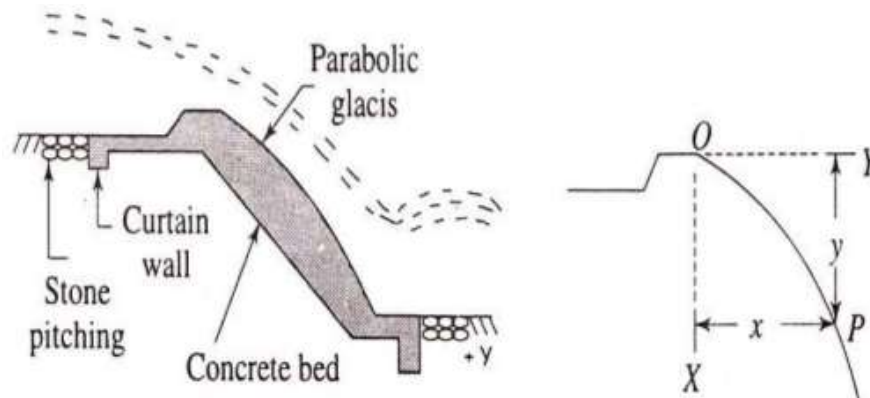
### 7. Straight glacis fall:



## Canal Fall/Drop:

### 8. Montague type fall:

It is the improvement over straight glacis fall by replacing the straight glacis by a parabolic glass.



## Canal Fall/Drop:

### 8. Montague type fall:

In the straight glacis type profile, energy dissipation is not complete. Therefore, montague developed this type of profile where energy dissipation takes place. His profile is parabolic and is given by the following equation,

$$X = U \sqrt{\frac{4Y}{g}} + Y$$

where X = The horizontal ordinate of any point of the profile measured from the d/s edge of crest.

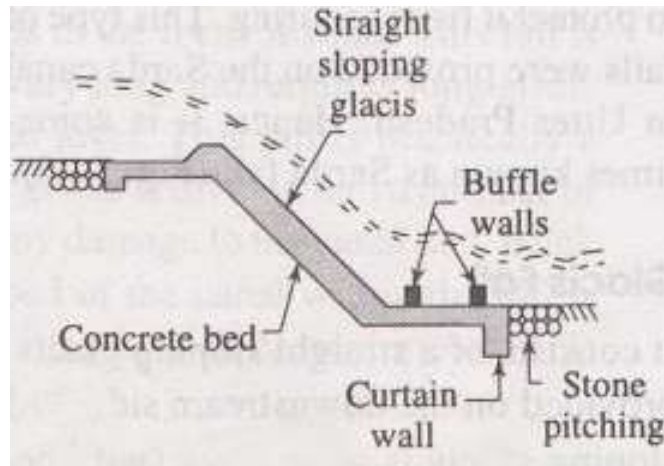
Y = Vertical ordinate measured from the crest level.

U = Initial velocity of water leaving the crest.

## Canal Fall/Drop:

### 9. Inglis fall or Baffle fall:

It is modified form of glacis fall by providing a baffle wall of a certain height at some distance downstream of toe of the glacis.



## Canal Fall/Drop:

### 9. Inglis fall or Baffle fall:

Here glacis is straight and sloping, but baffle wall provided on the downstream floor dissipate the energy. Main body of glacis is made of concrete. Curtain walls both at toe and heel are provided. Stone pitching are essential both at u/s and d/s ends

## Comparison of Different Types of Falls

- (i) Vertical drop falls are quite suitable for discharges upto 15 cumecs and drops up to 1.5 m. But this type of fall should not be flumed.
- (ii) Straight glacis type falls work satisfactorily for all conditions, if unflumed ; but in that case they become costly. Even then, they can be adopted suitably for discharges up to 60 cumecs and drops up to 1.5 m, and can even be flumed.
- (iii) Baffle fall or Englis fall may be used for all discharges when drop is more than 1.5 m. This type off all functions very satisfactorily, either flumed or unflumed, so long as it is undrowned.
- (iv) Well type falls are suitable and economical for high drops and very low discharges. They can hence be easily used, as tail escapes of small channels.

## Selection of Site for a Fall:

### Following points are given due consideration while selecting the site for a fall:

- So far as possible the fall should be combined with a rail or road bridge. The obvious reason is then abutments, wing walls and foundation are common to both the structures. Such combination makes a project economical.
- For achieving economy, a fall should be combined with a regulator which is required for bifurcating the canal.
- The site for a fall on the distributaries and minors should be selected in such a way that the command is not sacrificed in process of lowering the bed. Actually when the falls are provided the FSL below the fall goes below natural surface level for some length. Then it becomes necessary to provide outlet on the upstream side of the fall to command the area which ties immediately below the fall.

### 12.5. Design of Simple Vertical drop Fall

In a vertical drop fall, the energy of the flowing water is dissipated by means of impact and by sudden deflection of velocity from vertical to horizontal direction. A water cushion is provided at the toe of the drop, so as to reduce the impact of falling jet and thus to save the downstream floor from scour. The water cushion is formed by depressing the floor below the downstream bed of the canal, as shown in Fig. 12.15.

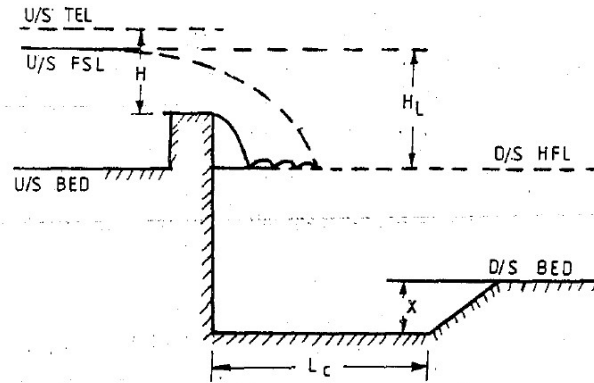
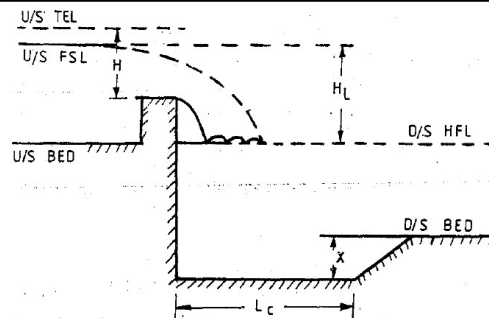


Fig. 12.15



The following dimensions for the cistern have been suggested by U.P. Irrigation Research Institute :

$$L_C = 5 \cdot \sqrt{H \cdot H_L} \quad \dots(12.8)$$

$$X = \frac{1}{4} \cdot (H \cdot H_L)^{2/3} \quad \dots(12.9)$$

where  $L_C$  = The length of the cistern in metres.

$X$  = Cistern depression below the downstream bed in metres.

$H$  = Head of water over the crest, including velocity head, in metres, i.e. = (u/s TEL - Crest level).

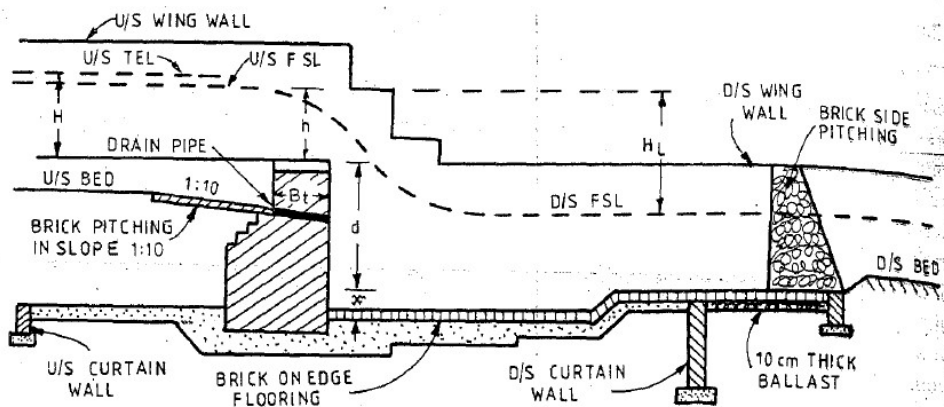
### 12.6. Design of a Sarda Type Fall

The design criteria for various components of such a fall, based on the recommendations of Bahadarabad Research Station, are given below :

**Length of the Crest.** Since fluming is not permissible in this type of falls, *the length of the crest is kept equal to the bed width of the canal.* Sometimes, for future expansion, the crest length may be kept equal to (bed width + depth).

**Shape of the Crest.** A rectangular crest with both faces vertical has been suggested for discharges under 14 cumecs. The top width is kept equal to  $0.55\sqrt{d}$  and the minimum base width is kept equal to  $\frac{h+d}{G}$  (Take  $G=2$  for masonry) where  $d$  is the height of the crest above the downstream bed level and  $h$  is the head over the crest [See Fig. 12.16 (a)].

For discharges over 14 cumecs, a trapezoidal crest with top width equal to  $0.55 \cdot \sqrt{H+d}$  with upstream side slope of 1 : 3 and downstream side slope of 1 : 8 is adopted [See Fig. 12.16 (b)].

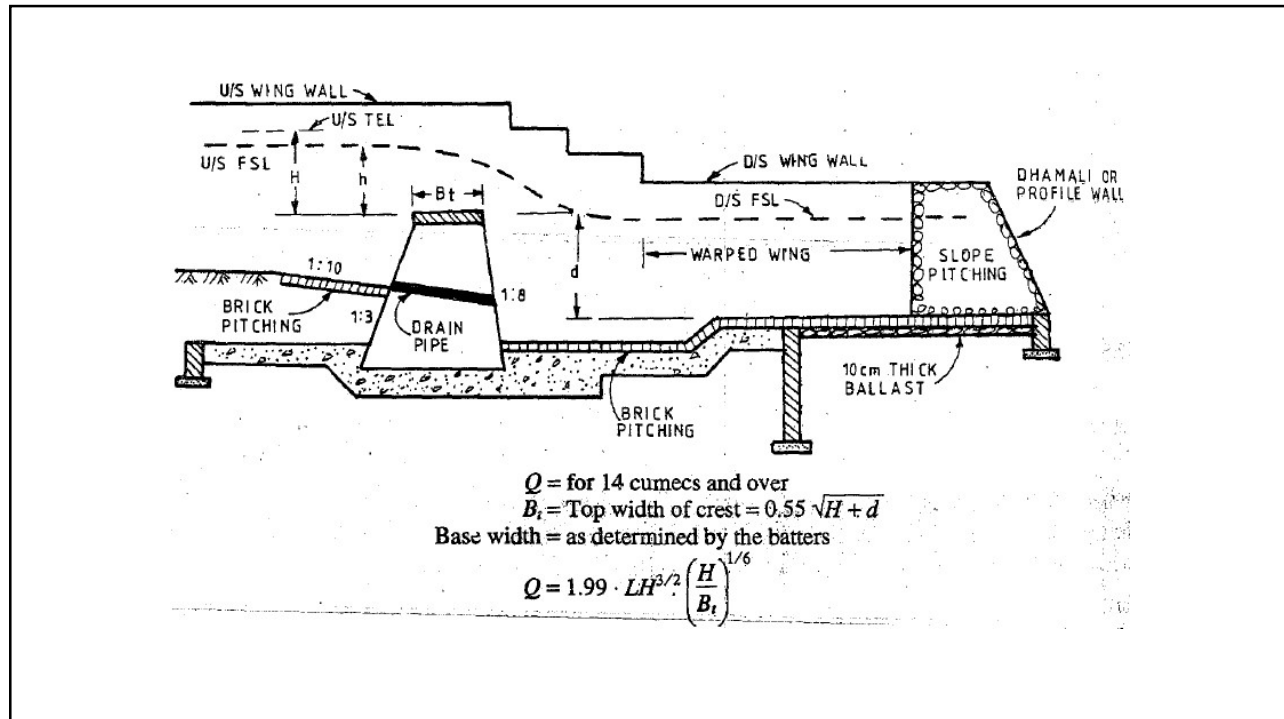


$$Q = \text{upto a maximum of 14 cumecs}$$

$$B_t = \text{Top width of crest} = 0.55\sqrt{d}$$

$$\text{Base width} = \frac{h+d}{2}$$

$$Q = 1.84 \cdot LH^{3/2} \left( \frac{H}{B_t} \right)^{1/6}$$



**Crest level.** The following discharge formula is used to determine the height of the crest.

$$Q = C_d \cdot \sqrt{2g} \cdot L \cdot H^{3/2} \cdot \left(\frac{H}{B_t}\right)^{1/6} \quad \dots(12.10)$$

where  $C_d = 0.415$  for rectangular crest  
 $= 0.45$  for trapezoidal crest

$L =$  Length of the crest

$B_t =$  Top width of crest.

Height of the crest above bed  $= y - h$

$\approx y - H$  (assuming  $h \approx H$  i.e. neglecting velocity of approach)

where  $y$  is the normal depth of channel (upstream).

**Upstream Wing Wall.** For trapezoidal crest, the upstream wing walls are kept segmental with radius equal to 5 to 6 times  $H$  and subtending an angle of  $60^\circ$  at centre, and then carried tangential into the berm as shown in Fig. 12.17. The foundations of the wing walls are laid on the impervious concrete floor itself.

For rectangular crest (*i.e.* discharge less than 14 cumecs), the approach wings may be splayed straight at an angle of  $45^\circ$ .

**Upstream Protection.** Brick pitching in a length equal to upstream water depth may be laid on the upstream bed, sloping towards the crest at a slope of 1 : 10. Drain pipes should also be provided at the u/s bed level in the crest so as to drain out the u/s bed during the closer of the channel.

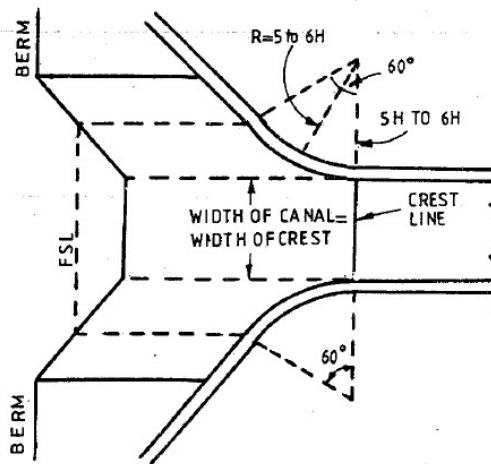


Fig. 12.17. Upstream wing walls for Trapezoidal crest of Sarda Type fall.

**Upstream Curtain Wall.**  $1\frac{1}{2}$  brick thick upstream curtain wall is provided, having a depth equal to  $\frac{1}{3}$ rd of water depth.

**Impervious Concrete Floor.** The total length of impervious floor can be determined by Bligh's theory for small works and by Khosla's theory for large works. The minimum length of floor on d/s of the toe of the crest wall should be =  $[2(\text{water depth} + 1.2 \text{ m}) + \text{drop}]$ . The balance can be provided under the crest and on upstream.

The floor thickness required on the downstream side can be worked out for uplift pressures (using minimum thickness of 0.4 m to 0.6 metre) and only a nominal thickness of 0.3 metre is provided on the upstream side. The maximum seepage head will occur when water is stored upto-top of crest on u/s side and there is no flow on the downstream side.

**Cistern.** The length and depth of cistern can be worked out from equations (12.8) and (12.9).

**Downstream Protection.** The d/s bed may be protected with dry brick pitching, about 20 cm thick resting on 10 cm thick ballast. The length of the d/s pitching is given by the values of Table 12.1; or 3 times the depth of downstream water, whichever is more. The pitching may be provided between two or three curtain walls. The curtain walls may be  $1\frac{1}{2}$  brick thick and of depth equal to  $\frac{1}{2}$  the downstream depth; or as given in Table 12.1 (minimum = 0.5 m).

Table 12.1

Head over the crest $H$ (metres)	Total length of $d/s$ pitching (metres)	Remarks	Curtain walls	
			No.	Depth in metres
Upto 0.3 m	3.0	All sloping down at 1 in 10	1	0.30
0.3 to 0.45	$3.0 + \text{Twice } H_L$	Horizontal up to end of masonry wings and then sloping down at 1 : 10	1	0.30
0.45 to 0.60	$4.5 + \text{Twice } H_L$		1	0.45
0.60 to 0.75	$6.0 + \text{Twice } H_L$		1	0.60
0.75 to 0.90	$9.0 + \text{Twice } H_L$		1	0.75
0.90 to 1.05	$13.5 + \text{Twice } H_L$	"	2	0.90
1.05 to 1.20	$18.0 + \text{Twice } H_L$	"	2	1.05
1.20 to 1.50	$22.5 + \text{Twice } H_L$	"	3	1.35

**Slope Pitching.** After the return wing, the sides of the channel are pitched with one brick on edge. The pitching should rest on a toe wall  $1\frac{1}{2}$  brick thick and of depth equal to half the downstream water depth. The side pitching may be curtailed at an angle of  $45^\circ$  from the end of the bed pitching, or extended straight from the end of the bed pitching.

**Downstream Wings.** Downstream wings are kept straight for a length of 5 to 8 time  $\sqrt{H} \cdot H_L$  and may then be gradually wrapped. They should be taken upto the end of the pucca floor.

All wing walls must be designed as retaining walls, subjected to full pressure of submerged soil at their back when the channel is closed. Such a wall generally has a base width equal to  $\frac{1}{3}$ rd its height.

**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 m
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

**Solution.**

**Length of crest.** Same as d/s bed width = 10 m

**Crest level.** A rectangular crest is provided, since the discharge is less than 14 cumecs. The discharge formula is given by

$$Q = 1.84 \cdot L \cdot H^{3/2} \left[ \frac{H}{B_1} \right]^{1/6}$$

Assume top width of the crest as 0.8 m.

$$\therefore 12 = 1.84 \times 10 \times H^{3/2} \times \frac{H^{1/6}}{(0.8)^{1/6}}$$

$$\text{or } H^{5/3} = \frac{12 \times 0.964}{1.84 \times 10} = 0.628$$

$$\text{or } H = (0.628)^{3/5} = 0.755 \text{ m ; Say } H = 0.76 \text{ m.}$$

Velocity of approach

$$= V_a = \frac{\text{Discharge}}{\text{Area}} = \frac{12}{(10 + 1.5) 1.5} \quad (\because \text{Depth of water} = 1.5 \text{ m})$$

$$= \frac{12}{11.5 \times 1.5} = 0.696 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V_a^2}{2g} = 0.025 \text{ m.}$$

$$\begin{aligned} \text{u/s TEL} &= \text{u/s FSL} + \text{Velocity Head} \\ &= 104.5 + 0.025 = 104.525 \text{ m} \end{aligned}$$

R.L. of the crest

$$\begin{aligned} &= (\text{u/s TEL} - H) \\ &= 104.525 - 0.755 = 103.77 \text{ m.} \end{aligned}$$

**Use crest level of 103.77 metres**

Height of the crest above d/s floor

$$= 103.77 - 103.0 = 0.77 \text{ m.}$$

*Shape of the crest.*

Width of the crest ( $B_c$ )

$$= 0.55 \cdot \sqrt{d}$$

where  $d$  = Height of the crest above d/s bed

$$= 103.77 - 101.5 = 2.27 \text{ m}$$

$$\therefore B_c = 0.55 \cdot \sqrt{d} = 0.55 \cdot \sqrt{2.27} = 0.825 \text{ m.}$$

**Keep 0.85 m width of the crest**

$$\text{Thickness at base} = \frac{h + d}{2}$$

$$= \frac{(0.755 - 0.025) + 2.27}{2} = \frac{0.73 + 2.27}{2} = 1.5 \text{ m.}$$

The top shall be capped with 20 cm thick C.C. 1 : 2 : 4

*Upstream wing wall.* It shall be splayed straight at an angle of  $45^\circ$  from the u/s edge of the crest and shall be embedded by 1.0 m into the berm. On the d/s side, wing walls are kept straight and parallel up to the end of the floor and joined to return walls, as shown in Fig. 12.19.

*Upstream protection.* 1.5 m long brick pitching (equal to u/s water depth) is laid on the u/s bed, sloping down towards the crest at 1 : 10, and *three drain pipes of 15 cm diameter at the u/s bed level should be provided in the crest so as to drain out the u/s bed during the closure of the canal.*

*Upstream curtain wall.* Maximum depth of u/s curtain wall

$$= \frac{y_u}{3} = \frac{1.5}{3} = 0.5 \text{ m.}$$

Provide 0.4 m  $\times$  0.8 m deep curtain wall on the u/s.

*Cistern.* Depth of cistern, is given by Eq. 12.9 as

$$X = \frac{1}{4} [H \cdot H_L]^{2/3} = \frac{1}{4} [0.76 \times 1.5]^{2/3} = \frac{1}{4} \times (1.14)^{0.667} \quad \dots(12.9)$$

$$= \frac{1}{4} \times 1.091 = 0.273 \text{ m ; Say-0.3 m deep.}$$

$$\therefore \text{R.L. of cistern} = 101.5 - 0.3 = 101.2 \text{ m.}$$

Length of cistern =  $5 \sqrt{H \cdot H_L}$

$$= 5 \times \sqrt{0.76 \times 1.5} = 5 \times \sqrt{1.14} = 5.34 \text{ m ; say 5.5 m.}$$

**Provide 5.5 m long cistern at R.L. 101.2 m.**

*Impervious floor.*

Maximum Static Head

$$= (\text{Crest level} - \text{d/s bed level})$$

$$= 103.77 - 101.5 = 2.27 \text{ m.}$$

Total floor length required

$$= C.H.; \text{ where } C \text{ is Bligh's coefficient}$$

$$= 6 \times 2.27 = 13.62 \text{ m. ; say 13.7 m.}$$

Minimum d/s floor length required

$$= [2 (\text{Water depth} + 1.2) + H_f]$$

$$= 2 (1.5 + 1.2) + 1.5 = 2 (2.7) + 1.5 = 5.4 + 1.5 = 6.9 \text{ m ; say 7 m.}$$

Provide 7 m d/s floor and the balance 6.7 m under and upstream of the crest, as shown in Fig. 12.18.

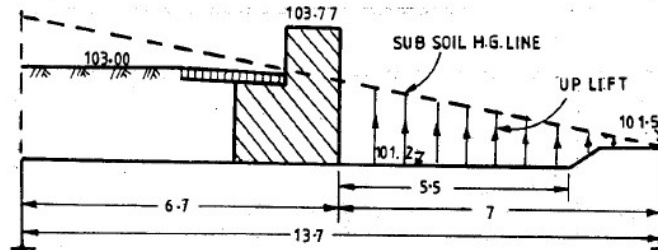


Fig. 12.18

**Floor Thicknesses.** H.G. line for the maximum static head is shown in Fig. 12.18.

*Maximum unbalanced uplift at the d/s toe of the crest*

$$= 0.3 + \frac{(103.77 - 101.5)}{13.7} \times 7 = 0.3 + 1.16 = 1.46 \text{ m}$$

Thickness required  $\frac{1.46}{1.24} = 1.29 \text{ m ; say 1.3 m.}$

Provide 1.1 m thick concrete overlain with 0.2 m thick brick pitching.

*Unbalanced head at 3 m from the toe of the crest*

$$= 0.3 + \frac{2.27}{13.7} \times 4 = 0.3 + 0.67 = 0.97$$

Thickness required  $= \frac{0.97}{1.24} = 0.78 \text{ m ; say 0.8 m.}$

Use 0.6 m thick concrete with 0.2 m brick layer.

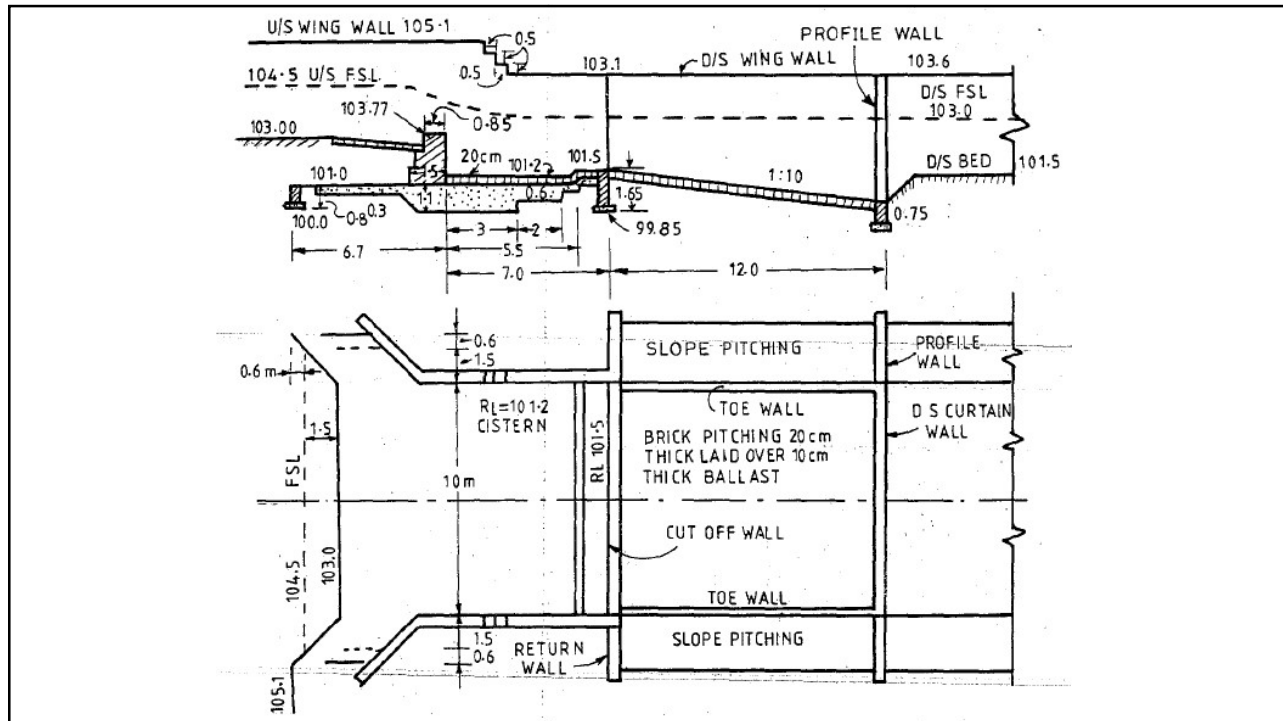
*Unbalanced head at 5 m from the toe*

$$= 0.3 + \frac{2.27}{13.7} \times 2 = 0.3 + 0.33 = 0.63 \text{ m.}$$

Thickness required

$$= \frac{0.63}{1.24} = 0.51 \text{ m ; Say 0.55 m.}$$

Use 0.35 m thick concrete with 20 cm thick brick layer, as shown in Fig. 12.19.



**D/s Curtain Wall.** The curtain wall at the d/s end of the floor should be 0.75 m deep (for  $H = 0.76$  m in Table 12.1)

Provide 0.4 m  $\times$  1.65 m deep curtain wall at d/s end of floor, i.e. upto a level of 101.5 - 1.65 = 99.85 metres, i.e. the deepest foundation level.

**Downstream pitching.** From Table 12.1,

Total length of d/s pitching  
 $= 9 + 2 \times 1.5 = 12$  metres.

Pitching is kept sloped at 1 : 10. A curtain wall of 0.4 m  $\times$  0.75 m shall be provided at the end of the pitching, as shown in Fig. 12.19.

**Example 12.4.** Design a 1.5 metres Sarda Type fall for a canal carrying a discharge of 40 cumecs with the following data :

Bed level upstream	= 105.0 m.
Bed level downstream	= 103.5 m.
Side slopes of channel	= 1 : 1
Full supply level upstream	= 106.8 m.
Full supply level downstream	= 105.3 m.
Berm level u/s	= 107.4 m
Bed width u/s and d/s	= 30 m.
Safe exit gradient for Khosla's Theory	= 1/5.

**Solution.**

*Length of the crest.* Is kept equal to bed width = 30 metres.

*Crest level.* A trapezoidal crest is provided, since the discharge is more than 14 cumecs.

The discharge formula is given by

$$Q = 1.99 LH^{3/2} \left( \frac{H}{B_t} \right)^{1/6}$$

Assume  $B_t = 1.0$  m.

$$\therefore 40 = 1.99 \times 30 \frac{H^{3/2} \cdot H^{1/6}}{(1)^{1/6}}$$

or  $H^{5/3} = \frac{40}{1.99 \times 30} = 0.796$

or  $H = (0.796)^{3/5} = 0.862$  m; say **0.865 m.**

Velocity of approach

$$\therefore V_a = \frac{40}{(30 + 1.8) 1.8} = 0.7 \text{ m/sec} \quad (\because \text{Full supply depth} = 1.8 \text{ m})$$

$$\text{Velocity head} = \frac{(0.7)^2}{2g} = 0.025 \text{ m.}$$

$$\begin{aligned} \text{u/s TEL} &= \text{u/s FSL} + \text{Velocity head} \\ &= 106.8 + 0.025 = \mathbf{106.825 \text{ m.}} \end{aligned}$$

$\therefore$  R.L. of the crest

$$\begin{aligned} &= \text{u/s TEL} - H \\ &= 106.825 - 0.862 = \mathbf{105.963 \text{ m.}} \end{aligned}$$

**Adopt a crest level of 105.97 m.**

*Shape of the crest.* Adopt crest width at top

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

where  $H = 0.865$  m

$d =$  Height of the crest above d/s bed

$$= 105.97 - 103.5 = 2.47 \text{ m.}$$

$$\therefore B_t = 0.55 \sqrt{0.865 + 2.47} = 0.55 \sqrt{3.335} = 1.0 \text{ m.}$$

Adopt a trapezoidal crest with top width of 1.0 m and u/s slope 1 : 3 and d/s slope 1 : 8.

**Upstream Wing Walls.** Radius of the wings should be 5 to 6 times the head over the crest =  $5 \times 0.865 = 4.325$ , to  $6 \times 0.865 = 5.19$  m. Use 5.0 m radius for the wings. U/s wing walls shall be kept segmental with 5 m radius subtending an angle of  $60^\circ$  at centre and then carried tangentially into the berm.

**Downstream Wing Walls.** The downstream wings shall be kept straight up to a distance of say  $6 \cdot \sqrt{H \cdot H_L}$ , i.e.  $6 \cdot \sqrt{0.865 \times 1.5} = 6.8$  m; say 7 m, and then warped in a slope of 1 : 1 and shall be taken upto the end of pucca floor.

**Upstream Protection.** Brick pitching equal to u/s water depth i.e. 1.8 m is laid on the u/s towards the crest at 1 : 10 slope. Provide 20 cm drain holes in the entire length at 3 m c/c to drain out the u/s bed during the closure of the canal.

**Upstream Curtain Wall.** The minimum depth of curtain wall =  $\frac{1}{3}$ rd water depth, i.e.  $\frac{1}{3} \times 1.8 = 0.6$  m. Provide 0.7 m deep masonry wall over 0.3 m thick concrete.

Thus, provide a curtain wall  $0.4 \text{ m} \times 1.0 \text{ m}$  deep on the u/s.

**Downstream Curtain Wall.** Minimum thickness

$$= \frac{\text{Depth}}{2} = \frac{1.8}{2} = 0.9 \text{ m.}$$

or from Table 12.1, it is equal to 0.75 m.

Provide a d/s curtain wall  $0.4 \text{ m} \times 1.4 \text{ m}$  over 0.3 m cement concrete. Thus, total depth of d/s curtain wall shall be 1.7 m with its bottom level at 101.8 m.

**Cistern.** Depth of cistern

$$= X = \frac{1}{4} (H \cdot H_L)^{2/3} \quad \dots(12.9)$$

$$\therefore X = \frac{1}{4} [(0.865 \times 1.5)^{2/3}] = \frac{1}{4} \times 1.19 = 0.3 \text{ m (say)}$$

$$\text{R.L. of Cistern} = 103.5 - 0.3 = 103.2 \text{ m.}$$

$$\text{Length of cistern} = 5 \sqrt{H \cdot H_L} = 5 \cdot \sqrt{0.865 \times 1.5} = 5 \times 1.14 = 5.7 \text{ m.}$$

Provide 5.7 m long Cistern

**Total Floor Length and Exit Gradient**

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi \sqrt{\lambda}}$$

Maximum static head ( $H$ ) is caused when water is stored upto the crest level and there is no water d/s.

$$H = 105.97 - 103.5 = 2.47 \text{ m.}$$

$$d = 1.7 \text{ m (i.e. Depth of d/s curtain wall)}$$

$$G_E = 1/5 \text{ (given)}$$



(2) *Toe of Crest.*

$$b_1 = 11 \text{ m}$$

$$b = 18 \text{ m}$$

$$\frac{b_1}{b} = \frac{11}{18} = 0.61$$

$$d = 103.2 - 101.9 = 1.3 \text{ m}$$

$$\alpha = \frac{b}{d} = \frac{18}{1.3} = 13.9$$

From Plate 11.1(b)

$$\phi_{D_2} = 36\%$$

(3) *Downstream Curtain Wall.*

$$d = 1.7 \text{ m}$$

$$b = 18 \text{ m}$$

$$\frac{d}{b} = \frac{1.7}{18} = 0.094$$

From Plate 11.1(a)

$$\phi_{E_3} = \phi_E = 29\%$$

$$\phi_{D_3} = \phi_D = 20\%$$

$$\phi_{C_3} = 0\%$$

$$\phi_{E_3} = \phi_E = 29\%$$

$$\phi_{D_3} = \phi_D = 20\%$$

$$\phi_{C_3} = 0\%$$

Correction for depth to  $\phi_{E_2}$

$$= \frac{29\% - 20\%}{1.7} \times 0.8 = \frac{9}{1.7} \times 0.8 = 4.2\% \text{ (-ve)}$$

$$\phi_{E_3} \text{ (corrected)} = 29 - 4.2 = 24.8\%$$

The levels of H.G. line for maximum static head are worked out in Table 12.2 and plotted in Fig. 12.20.

Table 12.2

Condition of flow	u/s W.L. in metres	d/s W.L. in metres	Head H in metres	Height/Elevation of H.G. line above datum						
				$\phi_{E_1}$ 100%	$\phi_{D_1}$ 84%	$\phi_{C_1}$ 80.5%	$\phi_{D_2}$ 36%	$\phi_{E_3}$ 24.8%	$\phi_{D_3}$ 20%	$\phi_{C_3}$ 0%
Maximum static head, i.e. Water up to crest level on u/s and no water downstream	105.97	103.50	2.47	2.47	2.08	1.99	0.89	0.61	0.50	0.90
				105.97	105.58	105.49	104.39	104.11	104.00	103.5

**Floor Thicknesses**

Provide a nominal thickness of 0.4 m under u/s floor.

Unbalanced head at d/s toe of glacis =  $104.39 - 103.2 = 1.19$  m

Thickness required =  $\frac{1.19}{1.24} = 0.97$  m ; Use 1.2 m.

Provide 1.0 m thick C.C. overlain by 0.2 m thick brick pitching.

Unbalanced head at d/s end of floor =  $104.11 - 103.5 = 0.61$  m.

Thickness required =  $\frac{0.61}{1.24} = 0.5$  m ; Use 0.7 m.

Use 0.5 m thick C.C. overlain by 0.2 m thick brick pitching.

Unbalanced head at 3 m from d/s toe of crest

$$= 0.91 + \frac{0.28}{7} \times 3 = 0.91 + 0.12 = 1.03 \text{ m}$$

Thickness required =  $\frac{1.03}{1.24} = 0.83$  m.

Provide 0.8 m thick C.C. laid over by 0.2 m thick bricks.

Thicknesses provided are shown in Fig. 12.21.

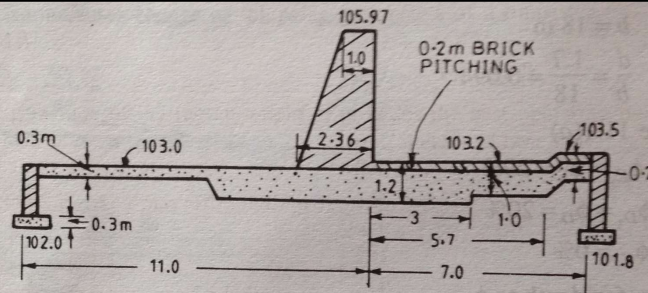


Fig. 12.21

**Downstream Pitching.** From Table 12.1, the length of d/s pitching required

$$= 9 + 2H_L = 9 + 2 \times 1.5 = 12 \text{ m.}$$

The pitching is kept sloping at 1 : 10 and a curtain wall of 0.4 m × 0.75 m is provided at the end of this pitching. The bed pitching and side slope pitching are separated by a toe wall 0.4 m × 0.75 m. Slope pitching is curtailed at an angle of 45° from the end of the bed pitching.

*Note :* Two rows of friction blocks of size 1.2 m × 0.6 m × 0.6 m staggered at a distance of 1.0 m from toe of the crest on the d/s cistern and two rows of cube blocks of size 0.25 m × 0.25 m × 0.25 m staggered at the d/s end of the floor, may be provided in conservative designs as an additional source of energy dissipation, as explained a little later under the heading of 'Roughening Devices'.

# RIVER TRAINING WORKS

## **River Training Works**

It implies various measures adopted on a river to divert and guide river flow, to train and control river beds, and to increase depth of flow.

### **Objectives (or Necessities):**

- To pass flood discharge safely and quickly
- To transport sediment load (suspended and bed) efficiently
- To prevent bank erosion and degradation of river bed
- To sustain the water depth up to minimum draft required for navigation purpose
- To guide or divert the flow through specified direction

### **Purpose:**

- To establish the channel along a certain alignment

## 10.2 Stages of rivers and their meandering process:

### Stages of rivers:

- A) upper reach
- B) middle reach
- C) lower reach

#### A) upper reach:

- i) Hilly reach (gradient <sup>1:100 to 1:500</sup> to 1:500) - incised rivers  
The rivers in this reach are characterized by the following attributes:
  - channel formation by the process of degradation
  - dissimilar character of river bed material and sediment transported.
  - steep bed slope.
  - swiftness of flow
  - formation of rapids along the courses
  - irregular pattern of meanders.

#### ii) Foothill submontane Reach (gradient 1:500 to 1:1000) - Boulder reach

The rivers in this reach are characterized by

- steepness of their bed slopes.
- bed materials consisting of a mixture of boulders, gravel, shingle and sand.
- high velocity flow.
- straighter, wide and shallow beds.
- braided and interlaced channels.

#### B) Middle reach (1:1000 to 1:10000) - Rivers in flood plains (alluvial rivers)

The rivers in this reach are characterized by

- similarity in characteristics of bed materials and materials transported.
- meandering free from one bank to another.
- constant erosion of concave banks and deposition.

Rivers in this reach are also classified as

Trough stage

- Aggrading - building of river bed due to heavy load, obstruction due to construction of dam/weir etc.
- Degrading - scouring of river bed at downstream of a dam/weir which results in lowering the water surface.
- Stable - neither scouring nor depositing the material of the alignment of the channel and its slopes as well as of its regime with very little variation from year to year. (1:10,000 to 1:20,000)

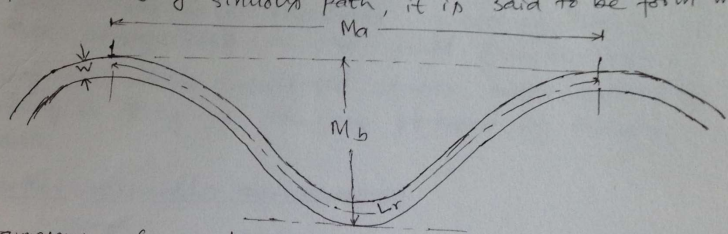
c) Lower reach - Tidal and delta rivers

The rivers in this reach

- the water levels change periodically due to tides.
- the tidal effect depends on the slope of the river, tidal range, freshet discharge, configuration of the river etc.
- river may split into branches and form a delta before becoming tidal.

Meandering of River:

When the course of river deviates from its straight flow and follow winding sinuous path, it is said to be form meanders.



Parameters of meander:

Meander:  
A meander has two consecutive loops, one flows clockwise and another anticlockwise.

Meander length (Ma):  
It is the axial length of one meander, i.e. the tangential distance between the corresponding points of a meander.

Meander belt (Mb):  
It is the distance between the outer edges of clockwise and anticlockwise loops of a meander.

Meander ratio:

It is the ratio of meander belt to the meander length,  
i.e.,  $M_b/M_a$

Tortuosity:

It is the ratio of the length of river channel to the axial length of the river i.e.,  $L_r/M_a$ .

Crossing (cross over):

The short straight reaches of the river, connecting two consecutive clockwise and anticlockwise, are called crossings.

Causes of meandering:

- due to earth's rotation (Lacey-1923)
- excessive slope and energy (Coxley-1938)
- changes in river stage (Russel-1936)
- steepness of bed slope (Lane-1957)

- excess sediment (Inglis 1947)
- local bank erosion (Fredkin-1945)
- local disturbance resulting in non uniformity and non-homogeneity in the fluid (Warner-1951)

Among the above mentioned causes, most convincing one according to Yang is the one proposed by Fredkin and Warner.

Relationships between meander parameters:

	<u>Trough</u>	<u>Incised</u>
$M_a$	$53\sqrt{Q}$	$46\sqrt{Q}$
$M_b$	$156\sqrt{Q}$	$102\sqrt{Q}$
$W$	$8.8\sqrt{Q}$	$4.5\sqrt{Q}$

where,  $M_a$ ,  $M_b$  and  $W$  are in m and  $Q$  in cumec.

### 10.3 Methods of river training:

#### Classification of river training works:

- i) High water training
- ii) Low water training
- iii) medium (mean) water training
- i) High water training - training for discharge

The primary purpose of high water training is to control flood. The river is trained to provide sufficient and efficient river cross-section for the safe passage of maximum flood. It concerns mainly with alignment and height of dykes or levees for a given flood discharge.

- ii) Low water training: - training for depth

The river is trained to provide sufficient depth for navigation during low stage of rivers. It is usually achieved by contraction of the width of the channel.

- iii) Mean water training: - training for sediment

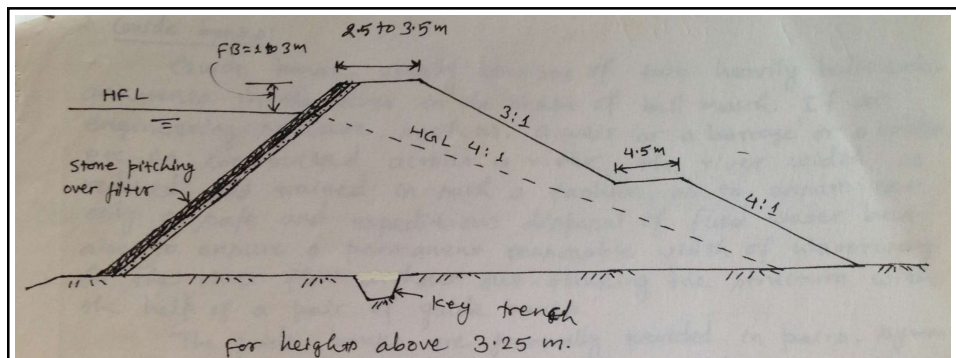
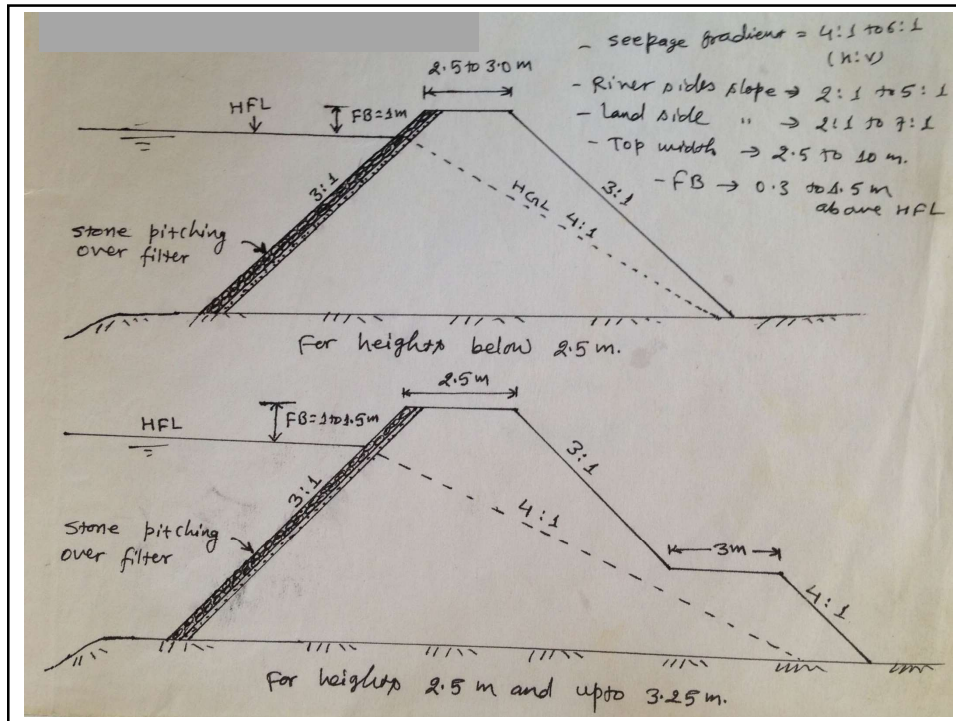
The river is trained to change river bed accordance with the stage of flood flow for the efficient transport of sediment load in order to keep the channel in good shape.

### Methods of river training

1. Marginal embankments or Levees or dykes
2. Guide banks or Bell bunds
3. Groynes or spurs
4. Artificial cut-offs
5. Pitching of banks and provision of launching aprons
6. Pitched islands
7. Miscellaneous methods such as sills, bandalling, etc.

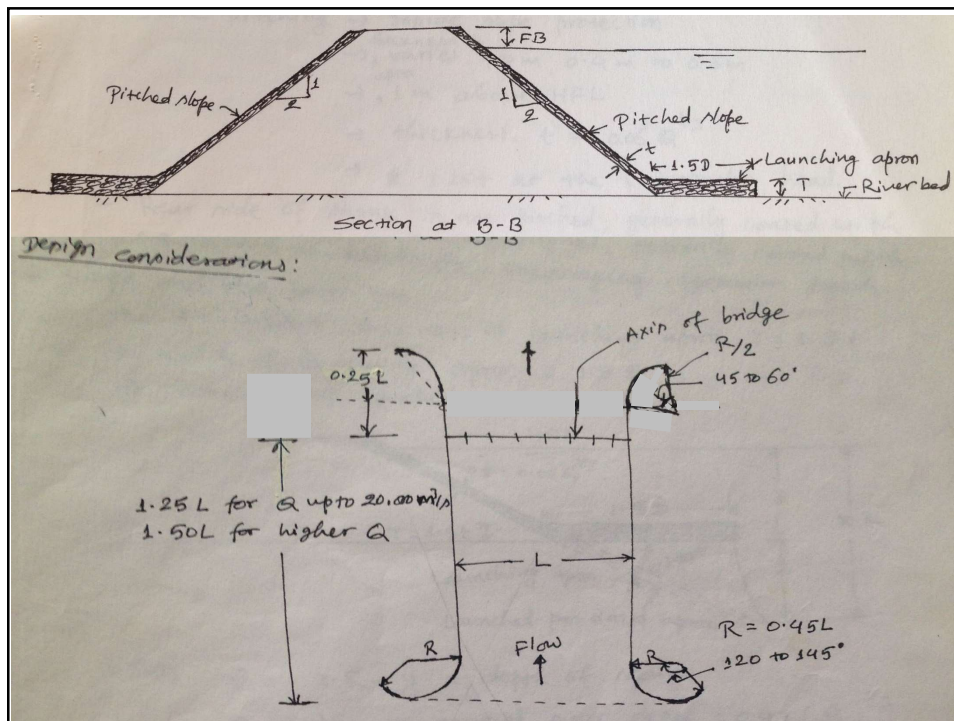
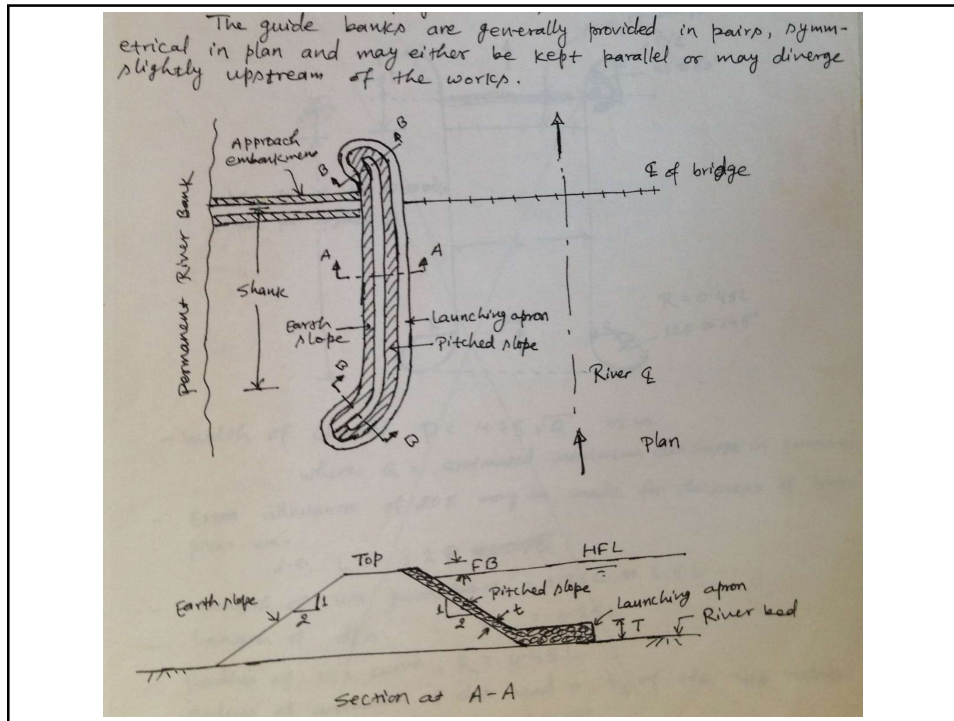
1. Marginal embankments or Levees or dykes:

Marginal embankments are generally earthen embankments, running parallel to the river, at some suitable distance from it. They may be constructed on both sides of the river or only on one side, for some suitable river length, where the river is passing through towns or cities or any other places of importance. These embankment-walls retain the flood water and thus, preventing it from spreading into the nearby lands or towns. A levee or a dyke is mainly used for flood protection by controlling the river and not by training the river.



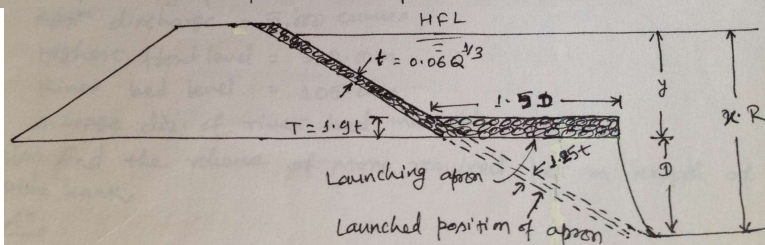
2 Guide banks:

Guide banks, usually consists of two heavily built embankments in the river in the shape of bell mouth. If an engineering structure, such as, a weir or a barrage or a bridge, etc., is constructed across a river, the river width is reduced and trained in such a fashion, as to ensure not only a safe and expeditious disposal of flood water but also to ensure a permanent reasonable width of waterway for the river flow without out-flanking the structure with the help of a pair of guide banks.



- width of channel,  $P = 4.75 \sqrt{Q}$  in m.  
where,  $Q =$  estimated maximum discharge in cumec.
- Extra allowance of 20% may be made for thickness of bridge piers, etc.  
i.e.,  $L = 1.2 P$   ~~$4.75 \sqrt{Q}$~~
- Length of u/s guide bank =  $1.25 L$  to  $1.5 L$
- Length of d/s " " =  $0.25 L$
- Radius of u/s curve,  $R = 0.45 L$ .
- Radius of curvature of d/s head =  $\frac{1}{2}$  of the u/s radius with a central angle of  $45^\circ$  to  $60^\circ$ .
- Side slope = 2:1 to 3:1
- Top width  $\neq 4$  m
- FB = 1.25 to 1.50 m

- Stone pitching  $\rightarrow$  inside slope protection
  - $\rightarrow$  thickness varies from 0.4 m to 0.6 m upto 1 m above HFL
  - $\rightarrow$  thickness,  $t = 0.06 Q^{1/3}$
  - $\rightarrow$   $1.25 t$  at the impregnable head.
- Rear side of plank is not pitched, generally coated with 0.3 m to 0.6 m earth for encouraging vegetation growth.
- Slope and toe protection:
  - $\rightarrow T = 1.25 t$  Thickness of launching apron,  $T = 1.9 t$
  - $\rightarrow$  width of launching apron =  $1.5 D$
  - $\rightarrow$  Thickness of launched apron =  $1.25 t$



where,  $D = R - y =$  depth of scour

$$R = \text{Lacey's normal scour depth} = 0.47 \left( \frac{Q}{f} \right)^{1/3}$$

$f =$  silt factor

values of  $K$  are tabulated as below:

S.N.		mean value of $K$	$D = KR - y$
1	Nose of guide banks	2.25	$2.25R - y$
2	Transition from noses, to straight portion	1.50	$1.5R - y$
3	straight reaches of guide banks	1.25	$1.25R - y$

- Assumed scour slope = 2:1

- ~~Thickness of launched apron = 1.25t~~

- Volume of stone required in the launched apron per unit length  
 $= \sqrt{2^2 + 1^2} * D * (1.25t) = 2.8tD$

- If width of the unlaunched apron is  $1.5D$ , then thickness of unlaunched apron,  $T = \frac{2.8tD}{1.5D} = 1.87t$  say  $1.9t$

i.e.,  $T = 1.9t$

\* Design a guide bank required for a bridge on a river having the following particulars:

Max<sup>m</sup> discharge = 5,000 cumecs

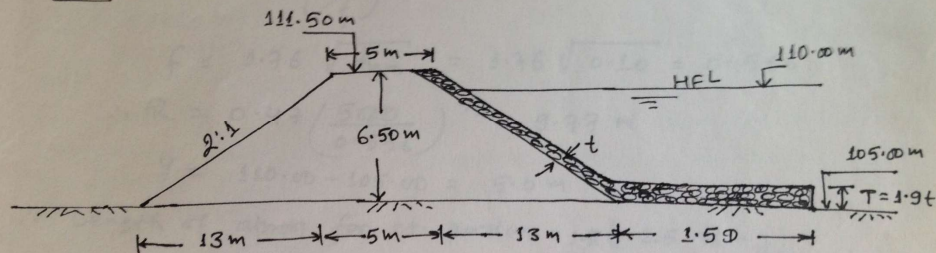
Highest flood level = 110.00 m

River bed level = 105.00 m

Average dia. of river bed material = 0.30 mm

Also find the volume of stone required per m length of the guide bank.

Sol<sup>n</sup>:





Length of apron for transition from noses to dt. portion,  
 $1.5 (1.5 R - y) = 1.5 (1.5 \times 9.77 - 5) = 14.48 \text{ m}$

Length of apron for noses =  $1.5 (2.25 R - y)$   
 $= 1.5 (2.25 \times 9.77 - 5) = 25.47 \text{ m}$

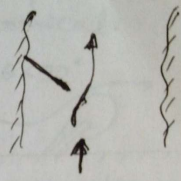
Volume of stone:  
 At ~~shank~~ <sup>slope portion</sup>, Volume =  $\sqrt{5} \times 6.50 \times t = \sqrt{5} \times 6.5 \times 1 = 14.53 \text{ m}^3/\text{m}$

~~at transition from~~ For apron  
 at shank, volume =  $2.8 t D = 2.8 \times 1 \times 7.21 = 20.19 \text{ m}^3/\text{m}$   
 at transition from dt portion =  $2.8 t D = 2.8 \times 1 \times 9.66 = 27.05 \text{ m}^3/\text{m}$   
 at noses, volume =  $2.8 t D = 2.8 \times 1 \times 16.98 = 47.54 \text{ m}^3/\text{m}$

3. Spur:

Spur is a linear structure, permeable or impermeable, projecting into a channel from a bank for the purpose of

- altering flow direction
- channel bank protection
- inducing deposition
- reducing flow velocity along the bank.



Spur types: <sup>materials</sup>

A) According to materials and/or construction

- 1) Retardance spur
- 2) Retardance/diverter spur
- 3) Diverter spur → impermeable spur

A) Based on materials of construction:

- 1) Permeable spur

It permits passage of restricted flow. It slows down the velocities over a portion of the channel area and thereby induces rapid deposition of sediments.

It is built of materials like ballies, baboos, timber brush, steel or wire etc.

ii) Impermeable spur:  
It does not permit passage of flow through it and such constitute solid obstruction. It is constructed in masonry or rockfill or earth core armoured with resistant material like stone, fascine mattress or saunpaga filled with stones.

B) Based on Function:

i) Retardance spur  
Retardance spur is designed to reduce the flow velocity in the vicinity of the channel bank or over the region of influence of the spur scheme.

ii) Retardance/diverter spur  
Retardance (diverter) spur is designed to function by retarding flow currents along the channel bank and providing flow deflection.

ii) diverter spur  
It is designed to function by diverting the primary flow currents away from the channel bank. It is most commonly constructed of dumped riprap since it is almost universally available and economical.

According to spur orientation for function (angle):

$\theta > 90^\circ$  Repelling spur       $\theta = 90^\circ$  Deflecting spur       $\theta < 90^\circ$  Attracting spur

- **Repelling spurs:** They force the flow away from themselves.
- **Deflecting spurs:** They change the direction of flow without repelling it. They are generally short and used for limited, local protection.
- **Attracting spurs:** They serve to attract the stream flow toward themselves and not repel the flow toward the opposite bank. They tend to maintain deep current close to the bank.

Some terminologies:

1) Permeability of spur  
 It is the percentage openness of surface of spur facing water flow.

2) Spur orientation ( $\theta$ )  
 Spur orientation is defined by spur angle, which is the angle between the flow tangential to spur tip and spur axis.

3) Expansion angle ( $\phi$ )  
 It is the angle between flow tangential to spur tip and line joining spur tip, and point in the nearby bank where the flow would have expanded after the impact of spur.

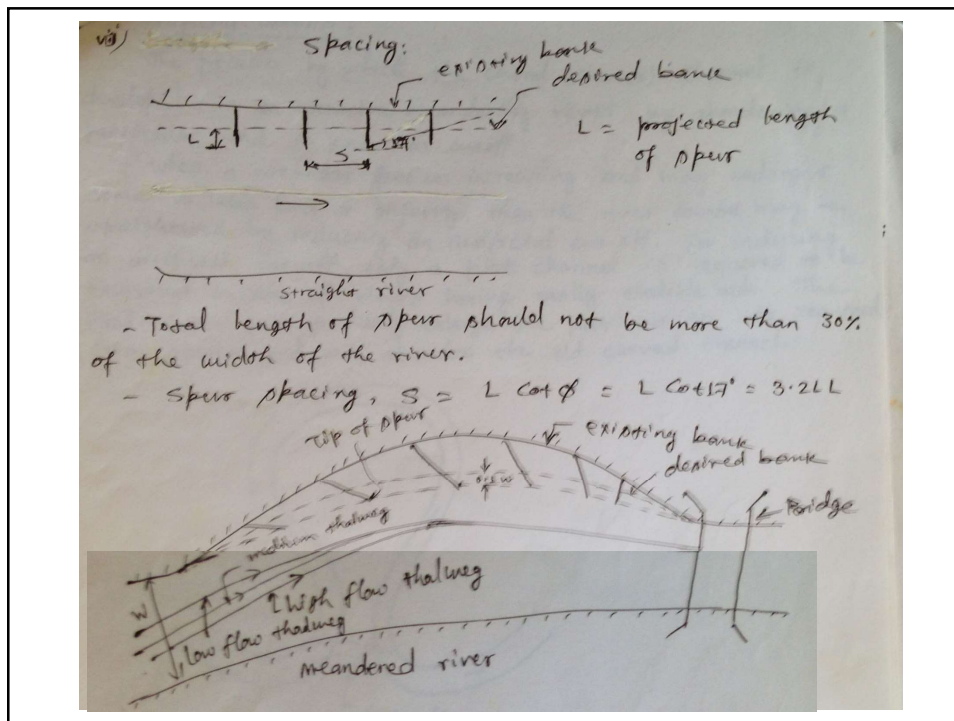
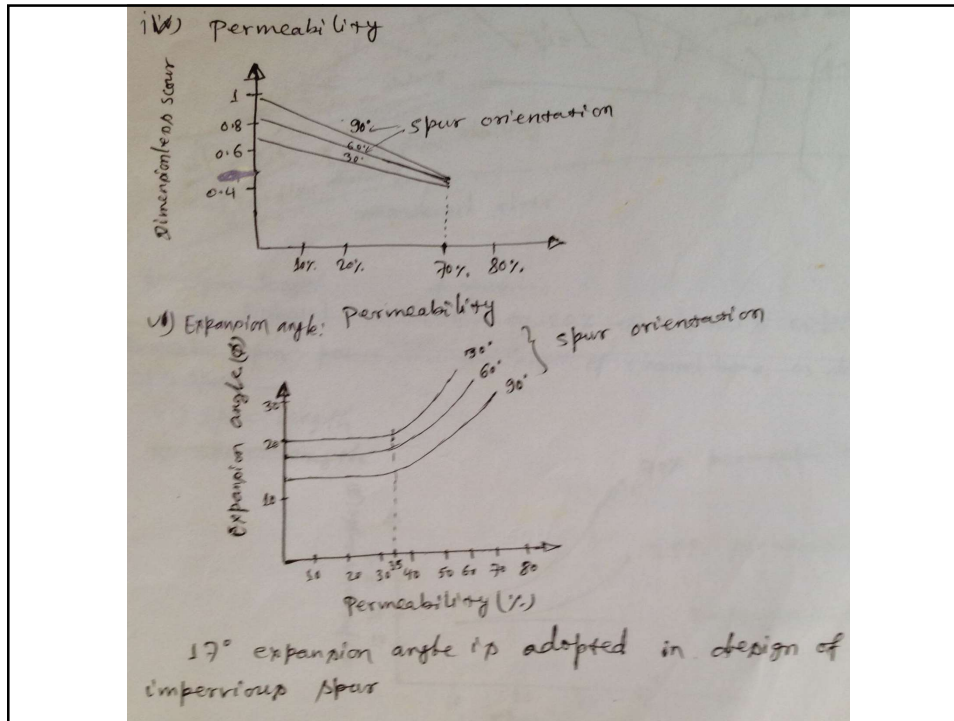
Design Considerations:

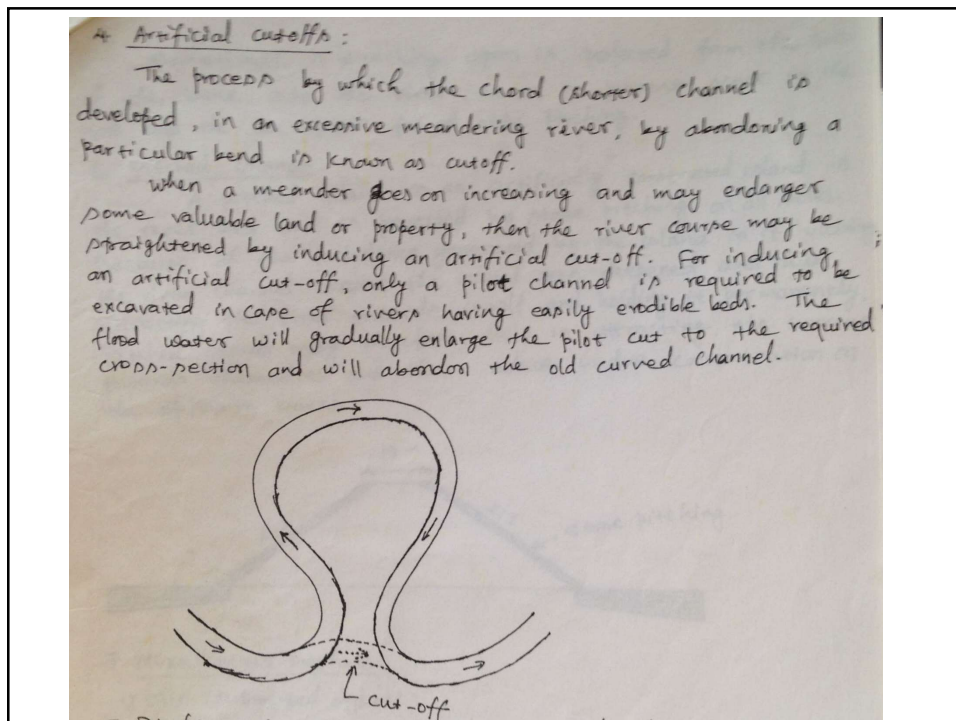
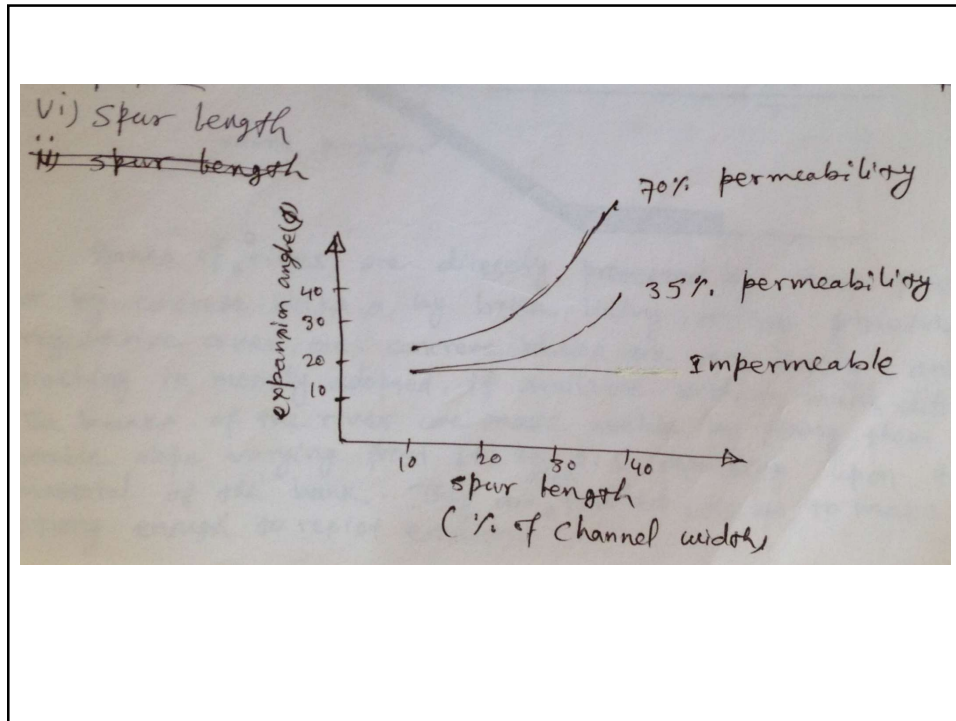
i) Identify longitudinal length of affected bank

ii) Spur adjustment at tip of spur  
 $\frac{y_s}{y_0} = F_s^{-0.33}$   
 $y_s = 4 y_0 F_s^{-0.33}$   
 $y_s = 4 y_0 Fr^{-0.33}$

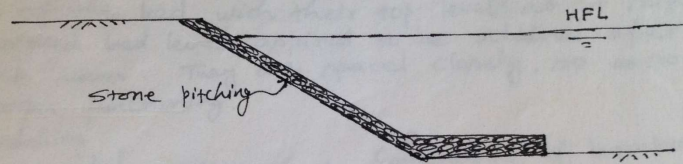
where,  
 $y_0$  = equilibrium scour depth from the normal bed level  
 $y_s$  = depth of water up of spur  
 $a$  = length of spur

iii) Permeability spur orientation  
 (Relative scour depth) / (Dimensionless scour)  
 $= \frac{\text{scour depth}}{\text{scour depth if spur is normal to the bank}}$





### 5. Pitching of banks and provision of launching aprons:

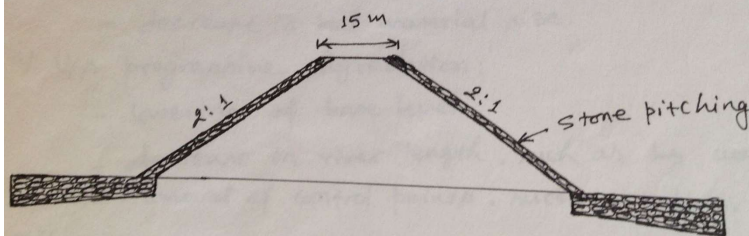


Banks of a river are directly protected by stone pitching, or by concrete blocks, or by brick lining, or by growing vegetative cover, etc. concrete blocks are very costly, and stone pitching is mostly adopted, if available without much difficulty. The banks of the river are made stable by giving them a stable slope varying from 1:1 to 2:1 depending upon the material of the bank. They are then pitched, so as to make them strong enough to resist erosion.

Sometimes a launching apron is projected from the toe of the bank into the river, so as to prevent scour at the toe and the consequent fall of slope pitching.

### 6. Pitched islands:

A pitched island is an artificially constructed island in the river bed and is protected by stone pitching on all sides. Because of the turbulence generated by the island in its vicinity, the river channel around the island gets deepened and thus, attracting the river towards itself and holding it permanently. Pitched islands may therefore, help in attracting the current towards themselves and thus, reduce undue concentration on the opposite banks.



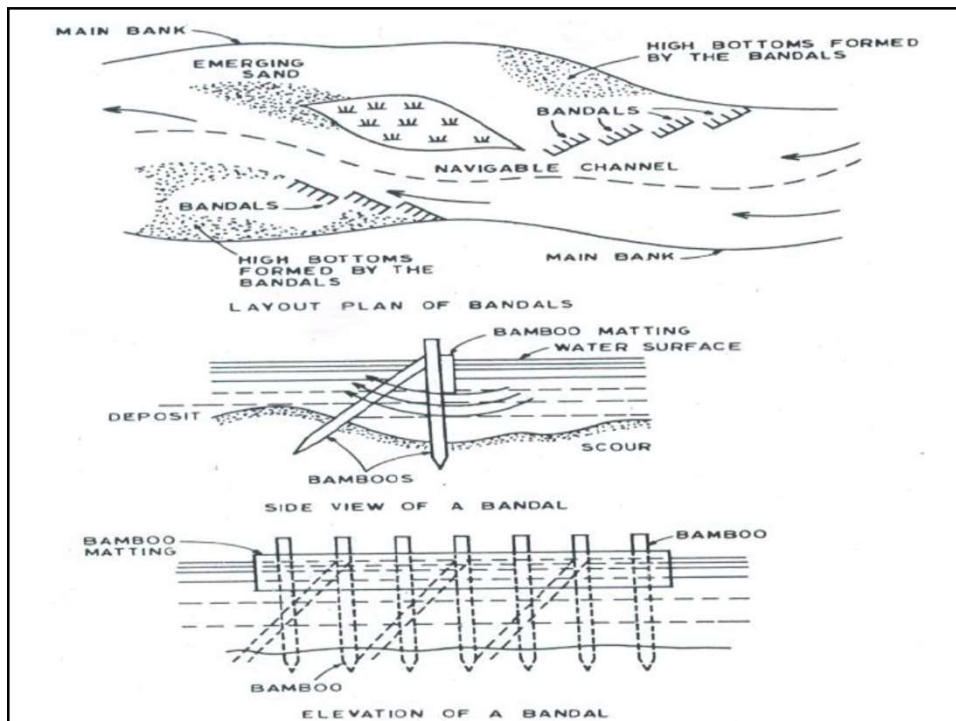
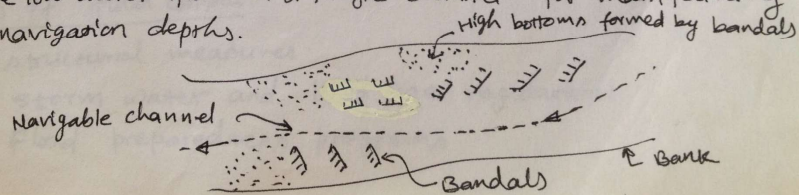
7. Mixellaneous methods:

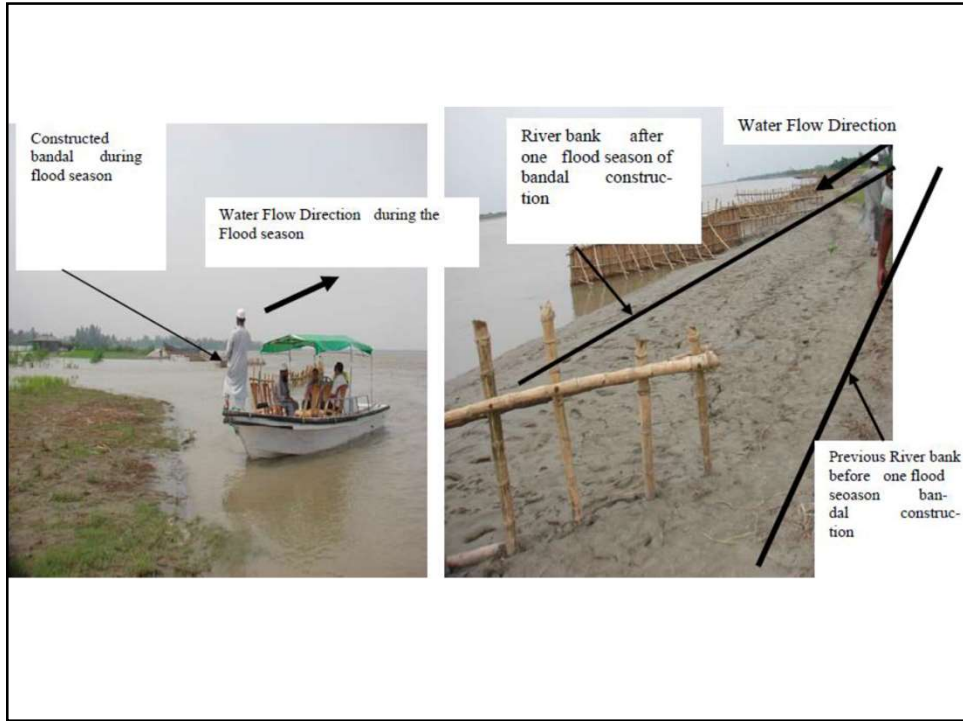
i) sills (submerged dykes):

Sometimes, a river may create deep channels in the vicinity of certain pucca structures and are required to be corrected. In such situations, sills are placed across the scoured portion of the bed, with their top levels at or slightly below the designed bed level aspired to be achieved after correcting the deep scours. They are spaced closely, so as to ensure their proper functioning.

ii) Bandalling:

A bandal consists of a framework of bamboo driven into river bed, set 0.6 m apart by means of horizontal ties supported by struts 1.25 m apart. Bandalling is designed to confine the low water flow in a single channel for maintaining required navigation depths.





# CROSS-DRAINAGE STRUCTURES

## CROSS DRAINAGE WORKS

- 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. The crossing of the canals with such obstacle cannot be avoided. So, suitable structures is 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**.

## CROOS DRAINAGE WORKS

- Irrigational Canals while carrying water have to cross few natural drainage streams, rivers, etc.. To cross those drainages safely by the canals, some suitable structures are required to construct. Works required to construct, to cross the drainage are called Cross Drainage Works (CDWs). At the meeting point of canals and drainages, bed levels may not be same. Depending on their bed levels, different structures are constructed and accordingly they are known by different names.

## NECESSITY OF CDWs

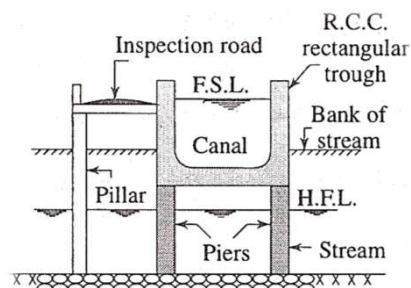
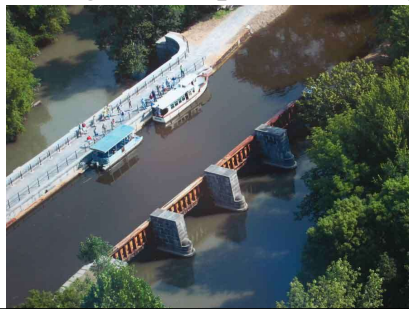
- The water-shed canals do not cross natural drainages. But in actual orientation of the canal network, this ideal condition may not be available and the obstacles like natural drainages may be present across the canal. So, the cross drainage works must be provided for running the irrigation system.
- At the crossing point, the water of the canal and the drainage get intermixed. So, far the smooth running of the canal with its design discharge the cross drainage works are required.
- The site condition of the crossing point may be such that without any suitable structure, the water of the canal and drainage can not be diverted to their natural directions. So, the cross drainage works must be provided to maintain their natural direction of flow.

## TYPES OF CDWs

- (1) Type I (Irrigation canal passes over the drainage)
  - (a) Aqueduct
  - (b) Siphon aqueduct
- (2) Type II (Drainage passes over the irrigation canal)
  - (a) Super passage
  - (b) Siphon or Canal siphon or Inverted siphon
- (3) Type III (Drainage and canal intersection each other of the same level)
  - (a) Level Crossing
  - (b) Inlet and outlet

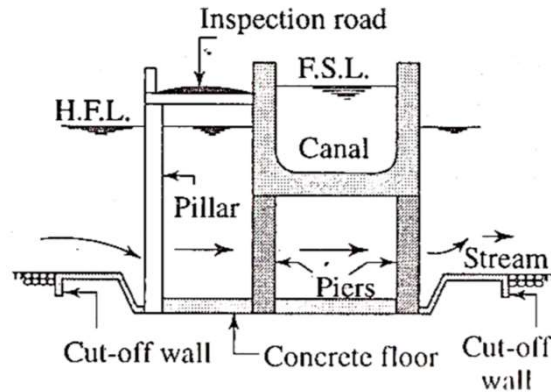
### • Aqueduct

The hydraulic structure in which the irrigation canal is taken over the drainage (such as river, stream etc..) is known as aqueduct. This structure is suitable when bed level of canal is above the highest flood level of drainage. In this case, the drainage water passes clearly below the canal.



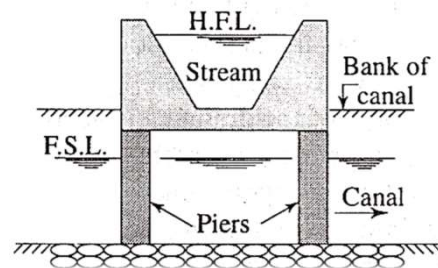
- **Siphon Aqueduct**

In a hydraulic structure where the canal is taken over the drainage, but the drainage water cannot pass clearly below the canal. It flows under siphonic action. So, it is known as siphon aqueduct. This structure is suitable when the bed level of canal is below the highest flood level.



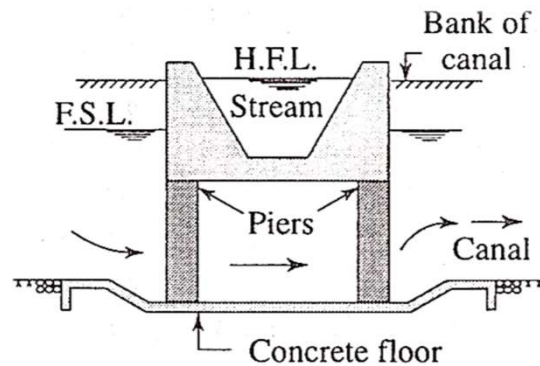
- **Super Passage**

The hydraulic structure in which the drainage is taken over the irrigation canal is known as super passage. The structure is suitable when the bed level of drainage is above the full supply level of the canal. The water of the canal passes clearly below the drainage.



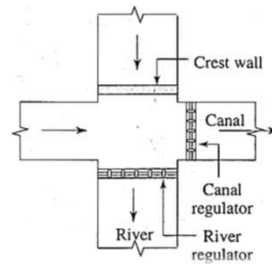
### • Canal Siphon

The hydraulic structure in which the drainage is taken over the irrigation canal, but the canal water passes below the drainage under siphonic action is known as canal siphon. This structure is suitable when the bed level of drainage is below the full supply level of the canal.



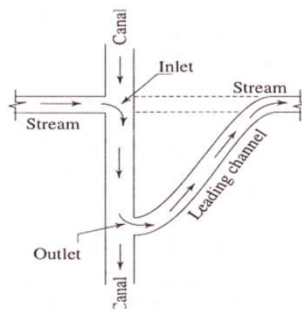
### • Level Crossings

When the bed 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. Level crossing consists of following components (i) crest wall (ii) Stream regulator (iii) Canal regulator.



### • Inlet and Outlet

When irrigation canal meets a small stream or drain at same level, drain is allowed to enter the canal as in inlet. At some distance from this inlet point, a part of water is allowed to drain as outlet which eventually meets the original stream. Stone pitching is required at the inlet and outlet. The bed and banks between inlet and outlet are also protected by stone pitching. This type of CDW is called Inlet and Outlet.



#### 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, 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 may be obtained by using the Lacey's equation, given by

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

where  $P$  = is the wetted 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 =  $0.8 P$ ).

*Size of the Barrels.* After having fixed the waterway, the size of the 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 openings is automatically fixed in aqueducts. However, in syphon-aqueducts, the required area of the waterway can be obtained by dividing the flood discharge by the permissible velocity through the barrels. This velocity through the barrels is generally limited to 2 to 3 m/sec. Knowing the area and then dividing it by the decided width of opening the height of opening can be fixed.

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 by

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

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

$$f_2 \cdot \frac{L}{R} \cdot \frac{V^2}{2g}; \text{ where}$$

$f_2$  is given by

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

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.

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

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 permitting 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

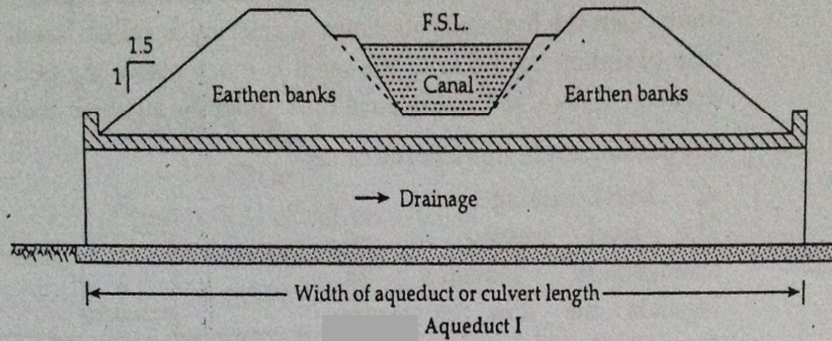
compromise obtained between the barrel area and afflux. Moreover, in order to reduce the afflux for the same velocity, the entry is made smooth by providing bell mouthed piers and surface friction is reduced by keeping the inside surface of the barrels as smooth as possible.

**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.

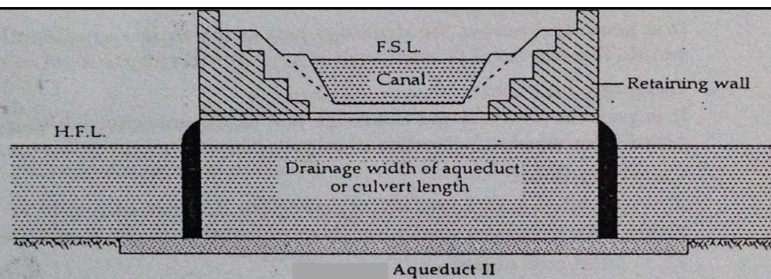
**Type I**

Sides of the aqueduct in earthen banks with complete earthen slopes. The length of culvert should be sufficient to accommodate both, water section of canal, as well as earthen banks of canal with aqueduct slope.



**Type II**

Sides of the aqueduct in earthen banks, with other slopes supported by masonry wall. In this case, canal continues in its earthen section over the drainage but the outer slopes of the canal banks are replaced by retaining wall, reducing the length of drainage culvert.



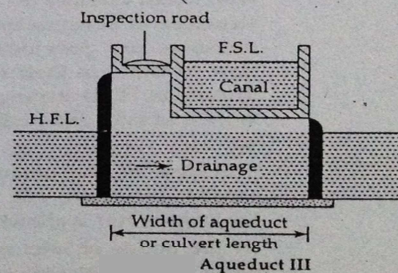
**Type III**

Sides of the aqueduct made of concrete or masonry. Its earthen section of the canal is discontinued and canal water is carried in masonry or concrete trough, canal is generally flumed in this section.

**Suitability**

Type I aqueduct or siphon will be economical only when length of aqueduct is small where cost of bank connections would be large in comparison to the savings obtained from the reduction in width of aqueduct.

In type III the width of the aqueduct is minimum but the cost of bank connections is maximum. This type is, therefore, suitable where the length of aqueduct is very large and where the cost of bank connection would be small in comparison to the saving obtained from the reduction in width of the aqueduct.



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

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{L_f B_n - (B_n - B_f) x} \quad \dots(14.3)$$

where  $B_n$  = Bed width of the normal channel section.

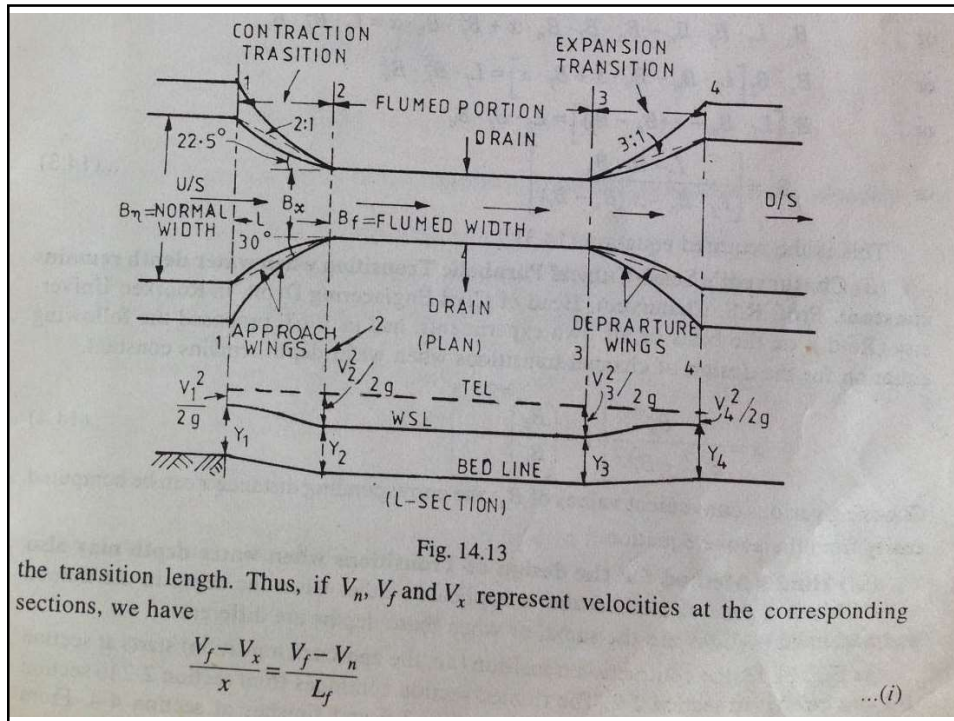
$B_f$  = Bed width of the flumed channel section.

$B_x$  = 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



the transition length. Thus, if  $V_n, V_f$  and  $V_x$  represent velocities at the corresponding sections, we have

$$\frac{V_f - V_x}{x} = \frac{V_f - V_n}{L_f} \dots (i)$$

Now, since depth  $y$  is assumed to be constant and the total discharge  $Q$  is also constant, we have

Velocity  $\times$  Area = Discharge

$$\therefore V_f \cdot B_f \cdot y = V_x \cdot B_x \cdot y = V_n \cdot B_n \cdot y = Q \quad (\text{assuming rectangular section throughout}).$$

$$\therefore V_f \cdot B_f = V_x \cdot B_x = V_n \cdot B_n$$

$$= \frac{Q}{y} = \text{constant} = K \text{ (say)}$$

or  $V_f = \frac{K}{B_f}$

$$V_x = \frac{K}{B_x}$$

$$V_n = \frac{K}{B_n}$$

Substituting these values in equation (i) we get

$$\left[ \frac{\frac{K}{B_f} - \frac{K}{B_x}}{x} \right] = \left[ \frac{\frac{K}{B_f} - \frac{K}{B_n}}{L_f} \right]$$

or  $\frac{B_x - B_f}{B_f \cdot B_x \cdot x} = \frac{B_n - B_f}{L_f \cdot B_f \cdot B_n}$

or  $B_x \cdot L_f \cdot B_f \cdot B_n - L_f \cdot B_f^2 \cdot B_n = B_n \cdot B_f \cdot B_x \cdot x - B_f^2 \cdot B_x \cdot x$

or  $B_x \cdot L_f \cdot B_f \cdot B_n - B_n \cdot B_f \cdot B_n \cdot x + B_f^2 \cdot B_x \cdot x = L_f \cdot B_f^2 \cdot B_n$

$$\begin{aligned}
 \text{or} \quad & B_x \cdot B_f [L_f \cdot B_n - B_n \cdot x + B_f \cdot x] = L_f \cdot B_f^2 \cdot B_n \\
 \text{or} \quad & B_x [L_f \cdot B_n - x (B_n - B_f)] = L_f \cdot B_f \cdot B_n \\
 \text{or} \quad & B_x = \left[ \frac{L_f \cdot B_f \cdot B_n}{L_f \cdot B_n - x (B_n - B_f)} \right] \quad \dots(14.3)
 \end{aligned}$$

This is the required equation (14.3).

(ii) **Chaturvedi's Semi-Cubical Parabolic Transition when water depth remains constant.** Prof. R.S. Chaturvedi, Head of Civil Engineering Deptt. in Roorkee University (Retd.), on the basis of his own experiments, had in 1963, proposed the following equation for the design of channel transitions when water depth remains constant.

$$x = \frac{L \cdot B_n^{3/2}}{B_n^{3/2} - B_f^{3/2}} \left[ 1 - \left( \frac{B_f}{B_x} \right)^{3/2} \right] \quad \dots(14.4)$$

Choosing various convenient values of  $B_x$ ; the corresponding distance  $x$  can be computed easily from the above equation.

(iii) **Hind's Method for the design of Transitions when water depth may also vary.** This is a general method and is applicable either when the depth in the flumed and unflumed portions are the same, or when these depths are different.

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 4-4 = Bed level at section 4-4 +  $y_4$  = (known)

$\therefore$  TEL at section 4-4 = FSL at section 4-4 +  $\frac{V_4^2}{2g}$  = (known)

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

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

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  $\frac{V_3^2}{2g}$  from TEL. Similarly, bed level at 3-3 can also be computed by subtracting  $y_3$  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 ( $H_f$ ) is the friction loss which can be computed with 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 subtracting  $y_2$  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

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

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.

**Water surface in Transition :** In the contraction transition between section 1-1 to 2-2, there will be a drop in water surface due to the drop in energy line and also due to the increased velocity head at 2-2. This drop in water surface has to be negotiated by a smooth curve tangential at both ends. This can be easily accomplished by using two parabolic curves meeting tangentially at the centre of the transition, as shown in Fig. 14.14.

Let  $2X_1 = L =$  The length in which fluming has been done.

Fig. 14.14. Water Surface profile for Transition Contraction.

$2Y_1 =$  Total difference in water levels between section 1-1 and 2-2.

The distance of the middle point of transition will be  $X_1$  and drop in water surface will be  $Y_1$ . The equation of the first parabolic curve, with origin at water surface of section 1-1 (i.e.  $O_1$ ) is given by

$$Y = C \cdot X^2$$

when

$$Y = Y_1, \text{ and } X = X_1$$

$$C = \frac{Y_1}{X_1^2}$$

Therefore, the equation of parabola becomes

$$Y = \left[ \frac{Y_1}{X_1^2} \right] X^2 \quad \dots(14.5)$$

Using the above equation, the first parabolic curve can be easily plotted. Similarly, the second parabolic curve can be plotted by taking the origin at  $O_2$  on section 2-2.

The water surface in the expansion transition between sections 3-3 and 4-4 can also be plotted in a similar fashion, where there will be a rise in the water surface from section 3-3 to 4-4, as shown in Fig. 14.13.

After having plotted the water surface profile over the entire length, the velocity head say ( $h_v$ ) can be found by measuring the vertical distance between TEL and water surface line at any point. The velocity head can then be converted into equivalent velocity by using  $V = \sqrt{2g \cdot h_v}$ . Hence, the velocity at each point can be known. The

CROSS DRAINAGE  
cross-sectional area required to pass the given discharge at each point can be found by dividing discharge by velocity at that point (i.e.  $A = \frac{Q}{V}$ ).

In trapezoidal channel of water depth  $y$ , the bed width  $B$ , and side slopes  $s : 1$ ; area is given by

$$A = BD + s \cdot y^2. \quad \dots(14.6)$$

In flared wings, the side slopes are generally brought to vertical from an initial slope of  $s : 1$  and, therefore, the side slopes at any point can be interpolated in proportion to the length of transition undergone. Thus at any point  $A$ ,  $y$  and  $s$  are known and hence the value of  $B$  can be worked out at this pt. by using the equation (14.6). The width of the canal at various points in the transition can thus be determined. Hence, all the dimensions of the transition are fully found out.

**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.

**Drainage**

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

**Solution.** Since the drainage is of a large size, work of type III will be adopted. Further, because the canal bed level (212.0 m) is much above the H.F.L. of drainage (i.e. 210.0 m) an **aqueduct** will be constructed. The earthen banks of the canal will be discontinued and the canal water taken in a concrete trough. For effecting economy, the canal shall be flumed.

**Step 1. Design of Drainage Waterway**

$$\text{Lacey's regime perimeter} = P = 4.75 \sqrt{Q}$$

where  $Q$  = High flood discharge of drain  
= 300 cumecs (given)

$$P = 4.75 \cdot \sqrt{300} = 82.3 \text{ m.}$$

Let the clear span between piers be 9 m and the pier thickness be 1.5 m.

Using 8 bays of 9 m each, clear waterway =  $8 \times 9 = 72$  m.

Using 7 piers of 1.5 each, length occupied by piers =  $7 \times 1.5 = 10.5$  m.

Total length of waterway =  $72 + 10.5 = 82.5$  m

**Step 2. Design of Canal Waterway**

Bed width of canal = 20.0 m.

Let the width be flumed to 10.0 m.

Providing a splay of 2 : 1 in contraction, the length of contraction transition

$$= \frac{20 - 10}{2} \times 2 = 10.0 \text{ m}$$

Providing a splay of 3 : 1 in expansion, the length of expansion transition

$$= \frac{20 - 10}{2} \times 3 = 15 \text{ m}$$

Length of the flumed rectangular portion of the canal between abutments = 82.5 m (provided).

In transitions, the side slopes of the canal section will be warped in plan from the original slope of  $1\frac{1}{2} : 1$  to vertical.

**Step 3. Head loss and bed levels at different sections.** (Fig. 14.20).

**At Section 4-4**

At section 4-4, where the canal returns to its normal section, we have

Area of trapezoidal canal section

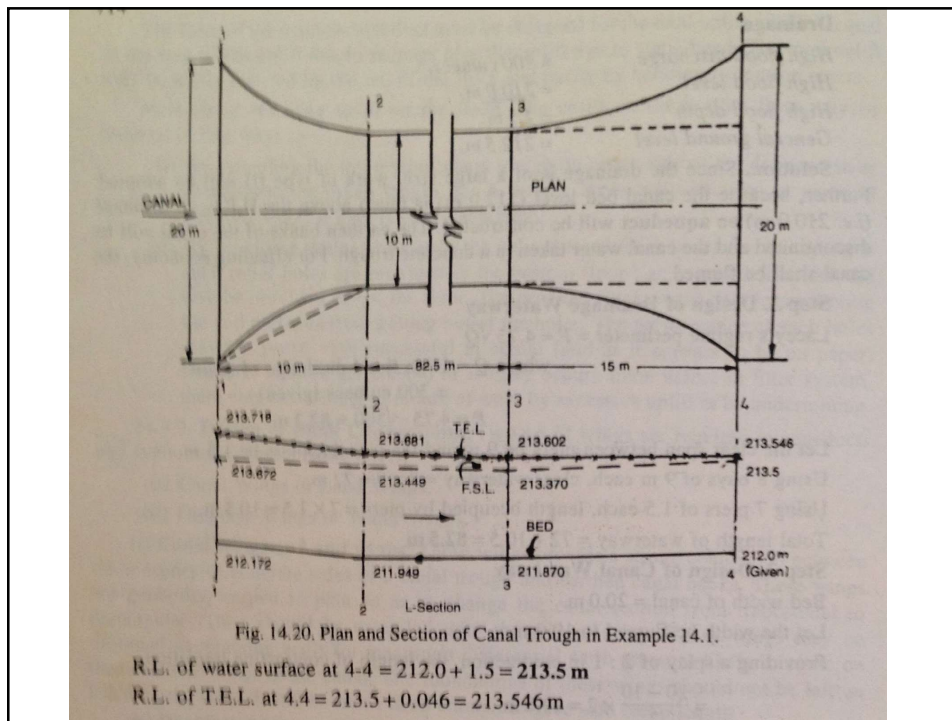
$$= (B + 1.5y)y$$

$$= (20 + 1.5 \times 1.5) 1.5 = 22.5 \times 1.5 = 33.75 \text{ m}^2$$

$$\text{Velocity} = V_4 = \left( \frac{Q}{A} \right) = \frac{32}{33.75} = 0.947 \text{ m/sec}$$

$$\text{Velocity head} = \frac{V_4^2}{2g} = \frac{(0.947)^2}{2 \times 9.81} = 0.046 \text{ m}$$

R.L. of bed at 4-4 = **212.0 m** (given)



The known condition of 4-4 shall now be utilised for finding the bed levels etc. at 3.3.

### At Section 3-3

Keeping the same depth of 1.5 m throughout the channel, we have at section 3.3 :

Bed width = 10 m

Area of channel =  $10 \times 1.5 = 15 \text{ sq m}$

$$\text{Velocity} = V_3 = \frac{32}{15} = 2.13 \text{ m/sec}$$

$$\text{Velocity head} = \frac{V_3^2}{2g} = \frac{(2.13)^2}{2 \times 9.81} = 0.232 \text{ m}$$

Assuming that the loss of head in expansion from section 3-3 to section 4-4 is taken as

$$\begin{aligned} &= 0.3 \left[ \frac{V_3^2 - V_4^2}{2g} \right] \\ &= 0.3 [0.232 - 0.046] \\ &= 0.3 \times 0.186 = 0.0558 \text{ m ; say } \mathbf{0.056 \text{ m}} \end{aligned}$$

$$\begin{aligned} \text{R.L. of T.E.L. at section 3-3} &= \text{R.L. of T.E.L. at 4-4} + \text{Loss in expansion} \\ &= 213.546 + 0.056 = 213.602 \text{ m} \end{aligned}$$

$$\begin{aligned} \therefore \text{R.L. of water surface at 3-3} &= \text{R.L. of T.E.L. at 3-3} - \text{Velocity Head} \\ &= 213.602 - 0.232 = \mathbf{213.370 \text{ m}} \end{aligned}$$

$$\begin{aligned} \text{R.L. of bed at 3.3} \\ &= 213.370 - 1.5 = \mathbf{211.87 \text{ m}} \end{aligned}$$

### At Section 2-2

From section 2-2 to 3-3, the trough section is constant. Therefore, area and velocity at 2-2 are the same as at 3-3. But from 2-2 to 3-3, there is a friction loss between 2-2 and 3-3 which may be computed by Manning's formula as equal to

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

where  $n$  is rugosity coefficient whose value in concrete trough may be taken as 0.016; and  $L$  is the length of trough = 82.5 m.

Area of trough section ( $A$ ) =  $10 \times 1.5 = 15 \text{ sq m}$

Wetted perimeter ( $P$ ) =  $10 + 2 \times 1.5 = 13 \text{ m}$

Hydraulic mean depth ( $R$ ) =  $\frac{A}{P} = \frac{15}{13} = 1.16 \text{ m}$

Velocity in trough =  $\frac{Q}{A} = \frac{32}{15} = 2.13 \text{ m/sec}$

$$\therefore H_L = \frac{(0.016)^2 \times (2.13)^2 \times 82.5}{(1.16)^{4/3}}$$

$$= 0.0787 \text{ m; say } \mathbf{0.079 \text{ m}}$$

$$\text{R.L. of T.E.L. at 2-2} = \text{R.L. of T.E.L. at 3-3} + \text{Friction loss in trough}$$

$$= 213.602 + 0.079 = 213.681 \text{ m}$$

$$\text{R.L. of water surface at 2-2}$$

$$= 213.681 - 0.232 = \mathbf{213.449 \text{ m}}$$

$$\text{R.L. of bed at 2-2}$$

$$= 213.449 - 1.5 = \mathbf{211.949 \text{ m}}$$

#### At Section 1-1

Loss of head in contraction transition from 1-1 to 2-2

$$= 0.2 \left( \frac{V_2^2 - V_1^2}{2g} \right)$$

$$= 0.2 \left[ \frac{(2.13)^2 - (0.947)^2}{2 \times 9.81} \right]$$

$$= 0.2 [0.232 - 0.046] = \mathbf{0.037 \text{ m}}$$

$$\text{R.L. of T.E.L. at 1-1} = \text{R.L. of T.E.L. at 2-2} + \text{Loss in contraction}$$

$$= 213.681 + 0.037 = \mathbf{213.718 \text{ m}}$$

$$\text{R.L. of water surface at 1-1}$$

$$= 213.718 - 0.046 = \mathbf{213.672 \text{ m}}$$

$$\text{R.L. of bed at 1-1}$$

$$= 213.672 - 1.5 = \mathbf{212.172 \text{ m}}$$

All the bed levels, F.S.L. and T.E.L. are plotted in Fig. 14.20.

#### Step 4. Design of Transitions

(a) *Contraction Transition.* Since the depth is kept constant, the transition can be designed on the basis of Mitra's Hyperbolic transition equation (14.2) given as :

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{L_f B_n - x (B_n - B_f)}$$

$$\text{where } B_f = 10 \text{ m}$$

$$B_n = 20 \text{ m}$$

$$L_f = 10 \text{ m}$$

Substituting we get

$$B_x = \frac{20 \times 10 \times 10}{10 \times 20 - x (20 - 10)} = \frac{2,000}{200 - 10x}$$

For various values of  $x$  lying between 0 to 10 m, various values of  $B_x$  are worked out, as shown below in Table 14.2. The distance  $x$  is measured from flumed section *i.e.* 2-2, as shown in Fig. 14.21.

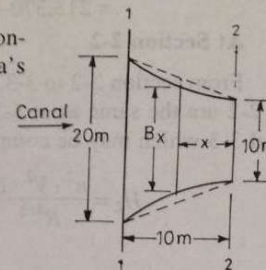
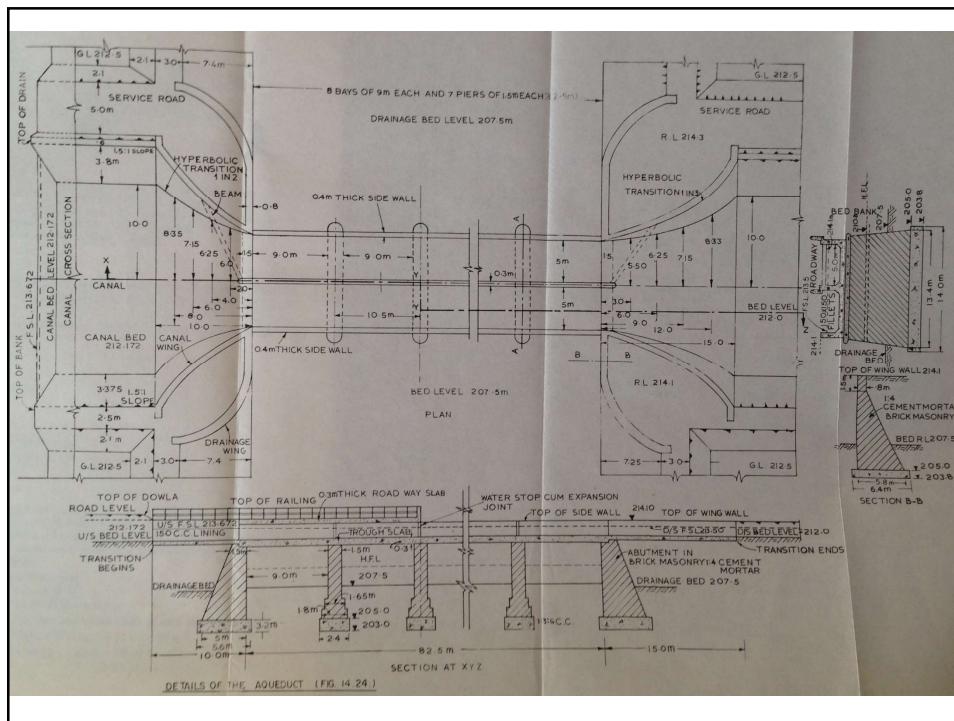
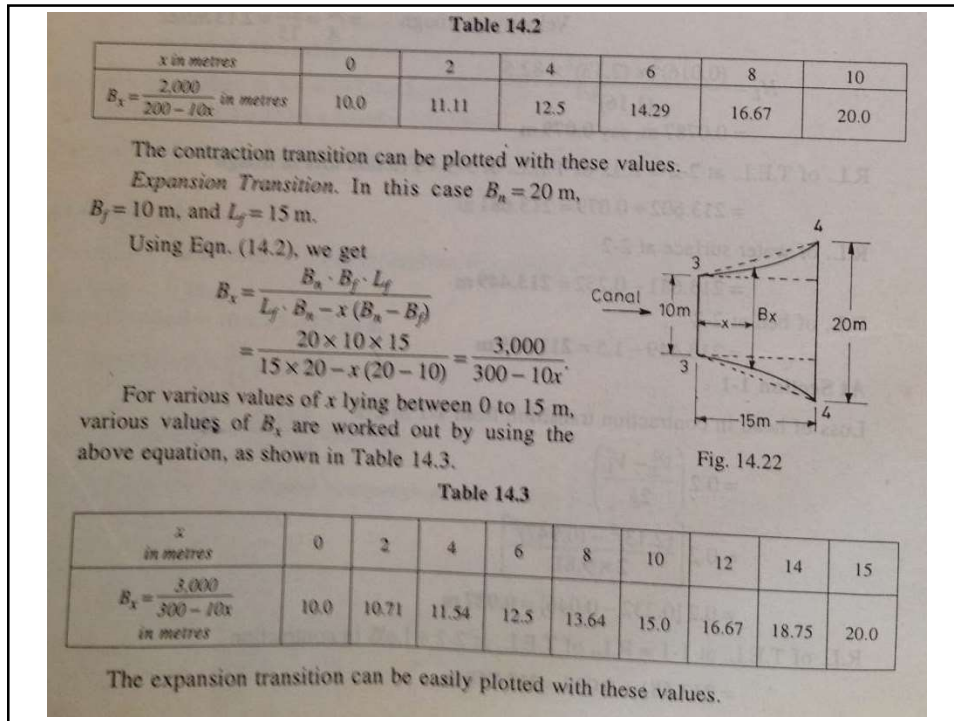


Fig. 14.21



**Example 14.2.** Design a syphon aqueduct if the following data at the crossing of a canal and a drainage are given :

- |                                       |                            |
|---------------------------------------|----------------------------|
| (i) Discharge of canal                | = 40 cumecs.               |
| (ii) Bed width of canal               | = 30 m.                    |
| (iii) Full supply depth of canal      | = 1.6 m.                   |
| (iv) Bed level of canal               | = 206.4 m.                 |
| (v) Side slopes of canal              | = $1\frac{1}{2} H : 1 V$ . |
| (vi) High flood discharge of drainage | = 450 cumecs               |
| (vii) High flood level of drainage    | = 207.0 m.                 |
| (viii) Bed level of drainage          | = 204.5 m.                 |
| (ix) General ground level             | = 206.5 m.                 |

**Solution.** Since the drainage is of a large size, work of type III will be adopted. Further, because the canal bed level (206.4 m) is slightly below the drainage HFL (207.0 m) ; a **syphon aqueduct** is required and is also asked for. The earthen banks of the canal will be discontinued and the canal water taken in a concrete trough. For affecting economy, the canal shall be flumed.

**Step 1. Design of Drainage Waterway**

$$\text{Lacey's regime perimeter} = P = 4.75 \sqrt{Q}$$

$$= 4.75 \sqrt{450} = 100.8 \text{ m}$$

Provide 11 clear spans of 8 m each and let the width of each pier be 1.5 m.

The length occupied by 11 bays of 8 m each =  $11 \times 8 = 88 \text{ m}$

The length occupied by 10 piers of 1.5 each =  $10 \times 1.5 = 15 \text{ m}$

Total length of waterway =  $88 + 15 = 103 \text{ m}$ .

Let us, now, limit the velocity through syphon-barrels, to a value, say 2 m/sec.

Height of barrels required

$$= \frac{\text{Discharge}}{\text{Velocity} \times \text{clear width of waterway}} = \frac{450}{2 \times 88} \text{ m} = 2.56 \text{ m}.$$

Hence, provide 11 rectangular barrels, each 8 m wide and 2.5 m high.

$$\text{Actual velocity through barrels} = \frac{450}{11 \times 8 \times 2.5} = 2.05 \text{ m/sec.}$$

**Step 2. Design of Canal Waterway**

Normal bed width of canal = 30 m

Let the width be reduced to 15 m.

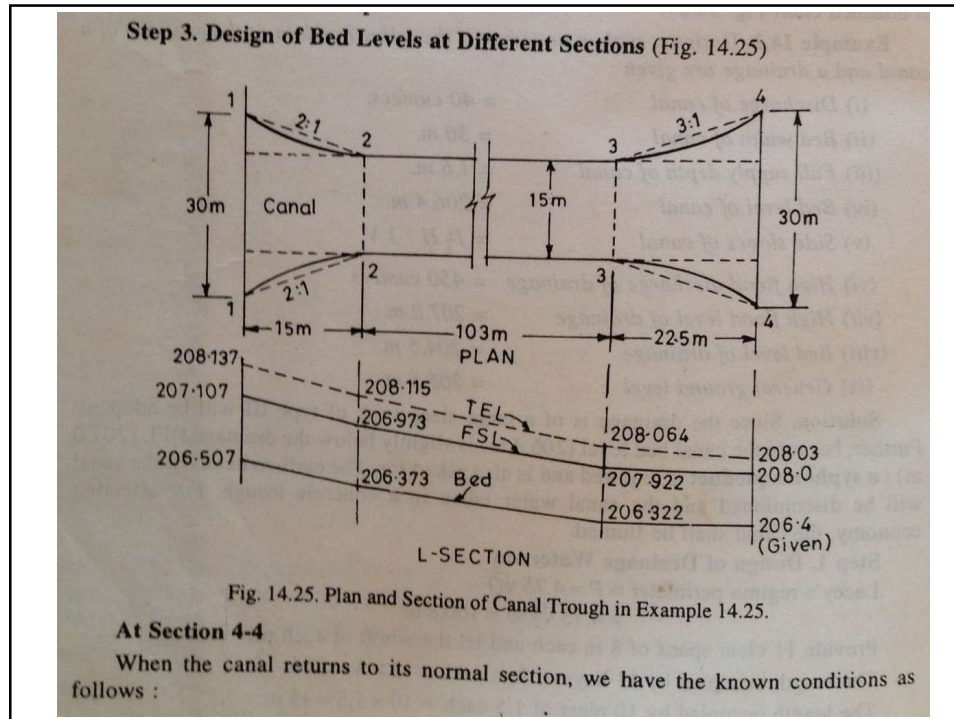
Providing a splay of 2 : 1 in contraction, the length of contraction transition

$$= \frac{30 - 15}{2} \times 2 = 15 \text{ m}.$$

Providing a splay of 3 : 1 in expansion, the length of expansion transition

$$= \frac{30 - 15}{2} \times 3 = 22.5 \text{ m}.$$

Length of flumed rectangular portion of the canal between abutments = 103 m (provided). In transitions, the side slopes of the canal section shall be warped in plan from the original slope of  $1\frac{1}{2} H : 1 V$  to vertical.



Area of trapezoidal canal section

$$= (B + 1.5y) y$$

where  $B = \text{Bed width} = 30 \text{ m}$

$y = \text{Depth} = 1.6 \text{ m}$

$$= [30 + 1.5 \times 1.6] 1.6$$

$$= 32.4 \times 1.6 = 51.84 \text{ sq. m.}$$

$$\text{Velocity of flow} = V_4 = \frac{Q}{A} = \frac{40}{51.84} = 0.77 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V_4^2}{2g} = \frac{(0.77)^2}{2 \times 9.81} = 0.030 \text{ m}$$

R.L. of canal bed at 4-4 = **206.4 m** (given)

Water depth = 1.6 m (given)

R.L. of water surface at 4-4 =  $206.4 + 1.6 = \mathbf{208.0 \text{ m}}$

R.L. of T.E.L. at 4-4 =  $208.0 + 0.03 = 208.03 \text{ m.}$

**At Section 3-3**

Assuming a constant depth of 1.6 m throughout the channel, we have at section 3-3, a rectangular channel, as follows :

Bed width = 15 m

Depth = 1.6 m (assumed constant)

Area =  $15 \times 1.6 = 24 \text{ sq. m.}$

Velocity =  $V_3 = \frac{40}{24} = 1.67 \text{ m/sec.}$

$$\text{Velocity head} = \frac{V_3^2}{2g} = \frac{(1.67)^2}{2 \times 9.81} = 0.142 \text{ m}$$

Assuming that the loss of head in expansion from section 3-3 to section 4-4 is taken as

$$= 0.3 \left[ \frac{V_3^2 - V_4^2}{2g} \right]$$

$$= 0.3 [0.142 - 0.030] = 0.3 \times 0.112 = 0.0336 \text{ m; say } \mathbf{0.034 \text{ m.}}$$

$$\begin{aligned} \text{R.L. of T.E.L. at 3-3} &= \text{R.L. of T.E.L. at 4-4} + \text{Loss in expansion} \\ &= 208.030 + 0.034 = 208.064 \text{ m.} \end{aligned}$$

$$\begin{aligned} \text{R.L. of water surface at 3-3} \\ &= 208.064 - 0.142 = \mathbf{207.922 \text{ m.}} \end{aligned}$$

$$\text{R.L. of bed at 3-3} = 207.922 - 1.6 = \mathbf{206.322 \text{ m.}}$$

#### At Section 2-2

From section 2-2 to section 3-3, the trough section is constant. Therefore, the area and velocity at 2-2 are the same as at 3-3. There is a friction loss between 2-2 and 3-3, which may be computed by Manning's formula, as equal to

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

where  $n$  is rugosity coefficient, whose value in a concrete trough may be taken as 0.016 and  $L$  is the length of channel = 103 m.

$$\begin{aligned} \text{Area of trough section (A)} &= 15 \times 1.6 = 24 \text{ sq. m} \\ \text{Wetted perimeter} &= 15 + 2 \times 1.6 = 18.2 \text{ m} \end{aligned}$$

$$\text{Hydraulic mean depth} = R = \frac{A}{P} = \frac{24}{18.2} = 1.32 \text{ m}$$

$$\text{Velocity in trough} = \frac{Q}{A} = \frac{40}{24} = 1.67 \text{ m/sec.}$$

$$\text{Head loss, } H_L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}} = \frac{(0.016)^2 \times (1.67)^2 \times 103}{(1.32)^{4/3}} = \mathbf{0.051 \text{ m.}}$$

$$\begin{aligned} \text{R.L. of T.E.L. at 2-2} &= \text{R.L. of T.E.L. at 3-3} + \text{Head loss in trough} \\ &= 208.064 + 0.051 = 208.115 \text{ m.} \end{aligned}$$

$$\text{R.L. of water level at 2-2} = 208.115 - 0.142 = \mathbf{207.973 \text{ m.}}$$

$$\text{R.L. of bed at 2-2} = 207.973 - 1.6 = \mathbf{206.373 \text{ m.}}$$

#### At Section 1.1

Loss of head in contraction transition from section 1-1 to section 2-2 may be taken as

$$= 0.2 \left[ \frac{V_2^2 - V_1^2}{2g} \right] = 0.2 \left[ \frac{(1.67)^2 - (0.77)^2}{2 \times 9.81} \right] = 0.0224 \text{ m ; say } \mathbf{0.022 \text{ m.}}$$

R.L. of T.E.L. at 1-1 = R.L. of T.E.L. at 2-2 + Loss in contraction  
 = 208.115 + 0.022 = 208.137 m

R.L. of water surface at 1-1  
 = 208.137 - 0.030 = **208.107 m.**

R.L. of bed at 1-1 required to maintain constant depth  
 = 208.107 - 1.6 = **206.507 m.**

All these levels are plotted and shown in Fig. 14.25.

#### Step 4. Design of Transitions

(a) *Contraction Transition.* Since depth is kept constant, the transition shall be designed on the basis of Mitra's Hyperbolic transition equation, (14.2) given by

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{B_n \cdot L_f - x(B_n - B_f)}$$

where  $B_f = 15$  m

$B_n = 30$  m

$L_f = 15$  m.

Substituting, we get

$$B_x = \frac{30 \times 15 \times 15}{30 \times 15 - x(30 - 15)} = \frac{6750}{450 - 15x} = \frac{450}{30 - x}$$

For various values of  $x$  lying between 0 to 15 m, various values of  $B_x$  are worked out, as shown in Table 14.4. The distance  $x$  is measured from the flumed section 2-2.

Table 14.4

$x$ in metres	0	2	4	6	8	10	12	14	15
$B_x = \frac{450}{30 - x}$ in metres	15.0	16.04	17.27	18.72	20.42	22.5	25.0	28.1	30.0

*Expansion Transition.* In this case, we have

$B_n = 30$  m

$B_f = 15$  m

$L_f = 22.5$  m

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{B_n \cdot L_f - x(B_n - B_f)} \quad \text{i.e. Eq. (14.2)}$$

$$= \frac{30 \times 15 \times 22.5}{30 \times 22.5 - x(30 - 15)} = \frac{675}{45 - x}$$

For various values of  $x$  lying between 0 to 22.5 m, corresponding values of  $B_x$  are worked out, as shown in Table 14.5. The distance  $x$  is measured from the flumed section 3-3.

Table 14.5

$x$ in metres	0	2	4	6	8	10	12	14	16	18	20	22.5
$B_x = \frac{675}{45-x}$ in metres	15.0	15.7	16.46	17.3	18.25	19.3	20.4	21.75	23.3	25.0	27.0	30.0

**Step 5. Design of Trough**

The trough shall be divided into three equal compartments, each 5 m wide, separated by 0.3 m thick partition walls (2 Nos.). The inspection road (5 m wide) shall be carried on the extreme left compartment, as shown in Fig. 14.28. A free-board of 0.6 m above the normal water depth of 1.6 m is sufficient, and hence, the bottom level of bridge slab may be kept at  $1.6 + 0.6 = 2.2$  m above the bed level of the trough. The height of the trough will also be kept as equal to 2.2 m. The entire trough section can be designed as monolithic reinforced concrete structure by the usual structural methods. The tentative thicknesses may be used as follows :

Outer walls = 0.4 m thick

Bottom slab of trough = 0.4 m thick.

The intermediate walls shall be extended into transitions, so as to provide the necessary clear width of 15 m.

Now, the overall outer width of trough (including walls)

$$= 15 + 2 \times 0.3 + 2 \times 0.4$$

$$= 15 + 0.6 + 0.8 = \mathbf{16.4 \text{ m.}}$$

Hence, the length of syphon barrel = 16.4 m

**Step 6. Head Loss Through the Syphon Barrels**

The head loss through the syphon barrels is given by Unwin's formula as equal to (neglecting vel. of approach)

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

where  $V$  = velocity through barrels = 2.05 m/sec

$f_1$  = coefficient of loss of head at entry,

= 0.505 for unshaped mouth.

$f_2 = a \left( 1 + \frac{b}{R} \right)$  where the values of  $a$  and  $b$  are taken from table 14.1 for cement plastered barrels as

$$a = 0.00316$$

$$b = 0.030$$

$R$  = Hydraulic mean depth for barrel.

$$= \frac{A}{P} = \frac{8 \times 2.5}{2(8 + 2.5)} = \frac{20}{21} = 0.953$$

$L$  = Length of barrel = 16.4 m.

Substituting these values, we get

$$f_2 = 0.00316 \left[ 1 + \frac{0.030}{0.953} \right] = \mathbf{0.00326}$$

$$\therefore h = \left[ 1 + 0.505 + 0.00326 \left( \frac{16.4}{0.953} \right) \right] \frac{(2.05)^2}{2 \times 9.81} = \mathbf{0.333 \text{ m}}$$

High flood level of Drainage is given = 207.0 m

∴ d/s H.F.L. = 207.0 m

Afflux ( $h$ ) = 0.333 m

u/s H.F.L. = d/s H.F.L. + Afflux (or loss of head)

$$= 207.0 + 0.333 = 207.333 \text{ m.}$$

### Step 7. Uplift Pressure on Roof of barrels

R.L. of bottom of trough = R.L. of canal bed – Slab thickness

$$= 206.4 - 0.4 = 206.0 \text{ m}$$

$$\text{Loss of head at entry of barrel} = 0.505 \frac{V^2}{2g} = 0.505 \times \frac{(2.05)^2}{2 \times 9.81} = 0.108 \text{ m.}$$

Uplift on the roof

= u/s H.F.L. – Loss at entry – Level of underside of roof slab

$$= 207.333 - 0.108 - 206.0$$

$$= 1.225 \text{ m of water} = 12.25 \text{ kN/m}^2 \text{ (1.225 t/m}^2\text{)}$$

(Assuming unit wt. of water =  $10 \text{ kN/m}^3$  or  $1 \text{ t/m}^3$ )

The concrete trough slab is 0.4 m thick and will thus exert a downward load of

$$0.4 \times 24 = 0.96 \text{ kN/m}^2$$

(assuming unit wt. of concrete =  $24 \text{ kN/m}^3$ )

\* Strictly speaking, unit wt. of water =  $9.81 \text{ kN/m}^3$ ; but to ease in calculations we have taken unit wt. of water =  $10 \text{ kN/m}^3$  ( $1 \text{ t/m}^3$ ) and of concrete =  $24 \text{ kN/m}^3$ .

The balance of the uplift pressure *i.e.*  $12.25 - 0.96 = 2.65 \text{ kN/m}^2$  has to be resisted by the reinforcement to be provided at the top in the roof slab. The roof slab has to be designed for full canal water load (1.6 m of water) plus self weight, when the drainage water is low and not exerting any uplift. Suitable reinforcement at bottom of the slab may be provided for this downward force.

# WATER LOGGING AND DRAINAGE

## **Water Logging:**

An agricultural land is said to be water logged if its productivity gets affected by the high water table. In fact, productivity decreases when root zone of the plant get flooded with water.

### **Causes of waterlogging:**

- Excessive amount of rains
- Not enough of natural drainage
- Obstruction to the natural drainage
- Obliteration of the natural drainage
- Obstruction of natural subsurface flow
- Water seepage through canal
- Development of reservoir
- Over and intensive irrigation
- Flood submergence
- High textured soil and black cotton soil

**Effects of water logging:**

- Inhabiting activity of soil bacteria
- Reduction in availability of capillary water
- Fall in soil temperature
- Rise of salts (alkaline salts)
- Defective air circulation
- Difficulties in cultivation
- Crop yields reduce
- Growth of wild flora
- Adverse effect on community health

**Preventive (Remedial) Measures:**

- Effective drainage system establishment
- Use of water in optimum quantity
- Crop rotation
- Lining of canals and watercourse

**Preventive (Remedial) Measures:**

- Introduction of intercepting drain
- Removing obstruction in natural drainage
- Utilization of underground water for irrigation by pumping
- Introduction of drip or sprinkler irrigation method

**Drainage:**

The process of removing and controlling excess water either on the surface or in the root zone beneath the soil by some engineering approach is called drainage.

**Objectives**

- To increase crop production to sustain high yields
- To prevent accretion of objectionable quantities of salts

**Importance**

- For satisfactory growth of crops as irrigation

### **Benefits of Drainage**

- Improves soil structure and increases and perpetuates the productivity of soil
- Water logged lands can be reclaimed
- Leads to early ploughing and planting
- Lengthens crop growing seasons of the year
- Reduces water table in the area
- Increases depth of root zone soil and provides more available plant food
- Increases soil ventilation
- Favors growth of soil bacteria
- Assures high soil temperature
- Decreases soil erosion and gullying by increasing water infiltration into soil
- Excess soil salt can be leached out

### **Types of Drainage:**

1. Land Drainage
2. Field Drainage

#### **1. Land Drainage**

This is large scale drainage where the objective is to drain surplus water from a large area by such means as excavating large open drains, erecting dykes and levees and pumping.

#### **2. Field Drainage**

This is the drainage that concerns in agriculture. It is the removal of excess water from the root zone of crops.

### **Types of Field Drainage:**

1. Surface Drainage (or Open Drainage)
2. Sub-surface Drainage (or Tile Drainage)

### 1. Surface Drainage (or Open Drainage)

The removal of excess rainwater falling on the field or the excess irrigation water applied to the field by shaping, grading, or management of the land surface to provide gradual removal or diversion of water off of the land surface.

Surface drainage is accomplished by smoothing out small depressions (land smoothing) or re-grading an undulating land surface to a uniform slope, and directing water to a natural or improved, constructed channel.

Ridge tillage is a form of surface drainage, providing excess water that accumulates between the ridges can flow away.

### 1. Surface Drainage (or Open Drainage)

Surface drainage is most advantageous on flat lands where slow infiltration, low permeability, or restricting soil layers prevent the ready infiltration of high intensity rainfall.

The surface drainages are classified as

- a) Shallow surface drains
- b) Deep surface drains

#### a) Shallow surface drains

Shallow surface drains are very useful in quick disposal of excessive water applied to the field or storm water, and thus prevent water percolating in the soil. The purpose of these drains is to counteract water logging rather than relieving water logged land.

### b) Deep surface drains

Deep surface drains are deeper than shallow surface drains and used in draining out the water logged area. These drains interfere with the agricultural operation and occupy considerable land.

#### Advantages of surface drainage

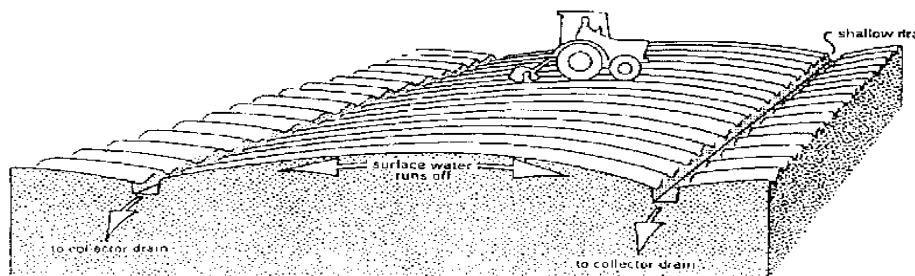
- Effectively dispose the rain water and excess irrigation water
- Economical
- Maintenance is easy

#### Disadvantages of surface drainage

- Minimal affect on reducing the saturated subsoil occuring as a result of high water table conditions
- Phosphorus and many herbicides may be transported in the surface drainage water
- Requires cross-drainage works at the crossing of canals
- Reduce cultivation area
- Regular silt clearance is required

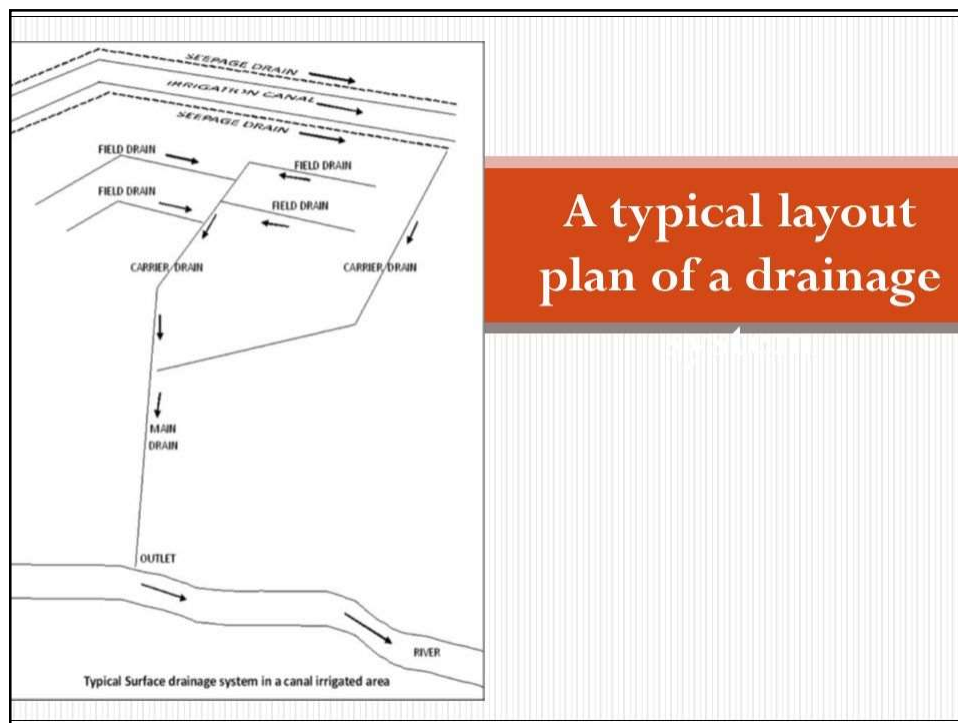
### Surface-Drainage Systems

- Surface drainage involves the removal of excess water from the surface of the soil.
- This is done by removing low spots where water accumulates by land forming or by excavating ditches or a combination of the two.



### Surface-Drainage Systems

- Land forming is mechanically changing the land surface to drain surface water.
- This is done by smoothing, grading, bedding or leveling.
- Land smoothing is the shaping of the land to a smooth surface in order to eliminate minor differences in elevation and this is accomplished by filling shallow depressions.
- There is no change in land contour. Smoothing is done using land levelers or planes



### Design of internal drainage of banded fields:

The design is made on the estimation based on water balance.

#### a) For Terai

This approach has been used in Nepal at Narayani and the Sunsari Morang Irrigation Projects.

The water balance incorporates the following assumptions:

- initial water level is 40 mm
- maximum water level is 300 mm which may persist for upto one day.
- depth in excess of 200 mm may persist for upto 3 days
- ~~3 days~~ 1 in 10 yrs maximum annual 3 days rainfall is taken as design rainfall.
- no rain follows the design rainfall for several days.
- losses due to evapotranspiration and deep percolation are replaced by ongoing irrigation and flood inflows.

The water balance equation may be expressed as:

$$h = 40 + \frac{P}{3}t - Q \cdot t, \text{ for } t \leq 3 \text{ days}$$

$$\text{and } h = 40 + P - Q \cdot t, \text{ for } t > 3 \text{ days}$$

where,  $h$  = depth of water in the field in mm

$P$  = design 3 days rainfall in mm.

$t$  = number of days that have elapsed since the rainfall began.

$Q$  = drainage runoff in mm/day.

b) For Hill:

The water balance incorporates the following assumptions:

- initial water level is 40 mm
- maximum water level is 100 mm
- 1 in 10 years maximum annual 24 hr rainfall is taken as design rainfall.
- no rain follows the design rainfall for several days.
- losses due to evapotranspiration and deep percolation are balanced by ongoing irrigation and/or flood inflows

The water balance equation may be expressed as

$$Q = P + 40 - 100 = P - 60$$

where,  $Q$  = drainage runoff in mm

and  $P$  = design 24 hr rainfall in mm.

Example:

Estimate the suitable design drainage rate for the internal drainage of banded fields with the following data:

- initial water level is 40 mm
- maxm water level is 300 mm which may persist for upto one day
- depths in excess of 200 mm may persist upto 3 days
- design mean annual maximum 3 days rainfall is 400 mm.
- no rain follows the design rainfall for several days
- neglect losses due to evapotranspiration and deep percolation

Solution: Here,  $P = 400$  mm

we have,

$$h = 40 + \frac{P}{3}t - Q \cdot t, \text{ for } t \leq 3 \text{ days}$$

$$\text{and } h = 40 + P - Q \cdot t, \text{ for } t > 3 \text{ days}$$

$$\text{Taking } Q = 1.1/0/ha = 8.64 \text{ mm/day}$$

For,  $t = 0$  days,

$$h = 40 \text{ mm}$$

$$\text{For } t = 3 \text{ days, } h = 40 + 400 - 8.64 \times 3 = 414.08 \text{ mm}$$

using For  $t = 6$  days,  $h = 40 + 400 - 8.64 \times 6 = 388.16 \text{ mm}$   
 which does not satisfy the design criteria.  
 Taking  $Q = 2 \text{ l/s/ha} = 17.28 \text{ mm/day}$

For  $t = 0$  days,  $h = 40 \text{ mm}$

For  $t = 3$  days,  $h = 40 + 400 - 17.28 \times 3 = 388.16 \text{ mm}$

For  $t = 6$  days,  $h = 40 + 400 - 17.28 \times 6 = 336.32 \text{ mm}$   
 using... which does not satisfy the design criteria.  
 Taking  $Q = 3 \text{ l/s/ha} = 25.92 \text{ mm/day}$

For  $t = 0$  days,  $h = 40 \text{ mm}$

For  $t = 3$  days,  $h = 40 + 400 - 25.92 \times 3 = 362.24 \text{ mm}$

For  $t = 6$  days,  $h = 40 + 400 - 25.92 \times 6 = 284.48 \text{ mm}$   
 which does not satisfy the design criteria.  
 Taking  $Q = 4 \text{ l/s/ha} = 34.56 \text{ mm/day}$

For  $t = 0$  days,  $h = 40 \text{ mm}$

For  $t = 3$  days,  $h = 40 + 400 - 34.56 \times 3 = 336.32 \text{ mm}$

For  $t = 6$  days,  $h = 40 + 400 - 34.56 \times 6 = 232.64 \text{ mm}$   
 which does not satisfy the design criteria.  
 Taking  $Q = 5 \text{ l/s/ha} = 43.2 \text{ mm/day}$

For  $t = 0$  days,  $h = 40 \text{ mm}$

For  $t = 3$  days,  $h = 40 + 400 - 43.2 \times 3 = 310.4 \text{ mm}$

For  $t = 6$  days,  $h = 40 + 400 - 43.2 \times 6 = 180.8 \text{ mm}$   
 which does not satisfy the design criteria.  
 Taking  $Q = 6 \text{ l/s/ha} = 51.84 \text{ mm/day}$

For  $t = 0$  days,  $h = 40 \text{ mm}$

For  $t = 3$  days,  $h = 40 + 400 - 51.84 \times 3 = 284.48 \text{ mm}$

For  $t = 6$  days,  $h = 40 + 400 - 51.84 \times 6 = 128.96 \text{ mm}$

using the  $h$  (mm) vs  $t$  (days) graph, a rate of  $6 \text{ l/s/ha}$  for the depths in excess of  $200 \text{ mm}$  restricts the period to  $2.65$  days, and reduces the peak depth to  $284.48 \text{ mm}$ . This satisfies the design criteria and makes any greater rate uneconomic. Hence ~~6 l/s/ha~~  $6 \text{ l/s/ha}$  is suitable design drainage rate.



### Time of Flood Concentration

The Time of flood concentration is derived from following formula:

$$T = \left( \frac{0.87L^3}{h} \right)^{0.385}$$

where, L = Stream length in km  
h = Vertical Distance in m

### Modified Dicken's Method

Dicken's method is an empirical, where the peak discharge is given by

$$Q_t = C_t A^{0.75}$$

where,  $C_t$  for the return period T is given by

$$C_t = 2.342 \log(0.6T) \log(1185/p) + 4$$

with  $p = 100 (A_s + 6) / A$

$A_s$  is snow covered area out of total catchment area A in  $\text{km}^2$ .

### • THE MANNING'S FORMULA

- Once the quantity of runoff is known, the design of ditches and similar structures is based on the principles of open channel flow.
- Mannings's formula assumes steady flow in a uniform channel.

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

$$Q = V \cdot A$$

Where:

- V = mean velocity (m/sec)
- R = hydraulic radius (m) = Area/wetted perimeter
- S = slope of the channel (m/m)
- n = Manning's roughness coefficient

### 9.5.3 Remodeling of existing natural drains

In areas where it is intended to use existing natural drains, it will be necessary to check that their capacity is sufficient to carry the drain design discharge. The first task would be to survey the channel, taking cross-section at a maximum spacing of 250m. These would be plotted and a longitudinal section of the drain produced at a scale of 1:5000, showing typical bed levels and ground levels adjacent to the drain. It is important that the bed levels plotted on the longitudinal section are representative of the average bed level in each section (M.9\_Drainage, 1990).

A straight line or series of straight lines would then be fitted to the bed levels on the longitudinal section. The cross section should be transformed into trapezoidal channels of similar form. The drain capacity below the design water level should then be calculated using the manning's equation.

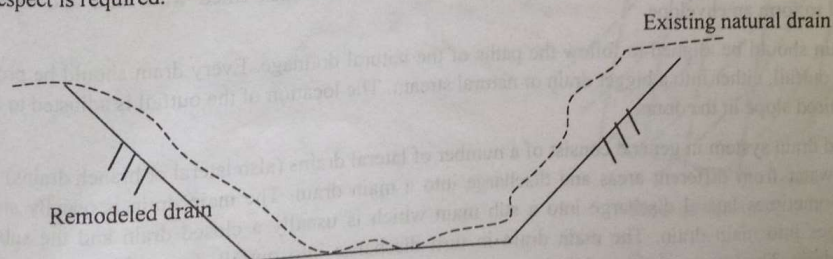
However, the existing natural drain may be very irregular in both level and in its cross-sectional shape and size, and any process of approximation to a uniform channel may be unrealistic. In this case it is necessary to carry out a back water analysis of the channel. The standard step is normally used for channels with a varying cross-section.

It is normal to start at a section where the water level for the design discharge is known. This is normally a structure at or near the downstream end of the drain. The calculation is performed by trial and error for successive steps (distances between cross-section) to give the water level at each cross-section, working from downstream to upstream. These water levels can then be compared with the required design water levels in the channel and hence used to determine if the channel has sufficient capacity to carry the design discharge.

If it is necessary to increase the capacity of the natural drain this can be done in a number of ways:

- Increase the depth of the drain.
- Increase the width on one side or both sides.
- Increase both the width and depth.

The choice of solution is dependent on the constraints of the existing channel. Any structures across the channel may make it difficult to deepen the channel. Buildings or structures located close to the banks of the channel may limit the increase in channel width. Should the channel be constrained by buildings say, through a village, it may be possible to culvert or flume the channel through that village. Each individual drains should be examined for the best solution and solution, and several method may be used on one drain in different reaches. Once the slope and levels of the new channel are decided, the channel is designed in the normal manner. It may not be possible to retain the standard B/D ratio and flexibility in this respect is required.



## 2. Sub-surface Drainage (or Tile Drainage)

The removal or control of salts and groundwater by means of drainage channel located at suitable depth below the ground surface, with the aim of lowering or controlling the water table depth below the crop root zone, is called sub-surface drainage.

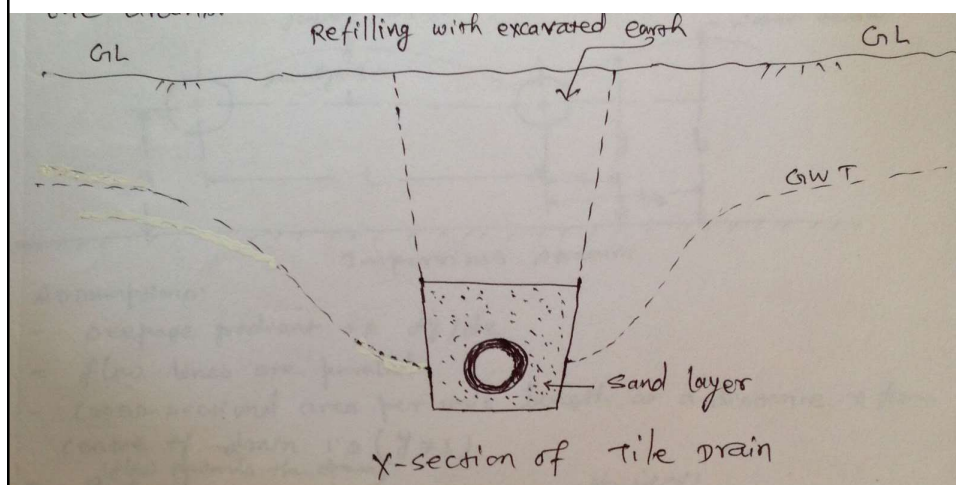
Subsurface drainage is usually implemented with the use of buried corrugated (and perforated) plastic or clay (*tile*) conduits, but it can be done also by creating an unlined pore (*mole drain*), constructing *blind (or French) drains*, excavating deep open drains, or by the use of *tubewells* (shallow groundwater wells).

### Advantages

- Drain the sub-soil water and help to control the raise of water
- Land above drains can be used for cultivation
- Reduce soil erosion
- Increase yield of crops

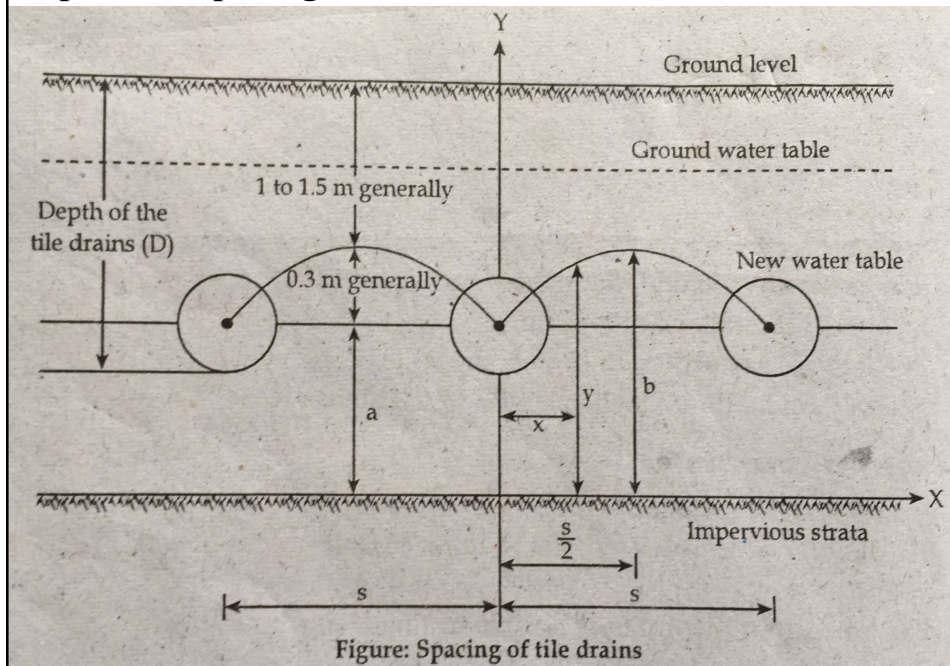
### Disadvantages

- High initial cost
- Requires skilled supervision in construction and maintenance



The drains usually consist of earthenware pipes of diameter 10 to 30 cm. The drains are laid below ground level, butting each other with open joints. These open joints between pipes are covered by tarred paper so as to prevent the entry of sand and earth into the pipes. When the tile drains are required to be placed in less pervious soils, a graded gravel filter is provided surrounding them. This filter is called envelope filter and its minimum thickness is 7.5 cm.

### Depth and Spacing of Tile Drains:



Assumptions:

- seepage gradient is  $dy/dx$
- flow lines are parallel
- cross-sectional area per unit length at a distance  $x$  from centre of drain is  $(y \neq 1)$
- $q \cdot d \frac{1}{x}$  (flow towards the drain) (drainage coefficient)
- design discharge = 1% for 24 hrs rainfall,

If  $Q_D$  = design discharge per unit length of drain  
then,  $q = \frac{1}{2} Q_D$  when  $x=0$

and  $q = 0$  when  $x = \frac{L}{2}$   
At a distance of  $x$  from the centre of drain,

$$\therefore q = \frac{1}{2} Q_D - \frac{1}{2} Q_D \cdot \frac{x}{L/2}$$

According to Darcy's Law i.e.  $q = \frac{Q_D}{2L} (L - 2x) \dots (1)$

where  $q = k i A = k \frac{dy}{dx} y$  (2)  
From equation (1) & (2) permeability of soil

$$\text{or } \frac{Q_D}{2kL} (L - 2x) dx = y dy \dots (3)$$

Integrating both sides

$$\frac{Q_D}{2kL} (Lx - x^2) = \frac{y^2}{2} + C \dots (4)$$

when,  $y = a$ ,  $x = 0$

$$\therefore C = -\frac{a^2}{2}$$

From eqn (4),

$$\therefore k = \frac{Q_D (Lx - x^2)}{2L(y^2 - a^2)} \dots (5)$$

when  $x = \frac{L}{2}$ ,  $y = b$ , eqn (5) reduces to

$$\therefore L = \frac{4k(b^2 - a^2)}{Q_D} \dots (6)$$

[Examples  $\rightarrow$  6.2 to 6.4]

**Example 6.2.** In a tile drainage system, the drains are laid with their centres 1.5 m below the ground level. The impervious layer is 9.0 m below the ground level and the average annual rainfall in the area is 80 cm. If 1% of the annual rainfall is to be drained in 24 hours to keep the highest position of the watertable to 1 metre below ground level, determine the spacing of the drain pipes. Coefficient of permeability may be taken as 0.001 cm/sec.

**Solution.** Although eqn. (6.12) can be directly used in this question, since that eqn. has been derived for designing the drains to take 1% of the average annual rainfall in 24 hours, which tallies with the given data, yet it would be prudent to use the basic eqn. (6.10) for determining the spacing of tile drains, and separately compute  $q$ , as:

$$q = \frac{\frac{1}{100} \times \left(\frac{80}{100}\right) \times (S \times 1)}{(24 \times 60 \times 60)} \text{ m}^3/\text{s}$$

$$= \frac{0.8S}{8.64 \times 10^6} \text{ m}^3/\text{s}/\text{m length of tile drain}$$

Using eqn. (6.10), we have

$$S = \frac{4K}{q} (b^2 - a^2)$$

where  $b$  = ht. of W.T. above the impervious layer

$$= 9 \text{ m} - 1 \text{ m} = 8 \text{ m}$$

$a$  = depth of impervious stratum below the centre of the drains =  $9 - 1.5 = 7.5 \text{ m}$

$$K = 0.001 \text{ cm/s} = \frac{0.001}{100} \text{ m/s}$$

Substituting values, we get

$$S \cdot q = 4 \times \frac{0.001}{100} (8^2 - 7.5^2)$$

$$\text{or } S \times \left(\frac{0.8S}{8.64 \times 10^6}\right) = \frac{4 \times 0.001}{100} (8^2 - 7.5^2)$$

$$\text{or } 0.8S^2 = \frac{4 \times 0.001}{100} \times (8.64 \times 10^6) (8^2 - 7.5^2) = 2678.4$$

$$\text{or } S = \sqrt{\frac{2678.4}{0.8}} = \sqrt{3348} = 57.86 \text{ m Ans.}$$

as in that case, the misalignment at joint

**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 =  $\left(\frac{600}{24 \times 3600}\right) = \frac{1}{144} \text{ m}^3/\text{s}$

$\therefore$   $Q = \frac{1}{144} \text{ m}^3/\text{s}$  Now,  $Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot S^{1/2}$

For a circular drain of diameter  $D$ , we have

$$A = \frac{\pi D^2}{4}, P = \pi D, R = \frac{D}{4}$$

or  $\frac{1}{144} = \frac{1}{0.011} \times \left(\frac{\pi D^2}{4}\right) \cdot \left(\frac{D}{4}\right)^{2/3} \cdot \left(\frac{0.3}{100}\right)^{1/2}$

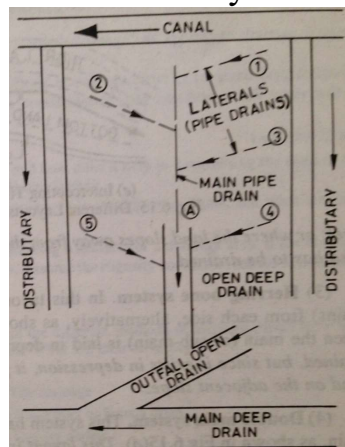
or  $\frac{1}{144} \times \frac{0.011 \times 4}{\pi} = \frac{D^2 \cdot D^{2/3}}{(4)^{2/3}} \times \frac{1}{\sqrt{333.3}}$

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.

### Layout of Tile Drains

The tile drains may be aligned in different fashions, depending on the topography of the area. Generally, laterals (branch drains) run through the most of the drainage area and join the mains, which in turn, discharge through the outlets into deep open drains. Various possible alternative layouts for the tile drainage are discussed below.



Various possible alternative layouts for the tile drainage system are shown in Fig. 6.15, and are discussed below:

(1) **Natural system.** The natural system is generally adopted in rolling topography, where drainage of isolated areas is required. The mains and the connected laterals are provided in natural course, as shown in Fig 6.15 (a). This system is suitable when the land is not to be completely drained. The system is quite flexible and permits location of drains where they are most needed.

(2) **Grid iron system.** This drainage system, consisting of laterals and mains (or submains), is shown in Fig 6.15 (b). In this system, the laterals are provided only on one side of the main, as shown. This system is adopted when the land is practically

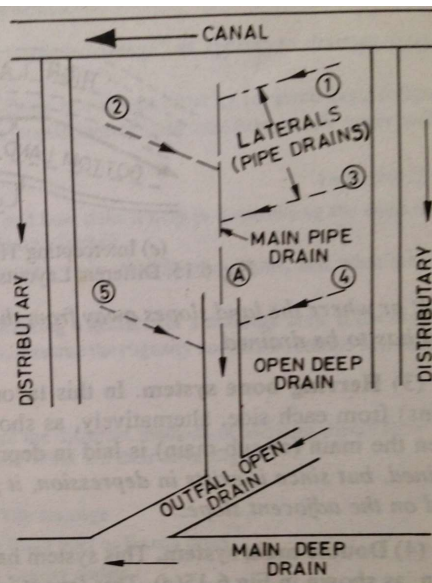
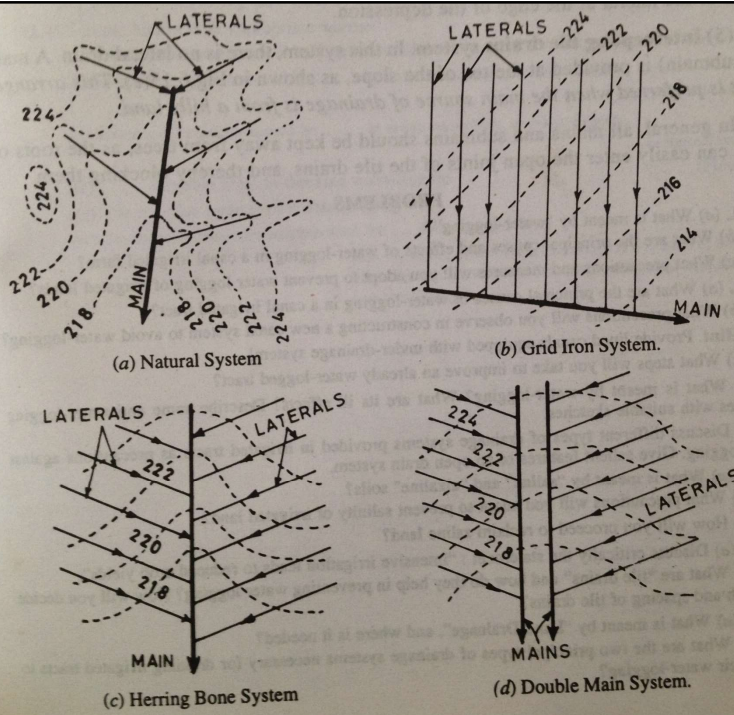


Fig. 6.14. General layout of a tile drain network.





(e) Intersecting Tile Drains System.

Fig. 6.15. Different Layouts of Tile Drainage Systems.

level; or where the land slopes away from the sub main on one side, and when the entire area has to be drained.

(3) **Herring bone system.** In this layout pattern, laterals join the mains (or sub-mains) from each side, alternatively, as shown in Fig 6.15(c). This layout is adopted, when the main (or sub-main) is laid in depression. *The land along the main is double drained, but since it exists in depression, it probably requires more drainage than the land on the adjacent slopes.*

(4) **Double main system.** This system has two mains with separate laterals for each main, as shown in Fig 6.15(d). *This layout is adopted when the bottom of depression is wide.* This arrangement reduces the length of the laterals and eliminates the break in slope of the lateral at the edge of the depression.

(5) **Intersecting tile drains system.** In this system, there is no lateral drain. A main (or submain) is provided at the toe of the slope, as shown in Fig 6.15(e). *This arrangement is preferred when the main source of drainage is from a hilly land.*

In general, all mains and submains should be kept away from trees, as the roots of trees can easily enter the open joints of the tile drains, and thereby blocking them.