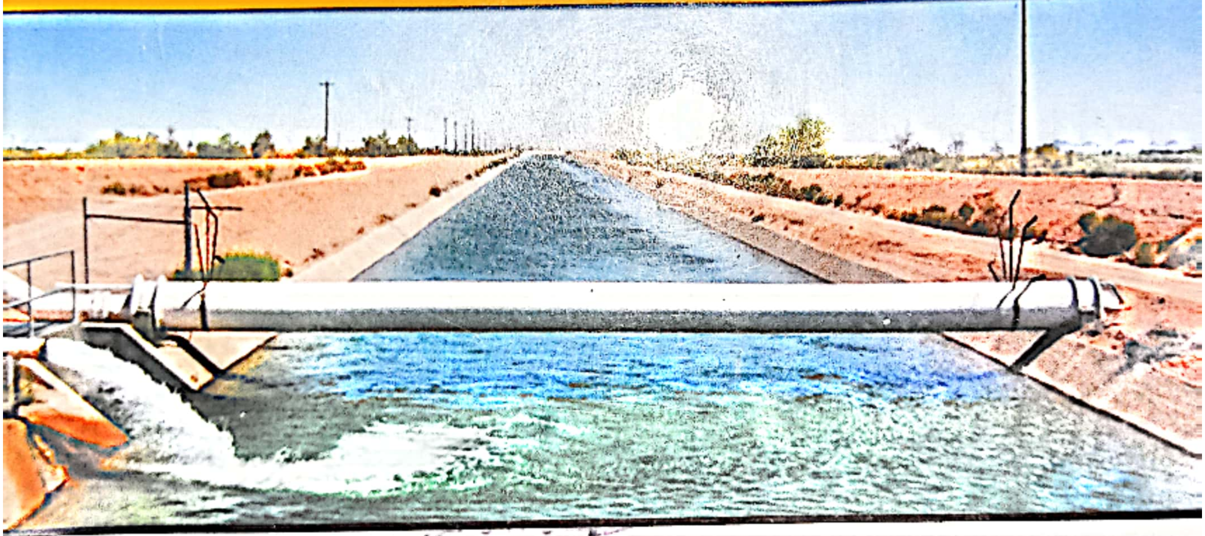
4th EDITION

G.L. Book House

IRRIGATION ENGINEERING

Bachelor of Engineering



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CHAPTER 1

INTRODUCTION OF IRRIGATION

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1.1 INTRODUCTION OF IRRIGATION

1.1.1 Definition of Irrigation

Irrigation is defined as the artificial application of water to the land for the purpose of the raising crops. A crop requires a certain amount of water at some fixed interval throughout its period of growth. The basic objective of irrigation is to supplement water to the land so as to obtain an optimum crop yields.

1.1.2 Necessity of irrigation

Inadequate rainfall

When rainfall at the place is in adequate to meet the water requirement of the crops, the irrigation is necessary. This is generally for the arid and semi arid region. When rainfall is less than 100 cm, irrigation water is essentially required.

	Rainfall (cm)	Irrigation requirement
i)	100	Rainfall need to be supplemented by irrigation
ii)	100-50	Rainfall is insufficient. Irrigation is essential
iii)	50-25	Irrigation is essentially required
iv)	Less than 25	No crop can be grown without irrigation

Uneven distribution of rainfall

Irrigation is necessary when the rainfall in region is adequate but it is not evenly distributed over the time.

Increasing the yields of crops

The yield of crops per hectare is substantially increased by irrigation as the supply of water is properly controlled. Although the rainfall may be adequate for the ordinary crops it may not cater the needs of high yielding varieties which require more water. In such case, the yield can be substantially increased by irrigation.

Growing number of crops

The rainfall in a region may be sufficient to growth only one crop in a year but it may be inadequate when a number of crop 2 to 3 are grown in the same year. Irrigation is necessary if a number of crops are grown during same year in different crops period.

Growing perennial crops

As rainfall is not regular irrigation is necessary for the perennial crops.

- Growing superior crops
- For controlled water supply

1.1.3 Advantages (function) of irrigation

Every irrigation project is designed, keeping in view of its economics. There is a capital investment on the project and future recurring charge for maintenance and operation of the project. The advantage of irrigation can be summarized in the points below:

- Increasing in food production
- Elimination of mixed cropping
- General prosperity
- Generation of hydropower
- Domestic water supply
- Facilities of communication (inspection roads acts as a link road for different villages)
- In land navigation

- Aforestation
- Flood control
- Insurance against drought

1.1.4 Disadvantages of irrigation

- Water pollution by seepage.
- Irrigation may result in colder and damper climate, resulting marshy lands
- Breeding of bacteria and mosquitoes causing outbreak of diseases like malaria.
- Water logging due to over irrigation.
- Procuring and supplying irrigation is complex and expensive in itself.

1.1.5 Scope of irrigation

Irrigation is the science of artificial application of water to the land in accordance with 'crop requirement' throughout the 'crop period' for full-fledged nourishment of the crops. Not only that, it deals with all dimensions extending from the watershed to the agriculture farms. Broadly the scope of irrigation can be divided into the following categories:

i) Engineering aspect

- Storage, diversion and lifting of water.
- Conveyance of water to agriculture lands.
- Application of water to agriculture lands.
- Development of water power.
- Drainage and relieving water logging.

ii) Agricultural aspect

- Depth of water
- Distribution of water
- Capacity of soil and flow of water
- Reclamation of lands

1.2 HISTORY OF IRRIGATION DEVELOPMENTS IN NEPAL

The development of irrigation has long history in Nepal. Annals show that irrigated agriculture was practiced in Nepal as early as during the era of Gautam Buddha and some story indicates he had involved himself in resolving disputes amongst irrigators. For systematic study, the history of irrigation development in Nepal can be divided into four phases as:

i) Primary phase or period prior to planned development *i.e.*, before 1956
ii) Infrastructure development phase (1957–1970)
iii) Intensive development phase (1971–1985)
iv) Integrated development phase (1986–date)

i) Primary phase

Irrigation facilities constructed in the Kathmandu valley during Licchavi and Malla period such as Raj kulos of which the traces are still found are fallen under primary phase. Well known king of Gorkha, King Ram Shah had special contribution in irrigation management aspect by empowering local people in irrigation dispute resolution. During Rana regime, Chandra

Shamsher had developed Chandra canal system in 1928 with assistance of British engineers. The main irrigation facilities during primary phase are Juddha canal in Sarlahi district, Jagadispur irrigation system in Kapilvastu district, and Pardi irrigation system in Pokhara.

ii) Infrastructure development phase

Irrigation facilities developed in first, second, third plan periods fall under this phase. Different irrigation system were built with the co-operation from India and USA. Some noteable irrigation system were Tika Bhairav, Mahadev Khola, Budhanilkantha irrigation system in Kathmandu valley, Vijayapu irrigation sytem in Pokhara valley, Khageri (Chitwan), Kamala and Hardinath (Dhanusha), Kodku-Godavari (Lalitpur), Pasupati (Kathmandu), Tinau (Rupandehi). Apart from these irrigation systems, which were developed under Koshi and Gandaki treaties with India, were also constructed during this phase.

iii) Intensive development phase

During fourth, fifth and sixth plan period multilateral donor agencies like World Bank and ADB came forward in aid of Nepal in irrigation development. Development of Kankai and Mahakali-I, irrigation projects initiation of command area development in Narayani zone irrigation system etc. were carried out with assistance of donor. During plan periods care Nepa had assisted to develop a number of small irrigation system covering a total of 10,000 ha. Bhairawa-Lumbini ground water, Marchawar lift and hill irrigation projects were also initiated in these phase.

iv) Integrated development phase

From the seventh plan onward *i.e.,* since the mid eighties there has been a major paradigm shift in irrigation development. Construction oriented development has been given less importance leaving Bagmati, Babai, Mahakali-II and very recent Sikta irrigation projects aside no other major projects were taken up. Rehabilitation of small farmers canals were given high priority under sectoral approach. Different projects like irrigation sector project, second irrigation sector project and community management irrigation projects were implemented on this phase.

1.2.1 Categories of Irrigation Scheme

i) Extensive development schemes (ED)

All schemes having primary and secondary canal but lacking tertiary canal system. Service block size 300 to 500 ha.

ii) Intensive development scheme (ID)

All schemes having primary and secondary canal and tertiary canal system. Service block size 30 to 50 ha.

iii) Command area development schemes (CAD)

All scheme ID plus quanternary, service block size 3 to 5 ha.

Large irrigation project

1. Mahakali irrigation project
2. Gandaki irrigation project

3. Koshi irrigation project

Nepal's largest project

- Sunsari Morang irrigation project

Medium irrigation project

- Babai
- Sikta
- Marchawa
- Kamala
- Bagmati
- kankai

More than 10000 farmer management irrigation systems in Hilly regions (FMIS).

1.3 STATUS AND NEED OF IRRIGATION IN NEPAL

Irrigation is a very ancient science. Since ancient times farmers are using their own source, knowledge and techniques for construction of different types of irrigation system in Nepal. However, the construction of modern irrigation system started in Nepal in 1922 A.D. all the irrigation system were developed, operated and maintained by farmers themselves and called farmers managed irrigation system (FMIS). From 1922 to 1957, government made little effort to develop irrigation infrastructures in Nepal. Chandra nahar, juddha nahar, Jagadispur jalasaya, Phewa badh are the few examples of projects develops in that periods. However, irrigation infrastructure development has given high priority since 1957, the milestone of the beginning of periodic plan in Nepal.

Out of the total 14.718 million hectare area of the country only 2.641 million hectare area is available and 1.766 million hectare land is irrigable. 76% of potential irrigable area lies in the terai region.

At the end of fiscal year 2065/066 out of the total irrigated land, the area covered by different methods is as follows:

Surface irrigation	= 6,66,873 ha
Underground irrigation	= 2,80,682 ha
Farmers water course	= 2,80,041 ha
Total irrigated area	= 12,27,596 ha
Total area	= 1,47,181 km^2
Total cultivated area	= 2.6 million ha
Total irrigable land	= 66% of cultivable land = 1. 7 Million ha
Total irrigation facilities	= 60% of irrigable land = 1.06 million ha

More than 80% people of country directly or indirectly depends upon agriculture for their existence irrigation facility are developed only in terai but small and negligible in hilly region.

1.4 CROPS THEIR SEASONS AND PERIODS (CROPPING PATTERN AND INTENSITY)

From the agricultural view, the year can be divided into two principle cropping season Rabi and Kharif crops season.

i) Rabi crops season (1st Oct to 31st March)

Rabi crops are wheat, barley, gram, mustard, potatoes etc.; also called winter crops.

ii) Kharif crop season (1st April to 30th Sept.)

Rice, maize, jower, cotton, tobacco, and groundnut, etc.; also called summer crops.

Kharif crops require about two to three times the quantity of water required by the Rabi crops.

Crop period is the period within which it covers the land space. The time period from seeding to harvesting of crops is called the crop period.

1.5 COMMANDED AREA AND IRRIGATION INTENSITY

Commanded area can be categorized in to two types.

i) Gross commanded area (GCA)

It is the total area bounded within the irrigation boundary of a project, which can be economically irrigated without considering the limitation of the quantity of available of water. It includes both the cultivable and uncultivable area. For example residential area, roads, grounds, ponds, etc. are the uncultivable area of the gross commanded area.

ii) Culturable or cultivable commanded area (CCA)

Culturable area is the cultivable part of the gross commanded area, and includes all the lands of Gross commanded Area (GCA) on which cultivation is possible. It will be thus, include pastures and fallow lands which can be made cultivable.

Intensity of irrigation

The percentage of CCA proposed to be irrigated in a given season is called irrigation intensity of that season, or seasonal intensity of irrigation. For examples the sanctioned intensity of irrigation under BIP (Bagmati irrigation project) is only 27.6% for Kharif season and 34.4% in Rabi season.

Annual Intensity of Irrigation (AII) = Sum of irrigation intensities of each season

In above example;

$$AII = 27.6 + 34.4 = 62\%$$

1.6 METHOD OF FIELD IRRIGATION AND THEIR SUITABILITY

Irrigation may be broadly divided into followings;

i) Surface Irrigation

In surface irrigation methods, the water is directly applied to the surface of the land. It is further classified as follows;

a) Flow Irrigation

When water is available at a higher level and it is supplied to lower level by mere action of gravity then it is called flow irrigation. It is further sub divided as follows;

1. Perennial irrigation

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

2. Flood irrigation

This type of irrigation is also called inundation irrigation. In this method of irrigation soil is kept submerged and thoroughly flooded with water so as to cause saturation of land.

b) Lift irrigation

If water is lifted up by some mechanical or manual means such as by pumps etc and then supplied for irrigation then it is called lift irrigation.

ii) Sub-surface irrigation

It is termed as sub-surface irrigation because in this type of irrigation water does not wet the soil surface. The underground water nourishes the plant root by capillarity. It may be classified as;

a) Natural sub-surface irrigation

When underground irrigation is achieved simply by natural processes without any additional extra efforts it is called natural sub surface irrigation.

b) Artificial sub-surface irrigation

When a system of open jointed drain is artificially laid below the soil so as to supply water to crops by capillarity then it is called artificial sub surface irrigation.

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

Free flooding

In this method, ditches are excavated in the field, and they may be either on the contour or up and down the slope. Water from these ditches, flow across the field. After the water leave the ditches no attempt is made to control the flow by means of levees.

- Suitable if the land is irregular and water is inexpensive, and abundantly available.
- Most inefficient resulting non uniform distribution and considerable loss of water.
- This method involves less labour because no land preparation is done in the form of leaves, field ditches, grading, etc.

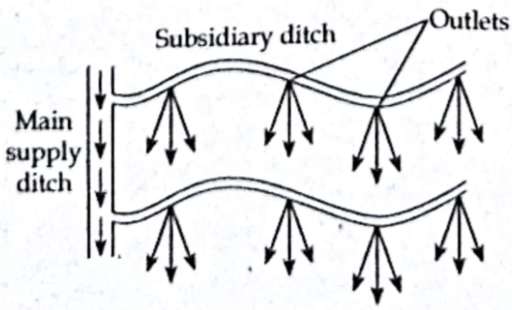


Figure: Free flooding

Border flooding

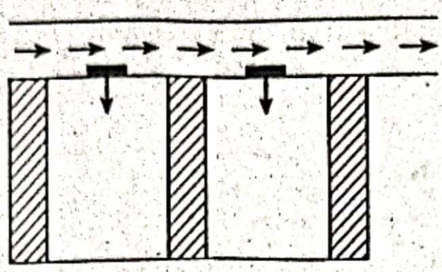


Figure: Boarder flooding

In this method land is divided into a number of strips with the help of low levees called borders. The length and slope of border strip are dependent on soil type, as the velocities should not be as large as to cause erosion and should not be small as to result in very uneven depth of infiltration from head to tail of the strip.

Water turned in to upper end of each strips flows slowly towards the lower end.

- Relatively efficient, rapid and easy.
- Suitable for some close growing crops and row crops.
- Suitable for wheat, leafy vegetables, barseem and other fodders, etc.

Check flooding

Check flooding is similar to free flooding except that the water is controlled by surrounding the area with low and flat levees. Levees are generally constructed along the contours of vertical interval 5–10 cm. Each compartment is essentially level. This result in more even spread of water and avoid wastage. However, some loss of cultivated area occurs. It requires constant attendance and work during irrigation.

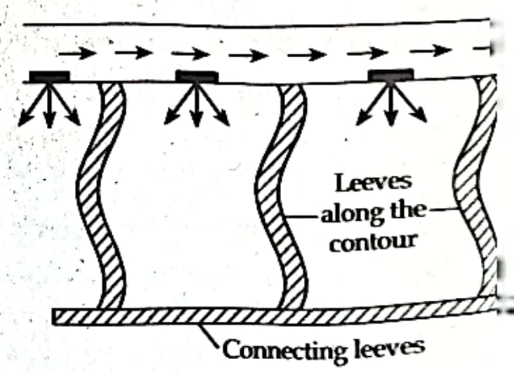


Figure: Check flooding

- Suitable for both permeable and impermeable soil.
- Water can hold for a certain time and allowed to percolate.
- Suitable for large discharge and level plots.

Basin flooding

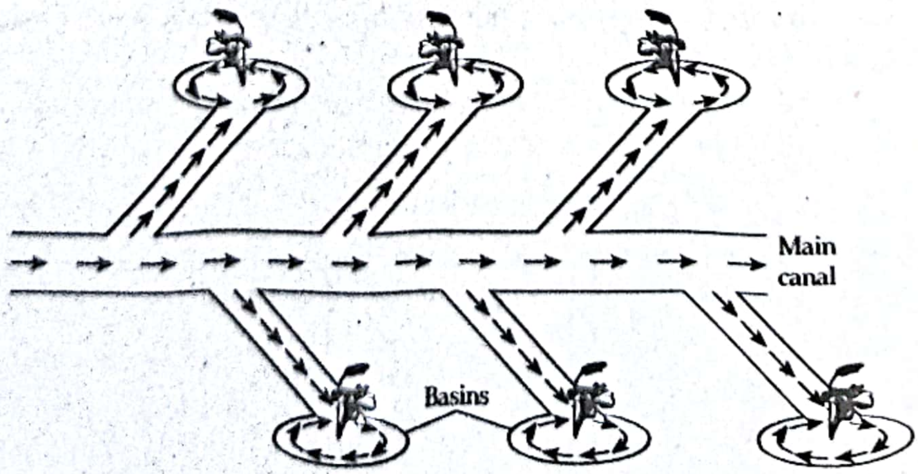


Figure: Basin flooding

This method is a special type of check flooding and is adopted specially for orchard trees. One tree is generally placed in the basin and surface is flooded by ditch water suitable for root of trees.

Furrow irrigation method

This method involves applying water to the furrows (a series of long narrow channels) in between the rows of the plant to be irrigated. Due to percolation water reaches to the root of plants. In this method, water is not applied to the entire surface area of land and hence evaporation losses are less. It is an excellent method for row crop like groundnut, tobacco, cotton potatoes, cauliflowers, etc. In this method, less evaporation takes place since $\left(\frac{1}{2}\right)^{th}$ to $\left(\frac{2}{5}\right)^{th}$ land surface is wetted by water.

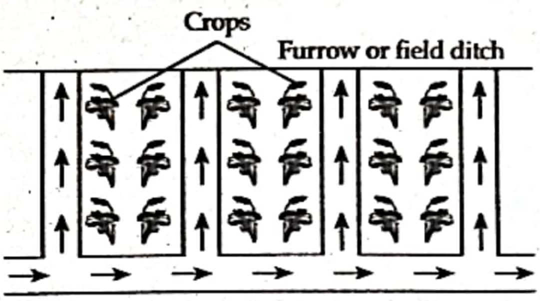


Figure: Furrow method

Sprinkler irrigation method

Applying water in the form of raindrop in case of quantity of water is low and pressure is high known as sprinkler irrigation method. It is costly method but irrigates different topography and slope efficiently.

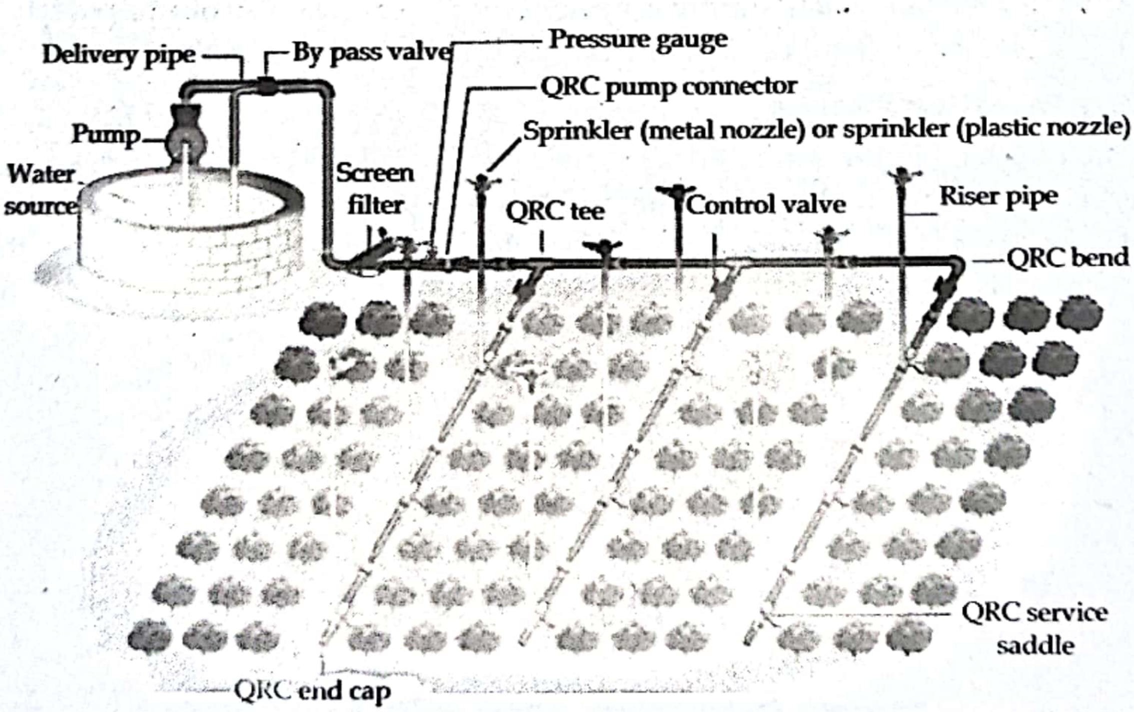


Figure: Sprinkler irrigation

Advantages of sprinkler irrigation method

- Seepage losses which occur in the earthen channel of surface irrigation are completely eliminated.
- Land leveling is not required.
- No cultivation area is lost for making ditches.

- Fertilizer can be uniformly applied.
- This method prevents water logging and salinity.
- Up to 80% efficiency can be achieved.

Disadvantages of sprinkler irrigation method

- High cost.
- Effect of pressure.
- Effect of wind.
- Leaf scorch due to salt.
- Clogging of nozzle.
- Energy or pressure is required.
- Frequent supervision required.

Limitations of sprinkler irrigation method

- High wind may distort sprinkler pattern causing non uniform spreading of water on the crops.
- In the areas of high temperature and high wind velocity, considerable evaporation losses of water may take place.
- They are not suited to crops requiring frequent and larger depths of irrigation such as paddy.
- Initial cost of system is very high, and system requires high technical skill.
- It requires larger electrical power.
- A constant water supply is needed for commercial use of equipment.
- Heavy soil with poor intake cannot be irrigated efficiently.

Drip irrigation method

Water applied in the form of drop directly near the base of the plant is known as drip irrigation method. This method is suitable where scarcity of water. It is also known as trickle irrigation. Drip nozzles are the small orifice made on the pipe network buried under the surface providing water to the root zone of the plants and crops.

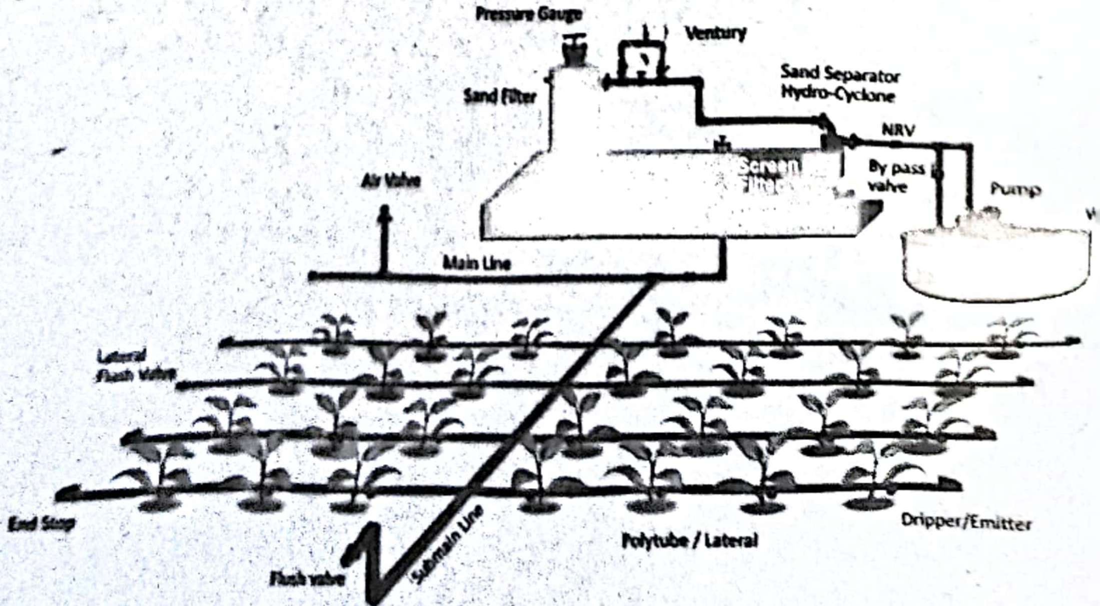


Figure: Drip irrigation

Advantages of drip irrigation method

- It has very high water application efficiency (more than 90%).
- The evaporation losses from the land surface are quite low.
- The deep percolation losses are entirely eliminated.
- It is quite suitable for small trees and widely spaced plants.

Disadvantages of drip irrigation method

- It is expensive method.
- There are maintenance problem because of clogging of small pipe and opening of emitter by clay and silt particles.
- It is not suitable for closely spaced crop such as wheat.
- Because only a part of soil is wetted, root growth is sometime inhibited.

1.7 ADVANTAGES AND DISADVANTAGES OF SURFACE AND SUB-SURFACE IRRIGATION METHOD

	Surface irrigation	Sub-surface irrigation
i)	**Advantages**	**Advantages**
	❒ Less initial cost	❒ No wastage of water
	❒ Not highly skilled man power is required	❒ Control over water
	❒ Not highly developed technology is required	❒ Less change of erosion of soil
	❒ Easy to supervise	❒ Chances of destruction is less
ii)	**Disadvantages**	**Disadvantages**
	❒ No control over water	❒ Initial cost is more
	❒ Wastage of water	❒ Highly skilled manpower is required
	❒ Chances of water logging	❒ Highly developed technology is required
	❒ Crops may suffer from excess water	❒ It is difficult to supervise
	❒ Level tract of land is required	
	❒ Chance of erosion of soil	

1.8 QUALITY OF IRRIGATION WATER

Just as every water is not suitable for human beings, in the same way, every water is not suitable for plant life. The water containing impurities which are injurious to plant growth is not satisfactory for irrigation and is called the unsatisfactory water.

The quality of suitable irrigation water is very much influenced by constituent of soil which is to be irrigated. Particular water may be harmful for irrigation on particular soil, but same water may be tolerable or even useful for irrigation on some other soil. The various impurities, which make the water unfit for irrigation are as follows:

i) Sediment concentration in water
ii) Total concentration of soluble salts in water
iii) Proportion of sodium ions to other cations
iv) Concentration of potentially toxic elements present in water
v) Bacterial contamination
vi) Bicarbonate concentration as related to concentration of calcium plus magnesium

1.9 PLANNING OF IRRIGATION PROJECT

- Main objective of planning is to match water availability with demand as closely as possible.
- Planning is always before the implementation or operation of work.
- Planning of irrigation requires study of;
  - Estimating water availability.
  - Crop planting technique.
  - Estimating irrigation demand for the planned crops.
  - Matching supply and demand.

1.9.1 Stages of planning

Planning is a stepwise process and it has the following stages.

i) Primary planning

- Reconnaissance and prefeasibility
- Location of head works
- Availability of water

ii) Feasibility study

- Economic analysis
- Sustainability study
- Benefit cost ratio (BCR) > 1
- Internal rate of return (IRR) $> i$ (Interest of bank)

iii) Final design

- Final design of irrigation structures and canals.

Factors to be considered during planning stages are as follows;

a) Type of the project and general plan to irrigation
b) Location, extend and types of irrigation land
c) Crop water requirement
d) Culturable area
e) Cost of work
f) Needs of immediate and future drainage
g) Evaluation of benefits
h) Method of financing
i) Type and location of engineering works
j) Annual cost of water to farmer

1.10 WORKED OUT PROBLEMS

PROBLEM 1

Justify the need of irrigation development in Nepal. Define cropping intensity and irrigation intensity.

Solution:

Nepal is an agricultural country. About 65% of the total populations are engaged in agricultural for survival. On the other hand, there is no scarcity of water in Nepal because of the presence of bulky natural water resources.

Based upon above two facts, and also the following scenario of present Nepal, irrigation is very much needed:

i) Uneven distribution of rainfall

Rainfall distribution in Nepal is not uniform all over the country. There may be surplus water at a region and deficit at the other. So, to manage this problem, irrigation development is necessary in Nepal.

ii) Increasing the yield of crops

Fertility characteristics of land alone are not sufficient for better yield of crops. There are such zones in Nepal where no irrigation facility but fertile soil is present. So, to increase the yield of crops through combine effort of irrigation water and fertile soil, well developed irrigation system is necessary.

iii) To grow number of crops

Seasonal variation within a year is better in Nepal *i.e.,* four seasons (summer, winter, spring and autumn). With well manipulated irrigation, water we can grow and earn more number of crops within a year.

iv) Inadequate rainfall

There might be scarcity of water at Terai belt due to inadequate rainfall. At the same instant there might be surplus water available at hilly region to irrigate the land of terai. So, to overcome the inadequate rainfall and fulfill the water requirement of crops, well grappled irrigation system is necessary.

v) Sound economic growth

With all the above problems solved through well addressed irrigation system, the economic growth can also be accountably increased along with the increase in crop production.

Moreover, other reasons for irrigation development are:

i) Growth of agro-based industry
ii) Utilization of natural water resources
iii) Increase the employment
iv) To conserve the cultivable land
v) To develop the scientific agricultural concept

Cropping intensity

It is defined as the ratio between net sown area and gross cropped area.
Mathematically,

$$\text{Cropping intensity} = \frac{\text{Gross cropped area}}{\text{Net sown area}} \times 100$$

The cropping intensity therefore, refers to raising a number of crops from the same field during one agricultural year.

Irrigation intensity

The percentage of CCA proposed to be irrigated annually is called irrigation intensity. By knowing the irrigation intensity for a crop season the area to be irrigated during that crop season can be determined.

PROBLEM 2

Discuss the advantages and disadvantages of sprinkler and drip irrigation. [2071 Magh]

Solution:

Applying water in the form of drop in case of quantity of water is low and pressure is high known as sprinkler irrigation method. It is costly method but irrigates different topographies and slope efficiently.

Advantages of sprinkler irrigation

i) Control over water
ii) Saving of water
iii) Saving of labour
iv) Efficient use of land
v) Protection of soil and crop from the extreme weather conditions
vi) Efficient use of land
vii) Frost and climatic control
viii) Prevent water logging and salinity

Disadvantages of sprinkler irrigation

i) High cost
ii) Effect of pressure
iii) Leaf scotch due to salt
iv) Clogging of nozzle
v) Energy or pressure is required
vi) Frequent supervision required

Drip irrigation

Water applied in the form of drops directly near the base of plant is known drip irrigation method. This method is suitable where water is insufficient.

Advantages of drip irrigation

i) It has very high water application efficiency (more than 90%).
ii) The evaporation losses from the land surface are quite low.
iii) The deep percolation losses are entirely eliminated.
iv) It is suitable for small trees and widely spaced plants.

Disadvantages of drip irrigation

i) It is an expensive method.

ii) There are maintenance problem because of clogging of small pipe and opening of emitter by clay and silt particles.

iii) Root growth is sometime inhibited because certain parts of soil are wetted.

PROBLEM 3

Discuss the status of irrigation in Nepal.	**[2072 Ashwin]**

Solution: See the definition part 1.3

PROBLEM 4

Define irrigation. Write the history of irrigation development in Nepal.
[2014 Semester fall: Po.U.]

Solution:

Irrigation

See the definition part 1.1.1

History of irrigation development in Nepal

See the definition part 1.1.2

PROBLEM 5

Write short notes on: FMIS in Nepal

Solution:

The abbreviation FMIS stands for farmer's Managed Irrigation System. FMIS have prevailed in Nepal for many centuries. The FMIS systems are operated and maintained solely by the community farmers or an individual family. The infrastructure in FMIS Nepal were mostly built from the local materials of mud, stone and forest product with rudimentary traditional methods, practiced over many centuries. Mostly, these FMIS are location specific, indigenous in their management practices and representative of the local organizational needs and services deliver. FMIS symbolizes grass root democratic institution where community takes responsibility of natural resource management and allocation. In this country, farmers are collectively engaged in irrigated agricultural development as an enterprise since time immemorial. There are thousands of FMIS in the country. Some notable examples of FMIS are Raj kulo of argali, and Chherlung of Palpa District and the Gyandhi irrigation system fo Palpa district. The FMIS provide irrigation services to 70% of country's total irrigation services to 70% of country's total irrigated area of little over of 1.2 million hectare. Hence, FMIS have gained on their own status that symbolic of the national heritage in Nepal, special feature of FMIS are as follows;

High mountain

The intake diversion is of rock fill with mud mortar walls and canals have slate lining with mud mortar base. A water reservoir is built in the vicinity of village, the main problems in the canal are leakages due to highly porous strata and only 10% of flow is received at the command area.

MID hills

The FMIS in the mid-hills consists of simple brushwood diversion and open earthen canal. Usually mud and shrubs are used to control leakage in canal system.

Terai

In the terai the diversion structures are usually built with an earthen bund with shrubs, logs and stones where available. Praganna irrigation system of Dang Deukhuri and Siyari irrigation system of Rupandehi Districts are some examples of FMIS in Terai region.

Problems in FMIS raised by farmer's are:

1. In the hills and mountains
   i) Water acquisition and protecting intake structure from flash food and river bed load.
   ii) Seepage problems in the feeder and main system
   iii) Siltation problems in the feeder canal
   iv) Landslide bank failure
   v) River encroachment
   vi) Water distribution
   vii) Large river crossing
2. In the lowland terai
   i) Abstraction from river
   ii) Canal bank overtapping
   iii) Siltation
   iv) Seepage
   v) Canal side slope instability
   vi) Inundation from natural command area and drainage from command area.
   vii) Water distribution

PROBLEM 6

Why is irrigation development important for Nepal? Define cropping pattern and cropping intensity. [2074 Bhadra]

Solution:

Importance of irrigation development in Nepal

See the solution of Q. no. 1

Cropping pattern

Cropping pattern is defines as the production of area under various crops at point of time. It is very dynamic parameter because there is no ideal pattern for all times to a particular region.

Rabi (winter) crops and kharif (summer) crops are the particular example of cropping pattern.

Cropping intensity

PROBLEM 7

The field capacity of soil is 40%, permanent wilting point is 20%, density of soil is 1.2 gm/cc, effective root depth is 90 cm, ET crop is 10 mm/day. Calculate the irrigation interval (IR) if the readily available moisture (RAM) is 75% of available soil moisture capacity and show AMC, RAM and irrigation interval of graph of available moisture and time.[2075 Baishakh]

Solution:

Given that;

Field capacity (F.C.) = 40%

Permanent wilting point (PWP) = 20%

Density of soil (γ_d) = 1.2 gm/cc

Effective root depth (d) = 90 cm = 0.9 m

ET_{crop} = 10 mm/day

RAM = 70% of AVM

Irrigation interval (IR) = ?

Now,

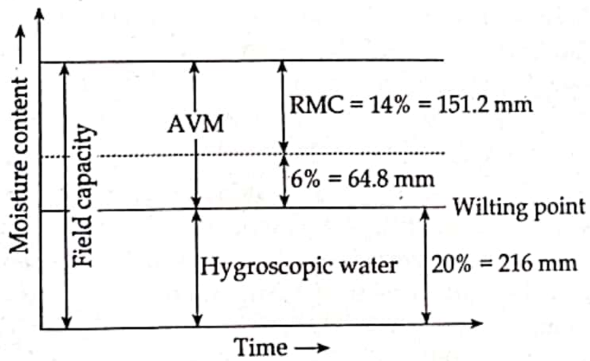
$$\text{Available moisture capacity} = \text{F.C.} - \text{PWP} = 40 - 20 = 20\%$$

$$\text{Readily available moisture} = 0.7 \times 20\% = 14\%$$

$$\text{Depth of water for consumptive use} = \frac{1.2 \times 0.9}{1.0} \times 0.14 = 0.1512 \text{ m} = 151.2 \text{ mm}$$

$$\text{Irrigation interval} = \frac{151.2}{10} = 15.12 \text{ days}$$

AVM = 151.2 + 64.8 = 216 mm



PROBLEM 8

Define irrigation. What are the advantages of irrigation? Describe the scope of irrigation in Nepal. [2076 Baishakh]

Solution: See the definition part 1.1.1, 1.1.2, 1.1.3 and 1.1.5

PROBLEM 9

Being an agriculture country Nepal, why it not developed as expected in this sector? Give our own reasons. [2076 Bhadra]

Solution:

Though Nepal is an agricultural Country, agricultural sector is not developed as expected.

Following are the reasons for our agricultural backwardness:

i) Lack of proper irrigation system
- Farmers have to depend highly on monsoon rain which is not reliable
- Hardly about 20% of total land under cultivation has good irrigation system
- In winter much of farm land remain unused due to lack of irrigation facilities.
- In monsoon landslide and floods damage the crop.

ii) Traditional farming
- Due to traditional farming work is very difficult and productivity is very low.
- Agricultural tools are not advanced

iii) Lack of transport and market.
- Due to lack of market, transportation or storage facilities agro based product rot in the place where they are grown.
- Difficult to provide facilities related to agriculture (tools, plats, equipment, etc.). Due to lack of transportation

iv) Unscientific distribution of land
- Those who are actual farmers don't own land at all while those who don't even tread hold large area of land and leave barren.
- Over fragmentation of agricultural land into small parcels resulting low productivity

v) Poor economic condition of farmers
- Most of Nepalese farmer are subsistence farmers compelled to take loan even to run their family, though it's not easy to get loan from bank for actual farmer without good mortgage, interest rate is high. In such condition most of farmer cannot afford new technology and machinery.

vi) Unavailability of fertilizer on time of demand

vii) Poor agricultural insurance mechanism

viii) Climate change-as new emerging problem

ix) Lack of research activities

Not enough researches and experiments to find the best crops and the best climate and best soil type.

PROBLEM 10

Write the different methods of surface and sub-surface irrigation and explain about furrow irrigation with their suitability. [2077 Chaitra]

Solution: See the definition part 1.6

PROBLEM 11

Explain the importance of irrigation development in Nepal. Describe about the method of surface and sub-surface irrigation and their suitability. [2078 Baishakh]

Solution: See the definition port 1.1, 1.2, 1.3 and 1.6

CHAPTER 2

IRRIGATION WATER REQUIREMENT

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2.1 GENERAL

Water requirement of crop

Every crop requires a certain quantity of water after a certain fixed interval throughout its period of growth. If the natural rain is sufficient and timely, no irrigation water is required for growth of crop.

Water requirement of a crop is defined as the total quantity and the way in which a crop requires water, from the time it is shown to the time it is harvested. The water requirement will vary with the crop as well as with the place. Depending upon the variations in climates, types of soil, method of cultivation and useful rainfalls, different crop will have different water requirement and some crop may have different water requirements at different places of the same country. Numerically, water requirement can be defined as sum of three entities viz. consumptive use (A), application and conveyance losses (B) and other special needs (C) *i.e.,*

$$WR = A + B + C$$

Crop period

The time period that elapses from the instant of its showing 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 showing to its last watering before harvesting is called base period or base of the crop. Base period is slightly less than crop period, but for all practical purposes they are taken as one and the same thing and expressed in days. The term growth period, crop period, base period are synonyms, each representing crop period and represented by B (in days). The base period of Kharif crop is 183 days and 182 days for Rabi crops.

2.1.1 Duty and Delta of crop

Delta (Δ)

Delta can be defined as the total depth of water required by a crop to come to maturity. It can be defined as the total quantity of water supplied to irrigate certain area to obtain maximum yield *i.e.,* the ratio of volume of water to the area of land irrigated by it is known as delta. Its unit is 'm' or 'cm'. It depends on the amount of each watering and the interval between successive watering the base period.

Delta is stated with the reference to the place at which it is measured, that is delta at farm, delta at outlet, delta at distributary head, delta at the head of main canal. Below table indicates the average value of delta for certain important crop.

Crop	Delta (Δ) on field
Sugarcane	120 cm (48")
Rice	120 cm (48")
Tobacco	75 cm (30")
Garden fruits	60 cm (24")
Cotton	50 cm (22")
Vegetables	45 cm (18")
Wheat	40 cm (16")
Barley	30 cm (12")
Maize	25 cm (10")
Fodder	22.5 cm (9")
Peas	15 cm (6")

Table 2.1: Average delta for certain crops

Duty (D)

Duty of water is the relation between the area irrigated or to be irrigated and the quantity of water used or required to irrigate it for purpose of maturing its crop. It can be defined as the number of hectares of land irrigated for full growth of a given crop by supplying of 1 cubic meter of water per second flowing continuously for the base period (B) of the crop. Its unit is hectares/cumecs.

Gross duty

It is the duty of water measured at the source of diversion of irrigation supplies.

Nominal duty

It is the duty sanctioned as per scheduled of an irrigation department.

Economic water duty

It is the duty of water which results in the maximum yield;

i) per unit area when land is limiting factor and
ii) per unit of irrigation water when water is the limiting factor.

Designed duty

It is the duty of water assumed in an irrigation project for designing capacities of the channels.

The average values of certain crops are tabulated in table 2.2.

Crop	Duty in hectares/cumec
Sugarcane	730
Rice	775
Other Kharif	1500
Rabi	1800
Perennials	1100
Hot fodder	2000

Table 2.2: Average approximate value of duty for certain crops

2.1.2 Relation between duty, delta and crop period

Let a crop of base period 'B' days and suppose that one cusecs of water be applied on this crop in the field for 'B' days.

Now, the volume of water applied to this crop during B days is;

$$V = QT$$
$$= 1\,m^3s^{-1} \times B \text{ days}$$
$$= 1\,m^3s^{-1} \times B \times 24 \times 60 \times 60 \text{ sec.}$$
$$\therefore \quad V = 86400\,m^3$$

By definition of duty (D) one cubic meter supplied for B days matures D hectares of land. This quantity of water (V) matures D hectare so, land or 10^4 D m^2 of area.

$$\text{Total depth of water applied on this land} = \frac{\text{Volume}}{\text{Area}}$$
$$= \frac{86400\,B}{10^4\,D} = \frac{8.64B}{D}\,m$$

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

$$\therefore \quad \Delta = \frac{8.64B}{D}\,m$$

$$\text{or,} \quad \Delta = \frac{864B}{D}\,cm$$

where, B is in day, D in hectares/cubic meter.

2.1.3 Factors affecting duty

The various factors affecting duty of water are as follows:

i) Type of crop

Different crops require different amount of water and duties for them are different. Duty of water for a crop requiring more water is less and vice versa. Obviously duty of water for rice is less than that for wheat.

ii) Useful rainfall

If the rain falling directly over the irrigated land is useful for the growth of the crop, then less irrigation water will be required to mature the crop. More useful rainfall less will be the requirement of irrigation water and hence more will be the duty of irrigation water.

iii) Type of soil

Evidently porous soil under a crop has less duty due to high absorption loss. Duty of water is more in heavy (clay) soil than light (sandy) soil.

iv) Cultivation method

Properly ploughed and tilled field before irrigation retains higher amount of water in its saturated zone then by reducing the number of watering and increasing the duty. More efficient the cultivation method, higher is the duty.

v) Method of water application

The more efficient the mode of application of water, higher is the duty. Flooding irrigation system has lesser duty than furrow irrigation.

vi) Topography of land

A level field means uniform application of water, but if it is not level, low portions receive more depth of water than higher level portions which implies wasteful use of water and hence less duty. Properly leveled field means more economical use of water and high duty.

vii) Condition, type and location of canal

If canal is in good condition and properly maintained the duty is more compared to that in different condition and poorly maintained.

2.1.4 Methods of improving duty

The various methods of improving duty are as follows:

i) Land should be properly ploughed and tilled.
ii) Correct quantity and timing of water application mean higher duty. Drip irrigation is most modern and efficient water application method.
iii) Canals should be lined to cut down transmission losses in the canal system. Higher velocity in lined section also results in reduced evaporation losses.
iv) The canal should be nearest to the command area so that idle length of the canal is minimum and hence reduced transmission losses.
v) Good quality of water should be used for irrigation.
vi) Rotation of crops should be practiced.
vii) Volumetric assessment of water is enforced so that efficient and economical use of the canal water is made by cultivators.

2.2 CROP WATER REQUIREMENT (PENMAN METHOD)

i) Consumptive use (Cu)

Considerable part of water applied for irrigation is lost by evaporation and transpiration. The two processes being difficult to separate are taken as one and called evapo-transpiration or consumptive use. Consumptive use for a particular crop may be defined as total water used by plants in transpiration (building plant tissues etc.) and evaporation from adjacent soil in any specified time. The consumptive use (Cu) varies with temperature, humidity, wind velocity, soil topography, sunlight hours, available moisture, method of irrigation, depth of water applied for irrigation, cropping pattern, seasons, mean monthly temperature.

Factors affecting consumptive use

Consumptive use or evapo-transpiration depends upon all those factors on which evaporation and transpiration depend; such as temperature, sunlight, humidity and wind movement, etc.

ii) Potential evapo-transpiration (PET)

When the sufficient moisture is available to completely meet the needs; the vegetation fully covering an area, the resulting evapo-transpiration is called PET. PET critically depends on the climatological factors rather than characteristics of plant and soil.

iii) Actual evapo-transpiration

The real PET occurring in the field is called actual evapo-transpiration (AET).The AET is largely affected by the characteristics of soil and plant.

2.2.1 Penman equation

Penman gives theoretical PET for his reference crops.

$$E_t = \frac{AH_n + E_a\gamma}{A + \gamma}$$

where, E_t = Daily potential evapo-transpiration.

A = Slope of the saturation vapor pressure versus temperature curve at the mean air temperature.

H_n = Net incoming solar radiation or energy expressed in mm of evaporable water per day.

E_a = Parameter including wind velocity and saturation deficit in mm/day.

γ = Psychometric constant = 0.49 mm of Hg/°C.

$$H_n = H_C(1-r)\left(a + b\frac{n}{N}\right) - \sigma T_a^4(0.56 - 0.92\sqrt{e_a})\left(0.1 + 0.9\frac{n}{N}\right)$$

where, H_C = Mean incident solar-radiation at the top of the atmosphere on a horizontal surface, expressed in mm of evaporable water per day.

r = Reflection coefficient (albedo) of the given area.

a = Constant depending upon latitude (ϕ) and given as; $a = 0.29 \cos\phi$

b = Constant having average value = 0.52

n = Actual duration of bright sunshine in hours.

N = Maximum possible hours of bright sunshine (mean value). This value is a function of latitude (ϕ).

σ = Stefan – Boltzmann constant = 2.01×10^{-9} mm/day.

T_a = Mean air temperature in K = 273 + °C

e_a = Actual mean vapour pressure in the air in mm of Hg.

The parameter E_a of the Penman's equation is estimated as;

$$E_a = 0.35\left(1 + \frac{V_2}{160}\right)(e_s - e_a) \text{ mm/day}$$

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

e_s = Saturation vapour pressure at mean air temperature in mm of Hg.

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

The above equation gives PET for given climatic condition for Penman's reference crop. To determine PET for any crop this value should be multiplied by corresponding crop coefficient.

$$ET_{crop} = K_{crop} \times ET_0$$

$K = 1.1 - 1.3$ for paddy

$K = 0.8$ for wheat

$K = 0.6$ for maize

ET_0 = Daily potential evapo-transpiration

The irrigation system is designed to fulfill the potential requirement of crop (PET).

2.3 IRRIGATION WATER REQUIREMENT

It is defined as the sum of crop water requirement plus all types of water losses.

Irrigation water requirement (IWR) = CWR + Losses

where, CWR is crop water requirement.

The irrigation water requirement is further studied as;

2.3.1 Consumptive irrigation requirement (CIR)

It is the quantity of water actually required by the plant. If a part of the consumptive use is provided by the natural rainfall, the consumptive irrigation requirement is given by,

$$CIR = C_u - R_e;$$

where, C_u = Consumptive use

R_e = Effective rainfall

i.e., the rainfall falling during the growth period of a crop and is available to meet evapo-transpiration needs of the crop exclusive of the rainfall lost through deep percolation below root zone or the water lost as surface run-off.

2.3.2 Net irrigation requirement (NIR)

It is defined as the Consumptive Irrigation requirement plus the water required for other purposes, such as leaching of alkaline or salty soils.

$$NIR = CIR + L_e = (C_u - R_e) + L_e$$

where, L_e is the water required for leaching and other purpose.

R_e is the effective rainfall.

2.3.3 Field irrigation requirement (FIR)

It is the amount of water required to be applied to the field. It is equal to the net irrigation requirements plus the amount of applied water lost as surface runoff, evaporation and deep percolation.

FIR = NIR + Water application losses

It is usually expressed as,

$$FIR = \frac{NIR}{\eta_a}$$

where, η_a is the water application efficiency.

2.3.4 Gross irrigation requirement (GIR)

The quantity of water required at the head of the canal is greater than the field irrigation requirement because there are always some transit (or conveyance) losses. Thus, the gross irrigation requirement is given by,

GIR = FIR + Conveyance losses

Alternatively, it can be expressed as,

$$GIR = \frac{FIR}{\eta_c}$$

where, η_c is the water conveyance efficiency.

2.4 EFFECTIVE RAINFALL (R_e)

The precipitation falling during growing period of a crop that is available to meet evapo-transpiration needs of crop is called effective rainfall. It includes the precipitation lost through deep percolation below the root zoon or water lost as surface runoff.

2.5 WATER LOSSES DUE TO SEEPAGE AND EVAPORATION

During the passage of water from main canal to the outlet at head of water course, water lost by evapo-transpiration from the surface or by seepage through precipitation of channels.

2.5.1 Evaporation losses

The water lost by evaporation is generally small as compared to the water lost by seepages in certain channels. Evaporation losses are generally 2–3% of total losses.

2.5.2 Seepage losses

There are two different condition of seepage which is as follows;

i) Percolation

In percolation; there exists a zone at continuous saturation from the canal to the water table and a direct flow is established. Almost all water lost from the canal joins the ground water reservoir. The loss of water depends upon the difference of top water surface level of the channel and level of the water table (*i.e.*, H) as shown in the figure.

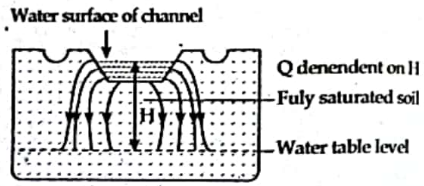

Figure: Precolation

ii) Absorption

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

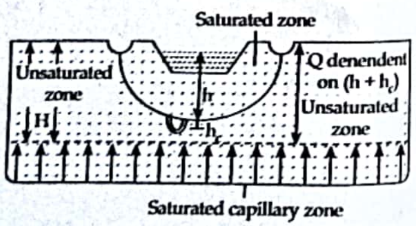

Figure: Absorption

In this case, the rate of loss is independent of seepage head (H) but depends only upon the water head 'h' (*i.e.*, distance between water surface level of canal and bottom of the saturated zone) plus capillary head (h_c), as shown in the figure.

The seepage loss depends on;

i) *Types of seepage: Percolation or absorption*
ii) *Soil permeability*
iii) *The condition of canal*

iv) Amount of silt carried by water
v) Velocity of canal water
vi) Cross-section of canal and its wetted perimeter

2.6 SOIL MOISTURE IRRIGATION RELATIONSHIP

The moisture (water) content within the soil may be divided in three parts.

2.6.1 Field Capacity (FC)

Field capacity is the water content of a soil after free drainage has taken place for sufficient period. It consists of two parts; one is attached to the soil molecules by surface tension against gravitation force, and can be extracted by plants by capillarity. This water is called capillary water. The other is that which is attached to the soil molecule by loose chemical bonds. This water which can't be removed by capillary is not available to the plants, and is called hygroscopic water.

$$\text{Field capacity} = \frac{\text{Weight of water retained in certain volume of soil}}{\text{Weight of same volume of dry soil}} \times 100$$

$$F = \frac{\text{Weight of water retained per unit area of soil}}{\gamma_d \times d}$$

where, d is the root zone depth in meter.

γ_d is the dry unit weight of soil.

γ_w is the unit weight of water.

F is the field capacity m.c.

Now,

$$\text{Weight of water retained in unit area of soil} = \gamma_d \times d \times F \text{ kN/m}^2$$

$$\therefore \quad \text{Volume of water in unit area of soil} = \frac{\gamma_d \times d \times F}{\gamma_w}$$

2.6.2 Permanent wilting point (PWP)

PWP is that water content at which plant can no longer extract the water for its growth, and wilts up. Hence available water or available moisture type may be defined as the difference in water content of soil between field capacity and PWP.

2.6.3 Readily available moisture (REM)

It is that portion of available moisture which is easily extracted by plants, and

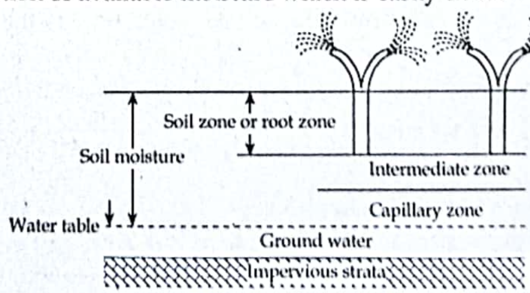

Figure: Soil moisture irrigation relationship

is approximately 75 to 80% of actual moisture content (AMC).

RAM = 75 to 80% of AMC

AMC = FC − PWP

Soil moisture deficiency

The water required to bring the soil moisture content of a given soil to its field capacity is called moisture deficiency.

Irrigation interval (II)

The time interval of irrigation water to re-irrigate the field is called irrigation interval.

i.e., $II = \frac{FC}{PET}$

2.7 DEPTH AND FREQUENCY OF IRRIGATION

The irrigation water should be applied as soon as the moisture falls up to the optimum level (greater then PWP) and its quantity should be just sufficient to bring the moisture content up to the field capacity, making allowance for application losses. The depth and frequency of irrigation can be shown in figure (i) and (ii).

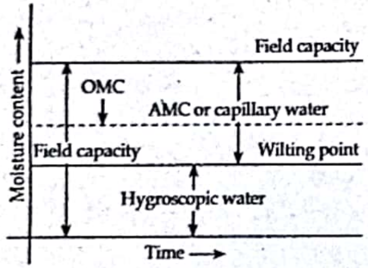

Figure: (i)

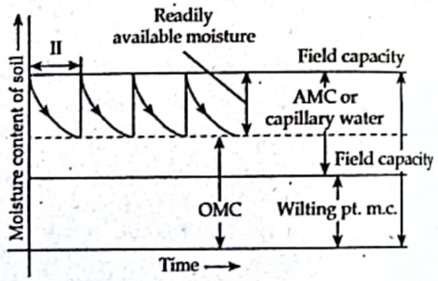

Figure: (ii)

2.8 IRRIGATION EFFICIENCIES

Efficiency is the ratio of water output to the water input, and is usually expressed as percentage. The efficiency of the irrigation is of different kinds:

i) Efficiency of water conveyance

It is the ratio of the quantity of water delivered to the field to the quantity of water diverted into the canal from the off take point. It takes the conveyance or transit losses in consideration.

$$\text{Water conveyance efficiency } (\eta_c) = \frac{\text{Quantity of water delivered to field}}{\text{Quantity of water diverted from offtake}} \times 100$$

ii) Efficiency of water application

It is the ratio of quantity of water stored at the root zone of the plants to the quantity of water applied to the field. It takes into consideration the water lost in farm.

$$\text{Water application efficiency } (\eta_a) = \frac{\text{Quantity of water stored at root zone}}{\text{Quantity of water applied to field}} \times 100$$

iii) Efficiency of water storage

It is the ratio of quantity of water stored at root zone during irrigation to the quantity of water needed to bring the moisture content of soil to the field capacity.

i.e., $$\text{Water storage efficiency } (\eta_s) = \frac{\text{Quantity of water stored in the root zone}}{\text{Quantity of water needed to bring the moisture to field capacity}} \times 100$$

iv) Efficiency of water use

It is defined as the ratio of the quantity of water used beneficially by the crop, including water required for leaching, to the quantity of water delivered to field.

i.e., $$\text{Water use efficiency } (\eta_u) = \frac{\text{Quantity of water used beneficially}}{\text{Quantity of water delivered to the field}} \times 100$$

v) Outlet discharge factor

The duty of the water varies, from one place to another, due to the various losses, and increases as soon as one moves downwards from the head of the canal towards the head of the branches or water courses. The duty at the head of water course is quite important, and is called the out let discharge factor.

2.9 WORKED OUT PROBLEMS

PROBLEM 1

Draw crop coefficient curve for rice crop. An irrigation project has 6000 ha of CCA and ET_0 is 150 mm/day, effective rainfall in 30 mm/month and overall efficiency of the project is 30%. Calculate the irrigation demand in cumecs.

Solution:

Given that;

CCA = 6000 ha

$ET_0 = 150$ mm/day

Effective rainfall = 30 mm/month = $\frac{30}{30}$ mm/day = 1 mm/day

Efficiency of the project = 30%

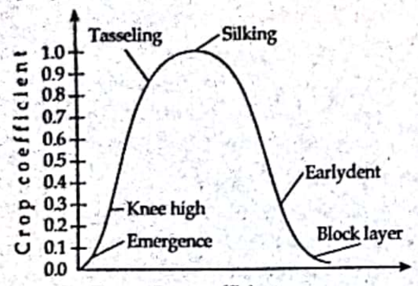

Figure: Crop coefficient curve

We know,

$ET_{crop} = ET_0 \times K_{crop}$

K = 1.1–1.3; for paddy (rice)

$ET_{crop} = 1.1 \times 150$ mm/day = 165 mm/day

Depth of irrigation water = (150 – 1) = 149 mm/day

Irrigation demand (Q) = 149 mm/day × 6000 ha

$$= \frac{149 \times 10^{-3}}{86400} \times 6000 \times 10^4 \text{ m}^3/\text{sec.}$$

$$= 103.472 \text{ m}^3/\text{sec.}$$

Project efficiency is 30%.

so, Irrigation demand $= \frac{103.472}{0.3} = 344.91$ cumec

PROBLEM 2

With following data:

FC = 80%	**PWP = 35%**
Root depth = 60 cm	**Soil density = 1.5 gm/cc**
ET_C = 5 mm/day	**Application efficiency = 80%**
Conveyance loss = 55%	**Distribution loss = 65%**

Calculate;

i) Available moisture content
ii) Readily available moisture contents
iii) Depth of irrigation at outlet of the field
iv) Irrigation interval
v) The requirement of head work

Solution:

Available moisture content = AMC

= Field capacity (F.C.) – Permanent wilting point (P.W.P.)

= (80 – 35)% = 45%

Readily available moisture content (RAM) = 80% of AMC

= 0.8 × 45% = 36%

Depth of water available for consumptive use $= \frac{\gamma_d \times d}{\gamma_w} \times 0.36$

$= \frac{1.5 \times 0.6}{1} \times 0.36$

$= 1.5 \times 0.6 \times 0.36$

= 0.324 m

= 32.4 cm

Irrigation interval $= \frac{0.324}{\frac{5}{1000}} = 64.8$ days

Field irrigation requirement $= \frac{\text{Consumptive use}}{\text{Water application efficiency}} = \frac{0.324}{0.8}$

= 0.405 m

Depth of water at outlet of field = 0.405 m

Irrigation requirement at head work $= \frac{0.405}{0.55 \times 0.65} = 1.133$ m

PROBLEM 3

The field capacity of soil is 60%, permanent wilting point is 25%, Density of soil is 1.2 gm/cc, effective root depth is 120 cm, ET_{crop} is 9 mm/day. Calculate the irrigation interval if the readily available moisture is 85% of available soil moisture capacity. [2069 Bhadra]

Solution:

Given that;

Field capacity = 60%

Permanent wilting point = 25%

Density of soil = 1.2 gm/cc

Effective root depth = 120 cm

ET_{crop} = 9 mm/day

Available moisture capacity = FC – PWP = (60 – 25)% = 35%

Readily available moisture = 0.8 × 35 = 28%

Depth of water available for consumptive use $= \frac{1.2 \times 1.2}{1.0} \times 0.28$

$= 0.403\text{ m} = 403.2\text{ mm}$

$$\text{Irrigation interval} = \frac{403.2}{9} = 44.8\text{ day}$$

PROBLEM 4

Calculate the design discharge of canal at 0, 1, 2, 3 and 4 km from head works. The GCA at the head of the canal is 45000 ha and after each km it is reduced by 5000 ha. Out of this command, the CCA is 80%. The intensities of irrigation for wheat and rice are 60% and 35% respectively. Assume total loss below km 4 = 0.45 m^3/s; channel losses per km is 3.5% of discharge at beginning of each km; Kor periods for wheat and rice are 4 and 3 weeks respectively. Kor depth for wheat and rice are 16 and 21 respectively.

Solution:

The gross commanded area and culturable commanded area at various km are first of all worked out.

Below	Gross commanded area in hectares	Gross culturable area in hectares
0	45000	36000
1	40000	32000
2	35000	28000
3	30000	24000
4	25000	20000

Outlet discharges for two crop seasons are determined as given,

i) For wheat,

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

where, $B = 4\text{ week} = 28\text{ days}$

$\Delta = 16\text{ cm} = 0.16\text{ m}$

$$\therefore D = \frac{8.64 \times 28}{0.16} = 1512\text{ hectare/cumec}$$

ii) For rice,

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

where, $B = 3\text{ week} = 21\text{ days}$

$\Delta = 0.21\text{ cm}$

$$\therefore D = \frac{8.64 \times 21}{0.21} = 864\text{ hectare/cumec}$$

Intensity of irrigation for wheat = 60%

Intensity of irrigation for rice = 35%

If 'G' is gross culturable area at any point, then 0.6G is the wheat is and 0.35G is the rice area.

$$\text{Discharge required for wheat} = \frac{0.6\text{ G}}{1512} = \frac{\text{G}}{2520}$$

Similarly,

$$\text{Discharge required for rice} = \frac{0.35\text{ G}}{864} = \frac{\text{G}}{2468.5}$$

Since, the discharge required for rice is more than that required for wheat. The outlet factor of rice becomes controlling factor.

Below km	Gross culturable area	Discharge required for rice $= \frac{G}{2468.5}$
0	36000	14.58
1	32000	12.96
2	28000	11.34
3	24000	9.72
4	20000	8.1

At 4 km;

Losses below 4 km = 0.45 m^3/sec.

Discharge required for crop at this point = 8.1 cumec

Total discharge required = 0.45 + 8.1 = 8.55 cumec

Design discharge = 10% more than required

$= 1.1 \times 8.55 = 9.405$ cumec

At 3 km;

Outlet discharge below 3 km = 9.72 cumec

Losses below 4 km = 0.45 cumec

Losses between 3 to 4 km = 3.5 % of 9.72 $= \frac{3.5}{100} \times 9.72 = 0.34$ cumec

Total losses below 3 km = 0.45 + 0.34 = 0.79 cumec

Total discharge required at 3 km = 9.72 + 0.79 = 10.51 cumec

Design discharge = $1.1 \times 10.51 = 11.561$ cumec

At 2 km;

Out let discharge required below 2 km = 11.34 cumec

Losses below 3 km = 0.79 cumec

Losses between 2 and 3 km $= \frac{3.5}{100} \times 11.34 = 0.397$ cumec = 0.4 cumec

Total losses below 2 km = 0.79 + 0.4 = 1.19 cumec

Total discharge required = 11.34 + 1.19 = 12.53 cumec

Design discharge = $1.1 \times 12.53 = 13.78$ cumec

At 1 km;

Out let discharge required below 1 km = 12.96 cumec

Losses below 2 km = 1.19 cumec

Losses between 1 and 2 km $= \frac{3.5}{100} \times 12.96 = 0.45$ cumec

Total losses = 1.19 + 0.45 = 1.64 cumec

Total discharge required = 12.96 + 1.64 = 14.6 cumec

Design discharge = $1.1 \times 14.6 = 16.06$ cumec

At 0 km;

Out let discharge required = 14.58 cumec

Losses below 1 km = 1.64 cumec

Losses between 0 to 1 km $= \frac{3.5}{100} \times 14.58 = 0.5103$ cumec

Total losses = (1.64 + 0.5103) cumecs = 2.15 cumec

Total discharge required = 14.58 + 2.15 = 16.73 cumec

Design discharge required = 1.1 + 16.73 = 18.403 cumec

PROBLEM 5

A minor commands 400 ha of irrigation area. It is proposed to consider wheat crop in the whole commanded area. The Kor period for wheat is considered 3 weeks. The Kor depth has been assumed to be 10 cm. In this period 2.75 cm of rainfall is normally expected with such intensity that 50% of this could be taken as superfluous (surface run off). Considering 10% conveyance loss find out; (a) duty of canal water at the field head and (b) discharge of minor at upstream head. [2070 Bhadra]

Solution:

Given that;

Commanded area = 400 ha

Kor period = 3 weeks = 21 days

Kor depth = 10 cm

Water available from rainfall = 50% of 2.75 cm = 1.375 cm

Depth of irrigation water = 10 − 1.375 = 8.625 cm

Duty of water on field $= \frac{8.64\ B}{\Delta} = \frac{8.64 \times 21}{.0.0862} = 2104.87$ ha/cumec

Here, conveyance loss is 10%. So, a discharge of 1 cumec at the head of water course, 0.9 cumec at the field.

Duty at head of water course = 2104.87 × 0.9 = 1894.36 ha/cumec

Discharge required $= \frac{400}{1894.36} = 0.211$ cumec

PROBLEM 6

A field channel has culturable command area of 2000 ha. The intensity of irrigation for gram is 30% and that for wheat is 50%. Gram has a Kor period of 18 days and Kor depth of 12 cm. While wheat has K_{or} period of 15 days and Kor depth of 15 cm. Calculate the discharge of the field channel. [2070 Magh]

Solution:

Given that;

Culturable commanded area (CCA) = 2000 ha

Intensity of irrigation for wheat = 50%

Intensity of irrigation for gram = 30%

Kor period for gram = 18 days

Kor period for wheat = 15 days

Kor depth for gram = 12 cm

Kor depth for wheat = 15 cm

Now,

Duty for gram $= \frac{864 \times 18}{12} = 1296$ ha/cumec

Duty for wheat $= \frac{864 \times 15}{15} = 864$ ha/cumec

Area of gram irrigation = 2000 × 0.3 = 600 ha

Discharge for gram $= \frac{600}{1296} = 0.46$ cumec

Area of wheat irrigated = 50% × 2000 = 1000 ha

Discharge for wheat $= \frac{1000}{864} = 1.16$ cumec

Total discharge required = 0.46 + 1.16 = 1.62 cumec

PROBLEM 7

Compute the flow discharge needed for a canal to irrigate dry seasons crops in 30000 ha and wet season crops in 40000 ha. Kor period and Kor depth for dry and wet season crops are 6 week and 14.8 cm and 4 weeks and 11.5 cm respectively. [2071 Magh: T.U.]

Solution:

For dry season

Dry season crop area = 30000 ha

Kor period = 6 weeks = 42 days

Kor depth = 14.8 cm

Now,

Duty for dry season $= \frac{864 \times 42}{14.8} = 2451.89$ ha/cumec

Discharge for dry season $= \frac{30000}{2451.89} = 12.23$ ha/cumec

For wet season

Wet season crop area = 40000 ha

Kor period = 4 weeks = 28 days

Kor depth = 11.5 cm

Now,

Duty for wet season $= \frac{864 \times 28}{11.5}$

= 2103.65 ha/cumec

Discharge for wet season $= \frac{40000}{2103.65}$

= 19.01 ha/cumec

∴ Total discharge required = 12.23 + 19.01

= 31.24 ha/cumec

PROBLEM 8

What is field capacity of soil? For distributary, GCA is 500 ha, CCA is 80%, intensity of irrigation is 30% for wheat and 20% for rice. The kor period is 3 weeks and 2.5 weeks for wheat and rice respectively. Calculate the outlet discharge and also determine the capacity of canal considering 10% loss in distribution.

Solution:

Field capacity of soil

Field capacity is the water content of a soil after free drainage has taken place for sufficient period. It consists of two parts, one is attached to the soil molecules by surface tension against gravitational force and can be extracted by plants capillarity, hence the water is called capillary water and the other water is that, which is attached to the soil molecule by loose chemical bonds. Such type of water which cannot be removed by capillary is not available to the plants and is called hygroscopic water.

$$\text{Field capacity} = \frac{\text{Weight of water retained in certain volume in soil}}{\text{Weight of same volume of dry soil}} \times 100$$

Here,

Gross commanded area (GCA) = 500 ha

Culturable commanded area (CCA) = 80% of GCA

= 0.8 × 500 = 400 ha

Now,

$$\text{Area to be irrigated under wheat} = 400 \times \frac{30}{100} = 120 \text{ hectares}$$

$$\text{Area to be irrigated under rice} = 400 \times \frac{20}{100} = 80 \text{ hectares}$$

Here, kor depth of wheat and rice is not given, so assuming;

Kor depth for wheat = 13.5 cm

Kor depth for rice = 19 cm

Kor periods for wheat = 3 weeks = 21 days

Kor periods for rice = 2.5 weeks = 17.5 days

Now,

$$\text{Duty for wheat} = \frac{864 \times B}{\Delta} = \frac{864 \times 21}{13.5} = 1344 \text{ ha/cumec}$$

$$\text{and, Duty for rice} = \frac{864 \times B}{\Delta} = \frac{864 \times 17.5}{19} = 795.79 \text{ ha/cumec}$$

Again,

$$\text{Outlet discharge required for wheat} = \frac{\text{Area}}{\text{Duty}} = \frac{120}{1344} = 0.089 \text{ cumec}$$

$$\text{and, Outlet discharge required for rice} = \frac{\text{Area}}{\text{Duty}} = \frac{80}{795.79} = 0.1 \text{ cumec}$$

Considering 10% loss in distribution; we have,

$$\text{Capacity of canal} = \frac{0.089 + 0.1}{0.9} = 0.21 \text{ cumec}$$

PROBLEM 9

What do you mean by crop water requirement? Calculate the discharge required at outlet when the area to be irrigated in Rabi is 4000 hectares and Kharif are 13.5 cm and 4 weeks and 19 cm and 2.5 weeks respectively.

Solution:

Crop water requirement

Crop water requirement is defined as the total quantity and the way in which a crop requires water from the time it is shown to the time it is harvested. The crop water requirement will vary with the crop as well as with the place.

Here, according to the given condition; we have,

$$\text{Duty for Rabi} = \frac{864 \times B}{\Delta} = \frac{864 \times 4 \times 7}{13.5} = 1792 \text{ ha/cumec}$$

$$\text{and, Duty for Kharif} = \frac{864 \times B}{\Delta} = \frac{864 \times 2.5 \times 7}{19} = 795.79 \text{ ha/cumec}$$

Now,

$$\text{Outlet discharge required for Rabi} = \frac{\text{Area}}{\text{Duty}} = \frac{4000}{1792} = 2.23 \text{ cumecs}$$

$$\text{and, Outlet discharge required for Kharif} = \frac{\text{Area}}{\text{Duty}} = \frac{4000}{795.79} = 5.02 \text{ cumecs}$$

PROBLEM 10

Determine the capacity of reservoir, it its cultivable area is 50000 hectares, from the following data: [2014 Semester fall: Po.U.]

Crop	Base period (days)	Duty at field (Ha/cumecs)	Intensity of irrigation (Percentage)
Wheat	120	1900	25
Rice	120	1000	10
Sugarcane	330	2500	15

Solution:

Here,

Area of each crop under irrigation = CA × Irrigation intensity

$$\text{Water required for wheat} = \frac{\text{Area}}{\text{Duty}} = \frac{50000}{1900} \times 0.25 = 6.58 \text{ cumecs}$$

$$\text{Water required for rice} = \frac{\text{Area}}{\text{Duty}} = \frac{50000}{1000} \times 0.1 = 5 \text{ cumecs}$$

$$\text{Water required for sugarcane} = \frac{\text{Area}}{\text{Duty}} = \frac{50000}{2500} \times 0.15 = 3 \text{ cumecs}$$

Water volume required by wheat in 120 days base periods = 6.58 × 120 × 24 × 3600

$= 68.22 \text{ Mm}^3$

Water volume required by rice in 120 days base periods = 50 × 120 × 24 × 3600

$= 51.84 \text{ Mm}^3$

and, Water volume required by sugarcane in 330 days base periods $= 3 \times 120 \times 24 \times 3600$

$= 31.1\ Mm^3$

$\therefore$ Capacity of reservoir $= 68.22 + 51.84 + 31.1 = 151.16\ Mm^3 \simeq 152\ Mm^3$

PROBLEM 11

Three distributories are used for irrigation. The details are given below. Find which one is more efficient. [2072 Ashwin]

	Distributary-1	**Distributary-2**	**Distributary-3**
Discharge	**15 m^3/s**	**20 m^3/s**	**25 m^3/s**
C.C.A.	**15000 ha**	**25000 ha**	**30000 ha**
Intensity of irrigation	**60%**	**80%**	**50%**
Base period	**200 days (cotton crop)**	**120 days (wheat crop)**	**365 days (sugarcane)**

Solution:

For Distributary-1

Area irrigated under cotton crops $= 15000 \times 0.6 = 9000$ ha

Discharge $= 15\ m^3/sec.$

Duty $= \frac{9000}{15} = 600$ hec/cumec

For Distributary-2

Area irrigated under wheat corps $= 25000 \times 0.8 = 20000$ ha

Discharge $= 20\ m^3/sec.$

Duty $= \frac{20000}{20} = 1000$ hec/cumec

For Distributary-3

Area irrigated under sugarcane crops $= 30000 \times 0.5 = 15000$ ha

Discharge $= 25\ m^3/s$

Duty $= \frac{15000}{20} = 600$ hec/cumec

Since, distributary-2 has higher duty so more efficient.

PROBLEM 12

Explain about soil-moisture-irrigation relationship. [2072 Ashwin]

Solution: *See the definition part 2.6*

PROBLEM 13

A farmer with his 40 m × 40 m plot plans to irrigate his field 4 sprinklers having a throw distance as 10 m and each placed 20 m apart. Prepare a sketch of wetting pattern of these sprinklers. Write your comments on the moisture pattern and suggest measure to improve it if required. [2072 Magh]

Solution:

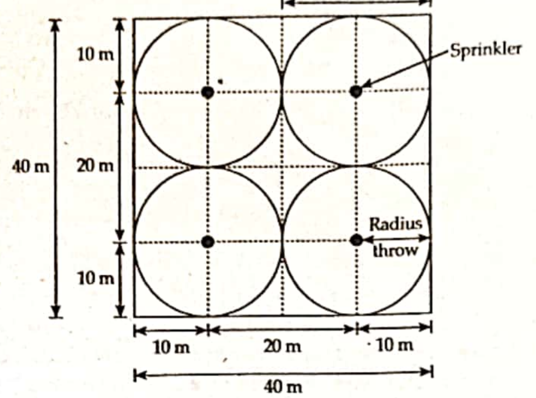

Figure: Sprinkler irrigation system (Plan)

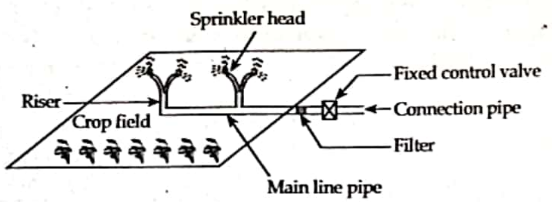

Figure: Sprinkler irrigation system (Side view)

From above figure, we can clearly see that some portion of field remains dry. To get desirable water distribution uniformly, two adjacent sprinklers should be spaced in such a way that there is some overlap of precipitation. As general rule, the spacing between the sprinkler is kept between 50–60% of wetting diameter. In this case, to improve moisture content we should decrease spacing between sprinklers and use more number of sprinklers.

PROBLEM 14

With the following data: FC = 35%, PWP = 12% root depth = 70 cm, soil density = 1.4 gm/cc, ET_c = 9 mm/day, RAM = 70% AMC, application efficiency = 85%, conveyance loss and distribution loss 20% where the abbreviations have their usual meanings. Calculate;

i) Available moisture content

ii) Readily available moisture content

iii) Depth of irrigation at the outlet of the field

iv) Irrigation interval and

v) Depth of irrigation water required at the headwork. [2072 Magh]

Solution:

i) AMC = FC − PWP = 35 − 12 = 23%

ii) RAM = 70% AMC = 0.7 × 23 = 16.1%

iii) Depth of water available for consumptive use $= \frac{\gamma_d \times d \times RAM}{\gamma_w}$

$= \frac{1.4 \times 0.7 \times 0.161}{1}$

$= 0.15778$ m

$= 157.78$ mm

Depth of water required at outlet $= \frac{157.78}{0.85} = 185.62$ mm

iv) Irrigation interval $= \frac{157.78}{9} = 17.53$ mm

v) Irrigation water required at headwork $= \frac{185.62}{0.2} = 928.1$ mm

PROBLEM 15

A stream of 150 litre per second was diverted from canal and 110 litre per second was delivered to the field. An area of 2.2 hectares was irrigated in 8 hours. Effective depth of root zone was 1.5 m. the runoff less in the field was 445 m³. the depth of water penetration varies linearly from 1.5 m at head end of field to holding capacity of the soil is 200 mm per meter depth of soil. Determine the water conveyance deficiency, water application efficiency station efficiency. Irrigation was started at moisture extraction level is 50%. [2073 Bhadra]

Solution:

i) Water conveyance efficiency $(\eta_c) = \frac{\text{Quantity of water delivered to field}}{\text{Quantity of water diverted from offtake}} \times 100$

$= \frac{110}{150} \times 100\% = 73.33\%$

ii) Water application efficiency $(\eta_a) = \frac{\text{Water stored in the root zone during irrigation}}{\text{Water delivered to the field}} \times 100\%$

Water supplied to the field during 8 hours @ 110 litre per second is;

$= 110 \times 8 \times 60 \times 60$ litres

$= 3168000$ litre $= 3168$ cu. m

Run off loss in the field $= 445$ cu. m

∴ Rhe water stored in the root zone $= 3168 - 445 = 2723$ cu. m

∴ Water application efficiency $(\eta_a) = \frac{2723}{3168} \times 100\% = 85.95\%$

iii) Water storage efficiency $= \frac{\text{Water stored in the root zone during irrigation}}{\text{Water needed in root zone prior to irrigation}} \times 100\%$

Moisture holding capacity of soil $= 200$ mm/m depth $\times 1.5$ m depth of root zone

$= 30$ cm

Moisture already available in root zone at the time of start of irrigation is;

$= \frac{50}{100} \times 30 = 15$ cm

Additional water required in root zone $= 30 - 15 = 15$ cm

$= \frac{15}{100} \times 2.2 \times 10^4$ cu. m

(*i.e.,* Depth × Plot area)

$= 3300$ cu. m

But, actual water stored in root zone $= 2723$ cu. m

∴ Water storage efficiency $(\eta_s) = \frac{2723}{3300} \times 100 = 82.51\%$

iv) Water distribution efficiency $(\eta_d) = \left(1 - \frac{d}{D}\right) = \left(1 - \frac{110}{150}\right) = \frac{1.5 + 1.1}{2}$

$= 1.3$ m

where, D is the mean depth of water stored in the root zone.

d can be calculated as below.

Deviation from mean while at upper end (absolute value) $= |1.5 - 1.3|$

$= 0.2$

Deviation from mean while at lower end (absolute value) $= |1.5 - 1.3|$

$= 0.2$

d = Average of absolute value of deviation from mean

$= \frac{0.2 + 0.2}{2} = 0.2$

so, $\eta_d = \left(1 - \frac{0.2}{1.3}\right) = 0.846 = 84.6\%$

PROBLEM 16

If daily consumptive use of the crop is 5 mm and the canal may operates from 6 am to 5 pm only. Available moisture for the given soil IS 220 mm per m and maximum depth of root zone for the crop is 1.2 m. assume that only 50% of soil moisture is available to crop. Application efficiency is 65%. Calculate the required discharge if CCA is 450 ha. Calculate irrigation interval and outlet discharge. [2074 Bhadra]

Solution:

Here,

Root zone depth $= 1.2$ m

Moisture holding capacity of soil $= 220$ mm/m

$= 220 \frac{mm}{m} \times 1.2$

$= 264$ mm

$= 0.264$ m

Here, only 50% of soil moisture is available to crop.

∴ Readily available moisture (RAM) $=$ (50% of 0.264 m) $= 0.132$ m

Daily consumption of water $= 0.005$ m/day

Now,

$$\text{Irrigation interval} = \frac{\text{RAM}}{\text{Daily consumption}} = \frac{0.132}{0.005} = 26.4 \text{ days}$$

We have,

$$\text{Required volume of water} = \frac{132}{100} \times 450 \times 10^4 \text{ m}^3 \text{ in } 26.4 \text{ days}$$

so, Required discharge when canal operates from 6 A.M. to 5 P.M. is;

$$= \frac{132}{100} \times \frac{450 \times 10^4}{26.4 \times 11 \times 3600}$$

$$= 0.568 \text{ m}^3/\text{sec.}$$

$$\text{Outlet discharge} = \frac{0.5682}{0.65} \text{ m}^3/\text{sec.} = 0.874 \text{ m}^3/\text{sec.}$$

PROBLEM 17

Define irrigation water requirement for rice crop. [P.U. 2014]

Solution:

Water requirement of a crop is defined as the total quantity and the way in which a crop requires water, from the time it is sown to the time it is harvested. Water requirement for rice crops:

Delta (Δ) = 120 cm

Duty (D) = 775 hectares/cumec.

PROBLEM 18

A certain crop is grown in on area of 3000 hectares. Which is fed by canal? The data pertaining to irrigation as follows;
Field capacity soil = 26%
Optimum moisture = 12%
Permanent wilting point = 10%
Effective depth of root zone = 80 cm
Apparent relative density of soil = 1.4
If frequency of irrigation is 10 days and overall efficiency is 22%. Find:
i) the daily consumptive use
ii) water discharge in m^3/sec. required in canal [P.U. 2015]

Solution: See the solution of Q. no. 14

PROBLEM 19

Explain in brief design principles of sprinkler irrigation system. A certain crop is grown in an area of 2600 hectares which is fed by a canal system. The data pertaining to imitation are as follows;
Field capacity soil = 28%
Optimum moisture = 13%
Permanent wilting point = 10 %
Effective depth of root zone = 75 cm
Apparent relative density of soil = 1.45
If the frequency of irrigation is 10 days overall efficiency is 42%. Find then;
i) Daily consumptive use and,
ii) Water discharge in m^3/sec. required in canal feeding the area [P.U. 2015 chance]

Solution:

Design procedure of sprinkler irrigation system

Step 1 : Indentify resource concern and problem

Step 2 : Make an inventory of available resources and operating conditions. Include information of soils, topography, water supply (quantity and quality), source of power (type and located crops, irrigator's desire for a type of sprinkler system labor availability farm operation schedules and water management skills.

Step 3 : Determine soil characteristics and limitations

Step 4 : Determine net irrigation water requirements for crops to be grown

Step 5 : Determine irrigation frequency, net and gross application

Step 6 : Determine sprinkler spacing nozzle sizes, head type, discharge, operating pressure, wetted diameter, average application rate, and performance characteristics

Step 7 : Determine number of sprinkler in an irrigation set (zone) required to meet system capacity requirements, numbers of laterals needed for selected time of set spacing, moves and frequency of irrigation in days.

Step 8 : Evaluate design. Does it meet the objectives and purposed indentified in step 1

Step 9 : Finalize sprinkler irrigation system design layout and management skills.

Step 10 : Determine lateral size (s) based on number of head, flow rate, pipeline length, determine minimum operating pressure, etc

Step 11 : Determine main line sized required to meet pressure and flow requirements according to number of operating laterals.

Step 12 : Prepare final layout and operation, maintenance and irrigation water management plans.

Depth of water used by plants for growth, which is supplemented by

$$\text{irrigation after every 10 days} = \frac{1.4 \times 0.75}{1}(0.28 - 0.13)$$

$$= 0.1575 = 15.75 \text{ cm}$$

Daily water used by plants *i.e.*,

$$\text{Consumptive use} = \frac{15.75}{10} = 1.575 \text{ cm}$$

Total irrigational water required including losses in field and conveyance is;

$$= \frac{\text{NRI}}{\eta_{\text{irrigation}}} = \frac{1.575}{0.22} \text{ cm/day}$$

$$= 7.16 \text{ cm/day}$$

$$\text{Field area} = 2600 \text{ ha} = 2600 \times 10^4 \text{ m}^2$$

$$\text{Discharge required in irrigation canal} = \frac{7.16}{100} \times \frac{2600 \times 10^4}{24 \times 60 \times 60} \text{ m}^3/\text{sec.}$$

$$= 21.54 \text{ m}^3/\text{sec.}$$

PROBLEM 20

A reservoir has to supply irrigation water to 40,000 hectares land. Calculate the storage required in the reservoir to meet the irrigation demand of various crops as detailed below: [2076 Baishakh]

Crops	Base period (days)	Duty (ha/cumec)	Intensity of irrigation
Wheat	120	2400	20%
Rice	140	1000	15%
Maize	100	1800	20%

Solution:

Calculate area to be irrigated

For wheat = Intensity of irrigation × Gross command area

= 20% of 40000

= 8000 ha

For rice = 15% of 40000 = 6000 ha

For maize = 20% of 40000 = 8000 ha

Now, precede same as the solution of additional problem 25

PROBLEM 21

Following data is given below; calculate the depth and frequency of irrigation.

Field capacity = 80% | **PWP = 30%**
Root depth = 70 cm | **Soil density = 1.5 gm/cc**
Consumptive use = 4 mm/day | **Application efficiency = 80%**
Readily available moisture (RAM) = 70% **[2076 Bhadra]**

Solution: Proceed as the solution of problem 3

PROBLEM 22

Define: duty, delta, base period and crop period [2077 Chaitra]

Solution: See the definition part 2.1 and 2.1.1

PROBLEM 23

After how many days will supply water to soil in order to ensure sufficient irrigation of the given crop if.

i) Field capacity of the soil = 28%

ii) Permanent wilting point = 13%

iii) Dry density of soil = 1.3 gm/cc

iv) Effective depth of root zone = 70 cm

v) Daily consumptive use of water for given crop = 12 mm. [2077 Chaitra]

Solution:

Assume readily available moisture (RAM) = 75% of available moisture content (AMC) and precede same as problem no. 3.

PROBLEM 24

Define frequency of irrigation and how do you calculate frequency of irrigation on the basis of soil moisture? Compute the flow discharge needed of a canal to irrigate dry season crops in 20,000 ha and wet season crops in 30000ha. Kor period and kor depth for dry and wet season crops are 5 weeks and 12.5 cm and 4 weeks and 9.5 cm respectively. [2078 Baishakh]

Solution:

The frequency of irrigation is defined as the number of days or time internal between irrigation.

The irrigation frequency can be calculated by following formula.

$$F = \frac{Y_d \times d \times RAM}{Y_W \times ET_C}$$

Here, d = Root zone depth

RAM = Readily available moisture content

ET_C = Crop water use rate or crop evapo-transpiration

RAM = Certain percent of AMC

AMC = Field capacity (FC) – Permanent wilting point (PWP)

For numerical part

Precede same as solution of problem no. 7

2.10 ADDITIONAL PROBLEMS

PROBLEM 1

After how many days will you supply water to the soil in order to ensure sufficient irrigation of the given crop if, FC = 28%, PWP = 13%, dry density of the soil = 1.3 gm/cc, daily consumption of water for the given crop = 12 mm, Effective depth of root zone = 70 cm.

Solution:

Available moisture content (AMC) = 28 − 13 = 15%

Let, RAM = 80%of AMC = 0.8 × 15 = 12

∴ OMC = 28

∴ Depth of stored water in root zone between 16% and 18% $= \frac{\gamma_d d(28-16)}{\gamma_w}$

$= \frac{0.7 \times 1.3 \times (0.28-0.16)}{1}$

= 0.109 m

= 10.92 cm

∴ Water available for consumptive use = 10.9 cm

Irrigation interval (II) $= \frac{10.92}{1.2} = 9.1$ days

Water should be applied in every 9 days.

Depth of water to applied = 9 × 1.2 = 10.8 mm

PROBLEM 2

After how many days you will supply water to a soil in order to ensure sufficient irrigation of the given crop if FC of the soil is 30%, PWP = 12%, density of the soil = 1.25 gm/c.c., effective depth of the root zone (d) = 60 cm, daily consumptive uses of water for the given crop ET_{crop} = 12.5 mm. Assume that is 80% of AMC.

Solution:

AMC = FC − PWP = 30 − 12 = 18%

Now,

RAM = 80% of AMC = 0.8 × 18 = 14.4%

Hence,

Depth of water available for consumptive use $= \frac{\gamma_d d \times RAM}{\gamma_w}$

$= \frac{1.25 \times 0.6 \times 0.144}{1}$

= 0.108 m = 108 mm

Irrigation frequency $= \frac{108}{12.5} = 8.64$ days

Water should be applied in every 8 days with equivalent water depth is;

= 8 × 12.5 mm = 100 mm

PROBLEM 3

Calculate the depth of water and interval of irrigation (water requirement) FC = 25%, PWP = 10%, γ_s = 1.451 gm/c.c., ET_{crop} = 0.1 mm/day. Root zone depth = 10 cm, assume irrigation is to be applied when soil moisture is applied to PWP.

Solution:

AMC = FC − PWP = 25 − 10 = 15%

According to the question irrigation is done after the moisture is depleted to PWP. So, AMC is consumed.

Depth of water available for consumptive use $= \frac{\gamma_d d \times AMC}{\gamma_w}$

$= \frac{1.55 \times 10 \times 0.15}{1}$

= 2.33

Irrigation interval $= \frac{2.33}{0.1} = 23.3$

∴ Irrigation should be done in each 23 days with water depth = 23 × 0.1

= 2.3 m

PROBLEM 4

Find FC of the soil with the following data, Root zone depth = 2.5 m, Existing water content = 7.5%, Dry density of the soil = 1.5 gm/c.c., water applied to the soil = 600 m³, Water loses due to evaporation = 15% area of plot = 1000 m².

Solution:

Volume of water consumed by plants = 0.85 × 600 = 510 m³

Hence,

Equivalent depth $= \frac{510}{1000} = 0.51$ m

But, $0.51 = \frac{\gamma_d d \times (FC - AMC)}{\gamma_w} = \frac{1.5 \times 2.5 \times (FC - 0.075)}{1}$

∴ FC = 0.211 = 21.1%

PROBLEM 5

Compute the frequency and depth of the irrigation if root zone depth = 100 cm, FC = 22%, efficiency of irrigation = 50%, γ_d = 1.5 gm/c.c., ET_{crop} = 25 mm/c.c., assume 50% depletion of available moisture before application of irrigation water at FC.

Solution:

AMC = FC − PWP = 22 − 12 = 10%

RAM = 0.5 × AMC = 0.5 × 10% = 5%

Depth of water available for consumptive use $= \frac{\gamma_d d \times RAM}{\gamma_w}$

$= \frac{1.5 \times 1 \times 0.05}{1}$

$= 0.075 \text{ m} = 75 \text{ mm}$

Irrigation interval $= \frac{75}{1 \times 25} = 3$ days

Depth of water $= \frac{75}{0.5} = 150$ mm

PROBLEM 6

With following data FC = 80%, PWP = 35%, root depth = 60 cm, soil density of the soil = 1.5 gm/c.c., ETC = 5 mm/day application efficiency = 80%, conveyance loss = 55%, and distribution loss = 65%. Calculate;
i) Available moisture content
ii) Readily available moisture content
iii) Depth of irrigation at the outlet of the field
iv) Irrigation interval
v) the irrigation requirement at the head works

Solution:

i) AMC = 80% − 35% = 45%

ii) RAM = 80% of AMC = 0.8 × 45% = 36%

iii) Depth of water available for consumptive use $= \frac{1.5 \times 0.6 \times 0.36}{1}$

$= 0.324$ m

$= 324$mm

Depth of water required at outlet $= \frac{324}{0.8} = 405$ mm

iv) Irrigation interval $= \frac{324}{5} = 64.8$ days

Irrigation water required at head works $= \frac{405}{0.45 \times 0.35} = 2571.43$ mm

PROBLEM 7

10 cumecs of water is delivered to a 32 Ha field, for 4 hrs. Soil prob in after the irrigation indicator that 0.3 mm of water has been started that 0.3 mm of water has been started in the root zone. Compute the water application efficiency.

Solution:

Volume of water applied $= 10 \times 4 \times 3600 \text{ m}^3$

$= 144000 \text{ m}^3$

$= \frac{144000}{10000}$

$= 14.4$ Ha-m

∴ Input = 14.4 Ha-m

Volume of water stored in root zone = 0.3 × 32

= 9.6 Ha-m

∴ Efficiency $= \frac{9.6}{14.4} \times 100\% = 66.67\%$

PROBLEM 8

A CCA for distributary is 15000 Ha. The intensity for rice y of irrigation is 40% for Rabi and 10% for rice. Determine the outlet discharge factor for Rabi and Rice may be assured to be 1800 Ha/cumecs and 775 Ha/cumecs.

Solution:

$$Q_{rabi} = \frac{A_{irrigable}}{D_{rabi}} = \frac{CCA \times II_{rabi}}{1800} = \frac{15000 \times 0.4}{1800} = 3.33 \text{ m}^3/\text{sec.}$$

$$Q_{rice} = \frac{A_{irrigable}}{D_{rice}} = \frac{CCA \times II_{rice}}{775} = \frac{15000 \times 0.1}{775} = 1.93 \text{ m}^3/\text{sec.}$$

PROBLEM 9

Water is released at the rate of 10 cumecs at H/R. If duty at the field is 125 Ha/cumecs and loss of water in transit is 30%. Find the area of the land that can be irrigated.

Solution:

Discharge reaching the field = 0.7 × 10 = 7 cumecs

∴ Area that can be irrigated = 7 × 125 = 875 Ha

PROBLEM 10

The gross area of an irrigation project is 50,000 Ha. Out of this 5000 Ha have been utilized for construction of dwellings roads, bridges, etc. The area to be cultivated during Rabi is 25000 Ha and during summer is 24000 Ha. The duty of canal water for winter crop is 5000 Ha/cumecs. Find the design discharge for the canal after leaving 10% allowance for peak discharge and loss of water in transit. What would be the annual intensity of irrigation?

Solution:

CCA = 50,000 − 5,000 = 45,000 Ha

$$Q_{winter} = \frac{25000}{5000} = 5 \text{ m}^3/\text{sec.}$$

$$Q_{summer} = \frac{24000}{3000} = 8 \text{ m}^3/\text{sec.}$$

∴ Design discharge $= 1.1 \times 8 = 8.8 \text{ m}^3/\text{sec.}$

$$II_{annual} = \frac{(24000 + 25000) \times 100\%}{45000} = 108.9\%$$

PROBLEM 11

A water course has a CCA of 1500 Ha .II for crop A is 45% and for B is 40%, both the crop being Rabi crops. Crop A has KOR period of 20 days and B has 15 days. Calculate the discharge of w/c if the Kor depth for crop A and B are 10 cm and 16 cm respectively.

Solution:

For crop A;

II = 45%

$$\text{Kor discharge} = \frac{II \times CCA \times \text{Kor depth}}{\text{Kor period}} = \frac{0.45 \times 1500 \times 0.1}{20}$$

$= 3.375$ Ha-m/day $= 0.391\ m^3/sec.$

For crop B;

II = 40%

$$\text{Kor discharge} = \frac{\text{II} \times \text{CCA} \times \text{Kor depth}}{\text{Kor period}} = \frac{0.4 \times 1500 \times 0.16}{15}$$

$= 0.4$ Ha-m/day

$= 0.741\ m^3/sec.$

Hence,

Total discharge of canal $= 0.391 + 0.741 = 1.132\ m^3/sec.$

PROBLEM 12

The gross commanded area for a distributary is 6000 hectares, 80% of which is culturable irrigable. The intensity of irrigation for Rabi season is 50% of that for Kharif season is 25%. If average duty at the head of the distributary is 2000 hectares/cumecs for Rabi season and 900 hectares/cumecs for Kharif season. Find out the discharge required at the head of the distributary from average demand considerations.

Solution:

$$\text{GCA} = 6000 \text{ hectares} = \frac{6000 \times 80}{100} = 4800 \text{ ha}$$

$= \text{CCA} \times \text{Intensity of irrigation}$

$$= \frac{4800 \times 50}{100}$$

$= 2400$ hectares

$$\text{Area to be irrigated in Kharif season} = 4800 \times \frac{25}{100} = 1200 \text{ hectares}$$

$$\text{Water required at head of distributary to irrigate Rabi area} = \frac{2400}{2000}$$

$= 1.2$ cumecs

$$\text{Water reqd. at head of distributary to irrigate Kharif area} = \frac{1200}{900} \text{ cumecs}$$

$= 1.33$ cumecs

Thus, the required discharge is maximum of this two *i.e.,* 1.33. Hence, the distributary should be designed for 1.33 cumecs.

PROBLEM 13

Determine the field capacity of a soil for the following data; Depth of root zone = 1.8 m, Existing moisture = 8%, Quantity of water applied to soil = 650 m3/s, Dry density of soil = 1450 kg/m³, Water lost due to deep percolation and evaporation = 10%, Area to be irrigated = 1000 m³.

Solution:

Here,

Volume of total water applied $= 650\ m^3$

Water wasted $= 10\%$ of $650\ m^3 = 65\ m^3$

Water used in raising m.c. up to the field capacity $= 650 - 65 = 585\ m^3$

Depth of water used in the raising m.c. up to field capacity from the existing $8\% = \frac{585}{1000} = 0.585\ m^3$

But, Water reqd. in root zone depth to increase m.c. $= \frac{\gamma_d}{\gamma_w}$(Upper limit m.c. −Lower limit m.c.)

or, $0.585 = \frac{1450}{1000} \times 1.8(\text{F.C.} - 0.08)$

or, $\text{F.C.} - 0.08 = 0.224$

or, $\text{F.C.} = 0.304 = 30.4\%$

Hence, field capacity is 30.4%.

PROBLEM 14

A certain crop is grown in an area of 3000 Ha, which is fed by a canal system. The data pertaining to irrigation is Field capacity = 26%, Optimum moisture content = 12% Permanent wilting point = 10%, Effective depth of root zone = 80 cm Apparent relative density of soil = 1.4. If the frequency of irrigation is 10 days and the overall irrigation efficiency is 22%, find;

i) daily consumptive use

ii) the water discharge in m³/sec. required in the canal feeding the area

Solution:

Depth of water used by plants for growth, which is supplemented by irrigation after every 10 days $= \frac{1.4 \times 0.8}{1}(0.26 - 0.12) = 0.1568 = 15.68$ cm

Daily water used by plants *i.e.,* consumptive use $= \frac{15.68}{10} = 1.568$ cm

Total irrigation water required including losses in field and conveyance is;

$$= \frac{\text{NRI}}{\eta_{\text{irrigation}}} = \frac{1.586}{0.22} \text{ cm/day} = 7.21 \text{ cm/day}$$

Field area $= 3000 \text{ Ha} = 3000 \times 10^4\ m^2$

$$\text{Discharge required in irrigation canal} = \frac{7.21}{100} \times \frac{3000 \times 10000}{24 \times 60 \times 60} m^3/sec.$$

$= 25\ m^3/sec.$

PROBLEM 15

If wheat required 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:

Assuming the base period is equal to crop period.

Now,

$$\text{Number of watering required} = \frac{140}{28} = 5$$

The depth of water required for each time = 7.5 mm

∴ Total depth of water required in 140 days $= 5 \times 7.5 = 37.5$ cm

Hence, delta for wheat is 37.5 cm.

PROBLEM 16

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

Solution:

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

where, 'B' in days and 'D' in hectares/cumecs.

Here,

$B = 120$ days

$D = 864$ hectares/cumecs

$$\therefore \quad \Delta = \frac{864 \times 120}{864} = 120 \text{ days}$$

PROBLEM 17

After how many days will you apply water to the soil in order to ensure sufficient irrigation of the given crop, if field capacity of the soil = 28%, Permanent wilting point = 13%, dry density of soil = 1.3 gm/c.c., Effective depth of root zone = 70 cm, Daily consumptive use of water for the given crop = 12 mm, assume necessary data if required.

Solution:

Available moisture = Field capacity – Permanent wilting point

= 28 % – 13% = 15%

Assuming readily available moisture is 80% of available moisture.

∴ Readily available moisture = 0.8 × 15% = 12%

Optimum moisture = 28 – 12 = 16%

It means the moisture will be filled by irrigation between 16% and 28%.

Depth of water stored in root zone between these two limits is;

$$= \frac{\gamma_d d}{\gamma_w}(\text{Field capacity m.c.} - \text{Optimum m.c.})$$

$$= \frac{1.3 \times 0.7}{1}(0.28 - 0.16)$$

$$= 0.129 \text{ m} = 10.92 \text{ cm}$$

Hence,

Water available for evapo-transpiration = 10.92 cm

1.2 cm of water utilized by plant in 1 day.

$$\therefore \quad 10.92 \text{ cm of water will be utilized by plant in } \frac{1 \times 10.92}{1.2} \text{ days} = 9.1 \text{ days}$$

Hence, after 9 days water should be supplied to the given crop.

PROBLEM 18

Wheat is to be grown in a field having a field capacity equal to 27% and the permanent wilting point is 13%. Find the storage capacity in 80 cm depth of the soil if the dry unit weight of the soil is 14.72 kN/m³. If irrigation water is to be supplied when the average soil moisture falls to 18%, find

the water depth required to be supplied to the field if the field application efficiency is 80%. What is the amount of water needed at the out let if the water lost in the water courses and the field is 15% of the out let discharge.

Solution:

Maximum storage capacity or moisture available is given by;

$$= \frac{\gamma_d d}{\gamma_w}(\text{Field capacity m.c.} - \text{Optimum m.c.})$$

$$= \frac{14.72 \times 0.8}{9.81}(0.27 - 0.13)$$

$$= 0.168 \text{ m} = 16.8 \text{ cm}$$

Since, moisture is allowed to vary between 27% to 18%;

i.e., Deficiency created in this fall $= \frac{14.72 \times 0.8}{9.81}(0.27 - 0.18)$

$= 0.108 \text{ m} = 10.8 \text{ cm}$

Quantity of water required to supply in field $= \frac{NRI}{\eta_a} = \frac{10.8}{0.8}$

$= 13.5 \text{ cm}$

Quantity of water needed at the canal outlet $= \frac{FRI}{\eta_c} = \frac{13.5}{0.85} = 15.88 \text{ cm}$

PROBLEM 19

During a particular stage of the growth of a crop, consumptive use of water 2.5 mm/day. Determine the interval in days between irrigation, and the depth of water to be applied when the amount of water available in the soil is 50% of the maximum depth of available water in the root zone which is 80 mm. Assume irrigation efficiency to be 60%.

Solution:

Readily available moisture in root zone of crop = 50% × 80 mm = 4 cm

Consumptive use = 2.5 mm/day = 0.25 cm/day

0.25 cm of water is consumed by crop in = 1 day

4 cm of water will be consumed by crop in $= \frac{1}{0.25} \times 4 = 16$ days

Depth of water recouped through irrigation = 4 cm

Now,

$$\text{Actual water required to be applied to field} = \frac{\text{Water required to be applied or absorbed}}{\text{Efficiency of water application}}$$

$$= \frac{4}{60}\% = \frac{4 \times 100}{60}$$

$$= 6.67 \text{ cm}$$

Hence, 6.67 cm depth of irrigation water shall have to be applied to the fields at an interval of 16 days.

PROBLEM 20

FC = 80% **PWP = 35%**
Root depth = 60 cm **Soil density = 1.5 gm/cc**
ET_c = 5 mm/day **Application efficiency = 80%**
RAM = 70% of AMC where, abbreviations have usual meanings. Calculate;
i) available moisture contents
ii) readily available moisture content
iii) depth of irrigation at the outlet of the field
iv) irrigation interval

Solution:

i) Available moisture content = FC – PWP = 80 – 35 = 45%

ii) Readily available moisture content = 70% of AMC

$$= \frac{70}{100} \times 45 = 31.5\%$$

iii) Depth water available for consumptive use $= \frac{\gamma_d \times d}{\gamma_w} \times \text{RAM}$

$$= \frac{1.5 \times 0.6}{1} \times 0.315$$

$$= 0.2835 \text{ m} = 28.35 \text{ mm}$$

Depth of water $= \frac{28.35}{0.8} = 35.44$

iv) Irrigation interval $= \frac{28.35}{5} = 5.67$ days

PROBLEM 21

A water course commands an irrigation area of 800 hectares. The intensity of irrigation in this area is 50%. The transplantation of rice crop takes 15 days and total depth of water required by crop is 60 cm on field during transplantation period, given that the rain falling on the field during this period is 15 cm.

i) Find duty of irrigation water for crop on field during transplantation.
ii) At the head of the distributary, assuming losses of water to be 20% in the water course.
iii) Calculate the discharge required in water course.

Solution:

Given that;

Commanded area = 800 hectares

Intensity of irrigation = 50%

Area of rice irrigated = 50% of 800 = 400 hectares

Depth of irrigation water = 60 – 15 = 45 cm

Duty of water on field $= \frac{8.64\text{ B}}{\Delta} = \frac{8.64 \times 15}{0.45} = 288$ hectare/cumec

Here, conveyance loss is 20%. Hence, a discharge of 1 cumec at the head of water course becomes 0.8 cumec at the field.

∴ Duty at head of water course (D) = 288 × 0.8 = 230.4 ha/cumec

Discharge required in water course $= \frac{400}{230.4} = 1.736$ cumec

PROBLEM 22

Determine the discharge of a distributary at the tail end from following data:
Gross commanded area = 20000 ha
Culturable commanded area = 70% GCA
Losses beyond the tail end = 1.0 cumec
Kharif (rice) Intensity of irrigation = 15%
K_{or} depth = 19 cm
K_{or} period = 2.5 weeks
Rabi (wheat) Intensity of irrigation = 30%
K_{or} depth = 13.5 cm
K_{or} period = 4.0 weeks
Sugarcane intensity of irrigation = 10%
K_{or} depth = 16.5 cm
K_{or} period = 4 weeks

Solution:

For Kharif;

Area irrigated = 0.15 × 0.7 × 20000 = 2100 ha

Outlet discharge factor (duty) $= \frac{8.64 \times (2.5 \times 7)}{0.19} = 796$ ha/cumec

Outlet discharge $= \frac{2100}{796} = 2.64$ cumec

For Rabi;

Area irrigated = 0.3 × 0.7 × 20000 = 4200 ha

Outlet discharge factor $= \frac{8.64 \times (4 \times 7)}{0.135} = 1792$ ha/cumec

Outlet discharge $= \frac{4200}{1792} = 2.34$ cumec

For sugarcane,

Area irrigated = 0.1 × 0.7 × 20000 = 1400 ha

Outlet discharge factor $= \frac{8.64 \times (4 \times 7)}{0.165} = 1466$ ha/cumec

Outlet discharge $= \frac{1400}{1466} = 0.95$ ha/cumec

Sugarcane usually requires irrigation along with Rabi;

Total outlet discharge = 2.34 + 0.95 = 3.29 cumec

Discharge required at tail end of distributor = 3.29 + 1.00 = 4.29 cumec

PROBLEM 23

The gross commanded area for a distributary is 6000 ha and culturable commanded area is 3600 ha. The intensity of irrigation for Kharif is 30% and that for Rabi is 60%. If the average duty at the head of distributor is 700 ha/cumec for Kharif and 1500 ha/cumec for Rabi. Determine discharge at head.

Solution:

Duty for Kharif = 700 ha/cumec

Area of Kharif irrigated = 30% of 3600 ha = 1080 ha

$$\text{Discharge for Kharif} = \frac{1080}{700} = 1.54 \text{ cumec}$$

Duty for Rabi = 1500 ha/cumec

Area of Rabi irrigated = 60% of 3600 ha = 0.6 × 3600 ha = 2160 ha

$$\text{Discharge for Rabi} = \frac{2160}{1500} = 1.44 \text{ cumec}$$

Hence, discharge at the head is 1.44 cumec.

PROBLEM 24

The culturable commanded area of a channel is 10000 ha. The intensity of irrigation is 35% for Rabi and 15% for Kharif (rice). If the K_{or} period is 4 weeks for former and $2\frac{1}{2}$ week for later, determine the discharge. Take K_{or} depth as 13.5 cm and 20 cm for the Rabi and Kharif crops, respectively.

Solution:

$$\text{Duty for Rabi} = \frac{8.64 \times (4 \times 7)}{0.135} = 1792 \text{ ha/cumec}$$

$$\text{Duty for Kharif} = \frac{8.64 \times (2.5 \times 7)}{0.2} = 756 \text{ ha/cumec}$$

Area of Rabi irrigated = 0.35 × 10000 ha = 3500 ha

$$\text{Discharge for Rabi} = \frac{3500}{1792} = 1.95 \text{ cumec}$$

Area of Kharif irrigated = 0.15 × 10000 ha = 1500 ha

$$\text{Discharge for Kharif} = \frac{1500}{756} = 1.98 \approx 2$$

Hence, out let discharge is 2 cumec.

PROBLEM 25

The base period, intensity of irrigation of various crops under a canal irrigation system is given in the table below. Find the reservoir capacity if the canal losses are 18% and reservoir losses is 14%. [2071 Bhadra: T.U.]

Crop	Base period (days)	Duty at field (Ha/cumecs)	Area under the crop (hectares)
Rice	120	850	3000
Wheat	120	1700	4500
Sugarcane	360	750	5400
Vegetables	120	650	1200
Cotton	200	1300	2200

Solution:

Here,

$$\text{Water required for rice} = \frac{\text{Area}}{\text{Duty}} = \frac{3000}{850} = 3.529 \text{ cumecs}$$

$$\text{Water required for wheat} = \frac{\text{Area}}{\text{Duty}} = \frac{4500}{1700} = 2.647 \text{ cumecs}$$

$$\text{Water required for sugarcane} = \frac{\text{Area}}{\text{Duty}} = \frac{5400}{750} = 7.2 \text{ cumecs}$$

$$\text{Water required for vegetables} = \frac{\text{Area}}{\text{Duty}} = \frac{1200}{650} = 1.846 \text{ cumecs}$$

$$\text{Water required for cotton} = \frac{\text{Area}}{\text{Duty}} = \frac{2200}{1300} = 1.692 \text{ cumecs}$$

Now,

Water volume required by rise in 120 days of crop period is given by;

$= 3.529 \times (120 \times 24 \times 3600) = 36588672 \text{ m}^3$

Water volume required by wheat in 120 days of crop period is given by;

$= 2.647 \times (120 \times 24 \times 3600) = 27444096 \text{ m}^3$

Water volume required by sugarcane in 360 days of crop period is given by;

$= 7.2 \times (360 \times 24 \times 3600) = 223948800 \text{ m}^3$

Water volume required by vegetables in 120 days of crop period is given by;

$= 1.846 \times (120 \times 24 \times 3600) = 19139328 \text{ m}^3$

Water volume required by cotton in 200 days of crop period is given by;

$= 1.692 \times (200 \times 24 \times 3600) = 19139329 \text{ m}^3$

∴ Total water volume required by all five crops $= 36588672 + 27444096 + 223948800 + 19139328 + 19139329$

$= 124706225 \text{ m}^3$

$= 124.71 \text{ Mm}^3$

Considering 18% canal losses and 14% reservoir losses; we have,

$$\text{Capacity of reservoir} = \frac{124.71}{0.82 \times 0.86} = 176.82 \text{ Mm}^3$$

PROBLEM 26

Define GCA, CCA, NCA, base period and crop period. Derive a relation between delta and duty. [P.U. 2015]

Solution: See the definition part 1.4 and 2.1.2

PROBLEM 27

How can you define irrigation? Mention the major advantages of irrigation. Explain briefly the factors affecting crop, water requirement. [P.U. 2015]

Solution:

The term irrigation can be defined as the artificial application of water to the land for purpose of raising the crops. A crop requires a certain amount of water at some fixed interval throughout its period of growth. The basic objectives of irrigation are to supplement water to the land so as to obtain an optimum crop yields.

Following are the some advantages of irrigation:

i) Increase in food production
ii) Elimination of mixed cropping
iii) General prosperity
iv) Domestic water supply
v) Facilities of communications (inspection acts as a link road for different village)
vi) Inland irrigation
vii) Afforestation
viii) Flood control
ix) Insurance against drought

The crop water requirements is defined as the total quantity and the way in which a crop requires water, from time it is shown to the time it is harvested. The water requirements will vary with crop as well as with the place.

The following ate the factors which effect on the water requirement of the crops;

i) Influence of climate
ii) Type of crop
iii) ground water table
iv) Ground slope
v) Intensity of irrigation
vi) Conveyance losses
  a) Type of soil
  b) Subsoil water
  c) Age of canal
  d) Position of F.S.L. with respect to N.S.L.
  e) Amount of silt came by canal
  f) Wetted perimeter
vii) Method of application of water
viii) Method of ploughing
ix) Crop period
x) Base period
xi) Delta of crop

I) Influence of climate

In hot climate the evaporation loss is more and the water requirement will be more and vice versa. Major climate factor which affect on crop water requirements are sunshine, temperature, humidity and wind speed.

Climate factor	Crop water need	
	High	Low
Sunshine	Sunny (no clouds)	Cloudy (no sun)
Temperature	Hot	Cold
Humidity	Low (dry)	High (humid)
Wind speed	Windy	Little wind

Table: Effect of major climatic factors on crop water needs

II) Influence of crop type of crop water needs

Different crops require different amount of water for maturity.
According to amount of water required plant can be classified into:

a) Low water requirements plants

Plants that require low levels of water are often called drought tolerant plants. *For example;* red cedar, live oak, crape myrtle and citrus trees

b) Mid-level water requirement plants

Most of plants can be categories in this group. These plants do not need to be water every day but they need to be watered when the soil has been dry for over a week or two. *For example;* geraniums, most roses, wisteria, sunflower and most flowering perennial shrubs

c) High water requirement plants

Some plants requires large amount of waters. These plants grow in marshy areas on bags or along the banks of river, streams and lakes. *For example;* iris plant, cannas, bee balms, ferns, etc.

Crop	Water requirements (mm)
Rice	900-2500
Wheat	450-650
Sorghum	450-650
Maize	500-800
Sugarcane	1500-2500
Cotton	700-1300
Tomato	600-800
Soyabean	450 -700
Potato	500-700

Table: Total water requirement of crops

iv) Water table

If water table is nearer to the ground source the water requirement will be less and vice versa.

v) Ground slope

If the slope of ground is steep the water requirement will be more intensity greater will be the water required for a particular crop.

vi) Intensity of irrigation

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

vii) Conveyance losses

Take place from canal intake to field (outlet). So design should be according to requirement of water plus losses.

viii) Method of application of water

In sprinkle method less water is required as it just moist the soil like rain water, whereas in flooding method more water is required.

ix) Method of ploughing

In deep ploughing less water is required and vice versa.

x) Crop period

It is time normally in days that a crop takes from instance of sowing to harvesting.

xi) Base period

It is the time in days between first watering and last watering to the crop before harvesting.

xii) Delta of crop

Total depth of water required by the crop in unit area during base period. In another words it is the total depth of water required for maturing crop.

$$\text{Volume} = \text{Depth} \times \text{Area}$$

$$\text{Volume} = \frac{\text{Volume}}{\text{Time}}$$

CHAPTER 3
CANAL IRRIGATION SYSTEM

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3.1 CLASSIFICATION OF CANAL

According to function; canal is classified into;

i) Irrigation canal

It carries water directly to the agricultural fields.

ii) Navigation canal

The canal which is utilized for navigation besides doing irrigation.

iii) Power canal

The canal which is utilized for power generation.

iv) Feeder canal

It is constructed with the idea of feeding two or more canal.

v) Carrier canal

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

Based on financial output, canal is classified into;

i) 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.

ii) Protective canal

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

Based on nature of source, canal is classified into;

i) 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. Permanent canal taken off from the source which is dry for part of the year is called seasonal permanent canal.

ii) Inundation canal

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

Based on discharge and relative importance;

i) Main canal
ii) Branch canal (Discharge > 5 cumec)
iii) Major distributary (Discharge $\frac{1}{4}$ to 5 cumec)
iv) Minor distributary (Discharge $< \frac{1}{4}$ cumec)
v) Field canal

Based on canal alignment;

i) Watershed canal
ii) Contour canal
iii) Side slope canal

Based on lining;

i) Unlined canal
ii) Lined canal

3.2 COMPONENTS OF CANAL IRRIGATION SYSTEM

The components of canal irrigation system are as follows;

i) Head work

The headworks comprise of all the works to store divert and controls the river water and regulate supplies into the canal.

ii) Canal network

It may consist of main canal branch canal, distributaries, water canals etc.

a) Main (major) canal

It is the channel taking off from the head work. No direct irrigation is usually carried out from the main canal.

b) Branch canal

When a main canal reaches the area which is to be irrigating, it gets divided into branches going to different parts of the area. These branches are called branch canal. In general branch canal also do not carry direct irrigation. Branch canal are usually feeder canal for distributaries. They usually carry a discharge of over 5 cumecs.

c) Major distributaries

These are small channels, taking off from the branch canals or sometimes from main canal. They supplies water through outlets to water courses for direct irrigation. Their discharge varies from 0.25 to 5 cumecs.

d) Minor distributaries

Their discharge is usually less than 0.25 cumecs. They supplies water to the water courses through the outlets provided along them.

e) Water courses

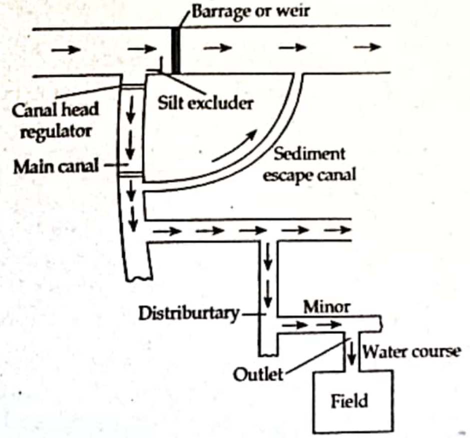
It is small channel connecting outlets to fields. Farmer normally constructs it.

iii) Structures in canals

It consists of cross drainage structures, canal falls, canal escapes, cross regulators, distributary head regulators, outlets, etc.

a) Cross drainage structures

These are structures constructed at the crossing of canal and natural drainage so as dispose of drainage water takes place without interrupting continuous canal supply.



b) Canal fall

Whenever the available natural ground slope is steeper than the design slope the difference is adjusted by a fall structures called canal drop or canal fall.

c) Cross regulators

It is a structure provided on the parent channel just downstream of the off take point of the off taking channel to raise water level in the parent channel so that the full supply can be taken into off taking channel even when the parent channel is running full.

d) Distributary head regulator

A distributary head regulator is provided at the head of each distributor and branch canal.

e) Outlet

An outlet is usually pipe embedded in the bank of channel with discharging capacity in proportion to the area to be controlled from that canal.

3.3 ALIGNMENT OF CANAL

Canal has to be aligned in such a way that it covers the entire area proposed to irrigated with shortest possible length and at the same time its cost including the cost of cross drainage is minimum. The alignment of canal should be done on the basis of following approach.

i) The alignment should be done such that (1) Minimum number of C/D works (2) Most economical way of distributing the water to the land (3) as high command area as possible.
ii) The alignment of canal on watershed is preferred since it is most economical.

iii) The length of main canal should be minimum between two points.
iv) Alignment should avoid villages, roads, cart tracks, cremation places, places of worship and other valuable properties.
v) Ideal length of canal should be minimum and branches should be economically planned.
vi) The alignment should not be made in a salty rocks or cracked strata.

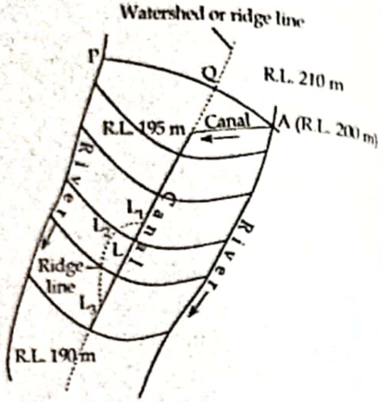


Figure: Ridge canal

Based on alignment of canal irrigation, canal can be of following three ways.

I) Watershed or ridge canal

Watershed or ridge line is highest line between two drainage areas. It is aligned along a watershed or runs for most of its length on watershed. When a channel is on the watershed it can command areas on both banks.

Advantages

1. It can command areas on both banks and so large area can be brought under cultivation.
2. No drainage can intersect watershed and hence the necessary of construction of cross drainage structures are avoided.

Disadvantages

1. The depression in the ridge line may also necessitate construction of canal bridge, siphons.
2. In head reach it is not economical to align in ridge.
3. If watershed is passing through village or towns this canal may have to leave the watershed for some distance.

II) Contour canal

A channel aligned parallel to contour except for the necessary longitudinal slope is called a contour canal. Such type of canal is constructed in hilly areas.

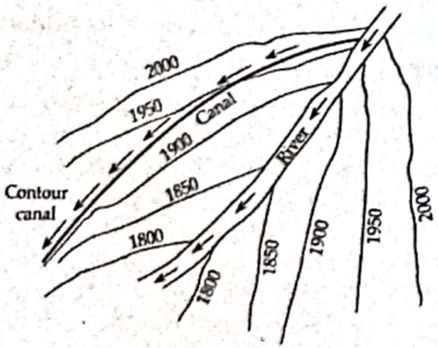


Figure: Contour canal

Advantages

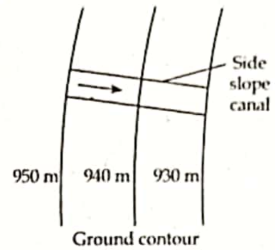
1. It is economical than ridge canal when ridge is very higher than river flows.
2. It is suitable for contour farming.
3. It is suitable for hilly region.

Disadvantages

1. It irrigates only one side of the canal so serve small area.
2. As the drainage flow is always right angle to contours such a canal would have more cross drainage structures.

III) Side slope canal

It is a channel aligned roughly at right angle to contours. Such a channel is roughly parallel to the natural drainage and hence does not intercept cross drainage and hence no cross drainage structure are required.



3.4 ALLUVIAL AND NON–ALLUVIAL CANALS

Alluvial canal

The soil, which is formed by transportation and deposition of silt through the agency of water, over a course of time, is called the alluvial soil. The canals when excavated through such soils are called alluvial canals. Canal irrigation (direct irrigation using a weir or a barrage) is generally preferred in such areas, as compared to the storage irrigation (*i.e.*, by using a dam).

Non–Alluvial canal

Mountainous regions may go on 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 do not pose much problems for designing irrigation structures on them. Canals, passing through such areas are called non-alluvial canals.

3.5 CANAL STANDARDS AND BALANCING DEPTH

Balancing depth

For a given canal section, the depth of cutting for which area cutting is equal to area of filling is called balancing depth.

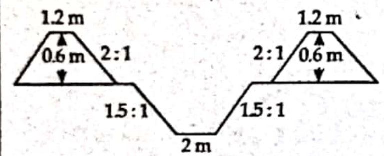
S.N.	Items	Size of canal					
		Minor distributaries		Major distributaries		Main and branch canals	
		Below 0.3 cumecs	Below 0.3 to 1 cumecs	1 to 5 cumecs	5 to 10 cumecs	10 to 30 cumecs	30 to 150 cumecs
1.	Maximum width of the bank crest	1	1.5	2	2.25	2.5	3
2.	Width of roadway	Nil	3.5	3.5	5	5	6
3.	Free board	0.3	0.4	0.5	0.6	0.75	0.9
4.	Depth of earth cover over saturation gradient	0.5	0.5	0.5	0.5	0.8	1

5.	Width of berms	Maximum berm width = 0.6 m + $\left(\frac{1}{4}\right)^{th}$ of the width of combined side slopes of cutting and embankment. Maximum berm width = 0.6 m + $\left(\frac{1}{2}\right)^{th}$ of the width of combined slopes		
6.	Width of land to be acquired (clear of banks) when canal is cutting is	Half the height of the bank above ground, subject to minimum of 1.5 m		
7.	Width of land to be acquired (clear of banks) when canal cutting is lesser than the balancing depth	Full height of bank above ground level plus 1.5	Full height of bank + 5 m	Full height of bank + 5 m

Table3.1: Canal standa

Example 3.1

Refer the canal section as given below.



Solution:

Area of cutting = $(2 + 1.5 \times h)\, h$

Area of filling = $(1.2 + 2 \times 0.6)\, 0.6 \times 2$

For balancing depth; we have,

Area of cutting = Area of filling

or, $(2 + 1.5 \times h)\, h = (1.2 + 2 \times 0.6)\, 0.6 \times 2$

or, $2h + 1.5h^2 = 1.44 + 1.44$

or, $1.5h^2 + 2h - 2.88 = 0$

$\therefore$ $h = 0.87$ m

Hence, balancing depth is 0.87 m.

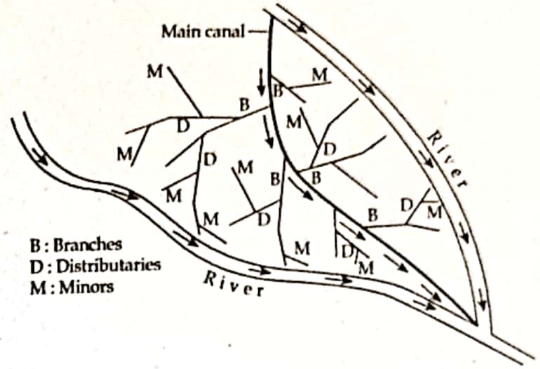
3.6 CANAL DISTRIBUTION SYSTEM

It has emphasized earlier that the direct irrigation scheme using a weir or barrage, as well as the storage irrigation scheme using a dam or a reservoir require a network of irrigation canals of different sizes and capacities. The entire network of irrigation channels is called the canal system. The canal system consists of;

i) Main canal
ii) Branch canals
iii) Distributaries

iv) Minors (minor distributaries)
v) Watercourses

In case of direct irrigation scheme, a weir or a barrage is constructed across the river, and water is headed up on the upstream side. The arrangement is known as Head works or diversion head works. Water is diverted into the main canal by means of a diversion weir. A head regulator is provided at the head of the main canal, so as to regulate the flow of water into the main canal. The canal distribution system is shown in the given figure.

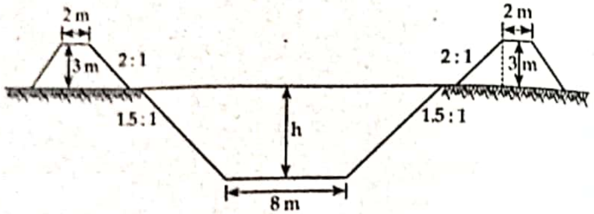


3.7 WORKED OUT PROBLEMS

PROBLEM 1

An irrigation canal has a bottom width of 8 m and side slopes of 1.5 H : 1V in cutting and 2 H : 1 V in filling. The width of the crest of bank is 2 m and its height above the ground level is 3 m. Compute depth and draw net x-section of canal illustrating various dimensions and level it. [2072 Ashwin]

Solution:



From the question,

Balancing depth (h) = ?

Now,

$$\text{Area of cutting} = 8h + 2\left(\frac{1}{2} \times h \times 1.5\,h\right) = 8\,h + 1.5\,h^2$$

$$\text{Area of filling} = 2\left[2 \times 3 + \left\{\frac{1}{2} \times (3 \times 2) \times 3\right\} \times 2\right] = 48\text{ m}^2$$

For balancing depth; we have,

Area of cutting = Area of filling

or, $8\,h + 1.5\,h^2 = 48$

or, $8\,h + 1.5\,h^2 - 48 = 0$

Solving; we get,

$h = 3.58$ m

Hence, the balancing depth is 3.58 m.

PROBLEM 2

Drawing a neat sketch, show the major components of an irrigation system from headwork to command area. [2072 Magh]

Solution: See the definition part 3.2

PROBLEM 3

Describe in what way you can align an irrigation canal for an agricultural land? Also write about canal standards and balancing depth. [2073 Bhadra]

Solution: See the definition part 3.3 and 3.5

PROBLEM 4

Describe with sketch about possible alignment of irrigation canal. [2073 Magh]

Solution: See the definition part 3.3

PROBLEM 5

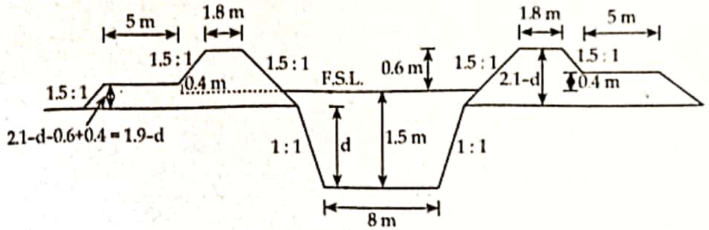
With a neat sketch, explain the canal distribution system suitable in Terai region of Nepal. [2074 Bhadra]

Solution: See the definition part 3.2

PROBLEM 6

A canal has bed width of 8 m. Full supply depth of water is 1.5 m, side slope in cutting 1 : 1 and filling 1.5 : 1. Top width of the bank is 1.8 m and service bank is 5 m. Free board is kept 0.6 m. Calculate balancing depth so as to get the most economical section. [2074 Bhadra]

Solution:



Let service bank (road) is provided 0.4 m above F.S.L.

$$\text{Area of cutting} = 8d + 2 \times \frac{1}{2} \times d \times d = 8d + d^2$$

$$\text{Area of filling} = \underbrace{\left[1.8 \times 0.2 + 2 \times \frac{1}{2}\{1.5 \times (0.2)^2\}\right]}_{\text{Upper part}}$$

$$+ \underbrace{\left[(5 + 1.8 + 2 \times 1.5 \times 0.2)(1.9 - d) + 2 \times \frac{1}{2} \times 1.5(1.9 - d)^2\right]}_{\text{Lower part}} \times 2$$

$$= \left[0.42 + 7.4(1.9 - d) + 1.5(1.9 - d)^2\right] \times 2$$

For balancing depth;

Area of cutting = Area of filling

or, $8d + d^2 = [0.42 + 7.4(1.9 - d) + 1.5(1.9 - d)^2] \times 2$

Solving using calculator; we get,

$d = 1.255$ m or 15.85 m (unfeasible)

so, Balancing depth = 1.25 m

PROBLEM 7

Explain with appropriate sketches the components of canal irrigation system. [2076 Bhadra]

Solution: See the definition part 3.2

PROBLEM 8

Derive balancing depth for a canal. Calculate balancing depth for a canal section giving a bed width equal to 20 m and side slopes of 1.5 : 1 in cutting and 2 : 1 in filling. The bank embankments are kept 4 m higher than ground level (berm) and crest width of banks is kept 3 m. [2077 Chaitra]

Solution:

Here,

Area of cut = $by + zy^2 = y(b + zy)$

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

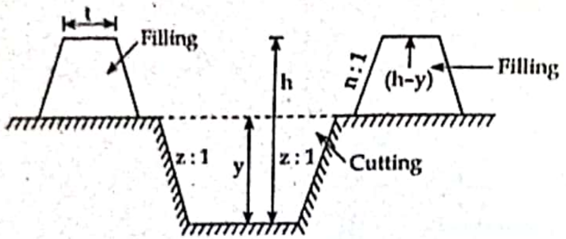
Equating the area of cut and fill

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

or, $by + zy^2 = 2ht + 2n(h^2 - 2hy + y^2)$

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

This is the required equation for balancing depth of canal; where, 'y' is depth of cutting 'n : 1' is side slope of canal in cutting 'z : 1' is side slope of filling, 't' is top width of canal bank, b is bed width of canal, 'h' is vertical height of top of bank from bed of canal.



Let's assume,

$A = (2n - z)$

$B = -(b + 4nh + 2t)$

$C = (2t + 2nh)h$

Balancing depth (y) = $\dfrac{-B \pm \sqrt{B^2 - 4AC}}{2A}$

From the question,

Bed width of canal = 20 m

Side slopes in cutting = 1.5 : 1

Side slopes in filling = 2 : 1

Height of embankment above berm = 4 m

Crest width of banks = 3 m

Balancing depth (y) = ?

Here;

Area of cutting = $20y + 1.5y^2$

Area of filling = $2[4 \times 3 + 2 \times (4)^2] = 2[12 + 32] = 88 \text{ m}^2$

For balancing depth,

Area of cutting = Area of filling

$1.5y^2 + 20y = 88 \text{ m}^2$

or, $y = 3.48$ m'

PROBLEM 9

Explain with appropriate sketches the components of canal irrigation system. [2078 Baishakh]

Solution: See the definition part 3.2

CHAPTER 4
DESIGN OF CANALS

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4.1 DESIGN CAPACITY OF CANALS

Following points should be taken into account while fixing the capacity of canals:

- Different types of crops that would be sown in any season should be known.
- The peak rate of water requirement of all crops in each season of a year should be known.
- The capacity of canal should be sufficient to fulfill the maximum of the peak demand of all the crops that are required to be irrigated at any one time amongst all the seasons.
- A canal designed on average outlet factor is proved to be very inadequate, as it fails to supply the required water to the crop at its peak period *i.e.,* at the time of kor watering.

- Canal should be designed for non-silting and non-scouring velocity.

4.2 SEDIMENT TRANSPORT IN CANALS

Flowing of water on the natural or artificial canal scour its bed. Silt or gravels or larger particles are detached from the bed of canal and are moved d/s of the canal with moving water. This phenomenon is called "sediment transport in canals".

Procedure of sediment transport (with respect to shear stress)

i) When the average shear stress τ_0 on the bed of an alluvial channel exceeds the critical shear τ_c the sediment particles start moving in different ways depending upon the flow condition, sediment size, fluid and sediment density and channel condition.

ii) At relatively low shear stress, *i.e.*, average shear stress just exceeds the critical shear, the particles on the bed start to roll and slide, along the bed remaining in contact with bed surface with discontinuous movement such sediment is called **contact load.**

iii) On increasing the shear stress, some sediment particles loose contact with the bed for some time with bounce. This type of sediment are called **saltation load.**

iv) It is difficult to distinguish clearly the bed load and saltation load. These two are grouped together and termed as bed load.

v) With further increase in shear stress, the particles may go in suspension due to the turbulent fluctuation. The particles in suspension move d/s such sediment are called suspension load.

vi) Both suspension load and bed load are originated from the bed of the channel and hence both are grouped under bed-material load.

vii) Actually suspended particle comprises the sediment particles originating from the bed and the sediment particles which are not available in the bed. The additional load is the product of erosion in the catchment and is appropriately called **wash load.**

viii) The concentration of wash load is related to the availability of the fine material in the catchment and their erodibility and is normally independent of hydraulic characteristics of the stream. So, it is not easy to make an estimate of wash load.

ix) Irrigation channel are designed to carry certain amount of water and sediment discharges. So, the sediment load transport effect the design of an alluvial channel.

Practical concept of sediment transport (with respect to velocity of flow)

There is movement of bed at low velocities. But as the velocity gradually increases the channel bed particles move resulting different shapes of channel bed. Following steps are relevant to describe.

i) When the velocity is gradually increased, then at 1st stage the bed particles (sediment load) comes just at the point of motion. This stage is called **threshold stage of motion.**

ii) On further increase in velocity, ripples of saw-tooth shape are formed as shown in figure 4.1.

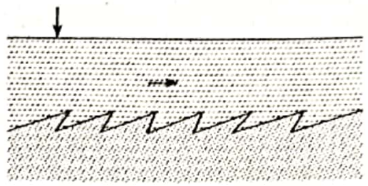


Figure 4.1: Saw tooth ripples

iii) As the velocity is increased further, larger periodic irregularities appear and are called dunes. Dunes are superimposed with ripples at their 1st stage of formation.

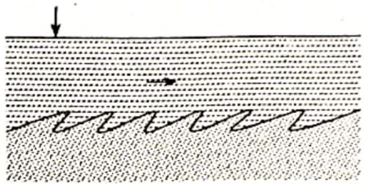


Figure 4.2: Dunes with ripples

iv) At higher velocity than (iii) ripples disappear and only the dunes are left.

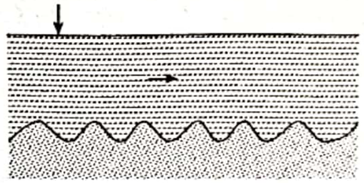


Figure 4.3: Dunes

v) On further increment of velocity beyond dunes the dunes are erased by the flow leaving very small undulation or virtually flat surface.

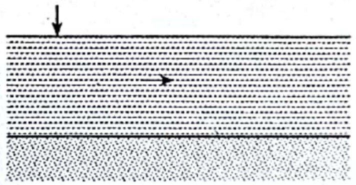


Figure 4.4: Flat surface

vi) Further increase in velocity result in the formation of sand waves in association with surface wave. *i.e.*, In this stage shape of bed changes the shape of flow.

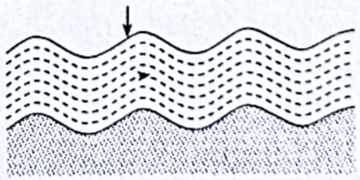


Figure 4.5: Sand waves in association with surface waves

vii) On further increment of velocity to make the Froude number $\left(\frac{V}{\sqrt{gy}}\right)$ greater than unity, the flow becomes super-critical. This makes surface wave so steep that they break intermittently and move upstream although the sediment particles move downstream.

Sand waves in this stage are called anti-dunes since direction of movement of bed forms in this regime is opposite to that of dunes. Anti-dunes rarely occur in case of irrigation canal.

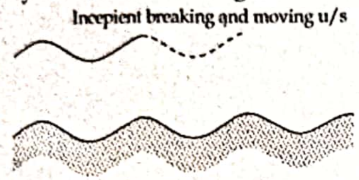


Figure 4.6: Antidunes

4.3 MECHANICS OF SEDIMENT TRANSPORT

Soil is assumed to be incoherent in the study of mechanics of sediment transport.

Incoherent soil

The type of soil in which no cohesive force is present between the particles *i.e.,* $c = 0$ like sand and gravels.

The basic mechanism of the sediment transport is the drag force exerted by water in the direction of flow on the channel bed. This force is pull of water on the wetted area and is known as tractive force or shear force or drag force.

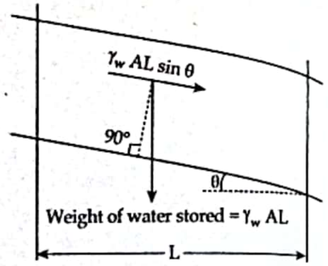


Figure 4.7: Longitudinal canal section

Let us consider a channel of length 'L' and x-section area 'A' as shown in the figure 4.7.

Volume of water stored = AL

Weight of stored water = $\gamma_w AL$

where, γ_w is the unit weight of water.

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

or, $S = \sin\theta \approx \tan\theta$ (slope of the bed)

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

Average tractive force per unit wetted area = Unit tractive force

or, $$\tau_0 = \frac{\gamma_w ALS}{\text{Wetted area}}$$

$$= \frac{\gamma_w ALS}{PL} = \frac{\gamma_w AS}{P} = \gamma_w RS$$

This average unit tractive force is also called shear stress $(\tau_0) = \gamma_w RS$.

It may be noted that this unit tractive force in channels, except for wide open channels, is not uniformly distributed along the wetted perimeter.

Incipient motion (Threshold point)

When the velocity of flow through a channel is very small, the channel bed does not move at all, and the channel behaves as a rigid boundary channel. As the flow velocity increases steadily, a stage is reached when the shear force exerted by the flowing water on the bed particles will just exceed the force opposing their movement. At this stage, a few particles on the bed will just start moving intermittently. This condition of motion is called the **incipient motion condition or simply the critical condition or the threshold point.**

4.4 TRACTIVE FORCE APPROACH OF CANAL DESIGN

The movement of the sediments at bed is caused by a force exerted on its grains by the flowing water. This force is known as the tractive force and is equal to the component of the weight of water in the direction of flow. For uniform flow, the mean tractive force per unit area is given by,

$$\tau = \gamma_w RS$$

where, τ is the tractive force per unit area (kNm^{-2}).

The permissible tractive stress or the critical tractive stress (τ_c) is defined as the maximum tractive stress that will not cause movement of the material forming the channel bed on a level surface. When tractive stress exceeds the critical tractive stress; the particles starts moving. The critical tractive stress is a function of the sediment concentration and the average particle size of the bed material in the case of channels in sandy soils.

Expressions for critical tractive stress

Shield's equation [for non scouring channel having protected side slopes]

Shield defined the critical tractive stress (τ_c) as that average shear stress (τ_0) at which the sediment particle on the bed of the channel just begins to move. He concluded that the bed particle begins to move when the drag force (F_1) exerted by the fluid on the particle, just equals or exceeds the resistance (F_2) offered by particle to its movement. These forces F_1 and F_2 are expressed as:

i) The drag force (F_1) exerted by the flow,

$$F_1 = K_1\left[C_D \cdot d^2 \cdot \frac{1}{2}\rho_w \cdot V_0^2\right] \quad (4.1)$$

where, K_1 = A factor depending on the shape of the particle.

C_D = Coefficient of drag.

d = Diameter of particle.

ρ_w = Density of water.

V_0 = Velocity of flow at top of particle, *i.e.,* at the bottom of channel.

ii) The particle resistance (F_2) is given by,

$$F_2 = K_2[d^3 \cdot (\rho_s - \rho_w)g] \quad (4.2)$$

where, ρ_s = Density of particle.

ρ_w = Density of water.

S_s = Specific gravity of particle = $\frac{\rho_s}{\rho_w}$.

K_2 = Factor dependent on shape of particle and internal friction of soil.

From equation (4.1); we have,

$$F_1 = K_1\left[C_D \cdot d^2 \cdot \frac{1}{2}\rho_w \cdot V_0^2\right]$$

According to Karman-Pradtt equation velocity of flow at the bottom of canal (V_0) is given by,

$$\frac{V_0}{V} = f_1 \cdot R_e \Rightarrow V_0 = V \cdot f_1 \cdot R_e \qquad (4.3)$$

where, V = shear friction velocity = $\sqrt{\frac{\tau_0}{\rho_w}}$.

R_e = particle Reynold's number = $V \cdot \frac{d}{\nu}$

ν = kinematic viscosity of water.

The drag coefficient (C_D) is given by,

$$C_D = f' \cdot \left(\frac{V_0 \cdot d}{\nu}\right) = f' \cdot \frac{V \cdot f_1 \cdot R_e \cdot d}{\nu}$$

$$C_D = f_2 \cdot \frac{V \cdot d}{\nu} \cdot R_e \qquad [f' f_1 = f_2]$$

or, $C_D = f_2 \cdot R_e^2$ (4.4)

Substituting (4.3) and (4.4) in equation (4.1); we get,

$$F_1 = K_1\left[f_2 \cdot R_e^2 \cdot d^2 \cdot \frac{1}{2}\rho_w \cdot V^2 \cdot f_1^2 \cdot R_e^2\right]$$

or, $F_1 = K_1\left[f_2 \cdot f_1^2 \cdot d^2 \cdot \frac{1}{2}\rho_w \cdot V^2 \cdot R_e^4\right]$ (4.5)

From equation (4.2); we get,

$$F_2 = K_2\left[d^3 \cdot \left(\frac{\rho_s}{\rho_w} - 1\right)\rho_w g\right] = K_2[d^3 \cdot (G_s - 1)\gamma_w] \qquad (4.6)$$

At critical condition equating F_1 and F_2,

$$K_1 \cdot f_2 \cdot f_1^2 \cdot \frac{1}{2}\rho_w \cdot V^2 \cdot R_e^4 = K_2[\gamma_w \cdot d^3(G_s - 1)]$$

or, $\frac{\rho_w \cdot V^2}{\gamma_w d(G_s-1)} = \left(\frac{2K_2}{K_1 f_2 f_1^2}\right) \cdot R_e^{-4}$

But, $\rho_w V^2 = \tau_c$

$\therefore$ $\frac{\tau_c}{\gamma_w d(G_s-1)} = F(R_e)$ [Function of R_e]

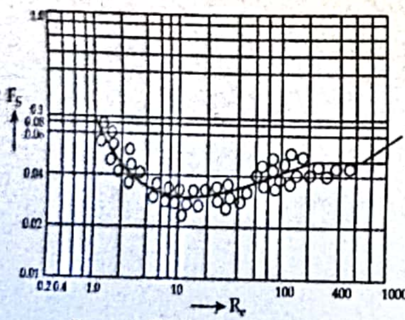


Figure 4.8: Shield's curve for incipient motion condition

The term $\frac{\tau_c}{\gamma_w d(G_s-1)}$ is a dimensionless number and is called the **Shield's Entraintment Function** and is usually denoted by F_s.

$$F_s = F(R_e)$$

The shield's entraintment function "F_s" is the function of R_e at critical stage of bed movement. Based on this concept shield have been plotted graph between 'F_s' and 'R_e' as shown in figure 4.8.

The application of this curve is more simple when $R_e > 400$ or F_s becomes constant and equal to 0.056. Mathematically,

$$F_s = 0.056$$

or, $\frac{\tau_c}{\gamma_w d(G_s-1)} = 0.056$

or, $\tau_c = 0.056\,\gamma_w d(G_s - 1)$ (4.7)

Shield also found that practical Reynold's number (R_e) exceeds 400 when the particle size is more than 6 mm (*i.e.,* for coarse alluvium soil). Therefore, equation (4.7) is valid only for d > 6 mm.

Hence, according to shield the critical tractive stress is proportional to the grain diameter and the submerged unit weight of sediment, and is given by;

$\tau_c = 0.056\gamma_w d(G_s - 1)$; for (d > 6 mm)

where, τ_c = Critical shear stress (kNm^{-2}) or critical tractive stress

G_s = Specific gravity of the sediment

d = Diameter of the sediment grain (m)

For no movement of bed particles;

Average tractive stress ≤ Critical tractive stress

or, $\tau_0 \le \tau_c$

or, $\gamma_w RS \le 0.056\gamma_w d(G_s - 1)$

or, $RS \le 0.056d(2.65 - 1)$

or, $RS \le \frac{d}{11}$

$\therefore$ $d \ge 11\,RS$ (4.8)

This gives the minimum size of the bed material that remains at rest *i.e.,* the particle less than the size of (11 RS) m are not transported on irrigation canal.

Above equation (4.8) is valid only for d > 6 mm. General equation for critical tractive stress worked out by Mittal and Swamee is,

$$\tau_c = 0.155 + \frac{0.409\,d_{mm}^2}{\sqrt{1 + 0.177d_{mm}^2}}\ N/m^2 \text{ [Valid for all condition]} \qquad (4.9)$$

This relation gives result within 5% of the values given by shield's curve for all the values of 'd'.

Design of non-scouring channels with unprotected side slopes (stability of channel slopes)

Shield's equation discuss about the stability of sediment particles only at horizontal bed of channel. At horizontal bed, there present only one disturbing force *i.e.,* shear stress, $\tau_0 = \gamma_w RS$.

Under this heading we are going to discuss about the stability of sediment particles at channel side slope. At side slope of channel, in additional to shear stress; there presence another disturbing force *i.e.*, the component of weight of the particle, unlike at horizontal bed as shown in figure 4.9.

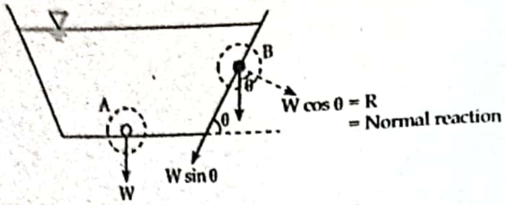


Figure 4.9: Forces on a grain at side slope

Consider a grain of weight 'W' represents by a dotted circle on a side slope and hollow circle on horizontal bed of channel in figure 4.9.

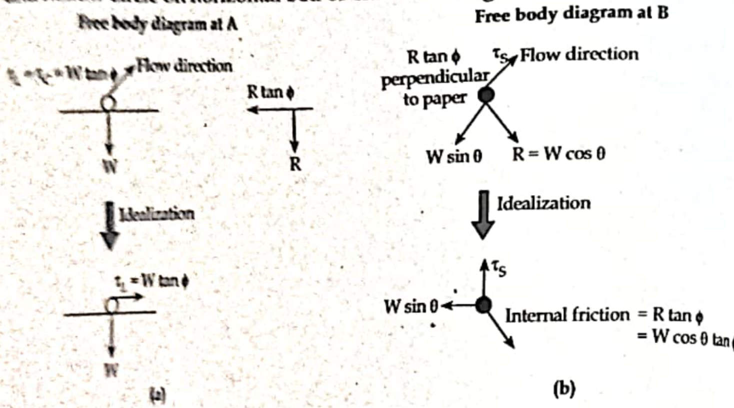


Figure 4.10: (a) Free body diagram at horizontal bed. (b) Free body diagram at sloping surface

where, $\tau_L = \tau_c$ = Critical shear stress at level surface or horizontal bed

$= W\tan\phi$

τ_s = Critical shear stress at sloping surface.

ϕ = Angle of repose of soil.

From figure 4.10 (b) (Idealized figure)

$$(\tau_s)^2 + (W\sin\theta)^2 = [W\cos\theta\tan\phi]^2$$

or, $$\tau_s^2 + \left[\frac{\tau_L}{\tan\phi}\sin\theta\right]^2 = \left[\frac{\tau_L}{\tan\phi}\cos\theta\tan\phi\right]^2 \qquad \left[\because W = \frac{\tau_L}{\tan\phi}\right]$$

or, $$\tau_s^2 + \frac{\tau_L^2}{\tan^2\phi}\sin^2\theta = \tau_L^2\cos^2\theta$$

or, $$\frac{\tau_s^2}{\tau_L^2} = \cos^2\theta - \frac{\sin^2\theta}{\tan^2\phi} \qquad (4.10)$$

or, $$\frac{\tau_s^2}{\tau_L^2} = \cos^2\theta\left[1 - \frac{\tan^2\theta}{\tan^2\phi}\right]$$

or, $$\frac{\tau_s}{\tau_L} = \cos\theta\sqrt{1 - \frac{\tan^2\theta}{\tan^2\phi}} \qquad (4.11)$$

Also, from equation (4.10); we have,

$$\left(\frac{\tau_s}{\tau_L}\right)^2 = \cos^2\theta + (\sin^2\theta - \sin^2\theta) - \frac{\sin^2\theta}{\tan^2\phi}$$

$$= 1 - \sin^2\theta\left[1 + \frac{1}{\tan^2\phi}\right]$$

$$= 1 - \sin^2\theta\left[\frac{1 + \tan^2\phi}{\tan^2\phi}\right]$$

$$= 1 - \sin^2\theta \cdot \frac{\sec^2\phi}{\tan^2\phi}$$

$$= 1 - \frac{\sin^2\theta}{\sin^2\phi}$$

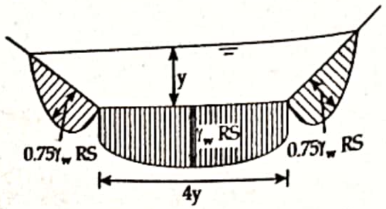


Figure 4.11: Distribution of tractive force in a trapezoidal channel section

∴ $$\left(\frac{\tau_s}{\tau_L}\right) = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} \qquad (4.12)$$

where, τ_s = Critical tractive stress on sloping surface.

τ_L = Critical tractive stress on level surface (at the bottom of canal).

θ = Angle of sloping surface.

ϕ = Angle of repose.

K = Tractive force ratio.

The above equation shows that $\tau_s < \tau_L$, which shows that the shear stress required to move a grain on the side slopes is less than the shear stress required to move the grain on canal bed. Moreover, on the canal bed, the average value of actual shear stress generated by the flowing water in a canal of given 'R' and 'S' is given by the equation;

$$\tau_0 = \gamma_w RS \qquad (4.13)$$

While on slopes, this value is given by;

$$\tau_0 = 0.75\gamma_w RS \qquad (4.14)$$

Design procedure by tractive force approach

i) The permissible tractive stress is obtained from the observed data of the existing channel based on the sediment concentration.

ii) A suitable bed slope 'S' of channel is then selected either with reference to average ground slope along alignment or on the basis of experience.

iii) The value of 'R' is determined from $\tau = \tau_c = \gamma_w RS$

iv) A suitable value of manning's rugosity coefficient 'N' is assumed and mean velocity 'V' is obtained from manning's formula.

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

$$\text{Stickler's formula (N)} = \left(\frac{1}{24}\right)d^{\frac{1}{6}} \qquad (4.15)$$

v) The area of cross section is then determined from the relation, $A = \frac{Q}{V}$

vi) Knowing the 'R' and 'A' the bed width 'B' and depth 'D' of the canal may be determined.

Example 4.1

Design a canal to carry a discharge of 50 cumec. The slope of the canal is 1 in 5000. The soil is coarse alluvium with having a grain size of 6 cm. Tractive stress not to exceed 2.35 N/m².

Solution:

Given that;

$Q = 50$ cumec

$S = 0.002$

$d = 6\text{ cm} = 0.06\text{ m}$

From Stickler's formula; we have,

$$n = \frac{1}{24} \times d^{\frac{1}{6}} = \frac{1}{24} \times (0.06)^{\frac{1}{6}} = 0.0260$$

Tractive stress on bed level of canal (τ_c) = 2.35 Nm^{-2}

$\tau = \tau_c = \gamma_w RS$

or, $2.35 = \dfrac{9810 \times R \times 1}{5000}$

or, $R = 1.198$ m

$$V = \frac{1}{N} R^{\frac{2}{3}} S^{\frac{1}{2}} = \frac{1}{0.026}(1.198)^{\frac{2}{3}}\left(\frac{1}{5000}\right)^{\frac{1}{2}} = 0.614$$

$$A = \frac{50}{0.614} = 81.433\text{ m}^2$$

$$P = \frac{A}{R} = 67.974\text{ m}$$

Now, assuming side slope of channel as 0.5 : 1; we have,

$BD + 0.5 D^2 = 81.433 = A$

or, $B + D\sqrt{5} = P = 67.974$

Solving these two equations; we have,

$67.974 D - 2.236 D^2 + 0.5 D^2 = 81.433$

or, $1.736 D^2 - 67.974 D + 81.433 = 0$

Solving this equation; we get,

$D = 37.918$ or 1.237

Considering the feasible value; we have,

$D = 1.237$ m

and, $B = 65.208$ m

Example 4.2

A canal is to be designed to carry a discharge of 56 cumec. The slope of the canal is 1 din 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, φ may be taken as 37°.

Solution:

Assuming the side slope of channel as; $\theta = 30°$, then,

$$\frac{\tau_s}{\tau_L} = \cos\theta\sqrt{1 - \frac{\tan^2\theta}{\tan^2\phi}} = \cos 30°\sqrt{1 - \frac{\tan^2 30°}{\tan^2 37°}} = 0.577$$

Minimum shear stress required to dislodge the grain on side slope is given by;

$\tau_s = 0.557\tau_L$

Hence, for the stability of the shear stress on the slopes of a channel must be less than or equal to $0.557\tau_L$.

i.e., $\tau_0 = 0.557\tau_L$

But, the shear stress actually generated on the side slope of the channel is;

$\tau_0 = 0.75\gamma_w RS$

or, $0.75\gamma_w RS \le 0.557\tau_L$

But, $\tau_L = \gamma_w RS = \gamma_w \dfrac{d}{11}$ $\left[\text{From (4.8) } \dfrac{d}{11} = RS\right]$

∴ $0.75\gamma_w RS \le 0.557\gamma_w \dfrac{d}{11}$

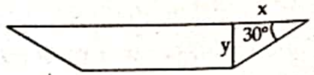
or, $RS \le 0.0676\, d$

or, $RS \le (0.0676 \times 0.05)$ m

or, $\dfrac{R \times 1}{1000} \le (0.0676 \times 0.05)$

or, $R \le 3.38$ m

or, $y \le 3.38$ m $(y \approx R)$



Adopting 20% safety factor y = 2.8 m; hence,

$A = y(b + x) = 2.8\,(b + 4.85)$

or, $P = b + 2\sqrt{x^2 + y^2} = b + 2\sqrt{23.6 + 7.84} = b + 11.22$

$$n = \frac{1}{24} \times d^{\frac{1}{6}} = \frac{1}{24} \times (0.05)^{\frac{1}{6}} = 0.0258$$

From the given figure; we have,

$\dfrac{y}{x} = \tan 30° = \dfrac{1}{\sqrt{3}}$

or, $x = 2.8\sqrt{3} = 4.85$ m

$$\therefore\ V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} = \frac{1}{0.0258} R^{\frac{2}{3}}\left(\frac{1}{1000}\right)^{\frac{1}{2}} = 1.223 R^{\frac{2}{3}}$$

Now, choosing trial values of b and proceeding to make Q equal to 56 cumecs.

b in metres	A	P	R	V	Q
3	21.980	14.220	1.546	1.635	35.937
4	24.780	15.220	1.628	1.693	41.942
5	27.580	16.220	1.700	1.742	48.053
6	30.380	17.220	1.764	1.786	54.248
7	33.180	18.220	1.821	1.824	60.514
6.3	31.220	17.520	1.782	1.798	56.121
6.29	31.192	17.510	1.781	1.797	56.058

Hence, use 6.3 m base width and 2.8 m depth.

4.5 DESIGN OF ALLUVIAL CANALS (KENNEDY'S AND LACEY'S THEORY)

The alluvial canals consist of alluvial soil which can be scoured easily. Moreover, the velocity is low which encourages silting. Therefore in an alluvial channel, both scouring and silting may occur if the channel is not properly designed.

The equation given by Chezy's and Manning's are applicable only for rigid boundary channels. For the design of alluvial soil channel In Nepal, the hypothetical theories given by Kennedy and Lacey are used.

4.5.1 Kennedy's theory

This theory is proposed by R.G. Kennedy, an executive engineer of Irrigation Department, Punjab. From the experiment and observation of 30 year he concluded the following.

- The silt supporting power in a channel cross section was mainly dependent upon the generation of eddies, rising to the surface.
- These eddies are generated due to friction of flowing water with channel surface.

Concept of critical velocity

The vertical component of these eddies try to move the sediment up, while the weight of the sediment tries to bring it down, thus keeping the sediment in suspension. Based on this concept, he defined the critical velocity (v_0) in a channel as the mean velocity which will just keep the channel free from silting or scouring and related to depth of flow by the equation,

$$V_0 = C_1 y^{C_2}$$

where, C_1, C_2 are constants depending upon silt charge and are found to be 0.55 and 0.64 (in M.K.S. or S.I. units) respectively.

$$\therefore \quad V_0 = 0.55 y^{0.64}$$

This equation is not valid for all type of soil or silt. Considering this limitation he introduced a factor 'm' in this equation to account for the type of soil through which the canal was to pass. This factor is dependent upon the silt grade and is named as critical velocity ratio (CVR). The equation was thus modified as,

$$V_0 = 0.55\, m y^{0.64}$$

where, V_0 = Critical velocity in the channel in m/s.

y = Water depth in channel in m.

m = CVR

Table 4.1

Recommended Values of C.V.R (m)

S.N.	Type of silt	Value of m
1	Light sandy silt, a little coarser	1.1
2	Sandy, loamy silt	1.2
3	Debris of hard soil	1.3

Design procedure by Kennedy's method

i) Determine the critical velocity by using the equation, $V_0 = 0.55\, m y^{0.64}$ by assuming the trial value of depth.

ii) Determine area by dividing the discharge by velocity *i.e.*, $A = \frac{Q}{V_0}$.

iii) Determine the channel cross-section, *i.e.*, area, wetted perimeter.

iv) Compute the actual mean velocity (V) that will prevail in the channel of this cross section, by using Kutter's formula, Manning's formula, etc.

a) Kutter's formula

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

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

b) Manning's formula

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

where, V is the velocity of flow in m/sec.

R is the hydraulic mean depth in m.

S is the bed slope.

n is the rugosity coefficient.

The value of 'n' in both of these equations depends upon the condition and also upon discharge. The values of 'n' may be taken as given in table below.

Table 4.2

Recommended values of manning's coefficient n for unlined channels

Condition of channel	Value of 'n'
Very good	0.0225
Good	0.025
Indifferent	0.0275
Poor	0.03

Discharge in cumec	Value of 'n' for unlined channels
14 to 140	0.025
140 to 280	0.0225
280 and above	0.02

c) Chezy's formula

$$V = C\sqrt{RS} \tag{4.13}$$

where, C = Chezy's constant depending upon the shape and surface of channel.

R = Hydraulic mean depth = $\frac{A}{P}$

S = Bed slope of channel.

Among these three formulas, generally Kutter's equation is used with Kennedy's theory.

v) If the two velocities V_0 and V work out to be the same, then assumed depth is all right, otherwise change it and repeat the procedure, till V_0 and 'V' becomes equal.

Example 4.3

Design an irrigation channel to carry 30 cumecs of discharge. The channel is to be laid at a slope of 1 in 5500. The critical velocity ratio for the soil is 1. Use manning's rugosity coefficient as n = 0.0225. Channel side slope is 1H : 2V.

Solution:

Given that;

$$Q = 30\ m^3/sec.$$
$$m = 1$$
$$n = 0.0225$$
$$S = 1{:}5500$$
$$\text{Side slope} = 1H : 2V \rightarrow 1V : \frac{1}{2}H \rightarrow z = 0.5$$

We have,

$$V_0 = 0.55\, my^{0.64}$$
$$A = \frac{Q}{V_0} = \frac{30}{V_0}$$
$$A = By + zy^2$$

or, $$B = \frac{A - zy^2}{y} = \frac{A - 0.5y^2}{y}$$

$$P = B = 2y\sqrt{1+z^2} = B + 2y\sqrt{1+(0.5)^2}$$
$$R = \frac{A}{P}$$
$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

Now, assuming first trial depth y = 1.5 m and proceeding the calculation in tabular form.

Trial no.	y	V_0	A	B	P	R	V	Remarks
1	1.5	0.713	42.078	27.302	30.656	1.373	0.740	$V_0 \neq V$
2	1.75	0.787	38.125	20.911	24.824	1.536	0.798	$V_0 \neq V$
3	2	0.857	35.003	16.501	20.973	1.669	0.843	$V_0 \neq V$
4	1.8	0.801	37.444	19.902	23.927	1.565	0.808	$V_0 \neq V$
5	1.81	0.804	37.312	19.709	23.756	1.571	0.810	$V_0 \approx V$
6	1.811	0.804	37.298	19.690	23.740	1.571	0.810	$V_0 \approx V$

Hence,

$$y = 1.811\ m$$

and, $B = 19.902\ m$

Example 4.4

Design an irrigation channel to carry 10.85 cumecs of discharge, with B/D ratio as 2.5. The critical velocity ratio is 1.0. Assume a suitable value of Kutter's rugosity coefficient and use Kennedy's method.

Solution:

Assume side slope of trapezoidal channel as 0.5 : 1.

$$V_0 = 0.55\, my^{0.64} = 0.55y^{0.64} \qquad [\because m = 1]$$
$$y = D$$
$$\therefore \quad V_0 = 0.55D^{0.64}$$
$$Q = AV$$
$$A = BD + 2 \times \frac{1}{2} \times D \times \frac{D}{2} = D\left[B + \frac{D}{2}\right]$$
$$Q = 40 = D\left[B + \frac{D}{2}\right]V_0 \qquad \left[\because \frac{B}{D} = 2.5\right]$$
$$40 = D[2.5D + 0.5D]V_0 = 3D^2V_0$$

But, $V_0 = 0.55D^{0.64}$

or, $40 = 3D^2 \times 0.55D^{0.64}$

$\therefore$ $D = 3.34\ m$

and, $B = 8.35\ m$

Now, determine the cross section,

$$A = 3D^2 = 3 \times (3.34)^2 = 33.5\ m^2$$
$$P = \left[B + \frac{2\sqrt{5}}{2}\right] = 15.81\ m$$
$$R = \frac{A}{P} = \frac{33.5}{15.81} = 2.12$$
$$V_0 = 0.55 \times (3.34)^{0.64} = 1.19$$

Assume n = 0.023 and using Kutter's formula,

$$V = \left[\frac{\frac{1}{0.023} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{0.023}{\sqrt{2.12}}}\right]\sqrt{2.12S} \qquad (4.14)$$

Assuming bed slope S = 1 in 4000

$$V = 1.114 < 1.19$$

or, $V < V_0$

To increase the value of 'V', we must increase the slope, *i.e.*, assume again the value of 'S' as 1 in 3700, then, from equation (4.14); we have,

$$V = 1.189 \approx 1.19 = V_0$$

So, use S = 1 in 3700

Hence, use the trapezoidal section as follows;

Depth = 3.34 m
Base width = 8.35 m
Side slopes = 0.5H : 1V
Bed slope = 1 in 3700

Example 4.5

Design an irrigation channel carrying a discharge of 20 cumecs with bed slope of 1 in 5000. The side slope of channel being 1 : 1 and manning's coefficient 0.014. The limiting velocity of channel to maintain non-scouring and non silting condition is 0.9 m/sec.

Solution:

$Q = 20$ cumecs
$S = 1$ in 5000
$Z = 1$
$n = 0.014$
Limiting velocity = 0.9 m/sec.

Now,

$$A = \frac{Q}{V} = \frac{20}{0.9} = 22.22 \text{ m}^2$$

$$A = By + zy^2$$

or, $$B = \frac{A - zy^2}{y} = \frac{22.22 - y^2}{y}$$

$$P = B + 2y\sqrt{1+z^2} = B + 2y\sqrt{1+(1)^2}$$

$$R = \frac{A}{P}$$

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

Trial no.	y	B	P	R	V	Remarks
1	1.5	13.313	17.556	1.266	1.182	V > 0.9
2	1	21.220	24.048	0.924	0.958	V > 0.9
3	0.9	23.789	26.334	0.844	0.902	V > 0.9
4	0.898	23.846	26.386	0.842	0.901	V ≈ 0.9
5	0.899	23.817	26.360	0.843	0.901	V ≈ 0.9

Since, limiting value of velocity is 0.9; we can stop the trial. Hence,

$y = 0.899$ m
and, $B = 23.817$ m

4.5.2 Lacey's theory

Kennedy's concept for the regime channel is that the channel with neither scouring nor silting velocity is regime channel which is not really true according to Lacey. He put on his theory for regime channel and explains that the channel performing non-scouring and non silting velocity may not be in regime condition. He, therefore, differentiated between the three regime conditions.

i) True regime

Following conditions should be satisfied for a channel to be in true regime;

- Discharge should be constant
- Flow should be uniform
- Silt charge should be constant; *i.e.*, amount of silt is constant
- Silt grade should be constant; *i.e.*, type and size of silt is always constant
- Channel should be 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.

But a real channel cannot be in true regime since all the above conditions can never be satisfied and so the real channel can be either in initial or final regime.

ii) Initial regime

If the cross section remains unchanged and only the bed slope of the channel varies due to dropping of silt, then the channel is said to in initial regime. When water flows through an excavated channel with narrower dimension and defective slopes, the silt carried by the water may get dropped in the upper reaches, thereby increasing the channel bed slope. Such channels are termed as channel in initial regime, and regime theory is not applicable to them. These channels appear to be regime but in fact they are not. They achieve working stability only due to rigidity of their banks.

iii) Final regime

If there is no resistance from the sides and all the variables such as perimeter, depth, slope, etc. are equally free to vary and finally get adjusted according to discharge and silt grade, then the channel is said to have achieved permanent stability, called final regime. Regime theory is applicable to these type of channel only and not to all regime channel.

Final regime channel has a tendency to assume a semi elliptical section. The coarser the silt, the flatter is the semi ellipse. The finer the silt, the more nearly the section attains the semi circle as shown in figure (4.12).

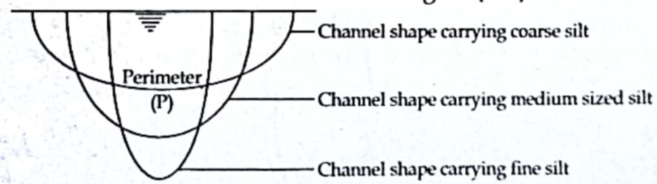


Figure 4.12

4.5.2.1 Lacey's Regime equation

On the basis of his own observation and research and also the earlier investigators, Lacey had developed the regime theory and evolved a number of equations as follows:

i) $$V = \sqrt{\frac{2}{5}fR} \tag{4.15}$$

or, $$R = \frac{2}{5}\frac{V^2}{f} \tag{4.16}$$

ii) $Af^2 = 140\,V^2$ (4.17)

where, V = Mean velocity of flow in m/sec.

f = Silt factor.

R = Hydraulic mean radius in m.

A = Cross-sectional area (m^2).

The above equations are also called Lacey's basic equations.

Other derived equations from Lacey's basic equations are as follows:

1. Silt factor–grain size relation

According to Lacey, silt factor (f) is the function of mean particle size (d) of the boundary material in the channel. Mathematically,

$$f = 1.76\sqrt{d} \quad (4.18)$$

where, f = Silt factor.

d = Mean particle size in mm

= d_{50} (Size of particle corresponding to 50% fineness)

2. Velocity–discharge–silt factor relationship

(V–Q–f) relationship

This can be obtained from equation (4.17). Multiplying both sides of equation (4.17) by v; we get,

$$AVf^2 = 140V^6 \quad (4.18)$$

We have, continuity equation,

$$Q = AV \quad (4.19)$$

From equation (4.18) and (4.19); we get,

$$Qf^2 = 140V^6 \quad (4.20)$$

$$\therefore \quad V = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}} \quad (4.21)$$

where, Q = discharge in cumecs.

3. Perimeter–discharge relation (P–Q relation)

Relation between P–Q can be obtained by combining equation (4.15) and (4.20).

Squaring both sides of equation (4.15) two times; we get,

$$V^4 = \frac{4}{25}f^2R^2$$

$$f^2 = \frac{25V^4}{4R^2} \quad (4.22)$$

Combining equations (4.20) and (4.22); we have,

$$Q\left(\frac{25V^4}{4R^2}\right) = 140V^6$$

$$\frac{25Q}{4R^2} = 140V^6$$

We have,

$$R = \frac{A}{P}$$

and, $Q = AV$

Then,

$$\frac{25Q}{4\left(\frac{A}{P}\right)^2} = 140V^2$$

or, $P^2Q = \frac{560}{25}(AV)^2$

or, $P^2Q = \frac{560}{25}Q^2$

$$\therefore \quad P = 4.75\sqrt{Q} \quad (4.23)$$

where 'P' is perimeter in m.

4. Regime flow equation (V–R–S Relation)

This is the relation between velocity, hydraulic mean depth and bed slope of channel. It forms basis for determining the bed slope after computing the channel X-section.

$$V = 10.8R^{\frac{2}{3}}S^{\frac{1}{3}} \quad (4.24)$$

S = Bed slope

This equation is also called **General regime equation.**

5. Regime slope equations

a) S–F–R relation

Cubing both sides of equation (4.24); we get,

$$V^3 = 1260R^2S \quad (4.25)$$

Also cubing both sides of equation (4.15); we get,

$$V^3 = \left(\frac{2}{5}\right)^{\frac{3}{2}} f^{\frac{3}{2}}R^{\frac{3}{2}} \quad (4.26)$$

From equation (4.25) and (4.26); we get,

$$1260R^2S = \left(\frac{2}{5}\right)^{\frac{3}{2}} f^{\frac{3}{2}}R^{\frac{3}{2}}$$

$$S = \frac{f^{\frac{3}{2}}}{4980R^{\frac{1}{2}}}$$

$$\therefore \quad S = \frac{0.0002f^{\frac{3}{2}}}{R^{\frac{1}{2}}} \quad (4.27)$$

b) S–f–Q relation

Equating equation (4.15) and equation (4.21); we get,

$$\sqrt{\frac{2}{5}fR} = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}}$$

$$R^{\frac{1}{2}} = \left[\frac{5}{2f}\right]^{\frac{1}{2}}\left(\frac{Qf^2}{140}\right)^{\frac{1}{6}}$$

$$R^{\frac{1}{2}} = \left(\frac{Q}{8.96f}\right)^{\frac{1}{6}} \quad (4.28)$$

Substituting equation (4.28) in equation (4.27); we get,

$$S = 0.0002f^{\frac{3}{2}}\left(\frac{8.96f}{Q}\right)^{\frac{1}{6}}$$

$$\therefore \quad S = \frac{f^{\frac{5}{3}}}{3340Q^{\frac{1}{6}}} \quad (4.29)$$

6. Regime scour depth equation

Squaring equation (4.15); we get,

$$V^2 = \frac{2}{5}fR$$

$$\text{or,} \quad R = \frac{5}{2}\frac{V^2}{f} \quad (4.30)$$

Again squaring both sides of equation (4.21); we get,

$$V^2 = \left(\frac{Qf^2}{140}\right)^{\frac{1}{3}} \quad (4.31)$$

Substituting (4.31) in (4.30); we get,

$$R = \frac{5}{2}\left(\frac{Qf^2}{140}\right)^{\frac{1}{3}}\frac{1}{f} = \frac{5}{2}\left(\frac{1}{140}\right)^{\frac{1}{3}}Q^{\frac{1}{3}}Q^{\frac{2}{3}-1}$$

$$\therefore \quad R = 0.47\left(\frac{Q}{f}\right)^{\frac{1}{3}} \quad (4.32)$$

Equation (4.32) gives hydraulic mean depth of a regime channel. In case of **wide channel the hydraulic mean depth (R) is almost same as the depth of flow.** Equation (4.32) is also used to determine the Scour depth of a channel.

Design procedure for Lacey's theory

i) Calculate the velocity from equation (4.21),

$$V = \left[\frac{Qf^2}{140}\right]^{\frac{1}{6}} \text{m/sec.}$$

where, Q is in cumecs and 'V' is in m/s.

'f' is the silt factor given by equation (4.18),

$f = 1.76\sqrt{d}$

ii) Find out the hydraulic mean depth 'R' from the equation (4.16);

$$R = \frac{5}{2}\left(\frac{V^2}{f}\right)$$

where, V is in m/sec. and 'R' is in m.

iii) Compute area of channel section $A = \frac{Q}{V}$.

iv) Compute wetted perimeter from equation (4.23),

$P = 4.75\sqrt{Q}$

where, P is in m and 'Q' is in cumecs.

v) Knowing these values, the channel section is known: and finally the bed slope S is determined by the equation (4.29),

$$S = \left[\frac{f^{\frac{5}{3}}}{3340Q^{\frac{1}{6}}}\right]$$

where, 'f' is the silt factor and 'Q' is the discharge in cumec.

Example 4.6

Design a regime channel for a discharge of 50 cumec and silt factor 1.1 using Lacey's theory.

Solution:

$Q = 50$ cumec

$f = 1.1$

i) $V = \left[\frac{Qf^2}{140}\right]^{\frac{1}{6}} = \left[\frac{50 \times 1.1^2}{140}\right]^{\frac{1}{6}} = 0.869 \text{ m/sec.}$

ii) $R = \frac{5}{2}\left(\frac{V^2}{f}\right) = \frac{5}{2}\left\{\frac{(0.869)^2}{1.1}\right\} = 1.675 \text{ m}$

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

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

For a trapezoidal slope with side slope, 0.5H : 1V

$\therefore \quad P = b + \sqrt{5}y$

and, $A = b + \frac{y}{2}$

$$\text{so,} \quad 33.56 = b + \sqrt{5}\,y \quad (4.33)$$

$$\text{and,} \quad 56.3 = by + \frac{y^2}{2} \quad (4.34)$$

From equation (4.33); we get,

$b = 33.56 - 2.24y$

Putting this value of 'b' in equation (4.34); we get,

$$\begin{aligned} 56.3 &= [33.56 - 2.24y]\,y + \frac{y^2}{2} \\ &= 33.56y - 2.24y^2 + 0.5y^2 \\ &= 33.56y - 1.74y^2 \end{aligned}$$

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

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

$\therefore \quad y = 1.65$ (neglecting unfeasible value)

$b = 33.56 - 2.24 \times 1.65 = 29.77$ m

$$S = \left[\frac{(1.1)^{\frac{5}{3}}}{3340 \times (50)^{\frac{1}{6}}}\right] = \frac{1}{5420}$$

Differences between Kennedy's and Lacey's theory

	Kennedy's Theory	Lacey's Theory
i)	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.
ii)	Relation between 'V' and 'D'.	Relation between 'V' and 'R'.
iii)	Critical velocity ratio 'm' is introduced to make the equation applicable to different channels with different silt grades.	Silt factor 'f' is introduced to make the equation applicable to different channels with different silt grades.
iv)	Kutter's equation is used for finding the mean velocity.	This theory gives an equation for finding the mean velocity.
v)	This theory gives no equation for bed slope.	This theory gives an equation for bed slope.
vi)	In this theory, the design is based on trial and error method.	This theory does not involve trial and error method.

4.6 DESIGN OF LINED CANALS WITH ECONOMIC ANALYSIS

Lining of canal means laying the stable (in erodible) material on the earthen surface of channel such as concrete, tiles, asphalt, etc. The seepage losses in channel reduce the potential of supplied water. So, main objective of lining is to reduce the seepage loss and maintain the potential of irrigation water.

The advantages of canal lining are as follows;

i) Seepage control

Seepage losses can be considerably reduced by lining the channels. But the cost of channel is increased by 2 to 2.5 times of the unlined channels. The heavy seepage losses also necessitate the higher capacity of the impounding reservoir and bigger dams. Lining of channels therefore reduces the impounding capacity of reservoir and also its cost.

ii) Prevention of water logging

Seepage losses through the unlined channels considerably increases the water table in the surrounding fields up to or near to the ground level as to bring the roots of crops within the capillary fringe. This enhances the alkali effects on the ground surface and hence makes the ground unfit for cultivation. Lining reduces the seepage and minimize this effect on the ground surface.

iii) Increase in channel capacity

Lining of channel produces the smooth surface and hence reduces the resistance of flow. The flow through unlined channel is resisted by the vegetation on the sides and bottom. The water on lined channel flows faster and greater discharge can be achieved through the same section. Hence lining the channel increases the channel capacity.

iv) Increase in commanded area

A lined channel can be designed not only smaller in cross section but also smaller in length. The steeper gradient can be provided so that short alignment can be selected. Flatter gradient can also be provided without silting on a lined channel. So the lined channel helps to bring high area under command.

v) Reduction in maintenance cost

Due to less seepage loss, no vegetation and less silting property of lined channel, its maintenance cost can be considerably reduced.

vi) Elimination of flood dangers

An unlined channel founded in weaker foundation is always in danger and there may also danger of outburst due to smaller cross-section in unlined channel. A strong concrete lining removes all such dangers.

Disadvantages of canal lining

i) Lining of channels requires high initial investment.
ii) Damaged lining is difficult to repair.
iii) The lining being permanent, it is difficult to shift the outlets if necessary.
iv) There is no berm in lined channel which do not provide additional safety for vehicles and pedestrians.

Requirements of good canal lining

The essential requirements of good canal lining are:

i) Durability (D)
ii) Repairability (R)
iii) Economy (E)
iv) Structural Stability (SS)

Shortly we can write as DRESS.

i) Durability

A good canal lining must be able to with stand the following:

- Natural wear and tear
- Effect of velocity of water
- Rain, sunshine, frost and thawing
- Thermal and moisture changes
- Chemical action of salts
- Damaging effect of cattle traffic, weed and rodent growth.

ii) Repairability

The lining should be such that it can be repaired easily and economically. For this Brick tile, boulder and precast slab linings are preferred to easily repairable than that of cast in situ concrete lining.

iii) Economy

The type of lining selected should not only be economic in initial cost but also in repair and maintenance cost.

iv) Structural stability

The lining should be strong enough to withstand the differential sub-soil water pressure due to sub-grade getting saturated through seepage or rain or due to sudden drawdown of channel.

The other additional requirements to above essential requirements are:

i) Impermeability

The lining should be such that the see-page losses are reduced to a considerable extent.

ii) Hydraulic efficiency

The lining should have low coefficient of rugosity so that the channel has a high discharge carrying capacity and channel section is hydraulically more efficient.

iii) Resistance to erosion

The lining should be able to withstand abrasion due to sediment transported by flowing water. Cement concrete and boulder linings provide better abrasion resistance as compared to brick tile lining.

Types of canal lining

The various types of canal lining can be grouped into following three categories.

i) Exposed and hard surface linings

1. Cement concrete lining (In-situ)
2. Precast concrete lining
3. Shortcrete lining.
4. Cement mortar lining
5. Hydraulic lime concrete lining
6. Brick tile lining or Burnt clay tile lining
7. Stone blocks or undress stone lining
8. Asphaltic concrete lining

ii) Buried membrane lining

1. Sprayed-in-place asphalt membrane lining
2. Prefabricated asphaltic membrane lining
3. Polyethylene film and synthetic rubber membrane lining
4. Bentonite and clay membrane lining
5. Road oil lining

iii) Earth lining

1. Thin compacted earth lining
2. Thick compacted earth lining
3. Loosely placed earth lining
4. Stabilized soil lining
5. Bentonite soil lining
6. Soil-cement lining

4.6.1 Cross-section of lined channels

i) Triangular cross-section

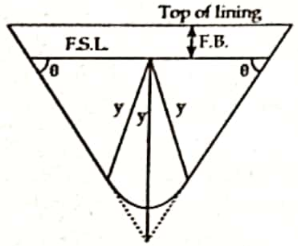


Figure 4.13: Triangular section

These are for smaller discharges.

Let, Central depth = Radius of circle = y

$$\text{Area} = \pi \times y^2 \times \frac{\theta}{\pi} + 2 \times 0.5 \times y \times y \times \cot\theta$$

or, $$A = y^2(\theta + \cot\theta) \tag{4.35}$$

$$\text{Perimeter} = 2\pi y \times \frac{\theta}{\pi} + 2y\cot\theta$$

or, $$P = 2y(\theta + \cot\theta) \tag{4.36}$$

$$\text{Hydraulic mean depth} = \frac{y^2(\theta + \cot\theta)}{2y(\theta + \cot\theta)} = \frac{y}{2}$$

ii) Trapezoidal section

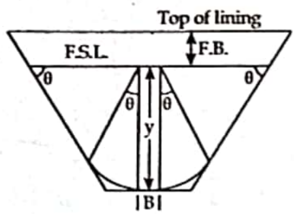


Figure 4.14 : Trapezoidal section

These are for larger discharges;

$$\text{Area} = By + 2\left(\pi y^2 \times \frac{\theta}{2\pi}\right) + 2(0.5 \times y \times y \times \cot\theta)$$

or, $$A = By + y^2\cot\theta$$

$$= y(B + y\theta + y\cot\theta) \tag{4.37}$$

$$\text{Perimeter} = B + 2\left(\frac{2\pi y\theta}{2\pi}\right) + 2y\cot\theta$$

or, $$P = B + 2y\theta + 2y\cot\theta \tag{4.38}$$

4.6.2 Design Standards for lined channels

i) Permissible velocities in lined channel

Permissible velocities for different type of linings as per Indian standards are tabulated as under,

Types of Lining	Permissible Velocity
Cement concrete lining (Unreinforced)	2.0 to 2.5 m/sec.
Burnt clay tile lining	1.8 m/sec.
Boulder lining	1.5 m/sec.

Table 4.3: Permissible velocity in different types of linings

II) Side slopes

The side slope should be nearly equal to angle of repose of sub-grade soil so that no earth pressure is exerted over the back of lining.

i.e., Side slope (θ) $\approx$ Angle of repose (ϕ)

III) Freeboard

Values of freeboard recommended for channels of different discharge carrying capacity are:

	Type of canals	Discharge (cumecs)	Freeboard (m)
1.	Main and Branch canals	> 10	0.75
2.	Branch canals and major distributaries	5–10	0.60
3.	Major distributaries	1–5	0.50
4.	Minor distributaries	< 1	0.30
5.	Water courses	< 0.06	0.10–0.15

Table 4.4: Values of free board for lined canals

The free board in lined canal is measured from full supply level to top of lining.

Iv) Bank widths

The minimum values recommended for top width of the bank are as follows

	Discharge (cumec)	Maximum top of width of bank	
		Inspection bank (m)	Non-inspection bank (m)
1.	0.15 to 0.75	5.0	1.5
2.	7.5 to 10.0	5.0	2.5
3.	10.0 to 15.0	6.0	2.5
4.	15.0 to 30.0	7.0	3.5
5.	30.0 and above	8.0	5.0

Table 4.5: Values of Bank width for lined canals

v) Rugosity coefficient

The values of rugosity coefficient for different types of lining are as follows:

	Surface characteristics	Value of N
1.	Concrete with	
	a) Formed no finish / pc tiles or slabs	0.018–0.02
	b) Trowel float finish	0.015–0.018
	c) Gunited finish	0.018–0.022
2.	Concrete bed trowel finish and sides	
	a) Hammer dressed stone masonry	0.019–0.021
	b) Coursed rubble masonry	0.018–0.02
	c) Random rubble masonry	0.02–0.025
	d) Masonry plastered	0.015–0.017
	e) Dry boulder lining	0.02–0.03
3.	Brick tile lining	0.018–0.02

Table 4.6: Values of Rugosity coefficient for lined canals

4.6.3 Design procedure for lined canals

Data required for design are:

i) Discharge, Q
ii) Rugosity coefficient, N
iii) Bed slope, S
iv) Side slopes
v) Limiting velocity, V

Equations used for design are:

i) Continuity equation

$$Q = AV$$

ii) Manning's equation

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

Procedure for design of lined channels (Trapezoidal section)

i) Knowing V, N and S. Find 'R' using Manning's equation
Manning equation is,

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

$$\therefore \quad R = \left(\frac{VN}{S^{\frac{1}{2}}}\right)^{\frac{1}{2}} \quad \textbf{(4.39)}$$

ii) Find area (A) of the section using continuity equation,

$$A = \frac{Q}{V} \quad \textbf{(4.40)}$$

iii) Find wetted perimeter,

$$P = \frac{A}{R} \quad \textbf{(4.41)}$$

iv) For trapezoidal section with rounded corners shown in figure 4.14.
From equation **(4.37)**; we have,

$$A = y(B + y\theta + y\cot\theta)$$

From equation **(4.38)**; we have,

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

Since, A and P are known form step (ii) and (iii). The value of 'B' and 'y' can be found form equation **(4.37)** and **(4.38)**.

Alternatively, if 'V' is not given but $\left(\frac{B}{Y}\right)$ ratio then following procedure is followed,

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

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

Then,

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

ii) $$V = \frac{Q}{A} = \frac{Q\ (\text{given})}{By + y^2(\theta + \cot\theta)}$$ (ii)

iii) From Manning's equation,

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

Substituting value of R and V from (i) and (ii) respectively,

$$\frac{Q\ (\text{given})}{By + y^2(\theta + \cot\theta)} = \frac{1}{N}\left(\frac{By + y^2(\theta + \cot\theta)}{By + 2y(\theta + \cot\theta)}\right)^{\frac{2}{3}} S^{\frac{1}{2}}$$ (4.42)

Using given $\left(\frac{B}{Y}\right)$ ratio in equation (4.42) value of 'B' and 'y' can be determined.

Procedure for design of Lined channels (Triangular section)

i) Knowing side slope (θ), find 'A' and 'P' and 'R' in term of depth 'y'.

From figure (4.13); we have,

From equation (4.35); we have,

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

From equation (4.36); we have,

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

and, $$R = \frac{A}{P} = \frac{y}{2}$$ (4.43)

ii) From continuity equation; we have,

$$V = \frac{Q}{A} = \frac{Q}{y^2(\theta + \cot\theta)}$$ (4.44)

iii) Manning's equation; we have,

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

Substituting the value of 'R' and 'V' from equations **(4.43)** and **(4.44)**; we get,

$$\frac{Q}{y^2(\theta + \cot\theta)} = \frac{1}{N}\left(\frac{y}{2}\right)^{\frac{2}{3}} S^{\frac{1}{2}}$$ (4.45)

The value of 'y' can be determined from equation **(4.45)** knowing Q, N, θ and S.

NOTE

Equation **(4.43)** shows that the triangular section with circular bottom is the best discharging section.

Example 4.7

Design a trapezoidal shaped concrete line channel to carry a distance of 250 cumec at a slope of 40 cm/km. The side slopes of the channel are 1.5 : 1. The value of 'N' may be taken as 0.017. Assume limiting velocity in the channel as 3 m/sec.

Solution:

Given that;

$Q = 250$ cumec

$$S = \frac{40\text{ cm}}{1 \times 1000 \times 100\text{ cm}} = 0.0004$$

Side slope 1.5 : 1, *i.e.*,

$\cot\theta = 1.5$

and, $\theta = \cot^{-1}(1.5) = 0.588$

$N = 0.017$

$V = 3$ m/sec.

i) From equation **(4.39)**; we have,

$$R = \left(\frac{3 \times 0.017}{(0.0004)^{\frac{1}{2}}}\right)^{\frac{3}{2}}$$

ii) $$A = \frac{Q}{V} = \frac{250}{3} = 83.33\text{ m}^2$$

iii) $$P = \frac{A}{R} = \frac{83.33}{4.072} = 20.465\text{ m}$$

iv) From equation **(4.37)**; we have,

$A = y(B + y\theta + y\cot\theta)$

or, $83.33 = y(B + 0.588\theta + 1.5y)$

or, $83.33 = y(B + 2.088y)$

or, $83.33 = By + 2.088y^2$ **(1)**

From equation **(4.38)**; we have,

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

or, $20.465 = B + 2y(0.588 + 1.5)$

or, $20.465 = B + 4.176y$ **(2)**

Solving equation **(1)** and **(2)**; we get,

From equation **(2)**; we have,

$B = 20.465 - 4.176y$

Substituting the value of 'B' in equation **(1)**; we get,

$83.33 = (20.465 - 4.176y)y + 2.088y^2$

or, $83.33 = 20.465y - 4.176y^2 + 2.088y^2$

or, $83.33 = 20.465y - 2.088y^2$

or, $2.088y^2 - 20.465y = 83.33$ **(3)**

Solving **(3)** for quadratic in 'y'; we get,

$y = 3.095$ or -12.89 (Not possible)

∴ $y = 3.095$ m

and, $B = 7.54$ m

Therefore, the required depth is $(3.095 + 1) = 4.095$ m and bed width is 7.54 m [1 m added as a free board].

Example 4.8

In example 4.7 if $\frac{B}{y}$ ratio = 2.5 and limiting velocity is not given, find B and y.

Solution:

i) $A = By + y^2(0.588 + 1.5) = By + 2.088y^2$

$P = B + 2y(0.588 + 1.5) = B + 4.176y$

$R = \frac{A}{P} = \frac{By + 2.088y^2}{B + 4.176y}$

ii) $V = \frac{Q}{A} = \frac{250}{By + 2.088y^2}$

iii) $V = \frac{1}{N} R^{\frac{2}{3}} S^{\frac{1}{2}}$

or, $\frac{250}{By + 2.088y^2} = \frac{1}{0.017}\left[\frac{By + 2.088y^2}{B + 4.176y}\right]^{\frac{2}{3}} (0.0004)^{\frac{1}{2}}$

Using B = 2.5y,

$$\frac{250}{2.5y^2 + 2.088y^2} = 1.176\left[\frac{2.5y^2 + 2.088y^2}{2.5y + 4.176y}\right]^{\frac{2}{3}}$$

or, $\frac{250}{4.588y^2} = 1.176\left[\frac{4.588y^2}{6.676y}\right]^{\frac{2}{3}}$

or, $46.335 = y^2 \times 0.779 \times y^{\frac{2}{3}}$

or, $59.5 = y^{\frac{8}{3}}$

∴ $y = 4.63$ m $= 4.63 + 1 = 5.63$ m

and, $B = 4.63 \times 2.5 = 11.575$ m

Example 4.9

Design a concrete lined channel to carry a discharge of 350 cumecs at a slope of 1 in 5000. The side slopes of the channel may be taken as 1.5 : 1. The value of n for lining is 0.014. Assume limiting velocity in the channel as 2 m/sec.

Solution:

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

or, $2 = \frac{1}{0.014} R^{\frac{2}{3}} \left(\frac{1}{5000}\right)^{\frac{1}{2}}$

Solving this we get;

$R = 2.79$ m (1)

For side slope of 1.5 : 1; we have,

$\cot\theta = 1.5$

or, $\theta = 34.1° = 0.59$ radians

Using,

$A = y\,(B + y\theta + y\cot\theta)$

and, $P = B + 2y\theta + 2y\cot\theta$

We get,

$A = y\,(B + 0.59y + 1.5y)$

and, $P = B + 2y\,0.59 + 2y \times 1.5$

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

∴ $\frac{175}{y} = B + 2.09y$

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

From equation (1); we get,

$2.79 = R = \frac{A}{P} = \frac{175}{B + 4.18y}$

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

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

Solving the above relation; we get,

$y = 3.2$ m

and, $B = \frac{175}{3.2} - 2.09 \times 3.2 = 48$ m

Using free board of 1 m,

Depth of lined canal = 3.2 + 1 = 4.2 m

∴ Depth of canal y = 4.2 m

and, Bed width B = 48 m

Example 4.10

Design a concrete lined channel to carry a discharge of 350 cumecs at a slope of 1 in 6400. The side slopes of the channel may be taken as 1.5 : 1. The value of n for lining material may be taken as 0.013. Assume limiting water depth of the channel as 4.0 m.

Solution:

For a trapezoidal channel,

$A = y\,(B + y\theta + y\cot\theta)$

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

For the channel side slope of 1.5 : 1;

$\cot\theta = 1.5$

or, $\theta = 0.59$ radian

∴ $A = y\,(B + 0.59y + 1.5y)$

$P = B + 1.18y + 3y = B + 4.18y$

Given that;

$y = 4$ m

so, $A = 4\,(B + 2.09 \times 4) = 4B + 33.44$

$P = B + 1.18\,y + 3y = B + 4.18 \times 4 = B + 16.72$

Hydraulic mean depth $(R) = \frac{A}{P} = \frac{4B + 33.44}{B + 16.72}$

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

or, $$350 = \frac{1}{0.013} \times (4B + 33.44) \times \left(\frac{4B + 33.44}{B + 16.72}\right)^{\frac{2}{3}} \left(\frac{1}{6400}\right)^{\frac{1}{2}}$$

or, $$364 = \frac{(4B + 33.44)^{\frac{5}{3}}}{(B + 16.72)^{\frac{2}{3}}}$$

Solving this equation; we get,

$B = 32.5$ m

Adopting free board of 1 m,

Depth of channel $y = 4 + 1 = 5$ m

Example 4.11

Design a lined channel to carry a discharge of 20 cumec. Assume suitable value of side slope and lining material as concrete with gunited finish. Longitudinal slope is 1 in 7000.

Solution:

Let the channel section be triangular, since discharge is small.

Let side slope be 1.5 : 1 then,

$\cot\theta = 1.5$

$\theta = \cot^{-1}(1.5) = 0.588$ radians

Now,

i) $A = y^2(\theta + \cot\theta) = y^2(0.588 + 1.5) = 2.088y^2$

$P = 2y(\theta + \cot\theta) = 4.176y$

$R = \frac{A}{P} = \frac{2.088y^2}{4.176y}$

ii) $V = \frac{Q}{A} = \frac{20}{2.088y^2}$

iii) $V = \frac{1}{N} R^{\frac{2}{3}} S^{\frac{1}{2}}$

or, $$\frac{20}{2.088y^2} = \frac{1}{N}\left(\frac{y}{2}\right)^{\frac{2}{3}} \left(\frac{1}{7000}\right)^{\frac{1}{2}}$$

Assume, rugosity coefficient $N = 0.018$ for concrete with gunited finish from table 4.6.

or, $$\frac{20}{2.088y^2} = \frac{1}{0.018}\left(\frac{y}{2}\right)^{\frac{2}{3}} \left(\frac{1}{7000}\right)^{\frac{1}{2}}$$

$\therefore$ $y = 3.235$ m

Providing 1 m free board, Total depth $= 4.235$ m.

4.6.4 Economic justification for the lining of canal

The lining of channel will be economically justified if the extra cost of providing the lining is less than or equal to the value of the benefits resulting from it. Thus, the annual cost of lining and its annual benefit are worked out to judge the economy of lining, which are as follows:

I) Annual benefits

Let, the rate of irrigation water to be sold to the cultivators per cumec be 'R_1'.

If 'm' cumec of water is saved by lining; then,

Money saved by lining $= mR_1$

Lining also reduces the maintenance cost. If average annual maintenance cost be R_2 and 'p' be the percentage fraction of saving achieved in maintenance cost then,

Amount saved $= pR_2$

$\therefore$ Total annual benefits $= mR_1 + pR_2$ (1)

II) Annual cost

Let the capital expenditure on lining be 'C' rupees. If the life of the lining is 'Y' years; then,

Annual cost of lining $= \frac{C}{Y}$

If 'r' be the interest rate,

Interest earned by capital 'C' per year $= C\frac{r}{100}$

Since, the capital value of asset decreases from 'C' to zero in 'Y' years, the average annual interest cost may be taken as $\frac{C}{2}\frac{r}{100}$ rupees.

$\therefore$ Total annual costs of lining $= \frac{C}{Y} + \frac{C}{2}\frac{r}{100}$

III) Benefit cost ratio

$$B/C \text{ ratio} = \frac{mR_1 + pR_2}{\frac{C}{Y} + \frac{C}{2}\frac{r}{100}}$$ (generally p is taken as 0.4.)

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

Example 4.12

An unlined canal giving a seepage loss of 3.3 cumecs per million sq. metres of wetted area is proposed to be lined with 10 cm thick cement concrete lining which costs Rs. 180.00 per 10 m². Given the following data; work out the economics of lining and benefits cost ratio.

Annual revenue per cumec of water from all crops = Rs. 3.5 lakhs

Discharge in the channels = 83.5 cumecs

Area of the channel = 40.8 m²

Wetted perimeter of the channel = 18.8 m

Wetted perimeter of the lining = 18.5 m

Annual maintenance cost of unlined channel per 10 m² = Rs. 1.00

Assume suitable data if required.

Solution:

Consider a 1 km reach of the channel.

∴ Wetted surface per km = 18.8 × 1000 = 18800 sq. m

i) Annual benefits

a) Seepage losses in unlined canal @ 3.3 cumecs per million sq. m is;

$$= \frac{3.3}{10^6} \times 18800 = 62040 \times 10^{-6} \text{ cumecs/km}$$

Assume seepage loss in lined channel @ 0.01 cumec per million sq m of wetted perimeter.

$$\therefore \text{ Seepage loss in lined canal } = 0.01 \times \frac{18800}{10^6}$$

$$= 188 \times 10^{-6} \text{ cumecs/km}$$

$$\text{Net saving } = 62040 \times 10^{-6} - 188 \times 10^{-6}$$

$$= 61852 \times 10^{-6} \text{ cumecs/km}$$

$$\text{Annual revenue saved per km of channel } = \text{Rs. } 61852 \times 10^{-6} \times 3.5 \text{ lakhs}$$

$$= \text{Rs. } 21648$$

b) Saving in maintenance

Annual maintenance cost of unlined channel for 10 sq. m is Re. 1.00

Wetted perimeter per 1 km length = 18800 sq. m

Annual maintenance charge for unlined channel per km is Rs. 1880

Assuming 40% of this is saved in lined channel,

Annual saving in maintenance charges = Rs. 0.4 × 1880

= Rs. 752

∴ Total annual benefits per km = Rs. 21648 + Rs. 752

= Rs. 22400

c) Annual costs

Area of lining per km of channel = 18.5 × 1000

= 18500 sq. m

Cost of lining per km of channel @ Rs. 180.00 per 10 sq. m is;

$$= \frac{18500 \times 180}{10} = \text{Rs. } 3{,}33{,}000$$

Assume life of lining as 40 yrs.;

$$\text{Depreciation cost per year} = \frac{\text{Rs. } 333000}{40} = \text{Rs. } 8325$$

Assume 5% rate of interest

$$\text{Average annual interest} = 0.5 \times C \times \frac{r}{100}$$

$$= 0.5 \times 333000 \times \frac{5}{100} = \text{Rs. } 8325$$

Total annual cost = Rs. 8325 + Rs. 8325 = Rs. 16,650

Now,

$$\text{B/C ratio} = \frac{\text{Annual benifits}}{\text{Annual costs}} = \frac{22400}{16650} = 1.35$$

Since, B/C ratio is more than unity, and hence, the lining is justified.

4.7 WORKED OUT PROBLEMS

PROBLEM 1

Using Lacey's basic equation, establish a Relationship between R, Q and f, where symbol have usual meanings.

Solution:

Lacey's basic equation is,

i) Here,

$$V = \sqrt{\frac{2}{5} fR}$$

or, $R = \frac{5}{2}\frac{V^2}{f}$ [Squaring on both sides] **(1)**

ii) $Af^2 = 140\,V^5$ **(2)**

From equation **(2)**; we have,

$$Af^2 = 140\,V^5$$

Multiplying both sides by 'V'; we have,

$$AVf^2 = 140\,V^6$$

or, $Qf^2 = 140\,V^6$ [Q = AV]

or, $V = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}}$ **(3)**

From equation **(1)** and **(3)**; we have,

$$R = \frac{5}{2} \times \frac{1}{f} \times \left(\frac{Qf^2}{140}\right)^{\frac{1}{3}} \quad \textbf{(4)}$$

or, $R = 0.481\left(\frac{Q}{f}\right)^{\frac{1}{3}}$

Generally, the constant is taken as 0.47.

$$\therefore \quad R = 0.47\left(\frac{Q}{f}\right)^{\frac{1}{3}}$$

This is the relation between 'R', 'Q' and 'f'.

PROBLEM 2

Design a canal using Kennedy formulation with the following data. Q = 10 cumec, manning's roughness coefficient = 0.0245 slope of bed 0.0002, m = 1 and side slope of canal 0.5 : 1 (H : V)

Solution:

Given that;

$Q = 30\ m^3/sec.$

m = 1

n = 0.245

Side slope = 0.5 : 1

i.e., $Z = 0.5$

We have,

$$V_0 = 0.55my^{0.64}$$

$$A = \frac{Q}{V_0} = \frac{30}{V_0}$$

$$A = By + Zy^2$$

$$B = \left(\frac{A - Zy^2}{y}\right) = \frac{(A - 0.5y^2)}{y}$$

$$P = B + 2y\sqrt{1 + Z^2} = B + 2y\sqrt{1 + 0.5^2}$$

$$R = \frac{A}{P}$$

$$V = \frac{1}{n}R^{\frac{2}{3}}S^{\frac{1}{2}} = \frac{1}{0.0245}R^{\frac{2}{3}}(0.0002)^{\frac{1}{2}}$$

Now assuming first trial depth, $y = 1.0$ m

Trial no.	Y	V_0	A	B	P	R	V	Remarks
1	1.0	0.55	54.545	54.045	56.282	0.969	0.565	$V_0 \neq V$
2	2	0.857	35.003	16.501	20.973	1.669	0.812	$V_0 \neq V$
3	1.2	0.618	48.538	39.848	42.532	1.141	0.63	$V_0 \neq V$
4	1.3	0.651	46.114	34.822	37.729	1.222	0.66	$V_0 \neq V$
5	1.5	0.713	42.078	27.302	30.656	1.373	0.713	$V_0 = V$

$\therefore$ $y = 1.811$ m

and, $B = 19.902$ m

PROBLEM 3

An irrigation canal carrying a discharge of 40 m³/sec. has to be constructed in alluvial soil. If the mean diameter of the soil is 0.5mm, design a suitable section and bed slope of such a canal.

Solution:

Given that;

$$Q = 40 \text{ Cumecs}$$

$$d_{mm} = 0.5 \text{ mm}$$

$$f = 1.76\sqrt{d_{mm}} = 1.76\sqrt{0.5} = 1.245$$

Using Lacey's theory; we have,

$$\text{Velocity (V)} = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}} = \left(\frac{40 \times 1.245^2}{140}\right)^{\frac{1}{6}} = 0.873 \text{ m/s}$$

$$A = \frac{Q}{V} = \frac{40}{0.873} = 45.819 \text{ m}^2$$

$$R = \frac{5}{2}\left(\frac{V^2}{f}\right) = \frac{5}{2}\left(\frac{0.838^2}{1.245}\right) = 1.41$$

$$P = 4.75\sqrt{Q} = 4.75\sqrt{40} = 30.042 \text{ m}$$

Assume side slope of channel, 0.5H : 1V; then,

$$P = b + \sqrt{5}y$$

and, $A = \left(b + \frac{y}{2}\right)y$

$$30.042 = b + \sqrt{5}y \quad (1)$$

and, $45.819 = by + \frac{y^2}{2}$ (2)

From equation (1); we have,

$$b = 30.042 - \sqrt{5}y$$

Putting the value of 'b' in equation (2); we get,

$$45.819 = (30.042 - \sqrt{5}y)y + \frac{y^2}{2}$$

or, $45.819 = 30.042y - \sqrt{5}y^2 + \frac{y^2}{2}$

or, $1.736y^2 - 30.042y + 45.819 = 0$

Solving this equation; we get,

$y = 1.69$ m (neglecting unfeasible value)

and, $b = 30.042 - \sqrt{5} \times 1.69 = 26.263$ m

$$S = \left(\frac{f^{\frac{5}{3}}}{3340Q^{\frac{1}{6}}}\right) = \left(\frac{1.245^{\frac{5}{3}}}{3340 \times 40^{\frac{1}{6}}}\right) = 0.00023 = 1 \text{ in } 4287$$

$y = 1.69$ m

$b = 26.263$ m

$S = 0.00023$ or 1 in 4287

PROBLEM 4

Draw neat sketch of cross-sections of a canal in cutting, filling and balanced mode, showing all features.

Solution:

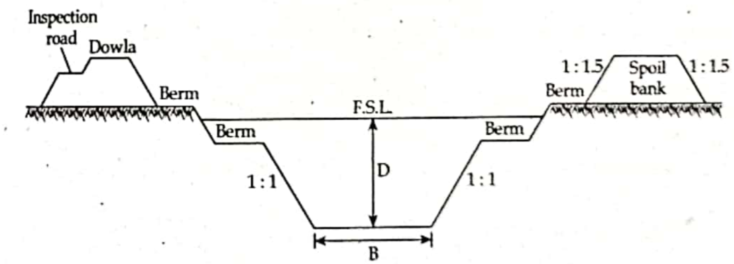


Figure: Channel in cutting (Fully cutting)

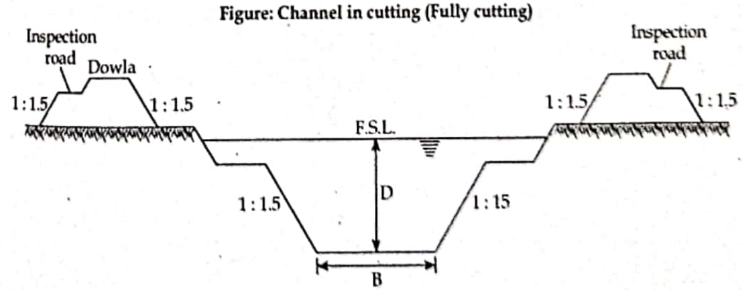


Figure: Channel in filling (Fully filling)

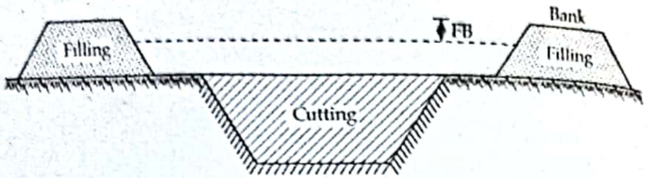


Figure: Balanced section (Cutting = Filling)

PROBLEM 5

From the data given below, design a stable irrigation canal, ensuring the stability of particles at bed as well as sides.

Discharge (Q) = 20 m³/sec.	**S = 0.001**
Particle size (d) = 4 cm	**Side slope = 30°**
Angle of repose of soil = 38°	

Solution:

Given that;

$$Q = 20 m^3/sec.$$
$$S = 0.001$$
$$d = 4 cm$$
$$\theta = 30°$$
$$\phi = 38°$$

Now,

$$\frac{\tau_S}{\tau_L} = \cos\theta\sqrt{1-\frac{\tan^2\theta}{\tan^2\phi}} = \cos\theta\sqrt{1-\frac{\tan^2 30°}{\tan^2 38°}} = 0.473$$

where, τ_S = Shear stress on slope.

τ_L = Shear stress on level surface.

Minimum shear stress required to carry away grain on side slope is given by,

$$\tau_S = 0.0473\,\tau_L$$

For stability of channel, shear stress must be less than or equal to $0.473\tau_L$

i.e., $\tau_0 \le 0.473\tau_l$

Also, the stress actually developed on the side slope of the channel is;

$$\tau_0 = 0.75\gamma_w RS$$

or, $0.75\gamma_w RS \le 0.473\tau_l$

But, $\tau_L = \gamma_w RS = \gamma_w \frac{d}{11}$ [d = 11RS]

$\therefore$ $0.75\gamma_w RS \le 0.473\gamma_w \frac{d}{11}$

or, $RS \le 0.057d$

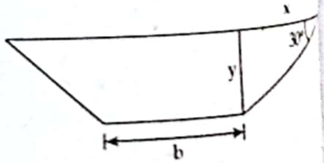
or, $RS \le (0.057 \times 0.04)$ m

or, $R \times 0.001 \le 0.002$

or, $R \le 2.28$ m

Assuming $y \approx R$; we have,

$$y \le 2.28 \text{ m}$$



Increasing by 20%; we have,

$$y = 2.74 \text{ m}$$

From the given figure; we have,

$$\frac{y}{x} = \tan 30°$$

or, $x = \frac{y}{\tan 30°} = 4.746$

Now,

$$A = y(b + x) = 2.74(b + 4.746)$$
$$P = b + 2\sqrt{x^2 + y^2} = b + 2\sqrt{22.52 + 7.51} = b + 10.96$$
$$n = \frac{1}{24}d^{\frac{1}{6}} = \frac{1}{24}0.04^{\frac{1}{6}} = 0.024$$

Using manning's formula; we have,

$$V = \frac{1}{n}\cdot R^{\frac{2}{3}}\cdot S^{\frac{1}{2}} = \frac{1}{0.024}\cdot R^{\frac{2}{3}}\cdot 0.001^{\frac{1}{2}} = 1.318R^{\frac{2}{3}}$$

Now, choosing trial values of 'b' and proceed to make 'Q' equal to 40 cumecs.

b	A = 2.74(b + 4.74)	P = b + 10.96	$R = \frac{A}{P}$	$V = 1.318R^{\frac{2}{3}}$	Q = AV	Remarks
3	21.224	13.96	1.52	1.743	36986	<40
4	23.964	14.96	1.602	1.804	43.241	>40
3.45	22.457	14.41	1.558	1.772	39.786	<40
3.49	22.567	14.45	1.562	1.774	40.036	≈40

$\therefore$ b = 3.49

and, y = 2.74 m

PROBLEM 6

For the given data below, design a regime channel

Q = 30 m³/sec.	**S = 1 in 4000**
Critical velocity ratio = 1.0	**Kutter's value = 0.022**

Solution:

Here,

$$V_0 = 0.55my^{0.64}$$

or, $V_0 = 0.55\,y^{0.64}$

$$A = \frac{Q}{V_0} = \frac{30}{V_0}$$
$$A = By + Zy^2$$

or, $B = \left(\frac{A - Zy^2}{y}\right) = \frac{(A - 1y^2)}{y} = \frac{(A - 1y^2)}{y}$ [Assume Z = 1]

$$P = B + 2y\sqrt{1 + Z^2} = B + 2y\sqrt{2} = B + 2.83y$$
$$R = \frac{A}{P}$$

Using Kutter's formula; we have,

$$V = \left[\frac{\frac{1}{0.022} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{0.022}{\sqrt{R}}}\right]\sqrt{RS}$$

Now, assuming value of y and proceeding with successive trial in the tabular form,

Trial number	Y	V_0	A	B	P	R	V	Remarks
1	1	0.55	54.545	53.545	59.205	0.921	0.679	$V_0 \neq V$
2	1.5	0.713	42.078	26.552	35.042	1.201	0.815	$V_0 \neq V$
3	1.6	0.743	40.376	23.635	32.691	1.235	0.831	$V_0 \neq V$
4	1.8	0.801	37.444	19.002	29.19	1.283	0.853	$V_0 \neq V$
5	1.9	0.829	36.171	17.137	27.891	1.297	0.859	$V_0 \neq V$
6	2	0.857	35.003	15.501	26.821	1.305	0.863	$V_0 \approx V$

$\therefore \quad y = 2\text{ m}$

and, $B = 15.501\text{ m}$

PROBLEM 7

The sides of an irrigation canal with the following design parameters are well protected. What will be the depth and bed width of such a canal?

$Q = 5$ cumecs

$d_{50} = 4\text{ cm} = 40\text{ mm}$

$i = \frac{1}{1000} = S =$ Bed slope

Solution:

For given size of particles; we have,

$$f = 1.76\sqrt{d_{mm}} = 1.76\sqrt{40} = 11.131$$

Now, using Lacey's equations; we have,

$$V = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}} = \left(\frac{5 \times 11.131^2}{140}\right)^{\frac{1}{6}} = 1.281\text{ m/sec.}$$

$$A = \frac{Q}{V} = \frac{5}{1.281} = 3.9\text{ m}^2$$

$$R = \frac{5}{2}\left(\frac{V^2}{f}\right) = \frac{5}{2}\left(\frac{1.281^2}{11.131}\right) = 0.369\text{ m}$$

$$P = 4.75\sqrt{Q} = 4.75\sqrt{5} = 10.621\text{ m}$$

Assume a trapezoidal canal with side slope 0.5H : 1V

$$P = B + \sqrt{5}y$$

and, $A = \left(b + \frac{y}{2}\right)y$

so, $\quad 10.621 = b + \sqrt{5}y$

$$b = 10.621 - \sqrt{5}y$$

$$A = by + \frac{y}{2}y$$

or, $\quad 3.9 = (10.621 - \sqrt{5}y)y + \frac{y^2}{2}$

or, $\quad 3.9 = 10.621y - \sqrt{5}y^2 + 0.5y^2$

or, $\quad 1.736y^2 - 10.621y + 3.9 = 0$

Solving; we get,

$$y = 5.726$$

and, 0.392 m

$\therefore \quad y = 0.392$ m (Neglecting unfeasible value)

and, $b = 10.621 - \sqrt{5} \times 0.392 = 9.744$ m

PROBLEM 8

Find out the depth of an irrigation canal having the following data.

Discharge (Q) = 30 m^3/sec.

Bed slope = 1 in 3000

Bed width = 6.0 m

Manning's rugosity coefficient = 0.019

What is the mean particle size of the canal bed material?

Solution:

Given that;

$$Q = 30\text{ cumecs}$$

$$S = 1\text{ in }3000 = 0.00033$$

$$B = 6\text{ m}$$

$$n = 0.019$$

Assume canal side slope 1.5H : 1V

$$A = By + Zy^2 = 6y + 1.5y^2$$

$$P = B + 2y\sqrt{1 + Z^2} = 6 + 2y \times 1.803$$

$$P = 6 + 3.606y$$

$$R = \frac{A}{P} = \frac{6y + 1.5y^2}{6 + 3.606y}$$

$$Q = AV = (6y + 1.5y^2)\cdot\frac{1}{n}\cdot R^{\frac{2}{3}}\cdot S^{\frac{1}{2}}$$

or, $\quad 30 = (6y + 1.5y^2)\cdot\frac{1}{0.019}\cdot\left(\frac{6y + 1.5y^2}{6 + 3.606y}\right)^{\frac{2}{3}}\cdot(0.00033)^{\frac{1}{2}}$

or, $\quad 30 = \frac{(6y + 1.5y^2)^{1.667}}{(6 + 3.606y)^{\frac{2}{3}}} \times 0.96044$

Solving this equation; we get,

$$y = 2.402\text{ m}$$

Again, we have,

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

$$= \frac{1}{0.019} \cdot \left(\frac{6y + 1.5y^2}{6 + 3.606y}\right)^{\frac{2}{3}} \cdot (0.00033)^{\frac{1}{2}}$$

$$= 1.23 \text{ m/sec.}$$

Using Lacey's formula; we have,

$$V = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}}$$

or, $(1.23)^6 = \frac{30 \times f^2}{140}$

or, $f = 4.02$

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

or, $d_{mm} = \left(\frac{4.02}{1.76}\right)^2 = 5.22 \text{ mm}$

∴ Average size of bed material = 5.22 mm

PROBLEM 9

Explain sediment transport and tractive force approach of canal design. [2069 Bhadra]

Solution: See the definition part 4.2 and 4.3 (Include procedure)

PROBLEM 10

Assuming the side slope 1 : 1, find the bed width and the depth of flow of an irrigation canal to carry a discharge of 21 m³/sec. with a velocity of 0.75 m/sec. Bed slope is 1 : 5000 and value of Chezy's constant C = 42. [2069 Poush]

Solution:

Given that;

$Q = 21 \text{ m}^3/\text{sec.}$

$Z = 1$

$S = 1:5000 = 0.0002$

Chezy's constant (C) = 42

Now,

Velocity (V) = 0.75 m/sec.

Area (A) $= \frac{Q}{V} = \frac{21}{0.75} = 28 \text{ m}^2$

$A = by + Zy^2$

or, $28 = by + 1y^2$

or, $b = \left(\frac{28 - y^2}{y}\right)$

$P = b + 2y\sqrt{1 + Z^2} = b + 2.83y$

$R = \frac{A}{P}$

Now, taking trial value for 'y', to get the value of 'V' equal to 0.75;

Trail number	y	b	P	R	V	Remarks
1	0.6	46.067	47.765	0.586	0.455	$V \neq 0.75$
2	0.9	30.211	32.758	0.855	0.549	$V \neq 0.75$
3	1.3	20.238	23.917	1.171	0.643	$V \neq 0.75$
4	1.8	13.756	18.850	1.485	0.724	$V \neq 0.75$
5	1.9	12.837	18.214	1.537	0.736	$V \neq 0.75$
6	2	12.000	17.660	1.586	0.748	$V \neq 0.75$
7	2.02	11.841	17.558	1.595	0.750	V = 0.75 m/sec.

∴ y = 2.02 m

and, b = 11.841 m

PROBLEM 11

Design of canal by Lacey's regime theory. [2069 Poush]

Solution: See the definition part. 4.5.2

PROBLEM 12

A stable channel is to be designed for a discharge of 40 m³/sec. and the silt factor of unity. Calculate the dimensions of the channel using Lacey's theory. What would be the bed width of this channel if it were to be designed on the basis of Kennedy's method with critical velocity ratio equal to unity and the ratio of bed-width to depth of flow the same as obtained from Lacey's method. [2070 Bhadra]

Solution:

Given that;

$Q = 40\text{m}^3/\text{sec.}$

$f = 1$

Using Lacey's theory; we have,

$$\text{Velocity of flow (V)} = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}} = \left(\frac{40 \times 1^2}{140}\right)^{\frac{1}{6}} = 0.812 \text{ m/sec.}$$

$$\text{Hydraulic depth (R)} = \frac{5}{2}\frac{V^2}{f} = \frac{5}{2} \times \frac{0.812^2}{1} = 1.648 \text{ m}$$

$$A = \frac{Q}{V} = \frac{40}{0.812} = 49.261 \text{ m}^2$$

$$P = 4.75\sqrt{Q} = 4.75\sqrt{40} = 30.042 \text{ m}$$

Assume side slope of channel 0.5H : 1V; then,

$A = BD + 0.5D^2$

$49.261 = BD + 0.5D^2 \quad (1)$

Also,

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

or, $30.042 = B + 2D\sqrt{1.25}$

or, $B = 30.042 - 2.236D$

Substituting the value of 'B' in equation (1); we get,

$$49.261 = 30.042D - 2.236D^2 + 0.5D^2$$

or, $1.736D^2 - 30.042D + 49.261 = 0$

Solving this equation; we get,

$D = 1.834$ m or 15.471 (not feasible)

$B = 30.042 - 2.236 \times 1.834 = 25.941$ m

$\therefore$ $B = 25.941$ m

and, $D = 1.834$ m

For second case,

$CVR = m = 1$

$$\frac{B}{D} = \frac{25.941}{1.834} = 14.144$$

$B = 14.144 \times D$

Now,

$V_0 = 0.55 \times 1 \times y^{0.64}$

$$A = \frac{Q}{V_0} = \frac{40}{V_0}$$

or, $BD + 0.5D^2 = \dfrac{40}{0.55D^{0.64}}$

or, $(BD + 0.5D^2) \times 0.55D^{0.64} = 40$

Solving equation (1) and (2); we get,

$D = 1.835$ m

and, $B = 14.144 \times 1.835 = 25.955$ m

PROBLEM 13

The banks of an irrigation channel conveying a flow of 12.0 m³/sec. are well protected with masonry structure with side slope as 0.5 : 1 (H : V). The canal has to pass through the medium having mean particles as 4 cm. If the bed slope of the canal is 1 : 2000, find the stable bed width. Draw the sketch of the designed section. Assume that the roughness is same along the whole perimeter of the section. [2070 Magh]

Solution:

Given that;

$Q = 12.0$ m³/sec.

$Z = 0.5$

$d_{mm} = 40$ mm

$f = 1.76\sqrt{d_{mm}} = 11.131$

$S = 1 : 2000$

Using Lacey's theory; we have,

$$\text{Velocity of flow (V)} = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}} = \left\{\frac{12 \times (11.131)^2}{140}\right\}^{\frac{1}{6}} = 1.483 \text{ m/sec.}$$

$$\text{Hydraulic mean depth (R)} = \frac{5}{2}\left(\frac{V^2}{f}\right) = \frac{5}{2}\left(\frac{1.483^2}{11.131}\right) = 0.494 \text{ m}$$

$$A = \frac{Q}{V} = \frac{12}{1.483} = 8.092 \text{ m}^2$$

$P = 4.75\sqrt{Q} = 4.75\sqrt{12} = 16.454$ m

Assuming the side slope 0.5H : 1V; we have,

$A = By + 0.5y^2$

$8.092 = By + 0.5y^2$ (1)

and, $P = B + 2y\sqrt{1+Z^2}$

or, $16.454 = B + 2.236y$

or, $B = 16.454 - 2.236y$ (2)

Substituting equation (2) in (1); we get,

$8.092 = 16.454y - 2.236y^2 + 0.5y^2$

or, $1.736y^2 - 16.454y + 8.092 = 0$

Solving this equation; we get,

$y = 0.52$ m (Neglecting the unfeasible value)

and, $B = 15.291$ m

PROBLEM 14

Sides of an irrigation canal with the following design parameters are well protected. What will be the stable depth and bed width of such a canal?
$Q = 5$ m³/sec., $d_{50} = 3$ cm, $i = 1$ in 500 **[2071 Bhadra]**

Solution:

For given size of particles we have,

$d = 1.76\sqrt{d_{mm}} = 1.76\sqrt{30} = 9.64$

Now, using Lacey's equation, we have,

$$V = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}} = \left[\frac{5 \times (9.64)^2}{140}\right]^{\frac{1}{6}} = 1.22 \text{ m/sec.}$$

$$A = \frac{Q}{V} = \frac{5}{1.22} = 4.1 \text{ m}^2$$

$$R = \frac{5}{2}\left(\frac{V^2}{f}\right) = \frac{5}{2}\left(\frac{1.22^2}{9.64}\right) = 0.386$$

$P = 4.75\sqrt{Q} = 4.75\sqrt{5} = 10.621$ m

Assume a trapezoidal canal with side slope 0.5 H:1 V.

$P = B + \sqrt{5}y$

and, $A = \left(B + \frac{y}{2}\right) \cdot y$

so, $10.621 = b + \sqrt{5}y$

$B = 10.621 - \sqrt{5}y$ (1)

and, $4.1 = \left(B + \frac{y}{2}\right) \cdot y$ (2)

Substituting equation (1) in (2); we get,

$$4.1 = \left(10.621 - \sqrt{5}y + \frac{y}{2}\right) \cdot y$$

or, $4.1 = 10.621y - 1.736y^2$

or, $1.736y^2 - 10.621y + 4.1 = 0$ (3)

Solving equation (3) for quadratic in 'y'; we get,

$y = 0.414$ or 5.70 (Not feasible)

$\therefore$ $y = 0.414$ m

$\therefore$ $B = 9.7$ m

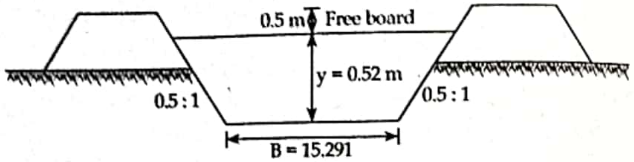


Figure: Designed channel section

PROBLEM 15

The slope of a channel in alluvium is $\frac{1}{6000}$. Find the channel section and maximum discharge which can be allowed to flow in it. Take f = 1.0. [2071 Bhadra]

Solution:

Given that,

$$S = \frac{1}{6000}$$

$$f = 1.0$$

Now, using equation (4.29); we get,

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

$$Q = \left(\frac{f^{\frac{5}{3}} \times 6000}{3340 \times 1}\right)^6 = 33.60 \text{ m}^3/\text{sec.}$$

$\therefore$ Discharge of 33.60 cumecs is allowed to flow in this channel.

Now, to find section:

i) $V = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}} = \left(\frac{33.6 \times 1^2}{140}\right)^{\frac{1}{6}} = 0.788$ m/sec.

ii) $A = \frac{Q}{V} = \frac{33.6}{0.788} = 42.64 \text{ m}^2$

$P = 4.75\sqrt{Q} = 4.75\sqrt{33.6} = 27.53$ m

$R = \frac{A}{P} = \frac{42.64}{27.53} = 1.53$ m

iii) *Assuming trapezoidal channel with side slope 0.5 H : 1V. Then,*

$$A = \left(B + \frac{y}{2}\right) \cdot y = 42.64$$

$$P = B + \sqrt{5}y = 27.53$$

or, $B = 27.53 - \sqrt{5}y$

Then,

$27.53y - \sqrt{5}y^2 + 0.5y^2 - 42.64 = 0$

or, $1.736y^2 - 27.53y + 42.64 = 0$

Solving; we get,

$y = 1.74$ m

and, $B = 23.64$ m

PROBLEM 16

What are Lacey's basic regime equations? Starting from these equations, derive the equation for wetted perimeter. [2071 Magh]

Solution: See the definition part 4.5.2.1 (3)

PROBLEM 17

Design an unlined channel is alluvial soil by the tractive force approach for a discharge of 50 cumec from the following data.

i) Bed slope = $\frac{1}{5000}$

ii) Side slope = 0.5:1

iii) Manning's N = 0.022

iv) Permissible tractive stress = 0.0025 kN/m². [2071 Magh]

Solution:

Given that;

$Q = 50 \text{ m}^3/\text{sec.}$

$S = 0.0002$

$N = 0.022$

$Z = 0.5(\tau_c)$

Permissible tractive stress $= 0.0025 \text{ kN/m}^2$

Now,

i) $\tau_c = \gamma_w RS$

or, $2.5 = 9810 \times R \times 0.0002$

$\therefore$ $R = 1.274$ m

ii) $V = \frac{1}{N} R^{\frac{2}{3}} S^{\frac{1}{2}} = \frac{1}{0.022}(1.274)^{\frac{2}{3}}(0.0002)^{\frac{1}{2}} = 0.755$ m/sec.

iii) $A = \frac{Q}{V} = \frac{50}{0.755} = 66.225 \text{ m}^2$

iv) $P = \frac{A}{R} = \frac{66.225}{1.274} = 51.98$ m

Now,

$$A = \left(B + \frac{y}{2}\right) \cdot y = 66.225 \quad (1)$$

$$P = B + \sqrt{5}y = 51.98$$

or, $B = 51.98 - \sqrt{5}y$ (2)

Substituting equation (2) in (1): we get,

$$51.98y - 1.736y^2 = 66.225$$

or, $1.736y^2 - 51.98y + 66.225 = 0$ (3)

Solving equation (3); we get,

$\therefore$ $y = 1.33$ m [Neglecting unfeasible value]

and, $B = 49$ m

PROBLEM 18

Design an irrigation channel to carry a discharge of 50 cumecs at a slope of 1 in 5000. Take Kutter's N = 0.0225 and m = 0.9.

Solution:

Given that,

$Q = 50$ cumecs

$S = 1 \text{ in } 5000 = 0.0002$

$N = 0.0225$

$m = 0.9$, Assume side slope 1 : 1, *i.e.*, $Z = 1$

Now,

$V_0 = 0.55\, my^{0.64}$

or, $V_0 = 0.55 \times 0.9 \times y^{0.64}$

or, $V_0 = 0.495y^{0.64}$ (1)

$$A = \frac{Q}{V_0} = \frac{50}{V_0} \quad (2)$$

$A = By + Zy^2$

or, $B = \left(\frac{A - y^2}{y}\right)$ (3)

$P = B + 2y\sqrt{2} = B + 2.83y$ (4)

$R = \frac{A}{P}$ (5)

Using Kutter's formula; we have,

$$V = \frac{\frac{1}{0.0225} + \left(23 + \frac{0.00155}{0.0002}\right)}{1 + \left(23 + \frac{0.00155}{0.0002}\right)\frac{0.0225}{\sqrt{R}}}\sqrt{RS}$$

Now, assuming value of 'y' and proceeding with successive trial in the tabular form.

Trial no.	Y	V_0	A	B	P	R	V	Remarks
1	1	0.495	101.010	100.010	102.84	0.982	0.621	$V_0 \neq V$
2	2	0.770	64.819	30.41	36.07	1.797	0.94	$V_0 \neq V$
3	3	1.00	50.004	13.668	22.158	2.257	1.094	$V_0 \neq V$
4	3.5	1.104	45.307	9.445	19.35	2.341	1.121	$V_0 \neq V$
5	3.6	1.124	44.497	8.760	18.948	2.348	1.123	$V_0 \approx V$

Hence, adopt $y = 3.6$ m and $B = 8.76$ m.

PROBLEM 19

Derive Lacey perimeter discharge relationship and regime slope equation. [P.U. 2014 Fall]

Solution: See the definition part 4.5.2.1 (3) and (5) b

PROBLEM 20

Mention types of lining. Write about advantage of lining in canal.

Solution: See the definition part 4.6

PROBLEM 21

Design on trapezoidal irrigation canal with side slope 0.5H : 1V to carry 5 cumec. The canal is to be laid at slope of 1 in 4000. The critical velocity ratio for the soil is 1.1. Use Kutter's rugosity coefficient as 0.023.

Solution:

Given that;

$Q = 5\ m^3/sec.$

$Z = 0.5$

$S = \frac{1}{4000} = 0.00025$

$m = 1.1$

$N = 0.023$

Now,

$V_0 = 0.55 \times 1.1 \times y^{0.64} = 0.605y^{0.64}$ (1)

$$A = \frac{Q}{V_0} = \frac{5}{V_0} \quad (2)$$

$$A = \left(B + \frac{y}{2}\right) \cdot y = By + \frac{y^2}{2}$$

$B = \left(\frac{A - 0.5y^2}{y}\right)$ (3)

$P = B + \sqrt{5}y$ (4)

$R = \frac{A}{P}$ (5)

Using Kutter's formula; we have,

$$V = \frac{\frac{1}{0.0225} + \left(23 + \frac{0.00155}{0.00025}\right)}{1 + \left(23 + \frac{0.00155}{0.00025}\right) \times \frac{0.023}{\sqrt{R}}} \times \sqrt{RS}$$

Now, Assuming 'y' and proceeding as below,

Trial no.	N	V_0	A	B	P	R	V	Remarks
1	0.7	0.482	10.384	14.4894	16.049	0.647	0.504	$V_0 \neq V$
2	0.8	0.524	9.53	11.516	13.305	0.716	0.542	$V_0 \neq V$
3	0.9	0.566	8.841	9.373	11.386	0.776	0.575	$V_0 \neq V$
4	0.95	0.585	8.54	8.515	10.639	0.803	0.588	$V_0 \approx V$

Hence, adopt $y = 0.95$ m and $B = 8.515$ m.

PROBLEM 22

A canal has to be designed to carry a design discharge as 50 m^3/s. The slope of the canal is 1 : 1000 and passes through medium with mean particles as 50 mm. Assuming a trapezoidal section, determine the stable depth of the canal assuming angle of repose of canal bed/side particles as 36°.[2072 Ashwin]

Solution: Proceed same as example 4.2

PROBLEM 23

Design a stable irrigation canal carrying a discharge of 50 m^3/s, which passes through alluvium (d_{mean} = 0.50 mm). Draw a sketch of the designed section. [2072 Ashwin]

Solution:

Given that,

$$Q = 50 \text{ cumec}$$
$$d = 0.5 \text{ mm}$$

We have,

$$f = 1.76\sqrt{d} = 1.76\sqrt{0.5} = 1.25$$

Using Lacey's theory; we have,

$$\text{Velocity of flow (V)} = \left[\frac{Qf^2}{140}\right]^{\frac{1}{6}} = \left[\frac{50 \times (1.25)^2}{140}\right]^{\frac{1}{6}} = 0.907 \text{ m/sec.}$$

$$\text{Hydraulic mean depth (R)} = \frac{5}{2}\left(\frac{V^2}{f}\right) = \frac{5}{2}\left\{\frac{(0.907)^2}{1.25}\right\} = 1.645 \text{ m}$$

$$\text{Area (A)} = \frac{Q}{V} = \frac{50}{0.907} = 55.13 \text{ m}^2$$

$$\text{Perimeter (P)} = 4.75\sqrt{Q} = 4.75\sqrt{50} = 33.6 \text{ m}$$

Assume side slope of channel 0.5H : 1V; then,

$$A = BD + 0.5D^2$$

or, $55.13 = BD + 0.5D^2$ (1)

Also,

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

or, $33.6 = B + 2D\sqrt{1.25}$

or, $B = 33.6 - 2.236D$ (2)

Substituting the value of B in the equation (1); we get,

$$55.13 = (33.6 - 2.236D)D + 0.5D^2$$

or, $1.736D^2 - 33.6D + 55.13 = 0$

Solving this equation; we get,

$$D = 1.81 \text{ m or } 17.54 \text{ m}$$

Taking feasible value only; we have,

$$D = 1.81 \text{ m}$$

∴ $B = 33.6 - 2.236D = 33.6 - 2.236 \times 1.81 = 29.55$ m [From equation (2)]

Hence, B = 29.55 m and D = 1.81 m.

PROBLEM 24

A canal is to be designed to carry a discharge of 32 cumecs. The bed slope is kept 1 in 1500. The soil is coarse alluvium having a grain size of 30 mm. Assuming the canal is trapezoidal and to be unlined with unprotected banks. Determine a suitable section for the canal. Assume $\phi = 37°$ [2072 Magh]

Solution: Proceed same as example 4.2

PROBLEM 25

Using Lacey's regime equation prove that $R = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}}$; where, R = hydraulic mean radius, q = discharge per unit wetted perimeter and f = silt factor. [2072 Magh]

Solution:

Lacey's regime equations are;

$$V = \sqrt{\frac{2}{5}fR} \quad (1)$$

and, $Af^2 = 140\ V^5$ (2)

Squaring equation (1); we get,

$$V^2 = \frac{2}{5}fR$$

or, $R = \frac{5}{2}\frac{V^2}{f}$ (3)

Multiplying both sides of equation (2) by 'v'; we get,

$$AVf^2 = 140\ V^6$$
$$Qf^2 = 140\ V^6 \quad [Q = AV]$$

∴ $V = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}}$ (4)

Squaring equation (4); we get,

$$V^2 = \left(\frac{Qf^2}{140}\right)^{\frac{1}{3}} \quad (5)$$

Substituting equation (5) in (3); we have,

$$R = \frac{5}{2}\left(\frac{Qf^2}{140}\right)^{\frac{1}{3}}\frac{1}{f}$$

∴ $R = 0.47\left(\frac{Q}{f}\right)^{\frac{1}{3}}$ (6)

Here,

$$q = \frac{Q}{P}$$

or, $q = \frac{AV}{\frac{A}{R}}$

or, $q = RV$

From equation (3) and (7); we have,

$$q = \frac{5}{2}\frac{V^3}{f} = \frac{5}{2}\left[\left(\frac{Qf^2}{140}\right)^{\frac{1}{6}}\right]^3 \frac{1}{f} = \frac{5}{2}\left(\frac{Qf^2}{140}\right)^{\frac{1}{2}} \frac{1}{f} = 0.21\,Q^{\frac{1}{2}}$$

$$\therefore \quad Q = \left(\frac{q}{0.21}\right)^2$$

From equation (6); we get,

$$\therefore \quad R = 0.47\left(\frac{q}{0.21}\right)^{\frac{2}{3}} \cdot \frac{1}{f^{\frac{1}{3}}} = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}}$$

PROBLEM 26

A canal is to be designed to carry a discharge of 40 cumecs. The bed slope is kept 1 in 1200. The soil is course alluvium having a grain size of 5 cm. assuming the canal is trapezoidal and to be unlined with unprotected banks. Determine a suitable section for the canal. Assume $\phi = 37°$ [2073 Bhadra]

Solution:

Assume,

Side slope = 30°

and, proceed same as problem 5

PROBLEM 27

The slope of a channel in alluvium is $\frac{1}{4000}$, Lacey's silt factor is 0.9 and side slopes are 0.5 : 1 (H : V). Find the channel section and maximum discharge which can be allowed to flow in it. [2073 Bhadra]

Solution: Proceed same as problem 15

PROBLEM 28

Describe about alluvial and non- alluvial canal. Design a canal using Kennedy's formula with the following data: $Q = 40\ m^3/sec.$, manning's roughness coefficient (n) = 0.018, bed slope (S) = 0.00020, m = 1.0 and side slope = 0.5 : 1 (H : V) [2073 Magh]

Solution:

Alluvial canal

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

Non–alluvial canal

The soil which is formed by disintegration of rock formation is known as non-alluvial soil. 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 do not pose many problems for designing irrigation structure on them. Canals, passing through such areas are called non–alluvial canals.

For numerical

See the solution of Q. no. 21

PROBLEM 29

Describe briefly semi theoretical approach in canal design. [2074 Bhadra]

Solution: See the definition part 4.4

PROBLEM 30

Design an economical trapezoidal lined channel to carry a discharge of 20 cumecs at a slope of 30 cm/km. The side slope of the channel is 1.5 : 1. The value of Manning's rugosity coefficient is 0.017 and limiting velocity in the channel is 1.5 m/sec. [2074 Bhadra]

Solution: Proceed same as an example 4.9

PROBLEM 31

Write down the concept of Kennedy and Lacey's silt theory. [2075 Baishakh]

Solution: See the definition part 4.5.1 and 4.5.2

PROBLEM 32

Proof using Lacey's theory that $P = 4.75(Q)^{0.5}$ [2075 Baishakh]

Solution: See the definition part 4.5.2.1

PROBLEM 33

Design a canal using Lacey's theory carrying a discharge of 20 cumec, silt factor is 1.5 and side slope is 0.5 : 1 (H : V) [2075 Baishakh]

Solution: Proceed same as an example 4.6

PROBLEM 34

What do you mean by canal loss? Explain causes of canal loss with their remedial measures to reduce it. Explain the advantages of a canal lining. [P.U. 2014]

Solution:

Canal loss

There is continuous loss of water from the headwork to the water course where water is supplied to the fields. This loss of water is called canal loss.

Causes of canal loss are:

I) Evaporation

Water loss by evaporation in canal is very small. It depends on temperature, wind velocity, humidity, etc. It should not exceed 7% of the water diverted into main canal.

II) Seepage

There are two different condition of seepage:

a) Percolation

If water table is close to the channel bed, there will be continuous flow of water from canal to the ground water reservoir creating a continuous saturation zone from canal bed to water table as shown in the figure below:

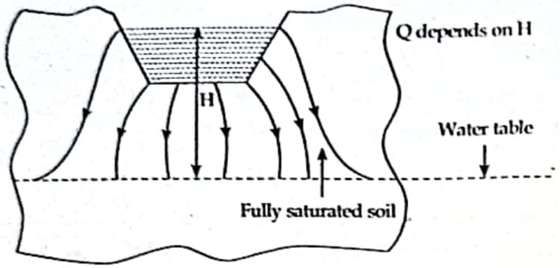


Figure: Percolation

b) Absorption

If water table is at a greater depth from channel bed, the water seeping from the channel is unable to reach the ground water reservoir but a small zone of complete saturation is formed round the channel section surrounded by a zone of partial saturation as shown in the figure.

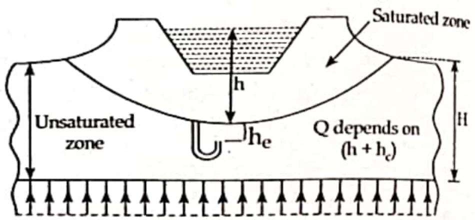


Figure: Absorption

Remedial measures to reduce canal loss:

i) Lining of channels.
ii) Lowering the full supply level of channels.
iii) Providing intercepting drains along channels in the reaches of high embankment and high water table.
iv) Improving the natural drainage of the area.
v) Removing silt

Advantages of canal lining

See the definition part 4.6

PROBLEM 35

Design a regime channel for a discharge of 45 cumec and silt factor as 1.1 using appropriate theory. [P.U. 2015]

Solution: See the example 4.6

PROBLEM 36

How will you justify economically the necessity of lining an existing canal, illustrate mathematically. Write short notes on:
i) spoil bank
ii) berm [P.U. 2015 chance]

Solution:

For economic justification of canal lining

See the definition part 4.6.4

i) Spoil bank

When earth work in excavation is more than earthwork in filling at a section of channel, it is necessary to transport the surplus earthwork economically. Instead of transporting this surplus earthwork, it is disposed on one or both sides of canal in form of heap. Such disposed earthwork is called spoil bank.

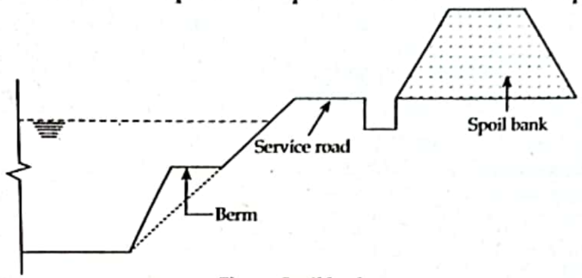


Figure: Spoil bank

ii) Berm

It is the horizontal distance left at ground level between the toe of the bank and the top edge of cutting. It is provided in such a way that the bed line and bank line remains parallel.

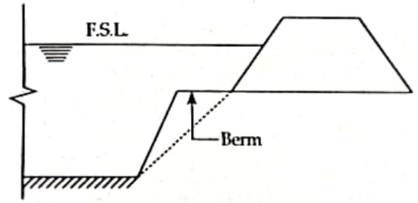


Figure: Berm

PROBLEM 37

Derive the relation among slope, silt factor and discharge from Lacey's theorem. [P.U. 2015 final]

Solution: See the definition part 4.5.2.1

PROBLEM 38

Design a regime canal (using Lacey's equation) for a discharge of 50 m^3/sec. with silt factor 1.1 assume trapezoidal section with side slope 0.5 : 1. [P.U. 2015 final]

Solution: See the example 4.6

PROBLEM 39

Design a main canal of a Babai irrigation project of discharge 20 cumecs. Assume necessary data [P.U. 2016 final]

Solution:

Assume, f = 1 and proceed as example 4.6

PROBLEM 40

Define tractive force. Explain sediment transport and tractive force approach of canal design **[P.U. 2017 final]**

Solution: See the definition part 4.2 and 4.4

PROBLEM 41

Describe briefly the semi-theoretical approach in canal design.

Solution: See the definition part 4.4 (tractive force approach)

PROBLEM 42

Design an economical trapezoidal lined channel to carry a discharge of 20 cumecs at a slope of 30 cm/km. the side slope of channel is 1.5 : 1. The value of manning's rugosity coefficient is 0.017 and limiting velocity in the channel is 1.5 m/sec.

Solution: See the solution of example 4.7

PROBLEM 43

Lacey's theorem is an advantage over Kennedy's theory, explain. **[2076 Baishakh]**

Solution:

Lacey's theorem is an advantage over Kennedy's Theory because of the following reason:

i) Kennedy selects Kutter's formula for finding the mean velocity. In Kutters formula value of N is arbitrarily fixed. Lacey has not fixed any value arbitrarily and gives an equation for finding the mean velocity.

ii) Kennedy has made use of term "CVR" (m) but he did not give any basis for calculating m. He simply states that it depends on silt charge and silt grade. Lacey has introduced a term "silt factor" (f). He related 'f' to mean diameter of the bed material and gave basis to calculate 'f'.

iii) Kennedy gives no clue for calculating longitudinal regime slope. Lacey produced a regime slope formula.

iv) Design based on Kennedy's theory can only be achieved after making while Lacey' theory does not involve trial and error method.

v) Lacey gave very important wetted regime perimeter equation, $P_w = 4.75\, Q^{\frac{1}{2}}$. He of course admitted that value of constant in the above equation is in no way constant and varies from 4 to 5.8 for regime channels.

PROBLEM 44

Design an irrigation canal using lacey's theory when Q : 15 cumec, mean diameter of silt particle is 0.33 mm; side slope of canal = $\frac{1}{2}$: 1 (H : V). **[2076 Baishakh]**

Solution: Proceed same as solution of problem no. 3

PROBLEM 45

Design a regime channel (using lacey's equation) for the following data. Discharge (Q) = 30 cumecs, silt factor (f) = 1.1, Side slope = 0.5 : 1, find also the longitudinal slope. **[2076 Bhadra]**

Solution: Proceed same as solution of problem no. 3

PROBLEM 46

Using tractive force approach, design a channel in alluvial soli for the following data:

Discharge (Q) : 45 cumecs

Bed slope = $\frac{1}{4500}$

Manning's n = 0.0225

Permissible tractive stress = 0.0035 kN/m²

Side slope = 0.5 : 1 **[2076 Bhadra]**

Solution: Proceed same as solution of example 4.1

PROBLEM 47

An irrigation channel is to carry a full supply discharge of 30 m³/sec. at a velocity of 1.75 m/sec. The side slopes are to be 1 : 1. The ratio of full supply depth to bed width is to be 1 : 6. Assuming the manning's n as 0.018, calculate the full supply depth, bed width, and bed slope of the channel using Kennedy's method. **[2077 Chaitra]**

Solution:

Given that;

$Q = 30\ m^3/sec.$

$V = 1.75\ m/sec.$

$z = 1$

$\frac{D}{B} = \frac{1}{6}$

$n = 0.018$

Full supply depth (D) =?

Bed width (B) =?

Bed slope (s) =?

Here,

$$V = \frac{Q}{A} = \frac{30}{BD + D^2} = \frac{30}{6D^2 + D^2} = \frac{30}{7D^2}$$

or, $1.75 = \frac{30}{7D^2}$

or, $D = 1.56\ m$

and, $B = 6D = 6 \times 1.56 = 9.39$ m

For bed slope (s), we have,

$$v = \frac{1}{n} R^{\frac{2}{3}} s^{\frac{1}{2}}$$

where, $R = \frac{A}{P} = \frac{BD + D^2}{B + 2D\sqrt{2}} = \frac{17.08}{13.8}$

$R = 1.23$ m

or, $1.75 = \frac{1}{0.018} \times (1.23)^{\frac{2}{3}} \times s^{\frac{1}{2}}$

or, $1.75 = 63.77 \times s^{\frac{1}{2}}$

or, $s = 7.53 \times 10^{-4} = \frac{1}{1328}$

PROBLEM 48

Design an irrigation channel to carry a discharge of 7 cumecs, assume n = 0.025 and critical velocity ratio m = 1.1. The channel has a bed slope of 0.3 m per kilometer. [2078 Baishakh]

Solution:

Assume side slope 1H : 2V and proceed same as solution of example 4.3

PROBLEM 49

Using tractive force approach design a channel in alluvial soil for the following data:

Discharge (Q) = 30 cumecs

Bed slope = $\frac{1}{3700}$

Manning's n = 0.0225

Permissible tractive stress = 0.0035 kN/m^2

Side slope = 0.5 : 1 [2078 Baishakh]

Solution:

Precede same as solution of example 4.1, using value of n directly from given

CHAPTER 5

DIVERSION HEADWORKS

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Diversion headwork is a structure constructed across a river for the purpose of raising water level in the river so that it can be diverted into the off taking canals. It is also known as canal headworks and performs the following functions.

- It raises the water level on upstream side.
- It regulates the supply of water into canals.
- It controls the entry of silt into canals.
- It provides some pondage creating small pond.
- It helps in controlling the vagaries of river.

5.1 COMPONENT PARTS OF WEIR/BARRAGE (DETAILED DRAWING)

A diversion headwork (or weir) usually consists of the following components.

i) Weir (or barrage) proper
ii) Under sluices
iii) Divide wall
iv) Fish ladder
v) Control head regulator
vi) Silt excluder, silt ejector
vii) River training works: guide banks, marginal bunds
viii) A typical layout of weir or barrage is shown in the figure

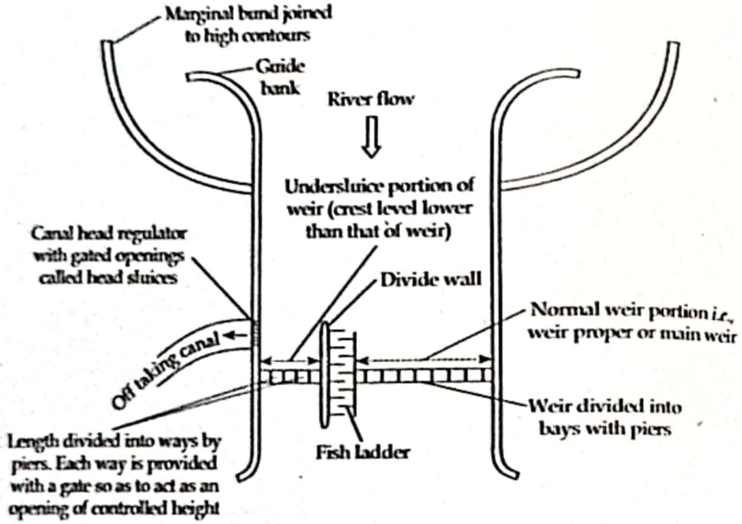


Figure 5.1: Typical layout of diversion headworks

5.1.1 Weir or barrage

Weir

A weir is a raised concrete (or masonry) crest wall constructed across the river width. It may be provided with a small shutter on its top. Most of the raising water (ponding) is done by solid wall and very little by shutters.

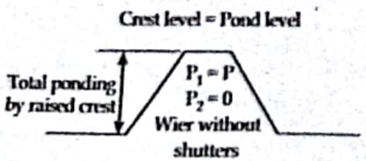


Figure 5.2: Weir without shutters

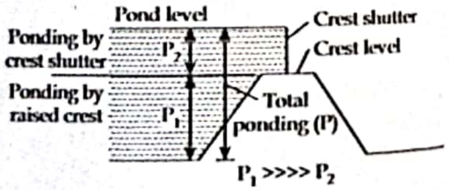


Figure 5.3: Weir with shutters

Types of weir depending upon floor design criteria

i) Gravity weir

Uplift pressure is resisted entirely by weight of wall.

ii) Non gravity weir

Uplift pressure is resisted by bending action of reinforced concrete floor.

Types of weir depending upon materials used

i) Masonry weir with vertical drop

This type of weir consists of a masonry wall with either both u/s and d/s faces vertical or both faces inclined or only u/s face inclined. It has an impervious horizontal floor or apron. Cutoff walls are provided at both u/s and d/s ends of the floor as shown in figure 5.4.

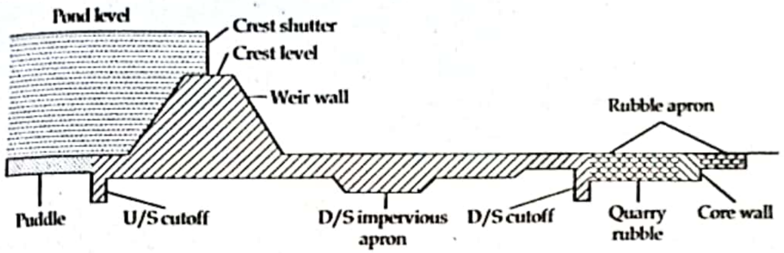


Figure 5.4: Masonry weir

ii) Rock fill weir with sloping apron

This type of weir consists of a masonry wall and dry pack boulders laid in the form of slope [*i.e.*, 1 : 20 at d/s and 1 : 4 at u/s] with weir wall at middle and few intervening core walls. The d/s slope is generally made very flat. It requires very large quantity of stone and would be economical only where stone is available in abundance in close proximity.

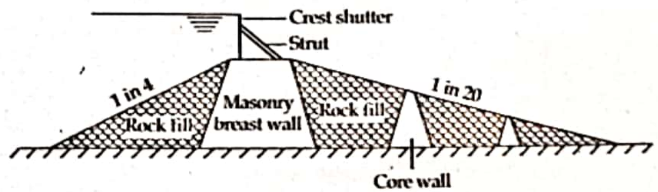


Figure 5.5: Rock fill weir

iii) Concrete weir with d/s glacis

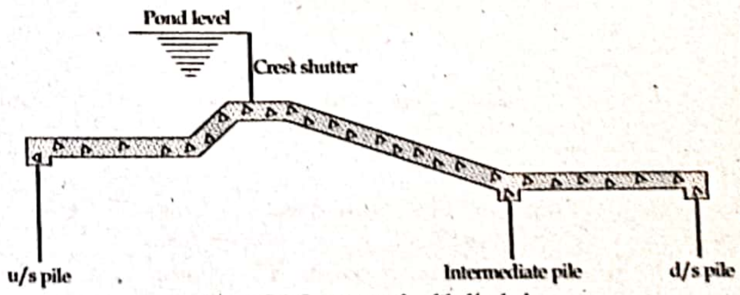


Figure 5.6: Concrete weir with d/s glacis

In this type of weir, sheet piles of sufficient depths are provided at u/s and d/s ends of the floor as shown in figure 5.6. Intermediate piles are also provided in some cases. Its design is based on modern concept of sub-soil flow, i.e., Khosla's theory. This type of weirs is constructed on the pervious foundation.

Barrage

If ponding of water is achieved by shutters or gates then it is called barrage. It has low crest wall with high gates.

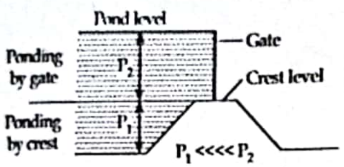


Figure 5.7: Barrage with a small raised crest

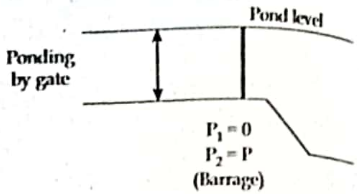


Figure 5.8: Without any raised crest

5.1.2 Undersluices

The weir proper is constructed in the middle portion of diversion head works. At the ends under sluices are provided adjacent to the canal head regulators. A comparatively less turbulent pocket of water is created near the canal head regulator by constructing under sluices portion of the weir. The undersluices are the openings provided in the weir wall with their crest at a low level. These openings are controlled by gates.

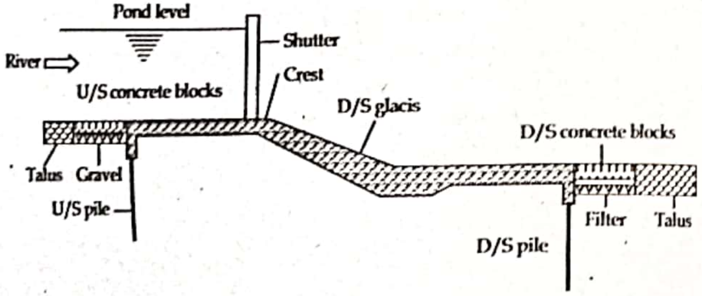


Figure 5.9: Cross-section of undersluice

Function of under sluices

- To maintain well maintained river channel near canal head regulator.
- To scour away silt deposited in front of head regulator.
- To pass a portion of flood (10 to 20%) of design flood during rainy season.
- Help in impounding fair amount of flood to secure full storage.
- They are used for quick lowering the u/s high flood level.

5.1.3 Divide wall

The divide wall is masonry or a concrete wall constructed at right angle to the axis of the weir and separates the weir proper from the under sluices. It extends from beyond the end of the head regulator on u/s side to loose protection of the under sluice on d/s side.

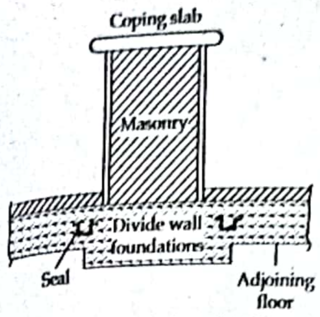


Figure 5.10: Cross-section of divide wall on Pucca floor

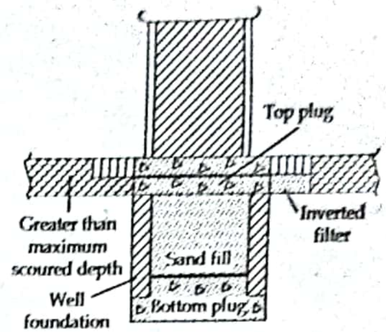


Figure 5.11: Cross-section of divide wall beyond Pucca floor

Function of divide wall

- To separate the under sluice portion from weir proper portion.
- Increase the effectiveness of the under sluices portion.
- To prevent cross current and flow parallel to the weir.
- Divide wall incidentally acts as one of the side walls of the fish ladder.
- To isolate pocket u/s of head regulator to facilitate scouring operation.

5.1.4 Fish ladder

Large rivers are generally inhibited by several types of fish, many of which are migratory such fish has found to be moving from u/s hill to d/s in the beginning of winter season in search of warmer water and return to their spawning ground u/s, slightly before monsoon in May and June. If no arrangement is made in weir or a dam to enable their migration their life goes in danger. So, for easy moment of the fish from u/s to d/s and again from d/s to u/s fish ladder is constructed. Typical plan of fish ladder is shown in the figure 5.12.

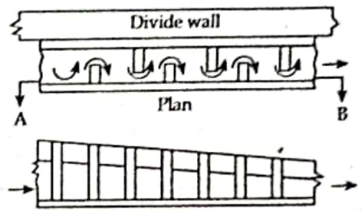


Figure 5.12: Fish ladder

5.1.5 Canal head regulator

A canal head regulator is provided at the head of each main canal off taking from diversion headwork. It should be so aligned that its axis makes an angle of 90° to 120° with the axis of weir as shown in the figure and should serve the following function.

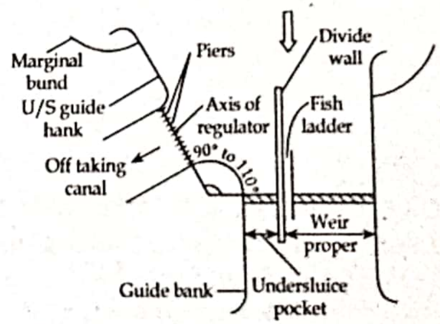


Figure 5.13: Alignment of a canal head regulator

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

It can be used to stop the canal supply when the silt charge in the river water exceeds a certain limit.

5.1.6 Silt excluder

Silt excluder is a structure constructed in the bed of river, u/s of head regulator to attack the river bed water, and divert the same into the d/s of the river. Its main function is to prevent the entry of silt into the canal. A typical silt excluder is shown in the figure 5.14.

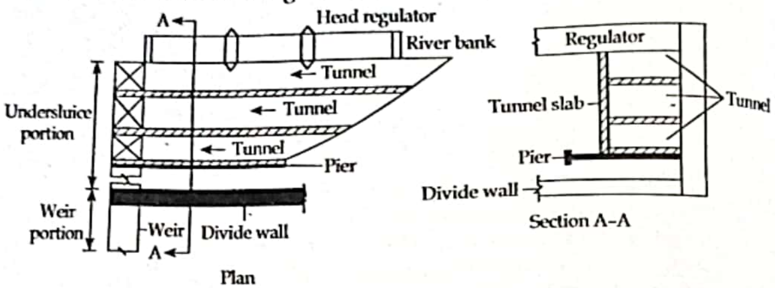


Figure 5.14: Silt excluder

5.1.7 Silt ejectors

They are also called silt extractors and are provided to extract silt from canal water after the silted water has travelled a certain distance in the off taking canal. These works are therefore constructed on the bed of the canal and a little distance d/s from the head regulator. They consist of curved tunnels located across the canal which starts along the axis of canal and turn towards the bank into the escape channel. The silted water is discharged into the d/s side of river from weir.

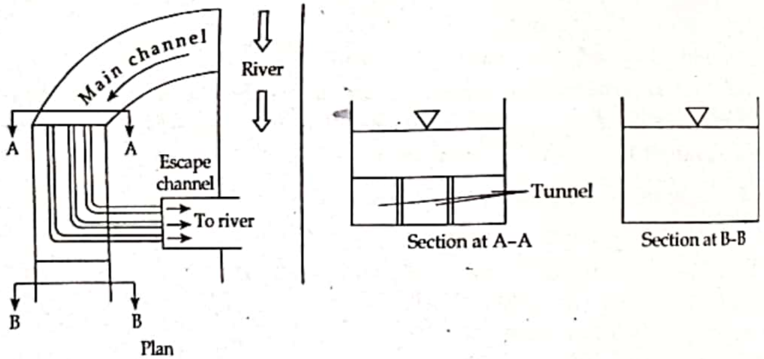


Figure 5.15: Silt ejector

5.2 BLIGH'S, LANE'S AND KHOSLA'S SEEPAGE THEORY

Failure mechanism of hydraulic structure

The seepage flow of water beneath the foundation of hydraulic structure causes the failure of structure either by piping or by uplift.

i) Piping or undermining

When the seepage water retains sufficient residual force at the emerging d/s end of the work, it may lift up the soil particles. This increases the porosity of the soil by removal of soil beneath the foundation. The structure may ultimately subside into the hollow so formed, resulting in the failure of the structure.

Prevention of failure due to piping

a) By providing sufficient length of impervious floor so that the path of percolation is increased and exit gradient is reduced.

b) Providing piles at u/s and d/s end of the impervious floor.

ii) Uplift

The water seeping below the structures exerts an uplift pressure on the floor of the structure. If this pressure is not counter balanced by the weight of concrete or masonry floor, the structure will fail by rupture of a part of floor.

Preventive measures of failure due to uplift pressure

a) Providing sufficient thickness of impervious floor

b) Providing pile at u/s end of impervious floor so that uplift pressure is reduced at d/s side

The surface flow may also cause the failure of hydraulic structure in following two ways:

i) By suction due to standing wave or hydraulic jump

Because of weir the standing wave or hydraulic jump is developed on its d/s side which causes suction or negative pressure and act in the direction of uplift pressure. This may result failure of the floor by rupture due to suction.

Preventive measures

a) Providing additional thickness of impervious floor to counterbalance the suction pressure due to hydraulic jump.

b) Constructing floor as monolithic concrete mass instead of different layers of masonry.

ii) By scour on u/s and d/s of the weir

The u/s and d/s ends of impervious floor may get scoured to considerable depth during floods. This may cause damage to impervious floor if no preventive measures are taken.

Preventive measures

i) Providing piles to below calculated scour depth both u/s and d/s ends.

ii) Providing launching apron of suitable length and thickness at both u/s and d/s end of impervious floor.

5.2.1 Bligh's seepage theory

Assumptions of Bligh's seepage theory

- According to Bligh, in a pervious foundation the water percolated seeps along the base profile of the structure which is in contact with the subsoil. The length of path thus traversed by the percolation water is known as length of creep or creep length.
- The loss of head per unit length of creep is called hydraulic gradient. The hydraulic gradient is constant throughout the seepage path.
- The loss of head is proportional to the length of creep.

Expression for hydraulic gradient according to Bligh's theory

Consider a section as shown in the figure.

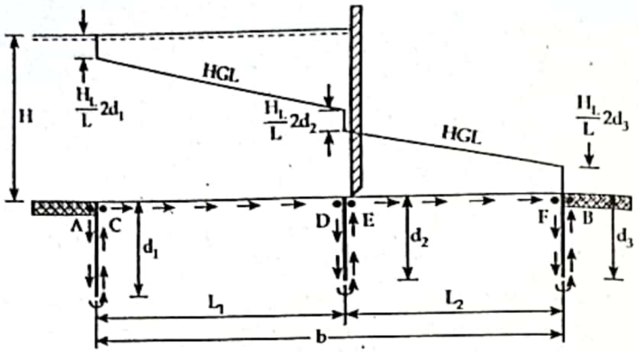


Figure 5.16: Bligh's theory

Let, H_L is the difference of water levels between u/s and d/s ends.

Water will seep along the bottom profile as shown in figure shown by arrows.

Seepage water starts from 'A' and emerges at 'B'.

$$\text{Total length of creep } (L) = d_1 + d_1 + L_1 + d_2 + d_2 + L_2 + d_3 + d_3$$
$$= (L_1 + L_2) + 2(d_1 + d_2 + d_3)$$
$$= b + 2(d_1 + d_2 + d_3)$$

$$\text{Head losses per unit length or hydraulic gradient} = \frac{H_L}{b + 2(d_1 + d_2 + d_3)}$$
$$= \frac{H_L}{L} \quad (5.1)$$

Head loss equal to $\left(\frac{H_L}{L} \times 2d_1\right)$, $\left(\frac{H_L}{L} \times 2d_2\right)$ and $\left(\frac{H_L}{L} \times 2d_3\right)$ occur respectively in the planes of three vertical cutoffs. The HGL can be drawn as show in figure.

Design criteria

Safety against piping

According to Bligh, the safety against piping can be ensured by providing sufficient creep length given by;

$$L = C \times H_L$$

or, $$\frac{1}{C} = \frac{H_L}{L}$$

From equation (5.1); we have,

$$\text{Hydraulic gradient} = \frac{1}{C}$$

According to Bligh, if hydraulic gradient $\le \left(\frac{1}{C}\right)$; there will be no danger of piping.

where, 'C' is Bligh's coefficient for the soil.

S.N.	Type of soil	Values of C	Safe hydraulic gradient should be lower than $\left(\frac{1}{C}\right)$
1.	Fine micaceous sand	15	$\frac{1}{15}$
2.	Coarse grained sand	12	$\frac{1}{12}$
3.	Sand mixed with boulder and gravel	5-9	$\frac{1}{5} - \frac{1}{9}$
4.	Light sand and mud	8	$\frac{1}{8}$

Table 5.1: Values of 'C' for different type of soil

Safety against uplift pressure

If the uplift head at any point is H_1 (meter of water) then uplift head has to be counter balanced by the weight of floor thickness.

$$\text{Uplift pressure} = \gamma_w H_1$$

where, γ_w = Unit weight of water = ρg

Downward pressure due to wt. of concrete floor thickness (t) = $\gamma_w G_c t$

where, G_c is the specific gravity of the floor material.

For equilibrium,

$$\gamma_w H_1 = \gamma_w G_c t$$

Subtracting 't' on both sides; we get,

$$H_1 - t = G_c t - t = t(G_c - 1)$$

or, $$t = \frac{H_1 - t}{G_c - 1}$$

$$\therefore \quad t = \frac{h}{G_c - 1} \quad (5.2)$$

where, h = $(H_1 - t)$ = Ordinate of HGL above top of the floor = Residual head

The thickness of floor is determined by using above equation (5.2). The calculated thickness should be increased by 33% so as to allow suitable factor of safety. To ensure safety against uplift the thickness of the floor must be ≥ t.

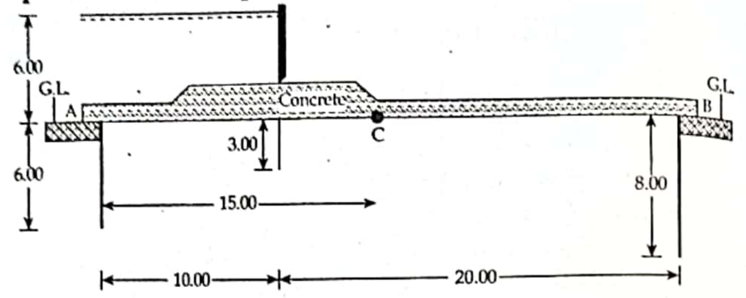
Limitations of Bligh's theory

- This theory has not made any distinction between vertical and horizontal creep.
- This theory did not explain any of the exit gradients. By considering a flat average gradient safety against undermining cannot simply be obtained, but by keeping this gradient will be low critical.
- Bligh's creep theory has no distinction between the outer and inner faces of sheet piles or the intermediate sheet piles. It is very clear from the investigation, that the outer faces of the end sheet piles are much more effective than inner ones.
- A loss of head does not take place in same proportions as creep length. The uplift pressure distribution is not linear but follows a sine curve.

- In case of two piles, the width between the two piles should be greater than twice the head or the piles are not effective.

Example 5.1

Find the average hydraulic gradient, uplift pressure and thickness of floor at a point 15 m from the upstream end of the floor in the figure 5.17.



Note: Dimensions are in metres

Figure 5.17

Solution:

Water percolates at point 'A' and emerges at point 'B',

Total creep length $L = 6 + 6 + 10 + 3 + 3 + 20 + 8 + 8 = 64$ m

Head on structure = 6 m = H_L

Hydraulic gradient $= \frac{H_L}{L} = \frac{6}{64} = \frac{1}{10.67}$

Creep length up to 15 m = $L = 6 \times 2 + 10 + 3 \times 2 + 5 = 33$ m

Head available for total creep length of 64 m = 6 m

Head available for total creep length of 1 m $= \frac{6}{64}$

Head available for total creep length of 33 m $(H_C) = \frac{6}{64} \times 33 = 3.094$ m

Residual head at point C $= 6 - 3.094 = 2.91$ m

i.e., $H = 2.91$ m

$\therefore$ Uplift pressure at 'C' $= \gamma h = 9810 \times 2.91 = 28.55$ kN/m^2

The residual head 'H' at 'C' can be directly found from the figure 5.18,

$$h = 6\left(1 - \frac{33}{64}\right) = 2.91 \text{ m}$$

Now,

$$\text{Thickness of floor (t)} = \frac{h}{G_c - 1}$$

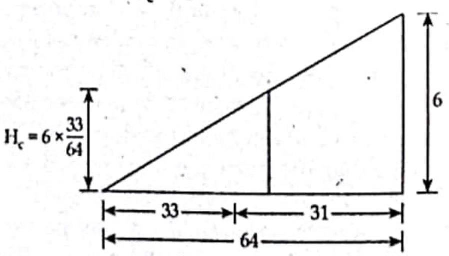


Figure 5.18

For concrete floor,

$G_c = 2.94$

$\therefore$ $t = \frac{2.91}{2.24 - 1} = 2.35$ m

Increasing by 33%; we get,

$t = 3.13$ m

5.2.2 Lane's weighted creep theory

From the analysis of 200 dams all over the world, Lane's concluded that horizontal creep is less effective in reducing uplift than vertical creep. Therefore, he suggested a factor of $\frac{1}{3}$ for horizontal creep against 1 for the vertical creep.

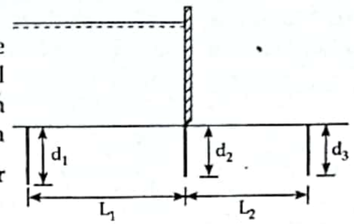


Figure 5.19: Lane's theory

For the structure in the figure;

$$L = d_1 + d_1 + \frac{L_1}{3} + d_2 + d_2 + \frac{L_2}{3} + d_3 + d_3$$

$$= \frac{L_1 + L_2}{3} + 2(d_1 + d_2 + d_3)$$

$$= \frac{b}{3} + 2(d_1 + d_2 + d_3) = \frac{1}{3}(N + V);$$

where, N = Sum of all the horizontal contain and the sloping contacts less than 45° to the horizontal.

V = Sum of all the vertical contacts and the sloping contacts greater than 45° to the horizontal.

For safety against piping 'L' must not be less than $H_L C_1$

where, C_1 is Lane's creep coefficient.

S. No.	Type of soil	Value of C_1	Safe hydraulic gradient $\left(\frac{1}{C_1}\right)$
1.	Very fine sand or silt	8.5	$\frac{1}{8.5}$
2.	Fine sand	7.0	$\frac{1}{7}$
3.	Coarse sand	5.0	$\frac{1}{5}$
4.	Gravel and sand	3.5 to 3.0	$\frac{1}{3.5}$ to $\frac{1}{3}$
5.	Boulders, gravel and sand	3 to 2.5	$\frac{1}{3}$ to $\frac{1}{2.5}$
6.	Clayey soil	3.0 to 1.6	$\frac{1}{2.5}$ to $\frac{1}{1.6}$

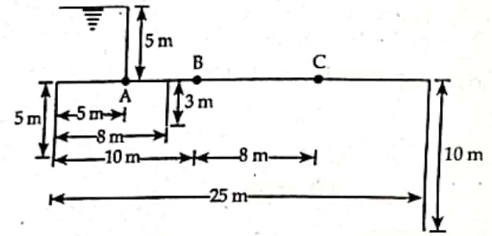
Table 5.2: Recommended values of Lane's coefficient of creep C_1 and safe hydraulic gradient $\left(\frac{1}{C_1}\right)$.

To ensure safety against piping the hydraulic gradient must be $\leq \frac{1}{C_1}$.

Being improvement over Bligh's theory also Lane's theory has most of the limitations of Bligh's theory. Since, Lane's theory is empirical and scarce any rational analysis, Bligh's theory is still being used in design. Lane's theory is not being used and is having only a theoretical importance.

Example 5.2

The figure below shows the section of hydraulic structure on permeable foundation. Calculate the average hydraulic gradient using,
i) Bligh's creep theory
ii) Lane's weighted creep theory
Also find uplift pressure and floor thickness required at point A, B and C from both the theories.



Solution:

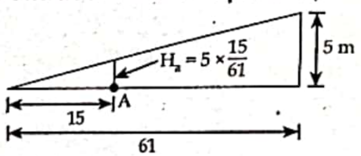
i) According to Bligh's theory

Total creep length (L) $= (5 \times 2) + 8 + (3 \times 2) + (25 - 8) + (10 \times 2)$
$= 61$ m

Total head available = 5 m

$\therefore$ Hydraulic gradient $= \frac{5}{69} = \frac{1}{12.2}$

a) Uplift pressure and floor thickness point A,



Creep length up to A $= (5 \times 2) + 5 = 15$

Residual head at point A,

$$h_1 = 5 - 5 \times \frac{15}{61} = 5\left(1 - \frac{15}{61}\right) = 3.77 \text{ m}$$

$\therefore$ Uplift pressure @ A $= 9.81 \text{ kN/m}^3 \times 3.77$ m
$= 36.984 \text{ kN/m}^2$

Thickness of floor at point A,

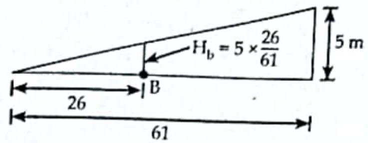
$$t_1 = \frac{h_1}{G_C - 1}$$ [From equation (5.2)]

Increasing by 33%;

$t_1 = 1.33 \times 3.04 = 4.04$ m

b) Uplift pressure and floor thickness at B,

Creep length up to B $= (5 \times 2) + 8 + (3 \times 2) + (10 - 8)$
$= 26$ m



Residual heat at point B,

$$h_2 = 5 - 5 \times \frac{26}{61} = 5\left(1 - \frac{26}{61}\right) = 2.87 \text{ m}$$

$\therefore$ Uplift pressure @ B $= 9.81 \text{ kN/m}^3 \times 2.87 \text{ m} = 28.155 \text{ kN/m}^2$

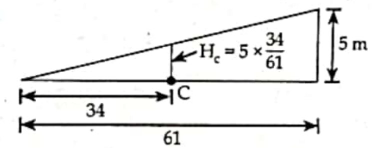
Thickness of floor at point B,

$$t_2 = \frac{h_2}{G_C - 1} = \frac{2.87}{2.24 - 1} = 2.315 \text{ m}$$

Increasing by 33%;

$t_2 = 1.33 \times 2.315 = 3.079$ m

c) Uplift pressure and floor thickness at C,



Creep length up to C $= (2 \times 5) + 18 + (3 \times 2) = 34$ m

Residual head at C,

$$h_3 = 5 - 5 \times \frac{34}{61} = 5\left(1 - \frac{34}{61}\right) = 2.213 \text{ m}$$

$\therefore$ Uplift pressure @ C $= 9.81 \times 2.213 = 21.710 \text{ kN/m}^2$

Thickness of floor at point C,

$$t_3 = \frac{h_3}{G_C - 1} = \frac{2.213}{2.24 - 1} = 1.785 \text{ m}$$

Increasing by 33%;

$t_3 = 1.33 \times 1.785 = 2.374$

II) According to Lane's theory

Total creep length (L) $= (5 \times 2) + (3 \times 2) + (10 \times 8) + \frac{1}{3} \times 25$
$= 44.333$ m

$\therefore$ Hydraulic gradient $= \frac{5}{44.333} = \frac{1}{8.867}$

a) Uplift pressure and floor at A,

Creep length up to A $= 5 \times 2 + \frac{1}{3} \times 5 = 11.667$ m

Residual head at A,

$$h_1 = 5\left(1 - \frac{11.667}{44.333}\right) = 3.684 \text{ m}$$

$\therefore$ Uplift pressure @ A $= 9.81 \times 3.684 = 36.142 \text{ kN/m}^2$

Thickness of floor at point A,

$$t_1 = \frac{h_1}{G_C - 1} = \frac{3.684}{2.24 - 1} = 2.971$$

Increasing by 33%;

$$t_1 = 1.33 \times 2.971 = 3.951 \text{ m}$$

b) Uplift pressure and floor thickness at B,

Creep length up to $B = (5 \times 2) + (3 \times 2) + \frac{1}{3} \times 10 = 19.333$ m

Residual head at B,

$$h_2 = 5\left(1 - \frac{19.333}{44.333}\right) = 2.82 \text{ m}$$

$\therefore$ Uplift pressure @ $B = 9.81 \times 2.82 = 27.664 \text{ kN/m}^2$

Thickness of floor at B,

$$t_2 = \frac{h_2}{G_C - 1} = \frac{2.82}{2.24 - 1} = 2.274 \text{ m}$$

Increasing by 33%;

$$t_2 = 1.33 \times 2.274 = 3.024 \text{ m}$$

c) Uplift pressure and floor thickness at C,

Creep length up to $C = 5 \times 2 + 3 \times 2 + \frac{1}{3} \times 18 = 22$ m

Residual head at C,

$$h_3 = 5\left(1 - \frac{22}{44.333}\right) = 2.519 \text{ m}$$

$\therefore$ Uplift pressure @ $C = 9.81 \times 2.519 = 24.711 \text{ kN/m}^2$

Thickness of floor at C,

$$t_3 = \frac{h_3}{G_C - 1} = \frac{2.519}{2.24 - 1} = 2.031 \text{ m}$$

Increasing by 33%;

$$t_3 = 1.33 \times 2.031 = 2.701 \text{ m}$$

5.2.3 Khosla's theory

The main principles of Khosla's theory are;

i) Seeping water below a hydraulic structure does not follow the bottom profile of impervious floor as stated by Bligh but each particle traces its path along a series of stream lines and equipotential lines as shown in

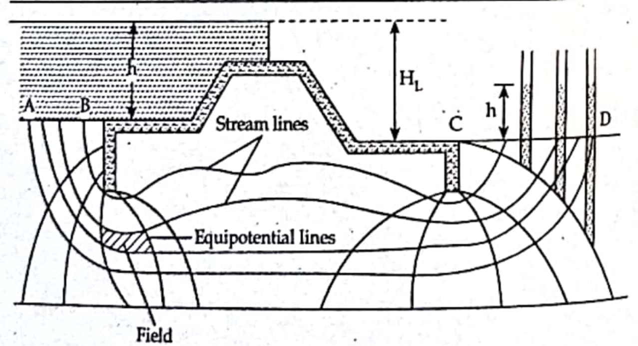


Figure 5.20: Khosla's flow net

the figure 5.21.

ii) The seepage water exerts force at each point in the direction of flow and tangential to the streamline as shown in the figure. This force has the maximum disturbing tendency at the exit end.

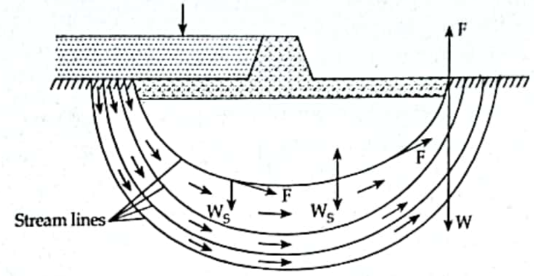


Figure 5.21: Seepage force

iii) The 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, ϕ is the flow potential = kh.

k is the coefficient of permeability of soil.

h is the residual head at any point within the soil.

iv) Undermining (piping) of the floor starts from the downstream end of the d/s pucca floor and if not checked it travels upstream towards the weir wall. It is therefore, absolutely necessary to have a reasonably deep vertical cutoff at the d/s end of floor to prevent undermining. Depth of d/s vertical cutoff is governed by;

a) Maximum depth of scour
b) Safe exit gradient

v) The outer faces of the end sheet piles are much more effective than the inner ones and the horizontal length of floor.

vi) The intermediate sheet piles, if smaller in length than the outer ones were ineffective except for local redistribution of pressure.

To find out the seepage flow by Khosla's theory it is necessary to plot the flow net and also to solve Laplacian equation mentioned in principal no (iii). But it is very difficult to solve Laplacian equation mathematically. So, for the design of hydraulic structures (such as weir, barrage, etc) and also to calculate the seepage flow and uplift pressure on pervious foundations, Khosla has developed simple profiles from practical complex profiles which is called the **Method of independent variables.** These simple profiles can be solved easily unlike the Laplacian equation.

5.2.3.1 Khosla's Method of Independent variables (Calculation of percentage pressure, *i.e.*, uplift pressure by Khosla's Theory)

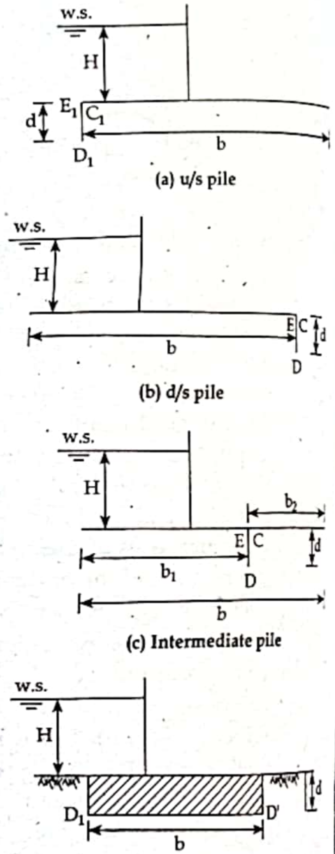
In this method the complex weir or barrage section (profile) broken into no. of simple section for which mathematical solution have been obtained to find the seepage flow in terms of percentage pressure. The percentage pressure later can be used to find uplift pressure due to seepage flow by directly multiplying with total head available for the flow. The uplift pressure thus obtained is in term of head of water.

The simple sections (profiles) given by Khosla consists of:

i) A straight horizontal either floor of negligible thickness with a sheet pile either at u/s end or d/s end. [Figure 5.22 (a) and 5.22 (b)]

ii) A straight horizontal floor of negligible thickness with a sheet pile at some intermediate point. [Figure 5.22 (c)]

iii) A straight horizontal floor, depressed below the bed but no vertical cutoff. [Figure 5.22 (d)]



(a) u/s pile

(b) d/s pile

(c) Intermediate pile

(d) Depressed floor

Figure 5.22: Khosla's simple profile for a weir of complex profile

u/s pile [Figure (a)]

$$\phi C_1 = 100 - \phi E$$
$$\phi D_1 = 100 - \phi D$$

d/s pile [Figure (b)]

$$\phi E = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$
$$\phi D = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$$

where, $\lambda = \dfrac{1+\sqrt{1+\alpha^2}}{2}$, $\alpha = \dfrac{b}{d}$.

Intermediate pile [Figure (c)]

$$\phi E = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$$
$$\phi D = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$
$$\phi C = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

where, $\lambda = \dfrac{\sqrt{1+\alpha_1^2}+\sqrt{1+\alpha_2^2}}{2}$.

$$\lambda_1 = \frac{\sqrt{1+\alpha_1^2}-\sqrt{1+\alpha_2^2}}{2}$$
$$\alpha_1 = \frac{b_1}{d}$$
$$\alpha_2 = \frac{b_2}{d}$$

Depressed floor [Figure (d)]

$$\phi_{D'} = \frac{2}{3}(\phi_E - \phi_D) + \frac{3}{\alpha^2}$$
$$\phi_{D_1'} = 100 - \phi_{D'}$$

In above simple profiles, the key points where percentage pressures are to be calculated are:

- Junction of floor and pile lines on either sides [E and C]

- Bottom point of pile line [D]
- Bottom corners in case of depressed floor [D_1' and D'].

The % pressure at these key points simple profiles will resemble the complex profiles only if the following corrections are applied.

- Correction for mutual interference of piles.
- Correction for thickness of floor.
- Correction for the slope of floor.

These corrections are described later.

The percentage pressure at key points can also be found out by using graph developed by Khosla which is known as Khosla's pressure curves. It is shown in figure 5.23 (end of chapter). The use of Khosla's pressure curve will be clear from example 5.3.

Correction for percentage pressure

I) Correction for the mutual interference of piles

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

where, b' is the distance between two pile lines.

D is 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 level at which interference is desired.

d is the depth of the pile on which the effect is considered.

b is the total floor length.

Sign convention

- Positive for the points in rear or backwater
- Negative for the points on forward direction of flow

II) Correction for thickness of floor

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.

In the figure 5.24;

$$\text{Thickness correction for } C_1 = \frac{\phi D_1 - \phi C_1}{\text{R.L. of floor level} - \text{R.L. of } D_1} \times \text{Thickness of floor at } C_1$$

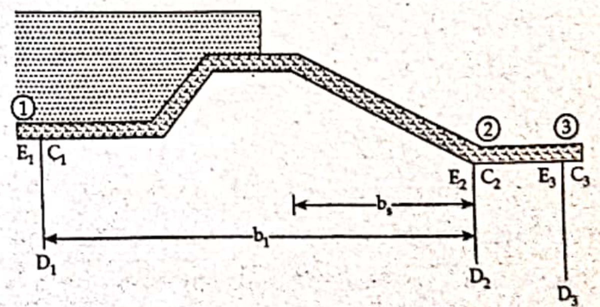


Figure 5.24: Thickness correction

$$\text{Thickness correction for } E_2 = \frac{\phi D_2 - \phi E_2}{\text{R.L. of floor level} - \text{R.L. of } D_2} \times \text{Thickness of floor at } E_2$$

Sign convention

- Positive for point in right side of pile
- Negative for point in left side of pile

In figure correction for C_1 is positive and E_2 is negative.

iii) Correction for slope

Slope	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

Table 5.2: Slope correction factor

It is given by;

$$C = C_s \times \frac{b_s}{b_1} \qquad \text{[For } E_2 \text{ in the figure 5.24]}$$

where, b_s and b_1 are as shown in the figure 5.24.

C_s is the slope correction factor and is given in table.

Sign convention

- Positive for the downward slope
- Negative for upward slope

Exit gradient

Exit gradient may be defined as the hydraulic or pressure gradient of subsoil flow or seepage flow at the d/s or the exit end of the floor.

It has been determined that for a standard form consisting of a floor length 'b' with a vertical cutoff of depth 'd', exit gradient at its downstream end is given by,

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}} \qquad (5.3)$$

where, $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$ and $\alpha = \frac{b}{d}$.

The value of exit gradient can also be found from figure 5.25 (end of chapter) developed by Khosla. In the figure (graph) for any value of $\alpha\left(=\frac{b}{d}\right)$ the corresponding value of λ can be obtained directly. Knowing the value of H, d and λ the value of G_E can be calculated using equation **(5.3)**.

Types of soil	Safe exit gradient
Shingle	$\frac{1}{4}$ to $\frac{1}{5}$ (0.25 to 0.2)
Coarse sand	$\frac{1}{5}$ to $\frac{1}{6}$ (0.2 to 0.17)
Fine sand	$\frac{1}{6}$ to $\frac{1}{7}$ (0.17 to 0.14)

Table 5.3: Safe exit gradient for different types of soil

Example 5.3

Figure 5.26 shows the profile of a weir. Determine the uplift pressure at key points and the exit gradient and find whether the section provided is safe against piping if it is founded on fine sand with permissible exit gradient of $\frac{1}{6}$. Also find uplift pressure at point X and check the safety against uplift.

Solution:

For u/s pile no. 1

Total length of floor (b) = 65 m

Depth of u/s pile line (d) = 100 – 91 = 9 m

$$\alpha = \frac{b}{d} = \frac{65}{9} = 7.222$$

$$\frac{1}{\alpha} = \frac{1}{7.222} = 0.138 \approx 0.14$$

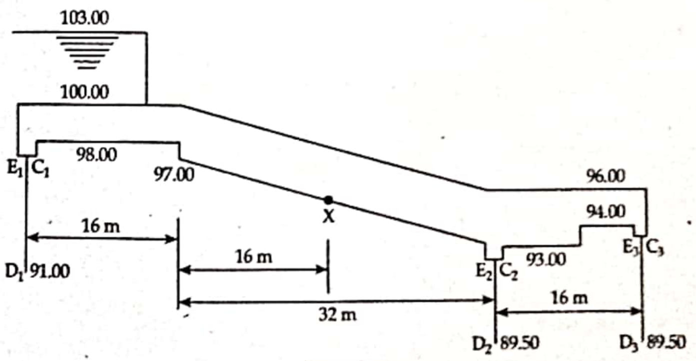


Figure 5.26

From figure 5.23 (a), for $\frac{1}{\alpha} = 0.14$,

$\phi_D = 23\%$

$\phi_E = 33\%$

$\therefore \quad \phi_{D_1} = 100 - \phi_D = 77\%$

$\therefore \quad \phi_{C_1} = 100 - \phi_E = 67\%$

Correction for ϕ_{C_1}.

I) **Correction of C_1 for mutual interference of pile 2,**

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

$d = 98 - 91 = 7\ m$

$b' = 48\ m$

$D = 98 - 89.5 = 8.5\ m$

$b = 65\ m$

$\therefore\quad C = 19\sqrt{\frac{8.5}{48}}\left(\frac{7+8.5}{65}\right) = 1.9\%$ (positive since pile 1 is rear to pile 2)

II) **Correction for floor thickness,**

$$= \frac{\phi_{D_1} - \phi_{C_1}}{\text{R.L. of floor level} - \text{R.L. of } D_1} \times \text{Thickness of floor at } C_1$$

$$= \frac{77-67}{100-91} \times 2$$

$= 2.2\%$ (Positive since C_1 is in right of the pile 1)

$\therefore\quad$ Corrected $\phi_{C_1} = 67 + 1.9 + 2.2 = 71.1\%$

For intermediate pile no. 2:

$d = 96 - 89.5 = 6.5\ m$

$b = 65\ m$

$b_1 = 48.5\ m$

$\alpha = \frac{b}{d} = \frac{65}{6.5} = 10$

$\frac{b_1}{b} = \frac{48.5}{65} = 0.746$

From the figure 5.23 (b), for $\frac{b_1}{b} = 0.746$ and $\alpha = 10$,

$\left(1 - \frac{b_1}{b}\right) = 0.254$

$\phi_{E_2} = 41\%$

$\phi_{D_2} = 34\%$

$\phi_{C_2} = 26\%$

Correction for ϕ_{E_2},

I) **Mutual interference due to pile 1,**

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

$d = 93 - 89.5 = 3.5\ m$

$b' = 48\ m$

$D = 93 - 91 = 2\ m$

$b = 65\ m$

$$C = 19\sqrt{\frac{2}{48}}\left(\frac{3.5+2}{65}\right) = 0.3\%$$

(Negative since pile lies forward from pile 1)

II) **Correction for floor thickness**

$$= \frac{\phi_{E_2} - \phi_{D_2}}{\text{R.L. of floor level} - \text{R.L. of } D_2} \times \text{Thickness of floor at } E_2$$

$$= \frac{41-34}{96-89.5} = 3.2\% \text{ (Negative)}$$

III) **Slope correction,**

$\text{Slope} = \frac{100+96}{32} = \frac{4}{32} = \frac{1}{8}$

$\text{Correction} = C_s \times \frac{b_s}{b_1}$

Here,

$C_s = 2$ (From the table 5.2)

$b_s = 32\ m$

$b_1 = 48\ m$

$\therefore\quad \text{Correction} = 2 \times \frac{32}{48} = 1.33$ (Positive)

$\therefore\quad$ Corrected $\phi_{E_2} = 41 - 0.3 - 3.2 + 1.33 = 38.83\%$.

Correction for ϕ_{C_2}.

I) **Mutual interference due to pile 3,**

$d = 93 - 89.5 = 3.5\ m;\ b' = 16\ m$

$D = 93 - 89.5 = 3.5\ m;\ b = 65\ m$

$\therefore\quad C = 19\sqrt{\frac{3.5}{16}}\left(\frac{3.5+3.5}{65}\right) = 1\%$ (Positive)

II) **Correction for floor thickness**

$$= \frac{\phi_{D_2} - \phi_{C_2}}{\text{R.L. of floor level} - \text{R.L. of } D_2} \times \text{Thickness of floor at } C_2$$

$$= \frac{34-26}{96-89.5} \times 3 = 3.7\% \text{ (Positive)}$$

III) **No slope correction for ϕ_{C_2},**

$\therefore\quad$ Corrected $\phi_{C_2} = 26 + 1 + 3.7 = 30.7\%$

For d/s pile no 3,

$d = 96 - 89.5 = 6.5\ m$

$b = 65\ m$

$\frac{1}{\alpha} = \frac{d}{b} = \frac{6.5}{65} = 0.1$

From figure 5.23(a),

$\phi_{E_3} = 29\%$

$\phi_{D_3} = 20\%$

Correction for ϕ_{E_3},

i) Mutual interference due to pile 2,

$d = 94 - 89.5 = 4.5$ m

$b' = 16$ m

$D = 94 - 89.5 = 4.5$ m

$b = 65$ m

$$C = 19\sqrt{\frac{4.5}{16}}\left(\frac{4.5 + 4.5}{65}\right) = 1.4\% \text{ (Negative)}$$

ii) Correction for floor thickness

$$= \frac{\phi_{E_3} - \phi_{D_3}}{\text{R.L. of floor level} - \text{R.L. of } D_3} \times \text{Thickness of floor at } E_3$$

$$= \frac{29 - 20}{96 - 89.5} \times 2 = 2.8\% \text{ (Negative)}$$

$\therefore$ Corrected $\phi_{E_3} = 29 - 1.4 - 2.8 = 24.8$

The corrected % pressure at key point are tabulated as under,

u/s pile 1	Intermediate pile 2	d/s pile 3
$\phi_{E_1} = 100\%$	$\phi_{E_2} = 38.8\%$	$\phi_{E_3} = 24.8\%$
$\phi_{D_1} = 77\%$	$\phi_{D_2} = 34.0\%$	$\phi_{D_3} = 20.0\%$
$\phi_{C_1} = 71.1\%$	$\phi_{C_2} = 30.7\%$	$\phi_{C_3} = 0\%$

Difference in head between pond level and d/s floor;

i.e., Maximum percolation head, $H = 103 - 96 = 7$ m

Uplift pressures $@E_1 = 1 \times 7 = 7$ m

$@D_1 = 0.77 \times 7 = 5.39$ m

$@C_1 = 0.711 \times 7 = 4.977$ m

$@E_2 = 0.388 \times 7 = 2.716$

$@D_2 = 0.34 \times 7 = 2.38$ m

$@C_2 = 0.307 \times 7 = 2.149$ m

$@E_3 = 0.248 \times 7 = 1.736$ m

$@D_3 = 0.20 \times 7 = 1.4$ m

$@C_3 = 0$ m

Exit gradient

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

$H = 7$ m

$d = 96 - 89.5 = 6.5$ m

$$\alpha = \frac{b}{d} = \frac{65}{6.5} = 10$$

From figure 5.25, for $\alpha = 10$,

$$\frac{1}{\pi\sqrt{\lambda}} = 0.135$$

$$\therefore \quad G_E = \frac{7}{6.5} \times 0.135 = \frac{1}{6.878}$$

$$\text{Permissible gradient} = \frac{1}{6} > \frac{1}{6.878}$$

Hence, safe.

Uplift pressure at X (Residual head at X) is;

$$P_X = P_{C_1} - \left(\frac{P_{C_1} - P_{E_2}}{48} \times 32\right) = 4.977 - \left(\frac{4.977 - 2.716}{48} \times 32\right)$$

$= 3.47$ m

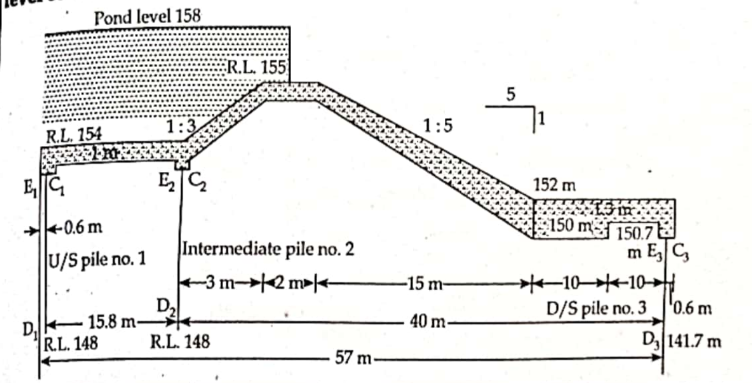
$$\therefore \quad \text{Floor thickness required at X} = \frac{P_X}{G_C - 1} = \frac{3.47}{2.24 - 1} = 2.8 \text{ m}$$

Thickness provided $= 96 - 93 = 3$ m > 2.8 m

Hence, safe against uplift.

Example 5.4

Determine the percentage pressure at various key points in figure. Also determine the exit gradient and plot the hydraulic gradient line for pond level on u/s and no flow d/s.



Solution:

1. For upstream pile line no. 1

Total length of the floor (b) = 57.0 m

Depth of u/s pile line, (d) = 154 − 148 = 6 m

$$\alpha = \frac{b}{d} = \frac{57}{6} = 9.5$$

Now,

$$\phi E = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

NOTE

The value should be changed into radian.

$$\text{where, } \lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + (9.5)^2}}{2} = 5.28.$$

$$\phi E = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) = \frac{1}{\pi} \times \cos^{-1}\left(\frac{5.28 - 2}{5.28}\right) \times \frac{\pi}{180}$$

$= 0.287 = 28.7\%$

$\phi C_1 = 100 - \phi E = 100 - 28.7 = 71.8\%$

$\phi D_1 = 100 - \phi D$

where, $\phi D = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) = \frac{1}{\pi} \times \cos^{-1}\left(\frac{5.28 - 1}{5.28}\right) \times \frac{\pi}{180}$

$= 0.199 = 19.9\%$

$\phi D_1 = 100 - 19.9 = 80.1\%$

Correction for C_1

i) Correction at C_1 for mutual interference of piles
ϕC_1 is affected by intermediate pile no. 2.

$$C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = 19\sqrt{\frac{5}{15.8}}\left(\frac{5+5}{57}\right) = 1.88\% \text{ (positive)}$$

where, D = Depth of pile no. 2 = 153 – 148 = 5 m
Since, the point C_1 is in the rear direction, the correction is positive.

ii) Correction at C_1 due to thickness of floor

$$\text{Thickness correction for } C_1 = \frac{\phi D_1 - \phi C_1}{\text{R.L. of floor level} - \text{R.L. of } D_1} \times 1$$

$$= \frac{80.1 - 71.3}{154 - 148} = 1.467\% \text{ (positive)}$$

Since, point C_1 is at right side of the pile correction is positive.

iii) Correction due to slope is zero since no slope starts or ends at this point.

Corrected $\phi C_1 = 71.3 + 1.88 + 1.467 = 74.647\%$

2. Intermediate pile no. 2

$$\phi E_2 = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$$

$$\phi D_2 = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$\phi C_2 = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

where, $\lambda = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2}$

$\lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2}$

$\alpha_1 = \frac{b_1}{d}$

and, $\alpha_2 = \frac{b_2}{d}$

From the figure,

d = Depth of intermediate pile = 154 – 148 = 6 m
b_1 = Floor length u/s of intermediate pile = 16.4 m
b_2 = Floor length d/s of intermediate pile = 40.6 m
$\alpha_1 = 2.73$
$\alpha_2 = 6.77$

$\lambda = 4.875$
$\lambda_1 = -1.968$
$\phi E_2 = 0.708 = 70.8\%$
$\phi D_2 = 0.632 = 63.2\%$
$\phi C_2 = 0.564 = 56.4\%$

Correction for ϕE_2

i) Correction at E_2 due to mutual interference due to pile 1

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

Here,
D = 153 – 148 = 5 m
d = 153 – 148 = 5 m
b′ = 15.8
b = 57 m

$\therefore\ C = 19\sqrt{\frac{5}{15.8}}\left(\frac{5+5}{57}\right) = 1.88\%$ (negative)

ii) Correction due to floor thickness (C) $= \frac{70.8 - 63.2}{154 - 148} \times 1$
$= 1.267\%$ (negative)

iii) Slope correction is nil.
$\therefore$ Corrected $\phi E_2 = 70.8 - 1.88 - 1.267 = 67.653\%$

Correction for ϕC_2

i) Correction at C_2 due to mutual interference due to pile 3

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

D = 153 – 141.7 = 11.3 m
d = 153 – 148 = 5 m
b′ = 40 m
b = 57 m
C = 2.89% (positive)

ii) Correction due to floor thickness

$C = \frac{70.8 - 63.2}{154 - 148} \times 1 = 1.267\%$ (positive)

iii) Correction due to slope

$C = \frac{C_s \times b_s}{b_1}$

$b_s = 3$ m
$b_1 = 40$ m
$C_s = 4.5$ for 3 : 1 from table.
C = 0.388% (negative for upward slope)
Corrected $\phi C_2 = 56.4 + 2.89 + 1.267 - 0.388 = 60.169\%$

3. **For d/s pile no. 3**

$b = 57$ m

$d = 152 - 141.7 = 10.3$ m

$\alpha = \frac{b}{d} = 5.53$

$$\lambda = \frac{\sqrt{1+\alpha^2}}{2} = \frac{\sqrt{1+(5.33)^2}}{2} = 3.31$$

$$\phi E_3 = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) \times \frac{\pi}{180} = 0.37 = 37\%$$

$$\phi D_3 = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) = 0.254 = 25.4\%$$

Correction for ϕE_3

i) Correction due to mutual interference of pile 2

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

$D = 150.7 - 148 = 2.7$ m

$d = 150.7 - 141.7 = 9$ m

$b = 57$ m

$C = 1.02\%$ (negative)

ii) Correction due to floor thickness

$$C = \frac{37 - 25.4}{152 - 141.7} \times 1.3 = 1.467\% \text{ (negative)}$$

iii) Correction due to slope is nil

Corrected $\phi E_3 = 37 - 1.02 - 1.464 = 34.516\%$

The corrected pressures at key points are tabulated below:

Upstream pile no. 1	Intermediate pile no.2	Downstream pile no. 3
$\phi E_1 = 100\%$	$\phi E_2 = 67.653\%$	$\phi E_3 = 34.516\%$
$\phi D_1 = 80.1\%$	$\phi D_2 = 63.2\%$	$\phi D_3 = 25.4\%$
$\phi C_1 = 74.647\%$	$\phi C_2 = 60.169\%$	$\phi C_3 = 0\%$

Exit gradient

We have,

$$\text{Exit gradient } (G_E) = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$$

H = Maximum seepage head = 158 − 152 = 6 m

d = Depth of d/s cutoff = 152 − 141.7 = 10.3 m

b = Total floor length = 57.0 m

$$\alpha = \frac{b}{d} = \frac{57}{10.3} = 5.53$$

$$\therefore \quad G_E = \frac{6}{10.3} \times 0.18 = 0.105$$

Hence, the exit gradient is equal to 0.105, *i.e.*, 1 in 9.53, which is very much safe referring table 5.3.

The elevation of H.G.L. can be calculated as;

Flow condition	u/s water level in meters	d/s water level in meter	Head in meter	Height/Elevation of Sub-Soil HGL above datum								
				u/s pile line			Intermediate pile line			d/s pile line		
				ϕE_1 100%	ϕD_1 80.1%	ϕC_1 74.65%	ϕE_2 67.653%	ϕD_2 63.2%	ϕC_2 60.169%	ϕE_3 34.516%	ϕD_3 25.4%	ϕC_3 0%
Pond level u/s with no flow d/s	158.0	152.0	6.0	6	4.81	4.48	4.06	3.79	3.61	2.07	1.52	0
				158.0	156.81	156.48	156.06	155.79	155.61	154.07	153.52	152

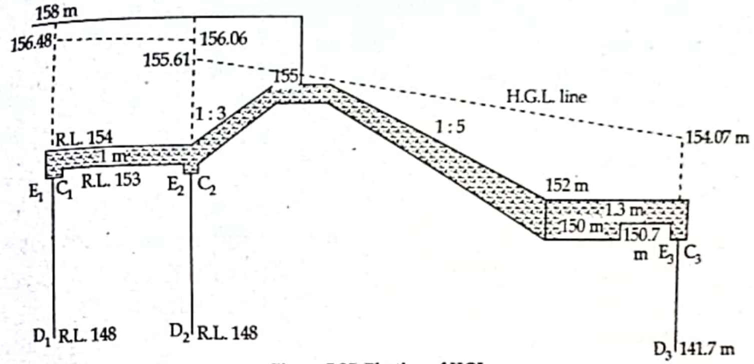


Figure 5.28: Plotting of HGL

5.3 DESIGN OF SLOPING GLACIS WEIR BAY (CREST, LENGTH AND THICKNESS OF FLOOR)

Some terms for design of sloping glacis weir,

i) Pond level

Pond level may be obtained by adding the working head to the design full supply level in the canal. It should include the head required for passing the design discharge into the canal and the head losses in the regulator.

ii) Afflux

Afflux may be defined as the rise in water level on the u/s of the weir or barrage above d/s under free flow condition as a result of construction of weir or barrage.

iii) Retrogression

Degradation of d/s river bed as a result of construction of weir or barrage is called retrogression. These results lowering of d/s river stages; so, retrogression must be considered in design.

iv) Discharge and Discharge intensity

Discharge over the crest of weir or barrage is given by,

$$Q = CLK^{\frac{3}{2}}$$

where, Q = Discharge in cumec

C = Coefficient of discharge usually taken as 1.7.

L = Length of clear waterway of weir or barrage in m.

K = Intercept between the u/s total energy line and the crest, or the static head + Velocity head over the crest, in m.

'K' represents the total head causing the flow over the crest of weir or barrage.

∴ The discharge intensity is given by;

$$q = \frac{Q}{L} = 1.7\,K^{\frac{3}{2}}$$

$$q = 1.7\,K^{\frac{3}{2}} \quad (5.4)$$

Some important design criteria

i) The u/s slope of glacis is taken as 1 : 2 and that of d/s slope 1 : 3 to 1 : 5 for the stability of the hydraulic jump.

ii) The d/s horizontal floor level is determined by considering that the hydraulic jump should not form lower than the toe of the glacis.

iii) The hydraulic jump is located with the help of Blench curves shown in figure 5.27 (end of chapter). For given 'q', the difference between u/s and d/s TEL is determined which gives the loss of head H_L. Knowing g and H_L, E_{f_2} (specific energy on the d/s side of the jump) can be determined from the Blench curves. Measuring Ef_2 below the d/s TEL, the position of the jump on the sloping glacis is obtained.

iv) The location of jump should be checked for different discharge intensities.

v) The length of the horizontal floor should not be less than five times the jump height *i.e.*, $5(D_2 - D_1)$. The maximum value of $(D_2 - D_1)$ which will occur during high flood condition should be used.

Example 5.5

Design a glacis weir for the following data,

i) Maximum discharge intensity on weir crest = 12.5 cumecs/m

ii) H.F.L. before construction of weir = 225.00 m

iii) River bed level = 218.75 m

iv) Pond level = 224.00 m

v) Height of crest shutters = 1.5 m

vi) Anticipated d/s water level in the river when the weir is discharge with pond level u/s = 221.5 m

vii) Bed retrogression = 0.5 m

viii) Lacey's silt factor = 0.9 m

ix) Permissible exit gradient = $\frac{1}{7}$

x) Permissible afflux = 1 m

Solution:

Step 1: Crest level

Pond level = 224.00 m

Height of shutters = 1.5 m

∴ Crest level = 224.00 − 1.50 = 222.50 m

Step 2: u/s T.E.L.

$$q = 1.7\,K^{\frac{3}{2}}$$

$$K = \left(\frac{q}{1.7}\right)^{\frac{2}{3}} = \left(\frac{12.5}{1.7}\right)^{\frac{2}{3}} = 3.78 \text{ m}$$

∴ Level of u/s T.E.L. = Crest level + K

= 222.50 + 3.78 = 226.28 m

Step 3: Scour depth, velocity and velocity head

Regime scour depth,

$$R = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35\left(\frac{(12.5)^2}{0.9}\right)^{\frac{1}{3}} = 7.5 \text{ m}$$

∴ Regime velocity $(V) = \frac{q}{R} = \frac{12.5}{7.5} = 1.67$ m/sec

Velocity head $\frac{V^2}{2g} = \frac{1.67^2}{2 \times 9.81} = 0.14$ m

Step 4: High flood condition

$$\text{u/s H.F.L.} = \text{u/s T.E.L} - \frac{V^2}{2g} = 226.28 - 0.14 = 226.14 \text{ m}$$

$$\text{d/s T.E.L.} = \text{HFL before construction} + \frac{V^2}{2g} = 225.00 + 0.14 = 225.14 \text{ m}$$

∴ Afflux = u/s HFL − d/s HFL = 226.14 − 225.00 = 1.14 m;

which is very near to the permissible afflux of 1 m.

d/s HFL after retrogression = HFL before construction − 0.50

= 225.00 − 0.5

= 224.5 m

d/s T.E.L. after retrogression = 225.14 − 0.5 = 224.64 m

∴ Loss of head at high flood is,

H_L = u/s TEL − d/s TEL after retrogression

= 226.28 − 224.64

= 1.64 m

Step 5: Pond level condition

When the weir is discharging with pond level u/s head over the crest,

= 224.00 − 222.5 = 1.50 m

∴ Discharge $(q) = 1.7\,(K)^{\frac{3}{2}} = 1.7\,(1.5)^{\frac{3}{2}} = 3.12$ cumec/m.

Loss of head at this time is,

H_L = Pond level − Anticipated d/s water level

= 224.00 − 221.5

= 2.50 m

Step 6: Hydraulic jump calculation

Hydraulic jump will be developed on the d/s sloping glacis. The calculations are done in the tabular form.

	Items	High flood condition	Pond level condition
1.	Discharge intensity, q	12.5 cumec/m	3.12 cumec/m
2.	Head loss, H_L	1.64 m	2.50 m
3.	d/s specific energy (Ef_2) from Blench curves [Figure 5.27(a)]	4.85 m	2.45 m
4.	Level at which jump will form $= \frac{d}{s}$ T.E.L after retrogression $- Ef_2$	224.64 − 4.85 = 219.79	221.5 − 2.45 = 219.05
5.	u/s specific energy $Ef_1 = Ef_2 + H_L$	4.85 + 1.64 = 6.49 m	2.45 + 2.5 = 4.95 m
6.	Pre-jump depth D_1 corresponding to Ef_1 from specific energy curves [figure 5.27(b)] *i.e.,* Montague curves or calculated from, $Ef_1 = D_1 + \frac{\left(\frac{q}{D_1}\right)^2}{2g}$	1.23 m	0.33 m
7.	Post jump depth D_2 corresponding to Ef_2 from specific energy curves [Figure 5.27(b)] or calculated from, $Ef_2 = D_2 + \frac{\left(\frac{q}{D_2}\right)^2}{2g}$	4.45 m	2.36 m
8.	Height of jump ($D_2 - D_1$)	4.45 − 1.23 = 3.22 m	2.36 − 0.33 = 2.03 m
9.	Length of concrete floor required beyond the jump = $5(D_2 - D_1)$	16.10 m Adopt 16.5 m	10.15 m

Table 5.4: Hydraulic jump calculation

Step 7: Design of glacis

Let, Crest width = 2 m

u/s slope 2 : 1

d/s glacis slope 3 : 1

Adopt d/s floor level = 218.85 m

(*i.e.,* about 0.2 m below the lower level at which jump will form)

Horizontal length of floor beyond the toe of the glacis = 16.5 m.

Step 8: Depth of sheet piles

Scour depth (R) = 7.5 m

Level of bottom of the possible scour hole at u/s end = u/s H.F.L − 1.5R

= 226.14 − 1.5 × 7.5

= 214.85 m

∴ The sheet pile at the u/s end may be taken up to a level of 214.85 m.

Level of bottom of the possible scour hole at d/s end = d/s H.F.L − 2R

= 224.50 − 2(7.5)

= 209.50 m

∴ The sheet pile at d/s end may be taken up to a level of 209.50 m.

Provide an intermediate pile at d/s toe of the glacis as the second line of defense with its bottom at the same level of 209.50 m.

Step 9: Length of impervious floor

Maximum percolation head = Pond level − d/s floor level

= 224.00 − 218.85

= 5.15 m

Depth of d/s pile = 218.85 − 209.50 = 9.35 m

Permissible exit gradient,

$$G_E = \frac{1}{7}$$

$$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$$

$$\frac{1}{\pi\sqrt{\lambda}} = \frac{d}{H}G_E = \frac{9.35}{5.15} \times \frac{1}{7} = 0.259$$

Hence, from figure 5.25 (end of chapter),

For $\frac{1}{\pi\sqrt{\lambda}} = 0.259$;

$$\alpha = \frac{b}{d} = 1.8$$

∴ Required length of impervious floor (b) = αd

= 1.8 × 9.5

= 16.83 m

Floor length already provided from other consideration

= (Horizontal length of u/s glacis) + (Length of crest)

+(Horizontal length of d/s glacis)

+(Length of floor beyond toe of d/s glacis)

= [2(222.5 − 218.75) + (2) + 3(222.50 − 218.85) + (16.5)]

= 7.5 + 2 + 10.95 + 16.5

= 36.95 m

Therefore, the floor length already provided is much more than that required from the consideration of permissible exit gradient. So, no impervious floor is required at u/s. However, a floor of length 1.05 m may be provided on u/s side so that the total length of impervious floor becomes 38 m.

Step 10: Percentage pressure at various key points

Having known the profiles of the glacis and the floor, the pressures at key points are determined by assuming the preliminary thickness of the floor at various points as under.

Floor thickness at u/s of crest = 1.0 m

Floor thickness at d/s end of glacis (*i.e.,* at intermediate pile) = 2.0 m

Floor thickness at d/s end of floor = 1.5 m

The calculation is shown in table 5.5 with reference to figure 5.29.

Pile line and pressure point	d (m)	$\frac{1}{\alpha}=\frac{d}{b}$	b_1	$\frac{b_1}{d}$	$\alpha=\frac{b}{d}$	Percentage pressure ϕ: Before correction	Correction for thickness	Correction for Slope	Correction for Interference	Correction values
u/s pile	3.9	0.103	-	-	-	-	-	-	-	-
Point D_1	-	-	-	-	-	80.0	-	-	-	80.0
Point C_1	-	-	-	-	-	71.0	+ 2.3	-	+ 3.5	76.8
Intermediate Pile	9.35	-	21.5	0.566	4.06	-	-	-	-	-
Point E_2	-	-	-	-	-	61.0	− 3.0	+ 2.3	−1.4	58.9
Point D_2	-	-	-	-	-	47.0	-	-	-	47.0
Point C_2	-	-	-	-	-	31.5	+ 3.3	-	+ 5.0	39.8
d/s pile	9.35	0.246	-	-	-	-	-	-	-	-
Point E_3	-	-	-	-	-	43.0	− 2.2	-	− 5.5	35.3
Point D_3	-	-	-	-	-	29.0	-	-	-	29.0

Table 5.5: Calculation of % pressure ($b = 38$ m; $b_s = 10.95$ m; $b' = 21$ m)

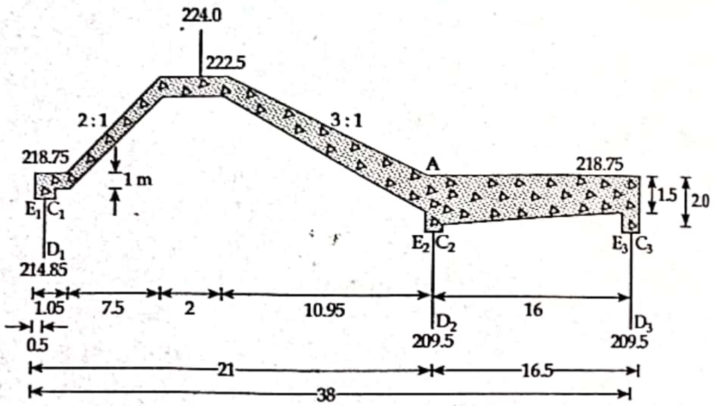


Figure 5.29

Step 11: Elevation of subsoil HGL for different condition

Knowing the % pressure at various key points, the elevation of H.G. line for following 3 conditions is shown in table 5.6.

i) Maximum static head
ii) High flood condition
iii) Pond level condition.

Step 12: Plotting the prejump water surface profile for high flood and pond level condition

The water surface profile on the d/s glacis before the formation of the jump can be obtained by finding depth D_1 at various points with the help of curves in figure 5.27(b). Or using formula,

$$Ef_1 = D_1 + \frac{\left(\frac{q}{D_1}\right)^2}{2g}$$

The calculations are shown in table 5.7.

Conditions	Datum (d/s floor water level or d/s water level) (m)	u/s water level		Height/Elevation of subsoil H.G line above datum: u/s pile ϕ_{E_1}	u/s pile ϕ_{D_1}	u/s pile ϕ_{C_1}	Intermediate pile ϕ_{E_2}	Intermediate pile ϕ_{D_2}	Intermediate pile ϕ_{C_2}	d/s pile ϕ_{E_3}	d/s pile ϕ_{D_3}	d/s pile ϕ_{C_3}
No flow condition head = 224 − 218.85 = 5.15 m	218.85	224	Height	5.15	4.12	3.96	3.03	2.42	2.05	1.82	1.49	0
			Elevation	224	222.97	222.81	221.81	221.27	220.90	220.67	220.34	218.85
High flood condition head = 226.14 − 224.5 = 1.64 m	224.50	226.14	Height	1.64	1.31	1.26	0.97	0.77	0.65	0.58	0.48	0
			Elevation	226.14	225.81	225.76	225.47	225.27	225.15	225.08	224.98	224.50
Flow at pond level head = 224 − 221.5 = 2.50 m	221.50	224.00	Height	2.50	2.00	1.92	1.47	1.18	1.00	0.88	0.73	0
			Elevation	224.00	223.50	223.42	222.97	222.68	222.50	222.38	222.23	221.50

Table 5.6: Elevation of HGL lines for various conditions

Distance from d/s end of crest (m)	Level of glacis (m)	High flood condition q = 12.5 cumec/m u/s T.E.L = 226.28 m: Ef_1 = 226.28 − Glacis level	D_1	Pond level condition q = 3.12 cumec/m u/s H.F.L = 224.00 m: Ef_1 = 224.00 − Glacis level	D_1
3.00	221.50	4.78	1.57	2.50	0.50
6.00	220.50	5.78	1.35	3.50	0.40
8.13	219.79 (Jump location for high flood)	6.49	1.23	-	-
9.00	219.50	-	-	4.50	0.35
10.35	219.05 (Jump location for pond level flow)	-	-	4.95	0.33

Table 5.7: Pre-jump profile (crest level 222.50)

Step 13: Calculation of floor thickness

Figure 5.30 and 5.31 shows the subsoil HGL for three conditions, *i.e.*,

i) No flow condition
ii) High flood condition
iii) Pond level flow condition

The floor is to be designed for the maximum unbalanced head found by subtracting elevation of water surface (or d/s level in case of no flow condition) from the elevation of corresponding HGL.

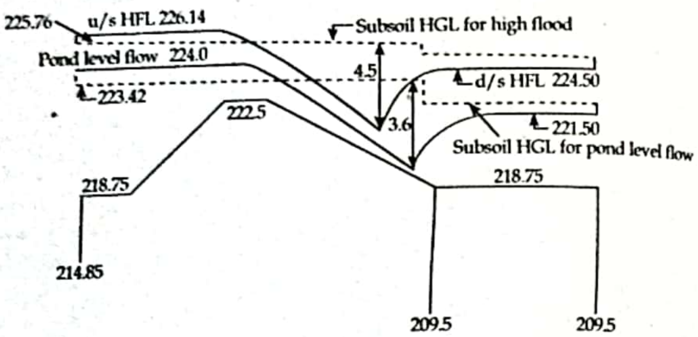


Figure 5.30: HG lines and water surface profiles

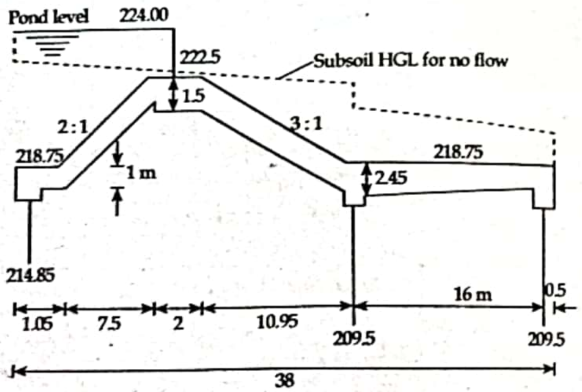


Figure 5.31: HGL for no flow condition

i) At the point of formation of hydraulic jump for high flood condition. From figure 5.30, unbalanced head for high flood condition = 4.5

$\frac{2}{3}$ of this head $= \frac{2}{3} \times 4.5 = 3.0$ m

Unbalanced static head for no flow condition = 2.21 m

Since, $\left(\frac{2}{3}\right)^{rd}$ of the unbalanced head for high flood condition is greater than the unbalanced static head for no flow condition, the former will be used for design. Thus,

Design head = 3.0 m

$\therefore$ Required thickness of floor $= \frac{3}{2.24 - 1} = 2.42$ m

ii) At the point of formation of hydraulic jump for flow at pond level condition.

Unbalanced head for flow at pond level condition = 3.6 m

$\frac{2}{3}$ of this head $= \frac{2}{3} \times 3.6 = 2.4$ m

Unbalanced static head for no flow condition = 2.86 m

$\therefore$ Design head = 2.68 m

$\therefore$ Required thickness of floor $= \frac{2.86}{2.24 - 1} = 2.31$ m

Hence, provide thickness of 2.45 m at the d/s end of the glacis decreasing it to 1.5 m at the end of the crest. A nominal thickness of 1 m is to be provided for the floor on the u/s of the glacis as shown in figure 5.31.

From figure 5.30, it may be seen that the design head for floor on the d/s of the glacis will be governed by the unbalanced static head which is greater than $\left(\frac{2}{3}\right)^{rd}$ of unbalanced head under the two flow conditions.

Pressure head at E_3 at d/s end of the floor = 1.82 m

$\therefore$ Required thickness of floor $= \frac{1.82}{2.24 - 1} = 1.47$ m

Hence, provide a thickness of 1.5 m at the d/s end of the floor.

Thus, the d/s floor has a thickness of 2.45 m at the d/s of glacis and a thickness of 1.5 m at the d/s end. This reduction in thickness may be done linearly as shown in figure 5.31.

5.4 DESIGN OF UNDERSLUICE AND SILT EXCLUDER

5.4.1 Design of Undersluice

Design procedure

i) Fix the discharge over the weir bay and undersluice bay sections. *i.e.,* 20% over sluice section and 80% over the weir bay section.

ii) Fix the crest levels of undersluice section and the weir bay section, *i.e.,* crest level of undersluice = Deepest bed level the slope of the d/s glacis is usually kept between 3 : 1 and 5 : 1. Crest level of weir bay = Crest level of undersluices + (1 to 1.5 m).

iii) Fix the waterways for the weir bay and undersluices sections.

iv) Determine the characteristic of the hydraulic hump for high flood condition and pond level condition with,

   a) No flow concentration and no retrogression.

   b) Flow concentration and with retrogression.

v) Calculate the normal scour depth and determine the bottom levels of the u/s and d/s piles. If necessary provide intermediate pile.

vi) Find the total length of impervious floor from exit gradient consideration.

vii) Calculate the percentage uplift pressure.

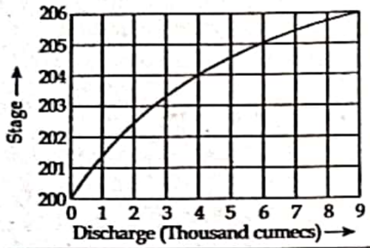
viii) Calculate the thickness of the floor at various points.

ix) Provide concrete block protection on the u/s and d/s sides.

x) Provide launching aprons on the u/s and d/s sides.

Example 5.6

Design the undersluice section of diversion headworks with following data.
Design flood discharge in the river = 9000 m^3/sec.
Deepest bed level of the river = 200.00 m
HFL before construction = 206.00 m
FSL of canal = 203.00 m
Permissible afflux = 1 m
Bed retrogression = 0.5 m
Discharge concentration factor = 20%
Lacey's silt factor = 1
Safe exit gradient = $\frac{1}{6}$
Pier contraction coefficient = 0.1
Full supply discharge of canal = 200 m^3/sec.
The stage discharge curve of the river at the site is shown in figure.
Assume other data if required.



Solution:

i) Crest level of undersluices = Deepest bed level = 200.00 m

Crest level of weir bay = 200 + 1.5 = 201.5 m

Pond level = FSL + Modular head = 203 + 1 = 204.00 m

NOTE

Modular head is equal to the sum of head loss in the regulator and the head required to pass full supply discharge into canal. It is generally taken as 1 m.

ii) Waterway

From Lacey's formula,

$$P = 4.75\sqrt{Q} = 4.75\sqrt{9000} = 450.62 \text{ m}$$

Let us fix the waterway as follows;

Undersluice portion

Number of spans = 5
5 bays of 16 m each = 80 m
4 piers of 2.5 m each = 10 m
Total = 90 m

Weir bay portion

Number of spans = 25
25 bays of 12 m each = 300 m

24 piers of 2 m each = 48 m
Total = 348 m
Width of fish ladder = 5 m
Width of divide wall = 3 m
Overall waterway between abutments = 90 + 348 + 5 + 3 = 446

This waterway is approximately equal to Lacey's waterway 'P'.

Discharge

Let us first determine the u/s TEL

u/s HFL after construction = HFL before construction + Afflux
= 206 + 1 = 207 m

Normal scour depth from lacey's formula,

$$R = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35\left\{\frac{(20.18)^2}{1}\right\}^{\frac{1}{3}} = 10 \text{ m}$$

$$\text{Velocity of approach} = \frac{q}{R} = \frac{20.18}{10} = 2.02 \text{ m/sec.}$$

$$\text{Head due to velocity of approach} = \frac{2.02^2}{2 \times 9.81} = 0.21 \text{ m}$$

u/s TEL = u/s HFL + Head due to velocity of approach
= 207 + 0.21
= 207.21 m

Undersluice

Head over the undersluice of the crest = 207.21 − 200 = 7.21 m

Since the u/s floor and the crest of the undersluice are at same level the width of crest is large and it will behave as a broad crested weir. Therefore, discharge is given by the equation,

$$Q = 1.705(L' - 0.1 \times nH_e)H_e^{\frac{3}{2}}$$

where, n is the number of end contractions.

Assuming that the end contractions near the divide wall and the abutment are suppressed, n = 8

$$Q = 1.705(80 - 0.1 \times 8 \times 7.21)(7.21)^{\frac{3}{2}}$$
$$= 2450.30 \text{ cumec}$$

Weir bay

Head over the weir bay crest = 207.21 − 201.5 = 5.71 m

Let us assume the crest of weir as 2.0 m. As the crest width B < $\frac{2H}{3}$, the weir will act as a sharp crest weir. Therefore, discharge is given by,

$$Q = 1.84(L' - 0.1 \times nH_e)H_e^{\frac{3}{2}}$$
$$= 1.84(300 - 0.1 \times 48 \times 5.71)(5.71)^{\frac{3}{2}}$$
$$= 6843.61 \text{ cumecs}$$

Total discharge Q = 6843.61 + 2450.30
= 9293.91 cumecs
> 9000 cumecs (OK)

Let us verify whether the discharge through undersluice is adequate,

Twice the design full supply discharge of canal = 400 cumecs

20% of the total design flood discharge = 1800 cumecs

The discharge capacity of undersluices is more than the minimum required by above two criteria and may be adopted.

Design of undersluice section

To find discharge intensity (q) and head loss (HL) for different conditions:

High flood conditions

i) **Without flow concentration and without retrogression**

$$q = 1.705H_e^{\frac{3}{2}} = 1.705 \times (7.21)^{\frac{3}{2}} = 33.00 \text{ cumecs/m}$$

d/s TEL = 206.00 + 0.21 = 206.21

u/s TEL = 207.00 + 0.21 = 207.21

Head Loss HL = 207.21 − 206.21 = 1.00 m

ii) **With 20% flow concentration and retrogression of 0.5 m**

$$q = 1.2 \times 33 = 39.6 \text{ cumecs/m}$$

$$\text{Head over the crest} = \left(\frac{39.6}{1.705}\right)^{\frac{2}{3}} = 8.14 \text{ m}$$

u/s TEL = 200.00 + 8.14 = 208.14 m

d/s TEL after retrogression = 206.21 − 0.5 = 205.71 m

Head Loss (H_L) = 208.14 − 205.71 = 2.43 m

Pond level conditions

i) **Without flow concentration and retrogression**

Head over the crest of undersluices = 204.00 − 200.00 = 4.00 m

At pond level condition, the velocity of approach can be found from the discharge which occurs at that level.

$$Q = 1.705\,(80 - 0.1 \times 8 \times 4.0)(4)^{\frac{3}{2}} + 1.84\,(300 - 0.1 \times 48\,(2.5)(2.5)^{\frac{3}{2}}$$
$$= 1047.55 + 2094.69$$
$$= 3142.24 \text{ cumecs}$$

$$\text{Average discharge intensity} = \frac{3142.24}{446} = 7.05 \text{ cumecs/m}$$

$$\text{Normal scour depth (R)} = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35\left\{\frac{(7.05)^2}{1}\right\}^{\frac{1}{3}} = 4.96 \text{ m}$$

$$\text{Velocity of approach } (V_a) = \frac{q}{R} = \frac{7.05}{4.96} = 1.42 \text{ m/sec.}$$

$$\text{Head due to velocity of approach} = \frac{(1.42)^2}{19.62} = 0.10 \text{ m}$$

u/s TEL = 204.00 + 0.1 = 204.10

$$\text{Discharge intensity (q)} = 1.705 \times (4.1)^{\frac{3}{2}} = 14.15 \text{ cumecs/m}$$

The water level on the d/s of the weir when a discharge of 3142.24 cumecs occurs on the river is obtained from stage curve shown in the figure above. *i.e.,* 203.50 m.

d/s TEL = 203.50 + 0.1 = 203.60 m

Head loss (H_L) = 204.10 − 203.60 = 0.50 m

ii) **With 20% flow condition and 0.5 m retrogression.**

Discharge intensity = 1.2 × 14.15 = 16.98 cumecs/m

$$\text{Head over the crest} = \left(\frac{16.98}{1.705}\right)^{\frac{2}{3}} = 4.63 \text{ m}$$

u/s TEL = 200.00 + 4.63 = 204.63 m

d/s water level after retrogression = 203.00 + 0.10 = 203.10 m

Head loss (H_L) = 204.63 − 203.10 = 1.53 m

5.4.2 Design of silt excluder

Design criteria

i) Silt excluders are designed for 15 to 20% of canal full supply discharge.

ii) A minimum velocity of 2 to 4.5 m/s should be maintained in order to eliminate silt deposition. Generally, 2 m/s for sandy rivers, 4 to 4.5 m/s for boulder stage rivers and 3 m/s for ordinary straight reaches.

iii) Knowing the discharge and velocity the cross section area of tunnel openings can be determined. $A = \frac{Q}{V}$

iv) The height (h) of the tunnel is equal to the height of the crest of head regulator above the u/s floor of undersluices minus thickness of roof slab.

v) Knowing the height (h), width is obtained as $B = \frac{A}{h}$.

vi) Total clear width is divided into number of tunnels such that the span is 1.8 m to 2.4 m for each tunnel separated by a divide wall of 0.6 m.

vii) Generally 4 to 6 tunnels are provided for each excluder.

viii) The length of the tunnel is different but head loss in each tunnel should be kept equal.

i.e., $h_f = \frac{v^2Ln^2}{R^{\frac{4}{3}}}$

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

$$S = \frac{h_f}{L}$$

ix) The height of tunnels varies from 0.5 to 0.6 m for sandy rivers and 0.8 to 1.2 m for boulder stage rivers.

x) At entrance, the tunnels are generally given a bell mouth shape and radius of bell mouthing generally varies from 2 to 6 times the tunnel width.

xi) At the exit, the tunnels are throttled for restricting the discharge to the desired value and to increase velocity to prevent deposition of silt.

Example 5.7

Design silt excluder for the diversion headwork for the data given below:
Full supply discharge of canal = 200 cumecs
Crest level of undersluices = 200 m
Crest level of head regulator = 202.00 m
Bay width of undersluices = 16 m

Solution:

Let design discharge of silt excluder be 20% of that of canal design discharge.

$$Q = 0.2 \times 200 = 40\ m^3/sec.$$

Let us assume the velocity of 2 m/s, then,

$$\text{Area of x-section (A)} = \frac{Q}{V} = \frac{40}{2} = 20\ m^2$$

Let the thickness of roof slab = 0.2 m

$$\text{Height of tunnel} = 202.00 - 0.2 - 200 = 1.8\ m$$

$$\text{Total clear width} = \frac{20}{1.8} = 11.11\ m^2$$

For clear span of 2.1 m,

$$\text{Number of tunnels} = \frac{11.11}{2.1} = 5.29\ (\text{say } 6.00\ m)$$

Let, thickness of divide wall = 0.65 m

$$\text{Overall width} = 6 \times 2.1 + 5 \times 0.65 = 15.85\ (\text{say } 16\ m)$$

Example 5.8

A sediment excluder is to be designed for a canal head off taking from a river with dominant discharge of 8000 cumecs. The other data of canal excluder are;
Canal discharge = 284 cumecs
Width of undersluice span of the barrage, where canal regulator is to be provided with an excluder = 15.35 m
River bed slope = $\frac{1}{4000}$
Average sediment diameter = 0.37 mm
Head available for design = 0.8 m
Manning's rugosity coefficient = 0.016

Solution:

Assuming 20% of canal design discharge as design discharge of silt excluder;

$$Q = 0.2 \times 284 = 56.8\ cumec$$

Design of tunnels

i) Number of tunnels; usually 4 to 6 tunnels are provided. Let us provide 6 tunnels.

ii) Width of undersluice bay is 15.35 m and assuming 0.6 m width of divide wall, the width of tunnel at exit $= \frac{15.35 - 5 \times 0.6}{6} = 2.06$ m

iii) Discharge through one tunnel $= \frac{56.8}{6} = 9.46$ cumec (say 9.45 cumec)

iv) The height of tunnel is chosen such that velocity through it is of the order of 2 m/sec. or more. At the exit the velocity may be taken higher say 3 m/sec.

$$\text{Area of the tunnel at exit} = \frac{Q}{V} = \frac{9.5}{3} = 3.15\ m^2$$

$$\text{and, Height} = \frac{3.15}{2.06} = 1.53\ m$$

Provide 1.53 m height of tunnel at the exit. This height is provided throughout the tunnel length.

v) Let the velocity at entry be 2 m/sec., then,

$$A = \frac{9.45}{2} = 4.73$$

$$\text{Tunnel width at entry} = \frac{4.73}{1.53} = 3.09\ (\text{say } 3.1\ m)$$

For better smooth entry,

$$\text{Entry width of tunnel} = 2 \times \text{Exit width}$$
$$= 2 \times 2.06 = 4.12\ m$$

∴ Entry width = 4.12 m
Exit width = 2.06 m
Height of tunnel = 1.53 m
Exit velocity = 3 m/sec.

Example 5.9

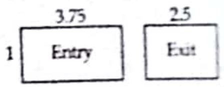
Design a silt excluder from the following data.
Canal discharge = 150 cumecs
Width of undersluice waterway = 13 m
Average sediment diameter = 0.37 m
Manning's roughness coefficient (η) = 0.016
Level difference between crest level of canal head regulator and u/s undersluice floor = 1.3 m
Length of tunnel = 70 m

Solution:

$$\text{Capacity of tunnel} = 20\% \text{ of canal discharge}$$
$$= 20\% \text{ of } 150$$
$$= 30\ cumecs$$

Assume,

Entry velocity = 2 m/sec.
Exit velocity = 3 m/sec.



$$\text{Entry cross section area} = \frac{30}{2} = 15\ m^2$$

$$\text{Exit cross section area} = \frac{30}{3} = 10\ m^2$$

Assume,

Slab thickness = 30 cm

Then,

Height of tunnel = 1.3 − 0.3 = 1 m [by 5.4.2 (iv)]

Width of tunnel entry = $\frac{15}{1}$ = 15 m

Width of tunnel exit = $\frac{10}{1}$ = 10 m

Let, number of tunnel = 4

∴ Each tunnel width = $\frac{15}{4}$ = 3.75 (at entry)

Let divide wall width = 30 cm = 0.3 m

Total width = 15 + 0.3 × 3 = 15.9 m

At exit,

Width of each tunnel = 2.5 m

5.5 DESIGN OF SILT EJECTOR

Design criteria

i) Silt ejectors are designed for 20 to 25% of the canal discharge.

ii) Bed of the canal is depressed by 0.3 m to 0.5 m at the mouth of the ejector so that approach velocities are reduced and the bed load may be trapped.

iii) The tunnels are of low height, about 0.5 to 0.6 m.

iv) The ideal distance between head regulator and silt ejector is usually between 150 m to 500 m.

v) The section of the sub-tunnel is gradually reduced such that there is an overall increase of 10 to 15 % in the velocity up to the exit.

vi) The section of main tunnel at exit is usually designed to attain the velocity of 2.5 to 6 m/s depending upon the grade of the sediment to be removed.

vii) The portion of the approach channel immediately upstream of the ejector should be pitched for a length of 3 to 4 times the depth of the water in the channel so that there is no erosion of the bed and sides of the channel.

Comparison between silt excluder and silt ejector

S.N.	Silt excluder	Silt ejector
i)	The silt excluder is located in the undersluice section of the river.	The silt ejector is located on the canal at some distance away from the head regulator.
ii)	The structure is heavy as it is subjected to large forces.	The structure is relatively light.
iii)	Good approach conditions are difficult to achieve.	Good approach conditions are easily achieved.
iv)	Working head is always available at the excluder.	Working head is reduced when the canal supply is low.
v)	The tunnels are quite large and are not liable to be clogged.	The tunnels are small and may be choked by debris.

5.6 WORKED OUT PROBLEMS

PROBLEM 1

Calculate the uplift pressure at key points in figure shown below using Khosla's theory. Check the exit gradient and thickness of the floor at A, B, C and D as shown in figure. The safe exit gradient is 0.15.

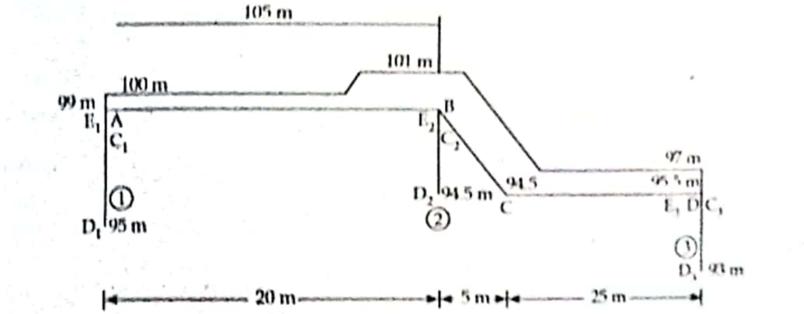


Figure 5.32

Solution:

For u/s pile line number 1:

Total length of floor, b = 20 + 5 + 25 = 50 m

Depth of u/s pile line, d = 100 − 95 = 5 m

$\alpha = \frac{b}{d} = \frac{50}{5} = 10$

Now,

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

NOTE

This value should be changed into radian.

where, $\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + 10^2}}{2} = 5.52$

$\phi_E = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) = \frac{1}{\pi} \times \cos^{-1}\left(\frac{5.52 - 2}{5.52}\right) \times \frac{\pi}{180} = 0.28 = 28\%$

$\phi_{C_1} = 100 - \phi_E = 100 - 28 = 72\%$

$\phi_{D_1} = 100 - \phi_D$

$\phi_D = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) = \frac{1}{\pi} \times \cos^{-1}\left(\frac{5.52 - 1}{5.52}\right) \times \frac{\pi}{180} = 0.195 = 19.5\%$

$\phi_{D_1} = 100 - \phi_D = 100 - 19.5 = 80.5\%$

Correction for ϕ_{C_1}

i) Correction at C_1 for mutual interference of piles

ϕ_{C_1} is affected by intermediate pile number 2

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

where, D = Depth of pile number 2.

i.e., $D = 99 - 94.5 = 4.5$ m

$d = 99 - 95 = 4$ m

$b' = 20$ m

$b = 50$ m

$$C = 19 \times \sqrt{\frac{4.5}{20}\frac{(4+4.5)}{50}} = 3.426\% \text{ (Positive)}$$

ii) Correction at C_1 due to thickness of floor

$$\text{Thickness correction for } C_1 = \frac{(\phi_{D_1} - \phi_{C_1})}{\text{R.L. of floor level} - \text{R.L. of } D_1} \times \text{Thickness of floor}$$

$$= \frac{80.5 - 72}{100 - 95} \times 1$$

$$= 1.7\% \text{ (Positive)}$$

iii) Correction due to slope is zero since, no slope starts or ends at this point.

∴ Corrected $\phi_{C_1} = 72 + 3.426 + 1.7 = 77.126\%$

For intermediate pile number 2

$$\phi_{E_2} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$$

$$\phi_{D_2} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$\phi_{C_2} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

where, $\lambda = \dfrac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2}$.

$$\lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2}$$

$$\alpha_1 = \frac{b_1}{d}$$

$$\alpha_2 = \frac{b_2}{d}$$

d = Depth of intermediate pile = 101 − 94.5 = 6.5 m

b_1 = Floor length u/s of intermediate pile = 20 m

b_2 = Floor length d/s of intermediate pile = 30

$$\alpha_1 = \frac{20}{6.5} = 3.077$$

$$\alpha_2 = \frac{30}{6.5} = 4.615$$

$$\lambda = \frac{\sqrt{1+3.077^2} + \sqrt{1+(4.615)^2}}{2} = 3.979$$

$$\lambda_1 = \frac{\sqrt{1+3.077^2} - \sqrt{1+(4.615)^2}}{2} = -0.743$$

∴ $\phi_{E_2} = 0.644 = 64.4\%$

$\phi_{D_2} = 0.560 = 56.0\%$

$\phi_{E_2} = 0.479 = 47.9\%$

Correction for ϕ_{E_2}

i) Mutual interference due to pile 1

$D = 99 - 95 = 4$ m

$d = 99 - 94.5 = 4.5$ m

$b' = 20$ m

$$C = 19\sqrt{\frac{4}{20}\frac{(4+4.5)}{50}} = 1.444\% \text{ (Negative)}$$

ii) Thickness of floor (C) $= \dfrac{64.4 - 56}{101 - 94.5} \times 2 = 2.584\%$ (Negative)

∴ Corrected $\phi_{E_2} = 64.4 - 1.444 - 2.584 = 60.372\%$

Correction for ϕ_{C_2}

i) Mutual interference with pile 3

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

where, $D = 99 - 93 = 7$ m

$d = 99 - 94.5 = 4.5$ m

$b' = 30$ m

$b = 50$ m

$$C = 19\sqrt{\frac{7}{30}\frac{(4.5+7)}{50}} = 2.111\% \text{ (Positive)}$$

ii) Correction due to thickness

$$C = \frac{56 - 47.9}{101 - 94.5} \times 2 = 2.492\% \text{ (Positive)}$$

iii) Correction due to slope

$$C = C_S \times \frac{b_S}{b_1}$$

$b_S = 5$ m

$b_1 = 30$ m

$C_S = 11.2$ (Let $C_S = 11.2$, for slope of 1 : 1.11)

$$C = 11.2 \times \frac{5}{30} = 1.867\% \text{ (Positive)}$$

∴ Corrected $\phi_{C_2} = 47.9 + 2.111 + 2.492 + 1.867 = 54.37\%$

For d/s pile number 3

$b = 50$ m

$d = 97 - 93 = 4$ m

$$\alpha = \frac{b}{d} = \frac{50}{4} = 12.5 \text{ m}$$

$$\lambda = \frac{1+\sqrt{1+12.5^2}}{2} = 6.77$$

$$\phi_{E_3} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = \frac{1}{\pi} \times \cos^{-1}\left(\frac{6.67-2}{6.67}\right) \times \frac{\pi}{180} = 0.253 = 25.3\%$$

$$\phi_{D_3} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{6.67-1}{6.67}\right) = 0.177 = 17.7\%$$

Correction for ϕ_{E_3}

i) **Mutual interference with pile 2**

$D = 95.5 - 94.5 = 1$

$d = 95.5 - 93 = 2.5$

$b = 50$ m

$b' = 30$ m

$$C = 19\sqrt{\frac{1}{30}}\frac{(1+2.5)}{50} = 0.243\%$$

ii) **Correction due to floor thickness**

$$C = \frac{25.3-17.7}{97-93} \times 1.5 = 2.85\% \text{ (Negative)}$$

iii) **No correction due to slope**

$\therefore$ Corrected $\phi_{E_3} = 25.3 - 0.243 - 2.85 = 22.207\%$

The corrected % pressure at key points is tabulated as under:

u/s pile 1	Intermediate pile 2	d/s pile 3
$\phi_{E_1} = 100\%$	$\phi_{E_2} = 60.372\%$	$\phi_{E_3} = 22.207\%$
$\phi_{D_1} = 80.5\%$	$\phi_{D_2} = 56\%$	$\phi_{D_3} = 17.7\%$
$\phi_{C_1} = 77.126\%$	$\phi_{C_2} = 54.37\%$	$\phi_{C_3} = 0\%$

Difference in head between pond level and d/s floor $= 105 - 97 = 8$ m

Uplift pressures $@D_1 = 0.805 \times 8 = 6.44$ m

$@C_1 = 0.771 \times 8 = 6.168$ m

$@E_2 = 0.6037 \times 8 = 4.83$ m

$@D_2 = 0.56 \times 8 = 4.48$ m

$@C_2 = 0.5437 \times 8 = 4.35$ m

$@E_3 = 0.222 \times 8 = 1.776$ m

$@C_3 = 0$

Exit gradient

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$$

$d = 97 - 93 = 4$ m

H = Maximum seepage head $= 105 - 97 = 8$ m

b = total floor length $= 50$ m

$$\alpha = \frac{b}{d} = \frac{50}{4} = 12.5 \text{ m}$$

$$\lambda = \frac{1+\sqrt{1+12.5^2}}{2} = 6.77$$

$$G_E = \frac{8}{4} \cdot \frac{1}{\pi\sqrt{6.77}} = 0.245 > 0.15 \text{ (Unsafe)}$$

Thickness of floor

$$\text{Thickness required at A} = \frac{\text{Uplift at A}}{G-1} = \frac{6.168}{2.24-1} = 4.974 > 1 \text{ m (Unsafe)}$$

$$\text{Thickness required at B} = \frac{4.35}{2.24-1} = 3.508 \text{ m} > 2 \text{ m (Unsafe)}$$

$$\text{Uplift pressure at C} = 1.776 + \frac{4.35-1.776}{30} \times 25 = 3.921 \text{ m}$$

$$\text{Thickness required at C} = \frac{3.921}{1.24} = 3.162 > 2.5 \text{ (Unsafe)}$$

$$\text{Thickness required at D} = \frac{1.776}{1.24} = 1.432 < 1.5 \text{ (Safe)}$$

PROBLEM 2

Find out the corrected pressure at u/s, downstream and Intermediate key points of a Hydraulic structure founded on permeable foundation as given below. Use factor for slope correction as 4%. Assume suitable data if necessary. Also calculate the exit gradient, and plot the hydraulic gradient line.

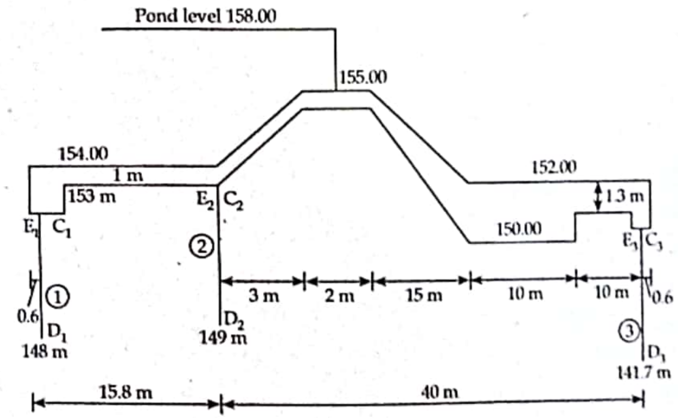


Figure 5.33

Solution:

For u/s pile number 1

Total length of floor $b = 56.4$ m

$d = 154 - 148 = 6$ m

$$\alpha = \frac{b}{d} = 9.4 \text{ m}$$

Now,

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

$$\text{where, } \lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+9.4^2}}{2} = 5.227$$

$\phi_E = 0.288 = 28.8\%$

$\phi_{C_1} = 100 - \phi_E = 100 - 28.8 = 71.2\%$

$\phi_D = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) = 0.2 = 20\%$

$\phi_{D_1} = 100 - 20 = 80\%$

Correction for ϕ_{C_1}

i) **Mutual interference due to pile 2**

$D = 153 - 149 = 4$ m

$d = 153 - 148 = 5$ m

$C = 19\sqrt{\frac{4}{15.8}}\frac{(4+5)}{56.4} = 1.526\%$ (Positive)

ii) **Correction due to floor thickness**

$C = \frac{80 - 71.2}{154 - 148} \times 1 = 1.467\%$ (Positive)

iii) **Slope correction is nil**

$\therefore$ Corrected $\phi_{C_1} = 71.2 + 1.526 + 1.467 = 74.193\%$

For intermediate pile number 2

$d = 154 - 149 = 5$ m

$b_1 = 15.2$ m

$b_2 = 40$ m

$\alpha_1 = \frac{b_1}{d} = \frac{15.2}{5} = 3.04$

$\alpha_2 = \frac{b_2}{d} = \frac{40}{5} = 8$

$\lambda = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2} = 5.631$

$\lambda_1 = -2.431$

$\phi_{E_2} = 0.709 = 70.9\%$

$\phi_{D_2} = 0.642 = 64.2\%$

$\phi_{C_2} = 0.582 = 58.2\%$

Correction for ϕ_{E_2}

i) **Mutual interference due to pile 1**

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

$D = 153 - 148 = 5$ m

$d = 153 - 149 = 4$ m

$b' = 15.2$ m

$b = 54.6$ m

$C = 19\sqrt{\frac{5}{15.2}}\frac{(4+5)}{54.6} = 1.796\%$ (Negative)

ii) **Floor thickness correction**

$C = \left(\frac{70.9 - 64.2}{154 - 149}\right) \times 1 = 1.34\%$ (Negative)

iii) **Slope correction is nil**

Corrected $\phi_{E_2} = 70.9 - 1.796 - 1.34 = 67.764\%$

Correction for ϕ_{C_2}

i) **Mutual interference due to pile 3**

$D = 153 - 141.7 = 11.30$ m

$d = 153 - 149 = 4.0$ m

$b' = 40$ m

$b = 56.4$ m

$C = 2.74\%$ (Positive)

ii) **Due to floor thickness**

$C = \frac{70.9 - 64.2}{154 - 149} \times 1 = 1.34\%$ (Positive)

iii) **Correction due to slope**

$C = C_S \times \frac{b_S}{b_1}$

$b_S = 3$ m

$b_1 = 40$ m

$C_S = 4\%$ (Given)

$C = 4 \times \frac{3}{40} = 0.3\%$ (Negative)

Corrected $\phi_{C_2} = 58.2 + 2.74 + 1.34 - 0.3 = 61.98\%$

For d/s pile number 3

$b = 54.6$ m

$d = 152 - 141.7 = 10.3$ m

$\alpha = \frac{b}{d} = \frac{54.6}{10.3} = 5.301$ m

$\lambda = \frac{1 + \sqrt{1 + (5.301)^2}}{2} = 3.197$

$\phi_{E_3} = 0.378 = 37.8\%$

$\phi_{D_3} = 0.259 = 25.9\%$

Correction for ϕ_{E_3}

i) **Mutual interference due to pile 2**

$D = 150.7 - 149 = 1.7$

$d = 150.7 - 141.7 = 9$

$b' = 40$ m

$b = 56.4$ m

$C = 0.743\%$ (Negative)

ii) **Floor thickness correction**

$C = \frac{37.8 - 25.9}{152 - 141.7} \times 1.3 = 1.155 \times 1.3 = 1.502\%$ (Negative)

iii) **Slope correction is nil**

Corrected $\phi_{E_3} = 37.8 - 0.743 - 1.502 = 35.555\%$

Percentage pressure (corrected) is tabulated as;

u/s pile 1	Intermediate pile 2	d/s pile 3
$\phi_{E_1} = 100\%$	$\phi_{E_2} = 67.764\%$	$\phi_{E_3} = 35.555\%$
$\phi_{D_1} = 80\%$	$\phi_{D_2} = 64.2\%$	$\phi_{D_3} = 25.9\%$
$\phi_{C_1} = 74.193\%$	$\phi_{C_2} = 58.2\%$	$\phi_{C_3} = 0\%$

Maximum seepage head available = 158 – 152 = 6 m

Uplift pressure @ key points,

$@E_1 = 6$ m
$@D_1 = 0.8 \times 6 = 4.8$ m
$@C_1 = 0.74193 \times 6 = 4.452$ m
$@E_2 = 0.6776 \times 6 = 4.066$ m
$@D_2 = 0.642 \times 6 = 3.852$ m
$@C_2 = 0.582 \times 6 = 3.492$ m
$@E_3 = 0.355 \times 6 = 2.13$ m
$@D_3 = 0.259 \times 6 = 1.554$ m

Elevation of HGL can be calculated starting from d/s end,

HGL $@C_3 = 152$ m
$@D_3 = 152 +$ Uplift pressure $@D_3$
$= 152 + 1.554 = 153.554$
$@E_3 = 153.554 +$ Uplift $@ E_3 -$ Uplift $@ D_3$
$= 153.554 + 2.13 - 1.554 = 154.13$
$@C_2 = 154.13 + (3.492 - 2.13) = 155.492$
$@D_2 = 155.492 + (3.852 - 3.492) = 155.852$
$@E_2 = 155.852 + (4.066 - 3.852) = 156.066$ m
$@C_1 = 156.066 + (4.452 - 4.066) = 156.452$ m
$@D_1 = 156.452 + (4.8 - 4.452) = 156.8$ m
$@E_1 = 156.8 + (6 - 4.8) = 158$ m

Plotting of HGL

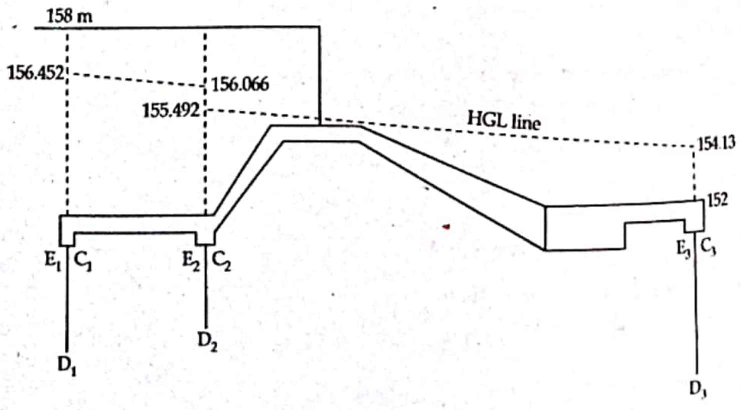


Figure 5.34: Plotting of HGL

Exit gradient

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$$

H = 6 m
d = 152 – 141.7 = 10.3 m
b = 54.6 m

$$\alpha = \frac{b}{d} = 5.301$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = 3.197$$

$G_E = 0.104 = 1$ in 9.643

PROBLEM 3

Drawing neat sketches. Explain how silt excluders and silt ejectors control the bed load in an irrigation system.

Solution:

Silt excluder and silt ejectors

See the definition part 5.1.6 and 5.1.7

Working principle

The basic principle of silt control is;

Most of the silt tries to settle down in water, thus, confining itself mostly in the bottom layers of water.

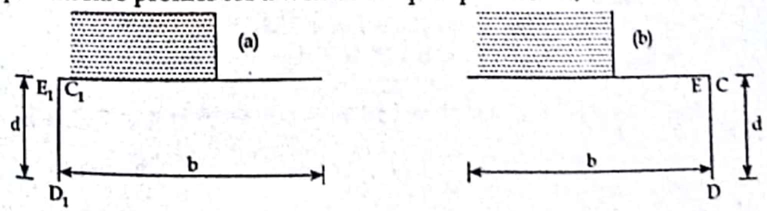
- The silt is also kept in suspension by the force of vertical eddies generated by the friction of the flowing water against the bed. The more the bed friction greater will be the upward force of eddies and lesser chance of silt settlement will exist. Hence, the main principle of silt excluder/extractor construction is to reduce this friction by constructing a smooth approach channel for efficient settlement of silt.
- The settled silt is then removed by separating top and bottom layer without causing disturbance to the flow. It can be better attained in a canal bed rather than in a river bed. Hence, work constructed in canal is more effective than the work constructed in river bed. *i.e.*, silt ejectors are better than silt excluder.

PROBLEM 4

Draw four simple Khosla's profiles for a weir of complex profiles. What corrections Khosla suggested to accommodate such simplifications?

Solution:

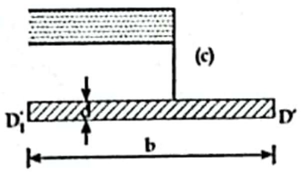
Simple Khosla's profiles for a weir of complex profiles are;

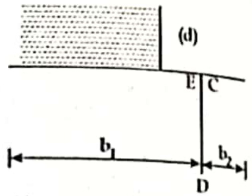


$$\phi_{C_1} = 100 - \phi_E \qquad \phi_E = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

$$\phi_{D_1} = 100 - \phi_D \qquad \phi_D = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$$

where, $\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2}$ and $\alpha = \frac{b}{d}$.





$$\phi_D' = \frac{2}{3}(\phi_E - \phi_D) + \frac{3}{\alpha^2} \qquad \phi_E = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$$

$$\phi_{D_1}' = 100 - \text{Ø}_D' \qquad \phi_D = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$\phi_C = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

Figure 5.36: Khosla's simple profiles

where, $\lambda = \frac{\sqrt{1 + \alpha_1^2} + \sqrt{1 + \alpha_2^2}}{2}$.

$$\lambda_1 = \frac{\sqrt{1 + \alpha_1^2} - \sqrt{1 + \alpha_2^2}}{2}$$

where, $\alpha_1 = \frac{b_1}{d}$ and $\alpha_2 = \frac{b_2}{d}$.

For correction given by Khosla

See the definition part 5.2.3 (i), (ii) and (iii)

PROBLEM 5

Write with definition sketch, how do you determine the pressure along the foundation of the structure and ensure safety against uplift pressure using Bligh's creep theory.

Solution:

Consider a section as shown in the figure below.

Let, H_L is the difference of water levels between u/s and d/s ends.

Water will seep along the bottom profile as shown above by arrows according to Bligh's theory. Seepage water starts from 'A' and emerges at 'B'.

$$\begin{aligned} \text{Total length of creep (L)} &= d_1 + d_1 + L_1 + d_2 + d_2 + L_2 + d_3 + d_3 \\ &= (L_1 + L_2) + 2(d_1 + d_2 + d_3) \\ &= b + 2(d_1 + d_2 + d_3) \end{aligned}$$

$$\begin{aligned} \text{Head losses per unit length or hydraulic gradient} &= \frac{H_L}{b + 2(d_1 + d_2 + d_3)} \\ &= \frac{H_L}{L} \end{aligned}$$

Head loss equal to $\left(\frac{H_L}{L} \times 2d_1\right)$, $\left(\frac{H_L}{L} \times 2d_2\right)$ and $\left(\frac{H_L}{L} \times 2d_3\right)$ occur respectively in the plane of three vertical cutoffs. The HGL is drawn as shown in figure.

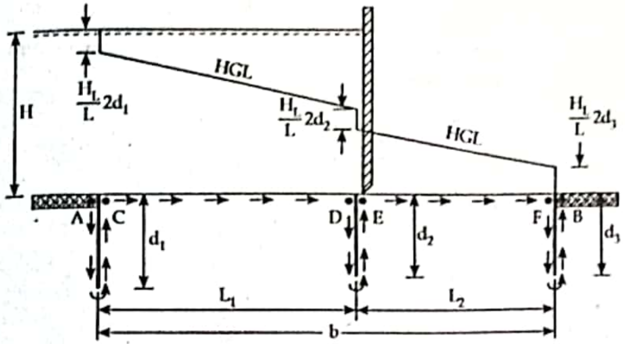


Figure 5.35

Safety against uplift pressure

If the uplift head at any point is H_1 (meter or water) then uplift head has to be counterbalanced by the weight of floor thickness. Then,

Uplift pressure $= \gamma_w H_L$

Pressure due to weight of floor thickness (t) $= \gamma_w \cdot G_C \cdot t$

where, G_C is the specific gravity of concrete.

For equilibrium,

$$\gamma_w \cdot H_L = \gamma_w \cdot G_C \cdot t$$

Subtracting 't' on both sides; we get,

$$H_L - t = G_C \cdot t - t = t(G_C - 1)$$

$$t = \frac{H_L - t}{G_C - 1} = \frac{H}{G_C - 1}$$

where, $H = H_L - t$ = ordinate of HGL above the top of floor.

By calculating the floor thickness using above equation and increasing by 33% the safety against uplift pressure is ensured.

PROBLEM 6

Draw neat sketches (plan and typical section through undersluice) of a barrage and show all components with sketch, explain how silt excluder works?

Solution:

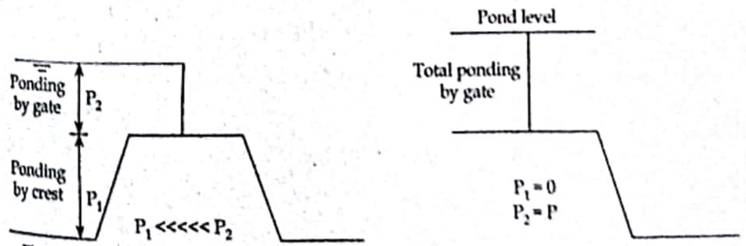


Figure: (a) Barrage with small raised crest   Figure: (b) Barrage without any raised crest

Figure 5.37

Principle of silt excluder and figure

See the solution of Q. no. 3

PROBLEM 7

Determine the thickness of u/s and d/s floor shown in figure below, using Khosla's and Bligh's methods.

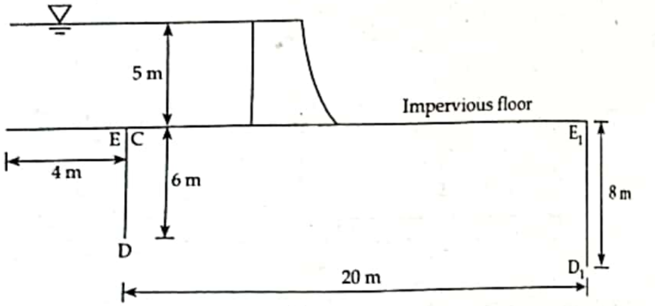


Figure 5.38

Solution:

Using Khosla's theory;

We have to find the thickness of floor at u/s and d/s. *i.e.,* at 'C' and E_1

At first; we should find out % pressure at 'C' and E_1.

At 'C'

For the figure

See the solution of Q. no. 5

$b_1 = 4$ m

$b_2 = 20$ m

$d = 6$ m

$$\alpha_1 = \frac{b_1}{d} = 0.667$$

$$\alpha_2 = \frac{b_2}{d} = 3.333$$

$$\lambda = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2} = 2.341$$

$$\lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2} = -1.139$$

$$\phi_C = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1+1}{\lambda}\right) \times \frac{\pi}{180} = 0.519 = 51.9\%$$

Correction due to mutual interference, by d/s pile

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

$D = 8$ m

$d = 6$ m

$b' = 20$ m

$b = 24$ m

$$C = 19\sqrt{\frac{8}{20}}\frac{(6+8)}{24} = 7.01\%\ \text{(Positive)}$$

$\therefore$ Corrected $\phi_{C_1} = 51.9 + 7.01 = 58.91\%$ [Neglecting thickness correction]

At E_1

For the figure

See the solution of Q. no. 5

$b = 24$ m

$d = 8$ m

$$\alpha = \frac{24}{8} = 3$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 2.081$$

$$\phi_{E_1} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) \times \frac{\pi}{180} = 0.488 = 48.8\%$$

Correction due to mutual interference due to u/s pile,

$D = 6$ m

$d = 8$ m

$b' = 20$ m

$b = 24$ m

$$C = 19\sqrt{\frac{6}{20}}\frac{(6+8)}{50} = 6.071\%\ \text{(Negative)}$$

$\therefore$ Corrected $\phi_{E_1} = 48.8 - 6.071 = 42.729\%$

Total seepage head available = 5 m

Then,

Uplift pressure at C = $0.5891 \times 5 = 2.946$ m

Uplift pressure at $E_1 = 0.4273 \times 5 = 2.137$ m

$$\text{Thickness of floor at C} = \frac{\text{Uplift pressure}}{G_C - 1} = \frac{2.946}{2.24-1} = 2.376\text{ m}$$

$$\text{Thickness of floor at } E_1 = \frac{2.137}{1.24} = 1.723\text{ m}$$

$\therefore$ Use 2.5 m thick floor at u/s

Use 1.8 m thick floor at d/s

Using Bligh's theory,

At 'C'

Total length of creep = $4 + 6 \times 2 + 20 + 8 \times 2 = 52$ m

Total seepage head = 5 m

Creep length up to C = $4 + 6 \times 2 = 16$ m

Seepage head available for 52 m creep length = 5 m

Seepage head available for 1 m creep length = $\frac{5}{52}$

Seepage head available for 16 m creep length = $\frac{5}{52} \times 16 = 1.538$ m

Residual head at C = 5 − 1.538 = 3.462 m

$\therefore$ Thickness of floor at C $= \dfrac{\text{Residual head(H)}}{G_C - 1} = \dfrac{3.462}{2.24 - 1} = 2.792$ m

At E_1

Creep length up to $E_1 = 4 + 6 \times 2 + 20 = 36$ m

Seepage head available for 36 m creep length $= \dfrac{5}{52} \times 36 = 3.462$

Residual head at $E_1 = 5 - 3.462 = 1.538$ m

Thickness of floor at $E_1 = \dfrac{1.538}{1.24} = 1.24$ m

$\therefore$ According to Bligh's theory,
Use 2.8 thick floor at u/s
and, 1.5 m thick floor at d/s

PROBLEM 8

Draw a neat sketch of the general layout of diversion headworks and cross-sections of undersluices, canal head regulator and weir with all details. **[2069 Bhadra]**

Solution:

General layout of diversion headworks

See the definition part 5.1

Cross-section of undersluice

See the definition part 5.1.2

Section through canal head regulator (Figure 5.39)

See the definition part 5.1.1 (Section of weir with details)

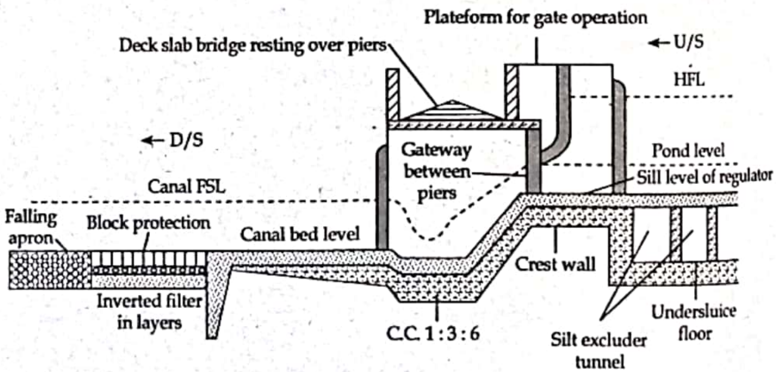


Figure 5.39: Section through canal head regulator

PROBLEM 9

Design height and width of silt excluder tunnels. Discharge of canal = 200 m³/sec; crest level of undersluice = 200 m, crest level of head regulator = 202 m, Bay width of undersluices = 16 m. Assume other data suitably. **[2069 Poush]**

Solution: See the solution of example 5.7

PROBLEM 10

Sketch the hydraulic gradient line of the weir profile shown below, $C_s = 6.5$ for (2 : 1) **[2070 Bhadra]**

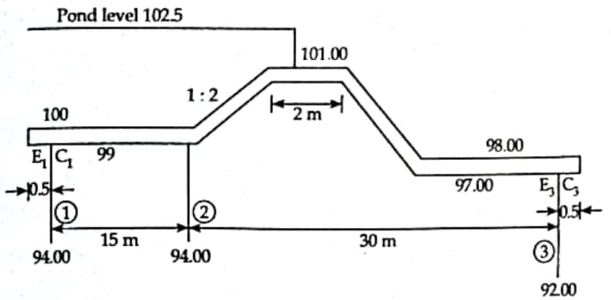


Figure 5.40

Solution:

For u/s pile number 1

Total length of floor (b) = 46 m

$d = 100 - 94 = 7$ m

$\alpha = \dfrac{46}{6} = 7.67$

$\lambda = \dfrac{1 + \sqrt{1 + 7.67^2}}{2}$

$\phi_E = \dfrac{1}{\pi} \times \cos^{-1}\left(\dfrac{\lambda - 2}{\lambda}\right) \times \dfrac{\pi}{180} = 0.32 = 32\%$

$\phi_{C_1} = 100 - \phi_E = 100 - 32 = 68\%$

$\phi_D = \dfrac{1}{\pi} \times \cos^{-1}\left(\dfrac{\lambda - 1}{\lambda}\right) = 0.22 = 22\%$

$\phi_{D_1} = 100 - 22 = 78\%$

Correction for ϕ_{C_1}

i) **Mutual interference to pile 2**

$C = 19\sqrt{\dfrac{D}{b'}}\dfrac{(d + D)}{b}$

$D = 99 - 94 = 5$ m

$d = 99 - 94 = 5$ m

$C = 19\sqrt{\dfrac{5}{15}}\dfrac{(5 + 5)}{46} = 2.38\%$ (Positive)

ii) **Correction due to thickness of floor**

$C = \dfrac{\phi_{D_1} - \phi_{C_1}}{\text{R.L. of floor level} - \text{R.L. of } D_1} = \dfrac{78 - 68}{100 - 64} = 1.67\%$ (Positive)

iii) **Slope correction is zero**

$\therefore$ Corrected $\phi_{C_1} = 68 + 2.38 + 1.67 = 72.05\%$

For intermediate pile number 2

$d = 100 - 94 = 6$ m

$b_1 = 15.5$ m
$b_2 = 30.5$ m
$\alpha_1 = \frac{15.5}{6} = 2.58$
$\alpha_2 = \frac{30.5}{6} = 5.08$
$\lambda = 3.97$
$\lambda_1 = -1.21$
$\phi_{E_2} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right) \times \frac{\pi}{180} = 0.69 = 69\%$
$\phi_{D_2} = 0.58 = 58\%$
$\phi_{C_2} = 0.51 = 51\%$

Correction for ϕ_{E_2}

i) Correction due to mutual interference due to pile 1
$D = 99 - 94 = 5$ m
$d = 99 - 94 = 5$ m
$b' = 15$ m
$b = 46$ m
$C = 2.38\%$ (Negative)

ii) **Correction due to floor thickness,**
$C = \frac{58-69}{100-94} \times 1 = 1.83\%$ (Negative)

iii) **Slope correction is nil**
∴ Corrected $\phi_{E_2} = 69 - 2.38 - 1.83 = 64.79\%$

Correction for ϕ_{C_2}

i) **Correction for mutual interference due to pile 3**
$D = 99 - 92 = 7$ m
$d = 99 - 94 = 5$ m
$b' = 30$ m
$b = 46$ m
$C = 2.39\%$ (Positive)

ii) **Correction due to floor thickness**
$C = \frac{58-51}{100-94} \times 1 = 1.17\%$ (Positive)

iii) **Correction due to slope**
$C = C_S \times \frac{b_S}{b_1}$
$b_S = 2$ m
$b_1 = 30$ m
$C_S = 6.5$
$C = 6.5 \times \frac{2}{30} = 0.433\%$
∴ Corrected $\phi_{C_2} = 51 + 2.39 + 1.17 - 0.433 = 54.127\%$

For d/s pile number 3
$b = 46$ m
$d = 98 - 92 = 6$ m
$\alpha = \frac{b}{d} = 7.67$
$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+7.67^2}}{2} = 4.37$
$\phi_{E_3} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) \times \frac{\pi}{180} = 0.32 = 32\%$
$\phi_{D_3} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{6.67 - 1}{6.67}\right) = 0.32 = 22\%$

Correction for ϕ_{E_3}

i) **Mutual interference**
$D = 97 - 94 = 6$ m
$d = 97 - 92 = 5$ m
$b = 46$ m
$b' = 30$ m
$C = 3.09\%$ (Negative)

ii) **Correction due to floor thickness**
$C = \frac{22-32}{98-92} \times 1 = 1.67\%$ (Negative)

ii) **Correction due to slope is nil**
∴ Corrected $\phi_{E_3} = 32 - 3.09 - 1.67 = 27.24\%$

The corrected % pressure at key points can be tabulated as;

u/s pile 1	Intermediate pile 2	d/s pile 3
$\phi_{E_1} = 100\%$	$\phi_{E_2} = 64.79\%$	$\phi_{E_3} = 27.24\%$
$\phi_{D_1} = 78\%$	$\phi_{D_2} = 58\%$	$\phi_{D_3} = 22\%$
$\phi_{C_1} = 72.05\%$	$\phi_{C_2} = 54.127\%$	$\phi_{C_3} = 0\%$

Diff. in head between u/s water level and d/s floor level $= 102.5 - 98 = 4.5$ m

Uplift pressure $@C_1 = 0.7205 \times 4.5 = 3.24$ m
$@D_1 = 0.78 \times 4.5 = 3.51$ m
$@E_2 = 0.6479 \times 4.5 = 2.92$ m
$@D_2 = 0.58 \times 4.5 = 2.61$ m
$@C_2 = 0.5412 \times 4.5 = 2.314$ m
$@E_3 = 0.2724 \times 4.5 = 1.226$ m
$@C_3 = 0.22 \times 4.5 = 0.99$ m

Elevation of HGL is calculated starting from d/s end
HGL $@C_3 = 98.00$ m
$@D_3 = 98 + 0.99 = 98.99$ m
$@E_3 = 98.99 + (1.226 - 0.99) = 99.226$ m
$@C_2 = 99.226 + (2.314 - 1.226) = 100.314$ m

$@D_2 = 100.314 + (2.61 - 2.314) = 100.61$ m
$@E_2 = 100.61 + (2.92 - 2.61) = 100.92$ m
$@C_1 = 100.92 + (3.24 - 2.92) = 101.24$ m
$@D_1 = 101.24 + (3.51 - 3.24) = 101.51$ m
$@E_1 = 101.51 + (4.5 - 3.51) = 102.5$ m

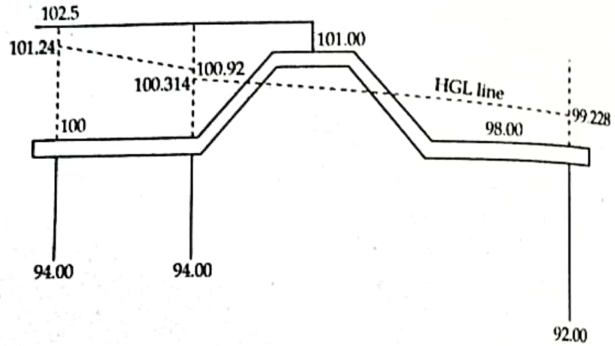


Figure 5.41: Plotting of HG line

Exit gradient

$H = 4.5$ m
$d = 98 - 92 = 6$ m
$b = 46$ m
$\alpha = \frac{b}{d} = 7.67$ m
$\lambda = 4.37$
$G_E = \frac{4.5}{6} \times \frac{1}{\pi\sqrt{4.37}} = 0.114 = 1$ in 8.756

PROBLEM 11

Using Khosla's method, check the safety of weir profile shown below against piping and uplift (at point 'A'). Safe exit gradient may be assumed to be 1 in 5. **[2070 Magh]**

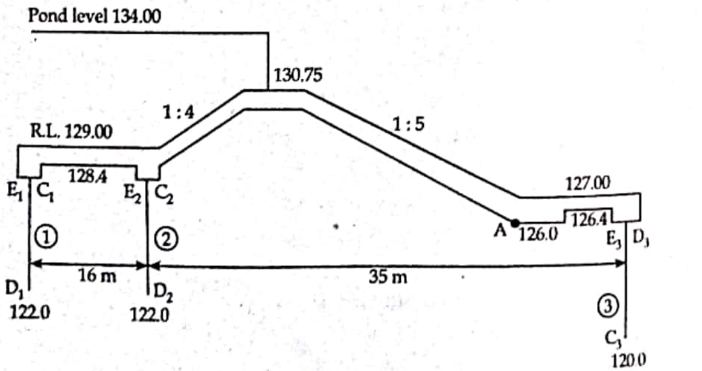


Figure 5.42

Solution:

For u/s pile number 1

$b = 51$ m
$d = 129 - 122 = 7$ m
$\alpha = \frac{51}{7} = 7.286$
$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = 4.18$
$\phi_E = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) = \frac{1}{\pi} \times \cos^{-1}\left(\frac{4.18 - 2}{4.18}\right) \times \frac{\pi}{180} = 0.33 = 33\%$
$\phi_{C_1} = 100 - \phi_E = 100 - 33 = 67\%$

We have,

$\phi_{D_1} = 100 - \phi_D$
$\phi_D = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) = \frac{1}{\pi} \times \cos^{-1}\left(\frac{4.18 - 1}{4.18}\right) \times \frac{\pi}{180} = 0.22 = 22\%$
$\phi_{D_1} = 100 - 22 = 78\%$

Correction for C_1

i) **Due to mutual interference of pile 2**

$D = 128.4 - 122 = 6.4$ m
$d = 128.4 - 122 = 6.4$ m
$b' = 16$ m
$b = 51$ m
$C = 19\sqrt{\frac{6.4}{16}}\frac{(6.4 + 6.4)}{51} = 3.02\%$ (Positive)

ii) **Correction due to thickness of floor**

$C = \frac{78 - 67}{129 - 122} \times 0.6 = 0.94\%$ (Positive)

iii) **Correction due to slope is zero**

$\therefore$ Corrected $\phi_{C_1} = 67 + 3.02 + 0.94 = 70.96\%$

For intermediate pile number 2

$d = 129 - 122 = 7$ m
$b_1 = 16$ m
$b_2 = 35$ m
$\alpha_1 = \frac{16}{7} = 2.29$
$\alpha_2 = \frac{35}{7} = 5$
$\lambda = \frac{\sqrt{1 + \alpha_1^2} + \sqrt{1 + \alpha_2^2}}{2} = 3.8$
$\lambda_1 = \frac{\sqrt{1 + \alpha_1^2} - \sqrt{1 + \alpha_2^2}}{2} = -1.3$
$\phi_{E_2} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right) = 0.71 = 71\%$

$$\phi_{D_2} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1}{\lambda}\right) = 0.61 = 61\%$$

$$\phi_{C_2} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right) = 0.53 = 53\%$$

Correction for ϕ_{E_2}

i) **Correction at E_2 due to mutual interference of pile 1**

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

$D = 128.4 - 122 = 6.4$ m

$d = 128.4 - 122 = 6.4$ m

$b' = 16$ m

$b = 51$ m

$\therefore$ $C = 3.02\%$ (Negative)

ii) **Correction due to floor thickness**

$$C = \frac{71-61}{129-122} \times 0.6 = 0.86\% \text{ (Negative)}$$

iii) **Slope correction is nil**

$\therefore$ Corrected $\phi_{E_2} = 71 - 3.02 - 0.86 = 67.12\%$

Correction for ϕ_{C_2}

i) **Mutual interference due to pile 3**

$D = 128.4 - 120 = 8.4$ m

$d = 128.4 - 122 = 6.4$ m

$b' = 35$ m

$b = 51$ m

$\therefore$ $C = 2.7\%$ (Positive)

ii) **Due to floor thickness**

$$C = \frac{61-53}{129-122} \times 0.6 = 0.69\% \text{ (Positive)}$$

iii) **Correction due to slope**

$$C = C_S \times \frac{b_S}{b_1}$$

$b_S = 1.75 \times 4 = 7$ m

$b_1 = 35$ m

$C_S = 3.3$ for 4 : 1 referring table of slope correction factor

$C = 0.66\%$ (Negative for upward slope)

$\therefore$ Corrected $\phi_{C_2} = 53 + 2.7 + 0.69 - 0.66 = 55.73\%$

For d/s pile number 3

$b = 51$ m

$d = 127 - 120 = 7$ m

$\alpha = \frac{b}{d} = 7.29$ m

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = 4.18$$

$$\phi_{E_3} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) \times \frac{\pi}{180} = 0.33 = 33\%$$

$$\phi_{D_3} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) = 0.22 = 22\%$$

Correction for ϕ_{E_3}

i) **Mutual interference due to pile 2**

$D = 126.4 - 122 = 4.4$ m

$d = 126.4 - 120 = 6.4$ m

$b = 51$ m

$b' = 35$ m

$C = 3.34\%$ (Negative)

ii) **Thickness correction**

$$C = \frac{33-22}{127-120} \times 0.6 = 0.94\% \text{ (Negative)}$$

iii) **Slope correction is nil**

$\therefore$ Corrected $\phi_{E_3} = 33 - 3.34 - 0.94 = 28.72\%$

The corrected % pressures at key points are tabulated as under:

u/s pile 1	Intermediate pile 2	d/s pile 3
$\phi_{E_1} = 100\%$	$\phi_{E_2} = 67.12\%$	$\phi_{E_3} = 28.72\%$
$\phi_{D_1} = 78\%$	$\phi_{D_2} = 61\%$	$\phi_{D_3} = 22\%$
$\phi_{C_1} = 70.96\%$	$\phi_{C_2} = 55.73\%$	$\phi_{C_3} = 0\%$

Diff. in head between u/s water level and d/s floor level $= 134 - 127$
$= 7$ m

Uplift pressure @$C_1 = 0.7096 \times 7 = 4.97$ m

@$D_1 = 0.78 \times 7 = 5.46$ m

@$E_2 = 0.6712 \times 7 = 4.7$ m

@$D_2 = 0.61 \times 7 = 4.27$ m

@$C_2 = 0.5573 \times 7 = 3.9$ m

@$E_3 = 0.2872 \times 7 = 2.01$ m

@$D_3 = 0.22 \times 7 = 1.54$ m

Uplift pressure at 'A' (h) $= 2.01 + \frac{3.9 - 2.01}{35} \times (35 - 7 - 2 - 18.75)$

Assuming;

Crest width = 2 m

h = 2.40 m

Thickness required at A $= \frac{h}{G_C - 1} = \frac{2.4}{2.24 - 1}$

$= 1.94$ m > 1 m (Unsafe)

Exit gradient

We have,

Exit gradient $(G_E) = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$

H = Maximum seepage head = 134 − 127 = 7 m

d = Depth of d/s cutoff = 127 − 120 = 7 m

$\alpha = \frac{b}{d} = \frac{51}{7} = 7.29$ m

$\lambda = \frac{1 + \sqrt{1+\alpha^2}}{2} = 4.18$

$G_E = \frac{7}{7} \times \frac{1}{\pi\sqrt{4.18}} = 0.16 < \frac{1}{5}$ (Safe)

The exit gradient is equal to 0.16 *i.e.*, 1 in 6.42, which is safe.

Hence, given weir profile is safe against piping and unsafe against uplift.

PROBLEM 12

Find whether the section provided is safe against uplift at A and B. **[2071 Magh]**

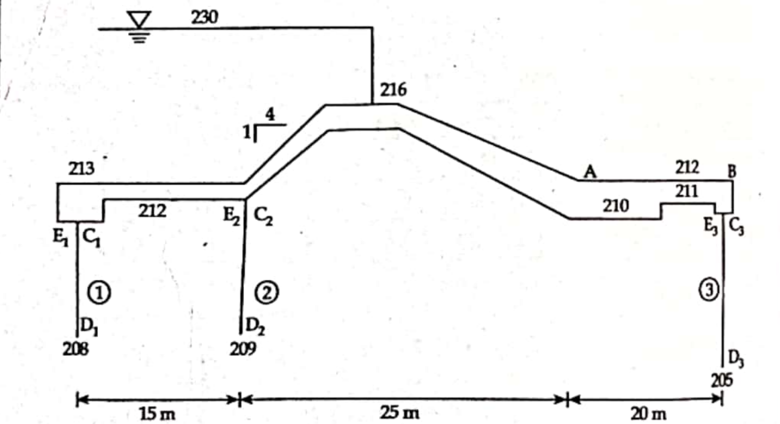


Figure 5.43

Solution:

For u/s pile no. 1

b = 60 m

d = 213 − 208 = 5 m

$\alpha = \frac{b}{d} = \frac{60}{5} = 12$

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

$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = 0.256 = 25.6\%$

$\phi_{C_1} = 100 - \phi_E = 100 - 25.6 = 74.4\%$

$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = 0.178 = 17.8\%$

$\therefore \quad \phi_{D_1} = 100 - \phi_D = 82.2\%$

Correction for ϕ_{C_1}

i) **Due to mutual interference of pile 2**

D = 212 − 208 = 4 m

d = 212 − 209 = 3 m

b′ = 15 m

b = 60 m

$C = 19\sqrt{\frac{4}{15}}\left(\frac{3+4}{60}\right) = 1.145\%$ (Positive)

ii) **Thickness correction**

$C = \frac{\phi_{D_1} - \phi_{C_1}}{213 - 208} \times 1 = 1.56\%$ (Positive)

$\therefore \quad$ Corrected $\phi_{C_1} = 74.4 + 1.145 + 1.56 = 77.105\%$

For intermediate pile no. 2

d = 213 − 209 = 4 m

$b_1 = 15$ m

$b_2 = 45$ m

$\alpha_1 = \frac{b_1}{d} = \frac{15}{4} = 3.75$

$\alpha_2 = \frac{b_1}{d} = \frac{45}{4} = 11.25$

$\lambda = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2} = 7.588$

$\lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2} = -3.707$

$\phi_{E_2} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right) = 71.3\%$

$\phi_{D_2} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right) = 66.2\%$

$\phi_{C_2} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right) = 61.6\%$

Correction for ϕ_{E_2}

i) **Due to mutual interference of pile 1**

D = 212 − 209 = 3 m

d = 212 − 208 = 4 m

b′ = 15 m

b = 60 m

$\therefore \quad C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = 0.991\%$ (Negative)

ii) **Thickness correction**

$C = \frac{71.3 - 66.2}{213 - 209} \times 1 = 1.275\%$ (Negative)

$\therefore$ Corrected $\phi_{E_2} = 71.3 - 0.991 - 1.275 = 69.034\%$

Correction for ϕ_{C_2}

i) Due to mutual interference of pile 2

$D = 212 - 209 = 3$ m

$d = 212 - 205 = 7$ m

$b' = 45$ m

$b = 60$ m

$\therefore$ $C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = 0.82\%$ (Positive)

ii) Thickness correction

$C = \frac{66.2 - 61.6}{213 - 209} \times 1 = 1.15\%$ (Positive)

iii) Slope correction

$C = C_s \times \frac{b_s}{b_1}$

$b_s = 4 \times 3 = 12$ m

$b_1 = 45$ m

$\therefore$ $C = 3.3 \times \frac{12}{45} = 0.88\%$ (Negative)

$\therefore$ Corrected $\phi_{C_2} = 61.6 + 0.82 + 1.15 - 0.88 = 62.69\%$

For d/s pile 3

$b = 60$ m

$d = 212 - 205 = 7$ m

$\alpha = \frac{b}{d} = 8.571$

$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = 4.815$

$\phi_{E_3} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) = 30.1\%$

$\phi_{D_3} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) = 20.9\%$

Correction for ϕ_{E_3}

i) Mutual interference due to pile 2

$D = 211 - 205 = 6$ m

$d = 211 - 209 = 2$ m

$b' = 45$ m

$b = 60$ m

$C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = 0.925\%$ (Negative)

ii) Thickness correction

$C = \frac{30.1 - 20.9}{212 - 205} \times 1 = 1.314\%$ (Negative)

$\therefore$ Corrected $\phi_{E_3} = 30.1 - 0.925 - 1.314 = 27.861\%$

The corrected % pressure at key points are tabulated as under:

u/s pile 1	Intermediate pile 2	d/s pile 3
$\phi_{E_1} = 100\%$	$\phi_{E_2} = 69.034\%$	$\phi_{E_3} = 27.861\%$
$\phi_{D_1} = 82.2\%$	$\phi_{D_2} = 66.2\%$	$\phi_{D_3} = 20.9\%$
$\phi_{C_1} = 77.105\%$	$\phi_{C_2} = 62.69\%$	$\phi_{C_3} = 0\%$

Maximum percolation head = 230 − 212 = 18 m

$\therefore$ Uplift pressure @$E_1 = 1 \times 18 = 18$ m

@$D_1 = 0.822 \times 18 = 14.796$ m

@$C_1 = 0.77105 \times 18 = 13.879$ m

@$E_2 = 0.69034 \times 18 = 12.426$ m

@$D_2 = 0.662 \times 18 = 11.916$ m

@$C_2 = 0.6269 \times 18 = 11.284$ m

@$E_3 = 0.27861 \times 18 = 5.015$ m

@$D_3 = 0.209 \times 18 = 3.765$ m

@$C_3 = 0 \times 18 = 0$ m

Uplift pressure at 'A' (h) $= 5.015 + \frac{11.284 - 5.015}{45} \times 20 = 7.801$ m

Thickness required at 'A' $= \frac{h}{G_C - 1} = \frac{7.801}{2.24 - 1}$

$= 6.291$ m > 2

Hence, unsafe.

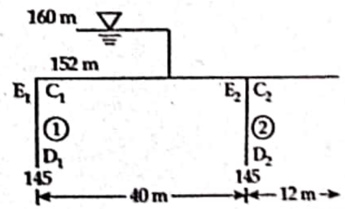
Uplift pressure at B (h) = 5.015 m = @E_3

Thickness required at B $= \frac{5.015}{2.24 - 1} = 4.044 > 1$ m

Hence, unsafe.

PROBLEM 13

Calculate % of uplift pressure at C_1 and E_2 with necessary corrections using Khosla theory. Assume thickness of floor is 1 m. **[P.U. 2014, Fall]**



Solution:

1. At C_1

$b = 52$ m

$d = 152 - 145 = 7$

$\alpha = \frac{b}{d} = 7.429$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 4.248$$

$$\phi_D = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$$

$\therefore \quad \phi_{D_1} = 100 - \phi_D = 22.3\%$

$$\phi_E = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = 32.2\%$$

$\therefore \quad \phi_{C_1} = 100 - \phi_E = 100 - 32.2 = 67.8\%$

Correction for ϕ_{C_1}

i) **Mutual interference due to pile 2**

Assume lower level of floor = 152 – Thickness (1 m) = 151 m

$D = 151 - 145 = 6$ m

$d = 151 - 145 = 6$ m

$b' = 40$ m

$b = 52$ m

$\therefore \quad C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = 1.698\%(+ve)$

ii) **Thickness correction**

$$C = \frac{(\phi_{D_1} - \phi_{C_1})}{152-145} \times 1 = 1.414\%(+ve)$$

$\therefore \quad$ Corrected $\phi_{C_1} = 67.8 + 1.698 + 1.414 = 70.912\%$

2. **At E_2**

$d = 152 - 145 = 7$ m

$b = 52$ m

$b_1 = 40$ m

$b_2 = 12$ m

$$\alpha_1 = \frac{b_1}{d} = 5.714$$

$$\alpha_2 = \frac{b_2}{d} = 1.714$$

$$\lambda = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2} = 3.893$$

$$\lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2} = 1.908$$

$$\phi_{E_2} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1-1}{\lambda}\right) = 42.5\%$$

$$\phi_{D_2} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda_1}{\lambda}\right) = 33.7\%$$

Correction for ϕ_{E_2}

i) **Mutual interference due to pile 1**

$D = 151 - 145 = 6$ m

$d = 151 - 145 = 6$ m

$b' = 40$ m

$b = 52$ m

$\therefore \quad C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = 1.698\%(-ve)$

ii) **Thickness correction**

$$C = \frac{(\phi_{E_2} - \phi_{D_2})}{152-145} \times 1 = \frac{(42.5-33.7)}{152-145} \times 1 = 1.257\%(-ve)$$

$\therefore \quad$ Corrected $\phi_{E_2} = 42.5 - 1.698 - 1.257$

$= 39.545\%$

PROBLEM 14

What are silt controlling structures? Explain any one with neat sketches.

Solution:

The structures made for the purpose of trapping silt present in the river and canal water to reduce sediment load on canal are called silt controlling structures. They are:

i) Silt excluder

ii) Silt extractor or ejector

See the definition part 5.1.6 and 5.1.7

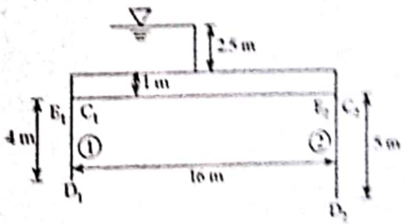
PROBLEM 15

An impervious floor of a weir on permeable soil is 16 m long and has sheet pile at both the ends. The u/s pile is 4 m deep and the d/s pile is 5 m deep. The weir creates a net head of 2.5 m neglecting the thickness of the weir floor; calculate the uplift pressure at the junction of the inner faces of the pile with the weir floor, by using Khosla's theory. [2071 Bhadra: T.U.]

Solution:

Assume;

Floor thickness = 1 m



For u/s pile 1

$d = 5$ m

$b = 16$ m

$$\alpha = \frac{b}{d} = 3.2$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 2.176$$

$$\phi_E = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = 47.4\%$$

$\therefore \quad \phi_{C_1} = 100 - \phi_E = 100 - 47.4 = 52.6\%$

and, $\phi_D = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = 31.8\%$

$\therefore \quad \phi_{D_1} = 100 - \phi_D = 100 - 31.8 = 68.2\%$

Correction for ϕ_{C_1}

i) **Mutual interference due to pile 2**

$D = 4$ m

$d = 5$ m

$b' = 16$ m

$b = 16$ m

$$\therefore \quad C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = 19\sqrt{\frac{4}{16}}\left(\frac{4+5}{16}\right) = 5.344\%(+ve)$$

ii) **Thickness correction**

$$C = \frac{(\phi_{D_1} - \phi_{C_1})}{5} \times 1 = \frac{(68.2 - 52.6)}{5} \times 1 = 3.12\%(+ve)$$

$\therefore$ Corrected $\phi_{C_1} = 52.6 + 5.344 + 3.12 = 44.136\%$

For d/s pile 2

$d = 6$ m

$b = 16$ m

$\alpha = 2.667$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+(2.667)^2}}{2} = 1.924$$

$$\phi_{E_3} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = \frac{1}{\pi} \times \cos^{-1}\left(\frac{1.924-2}{1.924}\right) = 51.3\%$$

$$\phi_{D_2} = \frac{1}{\pi} \times \cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = \frac{1}{\pi} \times \cos^{-1}\left(\frac{1.924-1}{1.924}\right) = 34.1\%$$

Correction for ϕ_{E_3}

i) **Mutual interference due to pile 1**

$D = 5$ m

$d = 4$ m

$b' = 16$ m

$b = 16$ m

$$\therefore \quad C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = 19\sqrt{\frac{5}{16}}\left(\frac{4+5}{16}\right) = 5.344\%(-ve)$$

ii) **Thickness correction**

$$C = \frac{(\phi_{E_3} - \phi_{D_2})}{5} \times 1 = \frac{(51.3 - 34.1)}{5} \times 1 = 3.44\%(-ve)$$

$\therefore$ Corrected $\phi_{E_2} = 51.3 - 5.344 - 3.44 = 42.516\%$

We have,

Maximum percolation head = 2.5 m

$\therefore$ Uplift pressures @ $C_1 = 0.44136 \times 2.5 = 1.103$ m

@ $D_1 = 0.682 \times 2.5 = 1.705$ m

@ $E_2 = 0.513 \times 2.5 = 1.283$ m

@ $C_1 = 0.341 \times 2.5 = 0.853$ m

PROBLEM 16

What is hydraulic structure? What are the basic principles of designing hydraulic structure? State the difference between Bligh's creep theory and Lanes weighted creep theory. [2071 Bhadra: T.U.]

Solution:

Hydraulic structure

Hydraulic structures are those, constructed for the purpose of either storage or diversion or both. Some of the important hydraulic structures are as follows;

i) Weir or barrage [Figure 5.2, 5.3, 5.7, 5.8]
ii) Dams
iii) Head regulator [Figure 5.13]
iv) Cross regulator
v) Cross drainage works
vi) Silt controlling devices (silt excluder, extractor), etc.

The basic principles of designing hydraulic structures are as follows:

i) Bligh's theory
ii) Lane's weighted creep theory
iii) Khosla's theory

Differences between Bligh's and Lane's theory

	Bligh's creep theory	Lane's weighted creep theory
i)	It makes no distinction between horizontal and vertical creep *i.e.,* $L = N + V$ where, N is the horizontal creep and V is the vertical creep.	It assumes that vertical creep is 3 times more effective in reducing uplift than horizontal creep *i.e.,* $L = \frac{1}{3}N + V$.
ii)	Bligh's coefficient (C) is distinguished for four types of soil.	Lane's coefficient C_1 is distinguished for six types of soil.
iii)	It is widely used for design of hydraulic structures.	Being empirical and lacking rational analysis Lane's theory is limited theoretically.

PROBLEM 17

An irrigation barrage has to be designed to pass a flood of 10000 m³/s, through alluvium media (median diameter of particles = 0.33 mm). The flood level, pond level and d/s floor level are 207.0 m, 204 m and 198.0 m respectively. If the safe exit gradient is $\frac{1}{6}$, compute minimum total impervious floor length required to safe guard the structure from piping. Prepare a conceptual section of the designed structure. [2072 Ashwin]

Solution:

Given that,

$Q = 10000\ m^3/sec.$

$d = 0.33$

$\therefore \quad f = 1.76\sqrt{d} = 1.76\sqrt{0.33} = 1.011$

Overall waterway,

$$P = 4.75\sqrt{Q} = 4.75\sqrt{10000} = 475 \text{ m}$$

$$\text{Discharge intensity, } q = \frac{Q}{P} = \frac{10000}{475} = 21.053 \text{ cumec/m}$$

$$\text{Scour depth (R)} = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35\left(\frac{(21.053)^2}{1.011}\right)^{\frac{1}{3}} = 10.26 \text{ m}$$

Assume, afflux = 1 m

Here,

u/s HFL = 207 m

$\therefore$ d/s HFL = u/s HFL − Afflux

Provide d/s cutoff at 1.5 R from d/s HFL.

$\therefore$ R.L. of d/s cutoff $= 206 - 1.5 \times 10.26 = 190.61$ m

Now,

Safe exit gradient $(G_E) = \frac{1}{6}$

Maximum static head (H) $= 204 - 198 = 6$ m

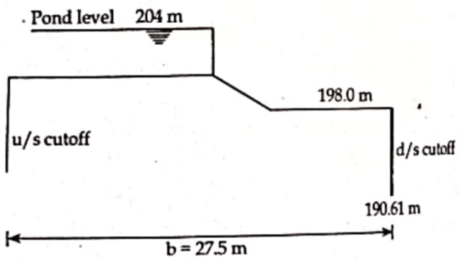
Depth of d/s cutoff (d) $= 198 - 190.61 = 7.39$ m

We have,

$$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$$

or, $\frac{1}{6} = \frac{6}{7.39}\frac{1}{\pi\sqrt{\lambda}}$

$\therefore$ $\lambda = 2.40$



Also,

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

or, $2.4 = \frac{1+\sqrt{1+\alpha^2}}{2}$

$\therefore$ $\alpha = 3.66$

Also,

$$\alpha = \frac{b}{d}$$

or, $3.66 = \frac{b}{7.39}$

$\therefore$ b = 27.09 m say 27.5 m

$\therefore$ Provide length of impervious floor of 27.5 m.

PROBLEM 18

What are silt ejectors and silt excluders in irrigation system? Write their design principles. [2072 Ashwin]

Solution: See the definition part 5.1.6, 5.1.7, 5.4.2 and 5.5

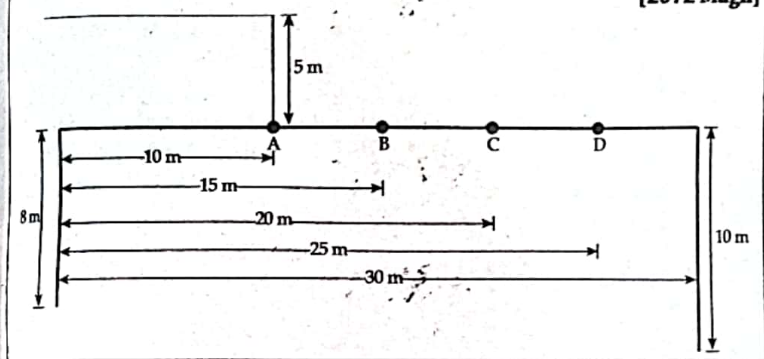
PROBLEM 19

Drawing a neat sketch of an irrigation headwork, draw a longitudinal section through head regulator showing u/s floor, regulator gates; energy dissipaters and protection works. [2072 Magh]

Solution: *See figure no. 5.1 and 5.39*

PROBLEM 20

A section of a hydraulic structure is shown in figure below, calculate the average hydraulic gradient. Also find the uplift pressures at point A, B, C and D. Find the thickness of the floor at these points. Take G = 2.24. [2072 Magh]



Solution: Proceed same as example 5.2

PROBLEM 21

A river carries a high flood discharge of 16000 m³/sec. with its average bed level at 200.0 m. A canal carrying 200 m³/sec. is to take off from the headworks. The full supply level of the canal at its head is 203.0 m. The high flood level before construction is 205.7 m and Lacey's silt factor is equal to unity. Fix suitable values for the waterway and crest levels of weir, undersluices and canal head regulator. Assume suitably any other data if required. [2073 Bhadra]

Solution:

I) **Waterway**

From lacey's formula; we have,

$$P = 4.75\sqrt{Q} = 4.75\sqrt{1600} = 600.83 \text{ m}$$

Let us fix the waterway as follows:

Undersluice portion

Number of spans = 6

6 bays of 16 m each = 96 m

5 piers of 2.5 m each = 12.5 m

Total = 108.5 m

Weir bay portion

Number of spans = 30

30 bays of 12 m each = 360 m

29 piers of 2 m each = 58 m

Total = 418

Width of fish ladder = 5 m

Width of divide wall = 3 m

Overall water way between abutments = 108.5 + 418 + 5 + 3

= 534.5 m

< 600.83 m

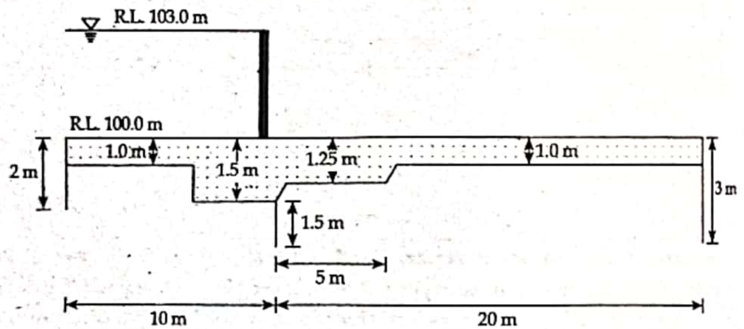
ii) Crest level of undersluice = Lowest water level given

= 200.00 m

Crest level of weir = 200 + 1.5 = 201.5 m

PROBLEM 22

Calculate the uplift pressure at key points of the pile of the structure shown in the figure below. Draw HGL and also check the thickness provided and safe exit gradient $G_E = \frac{1}{5}$. [2073 Bhadra]



Solution: See the solution of Q. no. 2

PROBLEM 23

Explain the design method to find the suitable size, length and thickness of floor of barrage. Using Khosla's seepage theory. Also draw the typical section of barrage showing the different component. [2073 Magh]

Solution: See the definition part 5.2.3.1 and 5.1.1

PROBLEM 24

What are the ways of controlling entry of sediments into canal from headworks? Differentiate between silt ejector and silt excluder in irrigation system? [2074 Bhadra]

Solution:

Silt excluder and silt ejector are the way of controlling entry of sediments in canals.

For differences between silt excluder and silt ejector

See the definition part 5.5

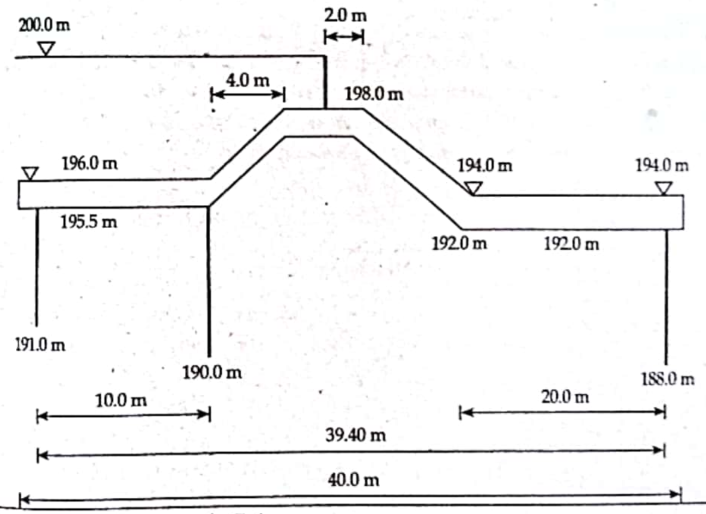
PROBLEM 25

A canal carrying 150 m^3/sec. is to take off from the headwork. The HFL and average bed level of river is 257 m and 250 m respectively. The canal bed level, full supply level and pond level are 249.5 m, 253.0 m, 254.0 m respectively and Lacey's silt factor is equal to unity. Fix the crest level and waterway of canal head regulator and also determine the length of impervious floor if safe exit gradient $G_E = \frac{1}{6}$. Draw conceptual sketch of canal head regulator. [2074 Bhadra]

Solution: See the solution of Q. no. 21

PROBLEM 26

Using Khosla's theory, find the seepage pressure head at the key points of the diversion head works section as shown in the figure below. Check the sufficiency of the thickness of concrete (S.G. 2.4) provided at the mid section of downstream launching apron of 20 m long. Take slope correction factor for slope 2 : 1 (H : V) = 6.5 [P.U. 2014]



Solution: See the example 5.4

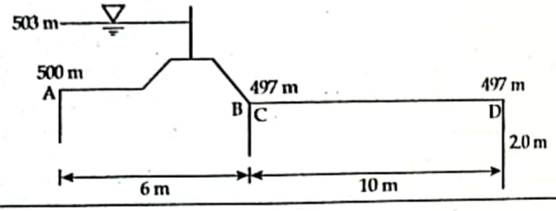
PROBLEM 27

Compare Bligh's creep theory and Lane's weighted creep theory. Explain how the safety of any hydraulic structures against uplift can be ensured using Bligh's theory. [P.U. 2017]

Solution: See the solution of Q. no. 16 and definition part 5.2.1

PROBLEM 28

For the figure shown below ignoring floor thickness and slope corrections, find the percentage pressure at the key points of the piles using Khosla's theory. The floor thickness may not be less than 30 cm anywhere. If permissible exit gradient is 0.15, check the floor against piping failure. Also, find the thickness of floor at A, B, C and D points. [2076 Baishakh]



Solution:

Proceed same as solution of problem 1, using only correction for mutual interference of piles

PROBLEM 29

Describe briefly the design step of silt excluder with net sketches. [2076 Bhadra]

Solution: See the definition part 5.1.6 and 5.4.2

PROBLEM 30

A weir has a solid horizontal floor length of 50 m with two lines of cut off 8 m depth below the river bed at two ends. The floor thickness is 1 m at u/s end and 2 m at d/s end, with the upper level being in flush with river bed for an effective head of 5 m over the weir, calculate the uplift pressure at the two inside corner points and also the exit gradient. [2076 Bhadra]

Solution:

Precede same as solution of problem 15 for two inside corner points

For exit gradient, we have,

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\lambda}$$

Here;

$H = 5\ m$

$d = 8\ m$

$\alpha = \frac{b}{d} = \frac{16}{8} = 2$

$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 1.618$

so, $G_E = \frac{5}{8} \times \frac{1}{\pi\sqrt{1.618}} = 0.156$

PROBLEM 31

How do you fix the design discharge of undersluices? [2076 Bhadra]

Solution:

The design discharge of undersluices is fixed in such a way that.

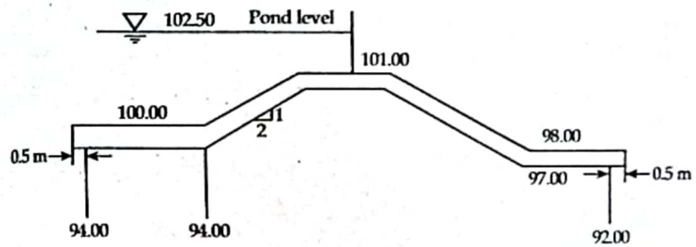
i) They should be able to ensure sufficient scouring capacity, for which the discharging capacity should be at least twice the full supply discharge of the main canal at it head.

ii) They should be able to pass the dry weather flow and low floods during the months excluding the rainy season, without necessity of dropping the weir structures.

iii) They should be able to dispose of 10 to 15% of the high flood discharge during severe floods.

PROBLEM 32

Using Khosla's method, calculate the uplift pressures at various key points in the figure below. Also, determine the exit gradient. Take the slope correction factor as 6.5. [2077 Chaitra]



Solution: See the solution of problem no. 10

PROBLEM 33

An irrigation barrage has to be designed to pass a flood of 11000 m³/sec., through alluvium media (median diameter of particles = 0.34 mm). The flood level, pond level and d/s floor level as : 208 m, 205 m and 199 m respectively. If the safe exit gradient is $\frac{1}{7}$, compute minimum total impervious floor length required to safe guard the structure from piping. Prepare a conceptual section of the designed structure. [2078 Baishakh]

Solution: Proceed same as the solution of problem no. 17

PROBLEM 33

Write down step by step design procedure of head regulator. [2078 Baishakh]

Solution: See the definition part 7.3

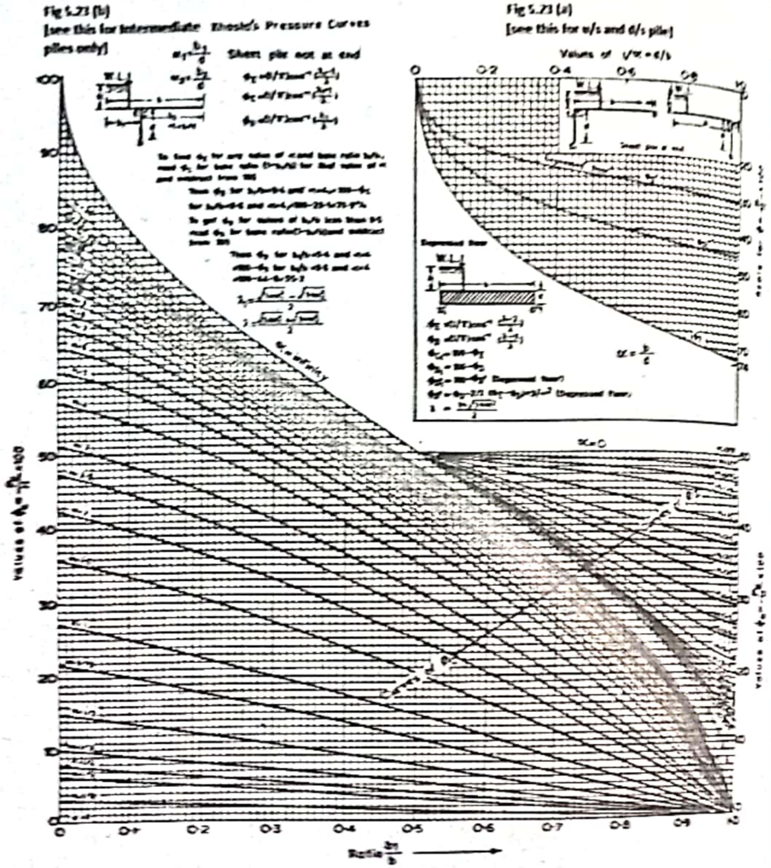


Figure 5.23: Khosla pressure curves

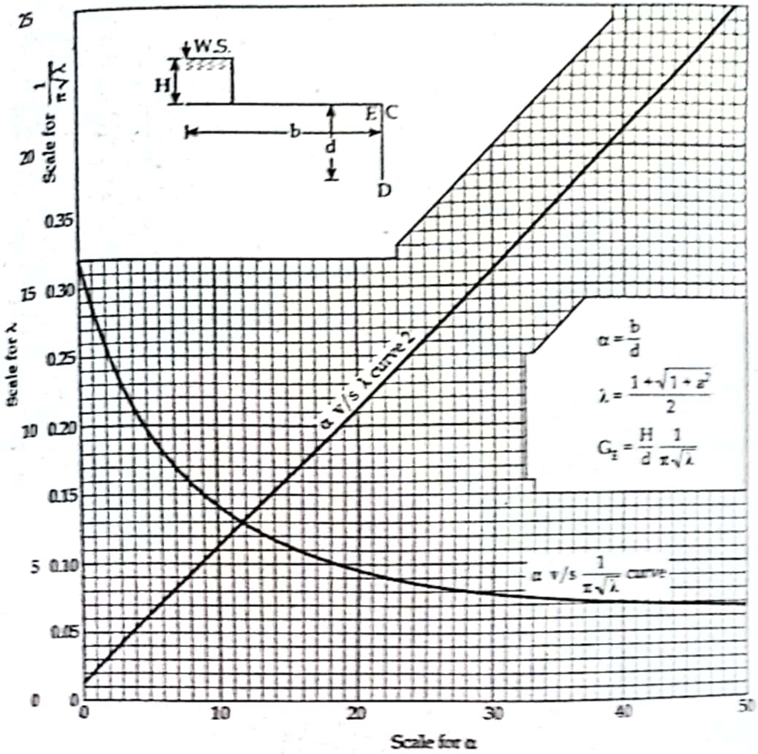


Figure 5.25: Exit gradient curve

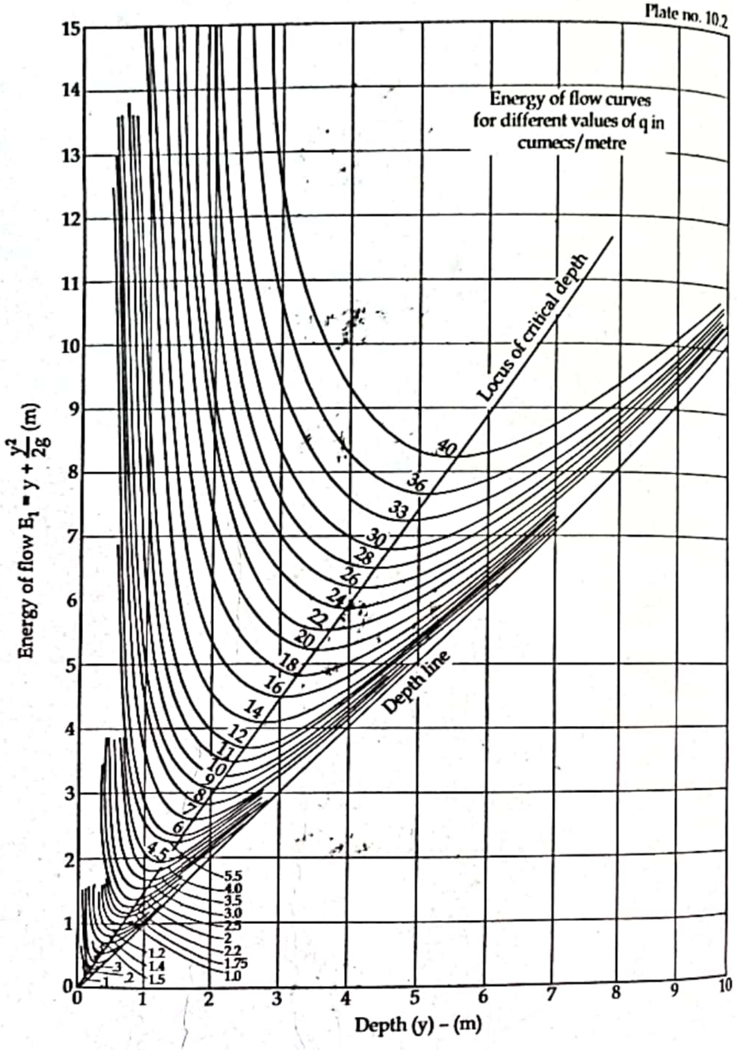


Figure 5.27:

5.7 OBJECTIVE QUESTION

1. A canal headworks has nothing to do with a ....
   a) weir b) guide bank
   c) head regulator d) safety ladder
2. In a diversion headworks project, the canal head regulator is usually aligned ....
   a) parallel to the barrage axis b) perpendicular to the divide wall
   c) parallel to the divide wall d) none of the above
3. The silt exclusion device, constructed on the bed of the main canal, taking off from a headwork is called ....
   a) silt excluder b) silt ejector
   c) both (a) and (b) d) none of the above
4. The safety of a hydraulic structure founded on pervious foundation can be ensured ....
   a) by providing sufficient length of its concrete floor
   b) by providing sufficient depth of its concrete floor
   c) by providing a downstream cutoff of some reasonable depth
   d) all of the above
5. Bligh's theory as applied to the design of weirs and barrages on permeable foundations, account for ....
   a) hydrostatic forces only b) hydrodynamic forces only
   c) both (a) and (b) d) none of the above
6. Khosla's safe exit gradient for design of weirs will be the lowest for the soil type ....
   a) fine sand b) coarse sand
   c) shingle and gravels d) none of the above
7. The critical exit gradient suggested in Khosla's theory of design of weirs and barrages is ....
   a) less for more porous soils b) more for more porous soils
   c) equal for all kinds of soils d) none of the above
8. According to Khosla's theory of independent variables for seepage below a hydraulic structure, the exit gradient, in the absence of a d/s sheet pile is ....
   a) 0 b) 1
   c) ∞ d) none of the above
9. The back water effect of a weir is best called ....
   a) retrogression b) afflux
   c) back water curve d) none of the above
10. A breast wall is usually provided ....
    a) in the weir section b) in the under-sluice section
    c) in the main canal section d) in the head regulator section
11. The safety against any possible scour, on u/s or d/s side of the pucca floor of a hydraulic structure is usually ensured by laying ....
    a) inverted filter b) toewall
    c) rock toe d) stone apron

12. The minimum thickness (t) of the downstream floor, as required in the design of weirs, can be expressed by the equation ....
    a) $\frac{h}{a+1}$ b) $\frac{h}{a-1}$ c) $\frac{h-t}{a-1}$ d) $1.33\left(\frac{h}{a-1}\right)$
13. While designing the floor thickness of a weir in its u/s length, the average head causing uplift is 2.8 m. The provided floor thickness here, with suitable factor of safety, could best be ....
    a) 2 m b) 2.5 m
    c) 0.8 m d) 4.5 m
14. The hydraulic jump that develops usually in barrages and canal head regulators is of the type ....
    a) strong jump b) steady jump
    c) oscillating and weak jump d) undular jump
15. Silt excluders are constructed ....
    a) on the river bed d/s of the head regulator
    b) on the river bed u/s of the head regulator
    c) on the canal bed d/s of the canal head regulator
    d) none of the above
16. Barrages constructed across alluvial rivers help in ....
    a) controlling floods b) restoring river regime
    c) ensuring monsoon storage d) all of them
17. The tunnel openings provided in front of a canal head regulator at a diversion headworks ....
    a) discharge sediment water into canal
    b) discharge sediment load into the undersluices, from where it ejects ... to the d/s river
    c) discharge clear water into the canal
    d) none of the above
18. A hydraulic jump involves ....
    a) sub-critical flow b) super critical flow
    c) critical flow d) all of them
19. Retrogression is ....
    a) the backwater effect of a weir
    b) the raising of the river bed u/s of the weir, during initial years of its construction
    c) the lowering of the river bed d/s of the weir, during initial years of its construction d) none of them
20. The phenomenon of hydraulic jump leads to ....
    a) evolution of energy b) dissipation of energy
    c) sometimes (a) and sometimes (b)
    d) none of them

Answer sheet

1	2	3	4	5	6	7	8	9	10
d	b	b	d	a	a	a	c	b	d
11	12	12	14	15	16	17	18	19	20
d	b	c	c	b	b	b	d	c	d

CHAPTER 6

RIVER TRAINING WORKS

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6.1 RIVER STAGES AND NEED OF RIVER TRAINING

6.1.1 River stages

As the river flow from its origin in a mountain to sea, it passes through various stages. A river generally has the following four stages.

I) Rocky stage (Incised stage)

It is first stage when the river takes off from mountain and hilly region. Flow channel are formed by degradation and cutting. Bed and bank consists of rocky strata. River reaches are highly steep. It is ideal for construction of dam.

II) Boulder stage

It is the second stage of the river. The river passes from rocky to boulder stage. Bed and bank in this stage consists of large boulders, gravels and shingles. River cross section is well defined and velocity is high but less than in straight course.

III) Trough and alluvial stage

The river in this stage flows in a zigzag manner known as the meandering. Cross-section is made up of sand and silt. Bed slope is flat and velocity is small in this stage. The river may be aggrading, degrading or stable type.

Iv) Deltaic stage

It is the last stage of river just before discharging into the sea. In this stage the river gets divided into no. of small branches and forms a delta. As the river approaches the sea the channel gets silted up and water level rises. There are no irrigation works in deltaic stage.

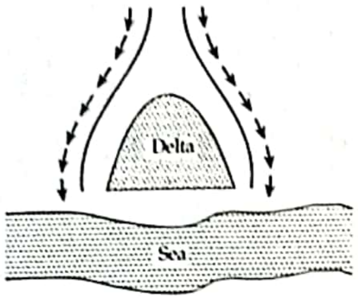


Figure 6.1: Delta

Need of river training

The importance of river training is as follows;

i) To provide a safe passage to pass flood without overflowing its banks and thus to prevent flooding of the adjoining areas.
ii) To prevent river from changing its course and eroding adjoining land.
iii) To prevent the erosion of banks and hence improve the alignment by stabilizing the river channel.
iv) To keep river transports the bed load and suspended load efficiently.
v) To provide minimum desired depth of flow required for navigation.
vi) To deflect the river flow away from a bank or to attract the river flow towards a bank.
vii) To provide a favorable curvature of flow at the weir to control entry of silt into the canal.
viii) To control and regulate the river bed configuration.

6.2 CLASSIFICATION OF RIVER TRAINING WORKS

High water training works (training for discharge)

High water training works are constructed for the purpose of quick disposal of maximum flood and to protect adjoining land against damage due to floods. It aims to provide sufficient river cross section for sage passage of maximum flood.

Low water training works (training for depth)

It is undertaken with the primary purpose of providing sufficient water depth for navigation during low flow period. Generally, spurs are constructed to contract the width of the channel and hence to increase the depth.

Mean water training works (training for sediment)

It is undertaken with the primary purpose of providing efficient disposal of sediment load and bed load and thus to preserve the channel in good shape. It is the most important training of the three types.

6.3 METHODS OF RIVER TRAINING

Following are generally adopted methods of training rivers,

I) Marginal embankments or levees

These are the earthen embankments which are constructed parallel to the river banks at some suitable distance from it. These embankment walls retain

the flood water and prevents from spreading into the nearby towns. The alignment of levees should follow the normal meandering pattern of the river.

Effect of marginal embankments

- Increase the water surface elevation and discharge
- Reduction in water surface slope
- Increase in velocity and scouring action
- Increase in bed levels
- Deposition of silt

II) Guide banks

Guide banks are the earthen embankments constructed for confining the alluvial river flow within a reasonable waterway and guiding it to a hydraulic structure, such as weir, a barrage or a bridge so that there is straight non tortuous approach to it.

Design of guide bank

a) Water way

The waterway is the actual width through which flow takes place at the structure. Generally, the length of clear waterway provided between the guide banks is taken equal to lacey's perimeter. *i.e.*, $P = 4.75\sqrt{Q}$

b) Top level

The top level of guide bank is kept equal to the u/s total energy level plus adequate free board. The u/s total energy level is equal to the high flood level before construction plus afflux and velocity head. Thus,

$$\text{Top level of bank} = \text{HFL before construction} + \text{Afflux} + \frac{V^2}{2g} + \text{Free board}$$

Afflux is rise in water level on the u/s of the structure.

Free board = 1.25 to 1.5 m

c) Length of guide banks

According to Gales the length of u/s guide bank is taken according to the river discharge as shown in table.

Discharge	Length of u/s guide bank	Convergence towards structure
Less than 20,000 cumecs	1.25 L	1 in 20
20,000 – 40,000 cumecs	1.25 L to 1.5 L	1 in 20 to 1 in 40
Greater than 40,000 cumecs	1.5 L	1 in 40

The length of d/s guide banks is recommended as 0.25L for all discharges.

d) Radius of heads

The guide banks are provided with u/s and d/s curve head as shown in the figure.

1. u/s curve head

The main function of u/s curve head is to guide the flow smoothly and axially to the structure and to keep the end spans active. A safe value of radius (R) for u/s curve head is taken as, R = 0.45L [L = width of river].

According to Gale,

R = 250 m for discharge between 7000 to 20000 cumecs
= 580 m for discharge between 40000 to 70000 cumecs

For discharge between 20000 to 40000 cumecs the value for R may be obtained by interpolation. The u/s curve head is extended to subtend an angle of 120° to 145° at its center as shown in figure 6.2.

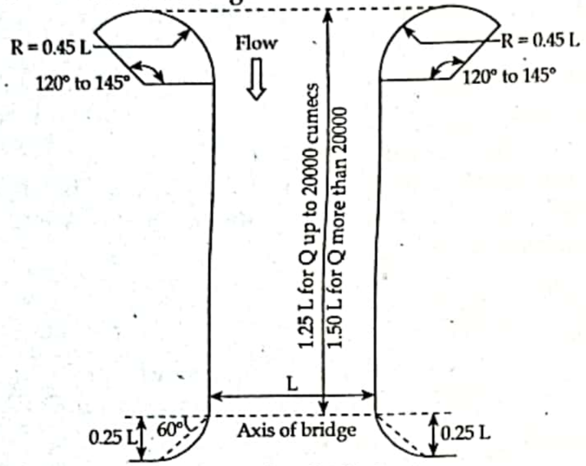


Figure 6.2: Plan of guide bank

2. **d/s curve head**

On the d/s, the river fans out so as to attain its normal width. The radius of d/s head is kept equal to one half of the u/s head radius.

i.e., $R = \frac{0.45\ L}{2} = 0.225\ L$

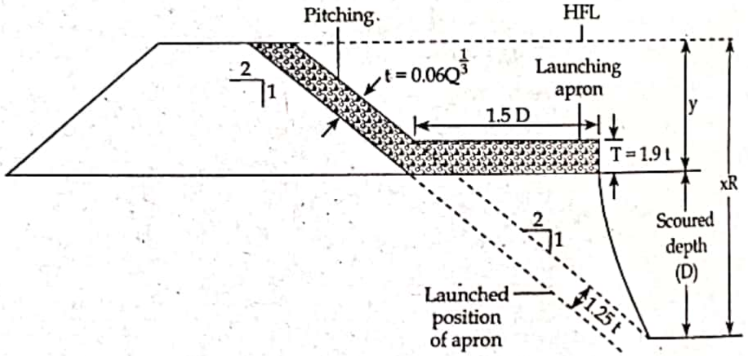


Figure 6.3: Cross-section of guide bank

e) **Cross–section of bund**

The top width of the guide bank should not be less than 4 m. Side slope should not be steeper than 2 : 1. A minimum free board of 1.2 to 1.5 m is generally provided.

f) **Slope protection**

The water face of the guide bank is protected by stone pitching to withstand erosive action of the water currents. The rear side is not pitched and is coated with a layer of earth about 0.3 to 0.6 m thick.

The thickness of pitching is given by;

$$t = 0.06\ Q^{\frac{1}{3}}$$

where, t = Thickness in m.
Q = Maximum discharge (cumecs).

A graded filter is 20 to 30 cm thick is usually provided below the pitching to prevent the failure of slope by sucking action of high velocity flow.

g) **Launching apron**

The stone pitching is extended beyond the toe in the form of packed stone which is called launching apron. If no such protection is provided, scour will scour at the toe with consequent undermining and collapse of the stone pitching. The stone pitching is extended on the horizontal river bed portion to obtain the launching apron. The launching apron is generally laid in a width equal to 1.5 times the scour depth 'D' below the original bed. Total scour depth below HFL is taken as 'xR'; where, 'R' is lacey's normal scoured depth given by,

$$R = 0.47 \left(\frac{Q}{f}\right)^{\frac{1}{3}}$$

where, 'Q' is the discharge and 'f' is silt factor.

S.N.	Location	Mean value of x	D = xR – water depth above bed
1.	Nose of the guide banks	2.25	2.25R – y
2.	Transitions from noses to straight portions	1.5	1.5R – y
3.	Straight reaches of guide banks	1.25	1.25R – y

Generally, a scour slope of 2 : 1 is assumed and the thickness of the launched apron should be 1.25t, where, 't' is the thickness of stone pitching. Then, the volume of stone required in the launched apron per unit length perpendicular to paper is;

$$\sqrt{2^2 + 1^2} \times D\ (1.25t) = 1.25t \times \sqrt{5}\ D$$
$$= 2.8\ tD$$

If the width of un–launched apron is 1.5 D, then the thickness of launched apron (T) is given as;

$$T = \frac{2.8\ t\ D}{1.5\ t\ D}$$
$$= 1.87t \text{ (say 1.9 t)}$$

Hence,

$$T = 1.9\ t$$

where, $t = 0.06\ Q^{\frac{1}{3}}$.

The width of the apron (1.5D) will be different in different portions of guide banks depending upon the value of D.

Example 6.1

Design the guide banks for a bridge site with the following data.
Maximum flood discharge = 10,000 cumecs
HFL = 200.00 m
River bed level = 195.00
Average diameter of the silt particle = 0.3 mm

Solution:

Waterway from lacey's formula,

$P = 4.75\sqrt{Q} = 4.75\sqrt{10000} = 475$ m

Allowing 20% extra for pier thickness, end contractions, etc.

Overall waterway, L = 1.2 × 4.75 = 570 m

Length of guide bank u/s = 1.25L = 712.5 m

Length of guide bank d/s = 0.25L = 142.5 m

Radius of u/s curve head = 0.45L = 256.5 m

Radius of d/s curve head = 0.225L = 128.25 m

Angle of sweep of u/s head = 135

Angle of sweep of d/s head = 60

I) Cross section

Assuming a free board of 1.5 m,

Top level of the guide bank = 200.00 + 1.5 = 201.5 m

Let us assume a top width of 5 m;

Height of guide bank above river bed = 201.5 – 195 = 6.5 m

Let the side slopes be 2 : 1

Thickness of stone pitching, $t = 0.06\, Q^{\frac{1}{3}} = 0.06 \times (10000)^{\frac{1}{3}} = 1.3$ m

Volume of stone pitching per meter length, $v = \sqrt{2^2+1} \times 6.5 \times 1.3$
$= 18.9\ m^3$

II) Launching apron

Lacey's silt factor (f) = $1.76\sqrt{0.3} = 0.96$

Lacey's regime scour depth (R) = $0.47\left(\frac{Q}{f}\right)^{\frac{1}{3}}$

$= 0.47\left(\frac{10000}{0.96}\right)^{\frac{1}{3}} = 10.26$ m

At nose of guide bank;

Depth of scour below river bed (D) = xR – y
= 2.25 × 10.26 – (200 – 195)
= 18 m

Length of launching apron = 1.5D = 1.5 × 18 = 27 m

Thickness of apron at the inner edge = 1.5 t = 1.5 × 1.3
= 1.95 m (say 2 m)

Thickness of apron at the outer edge = 2.25 t = 2.25 × 1.3
= 2.93 m (say 3 m)

At transition of guide bank;

Depth of scour below river bed (D) = xR – y
= 1.5 × 10.26 – 5
= 10.39 m (say 10.4 m)

Length of apron = 1.5D = 1.5 × 10.4 = 15.6m

Thickness of apron is kept the same as that of the nose.

At shank of guide bank;

Depth of scour below river bed = 1.25R – y
= 1.25 × 10.26 – 5
= 7.83 m

Length of apron = 1.5 D = 11.7 5m

Thickness of apron is kept same as that of nose.

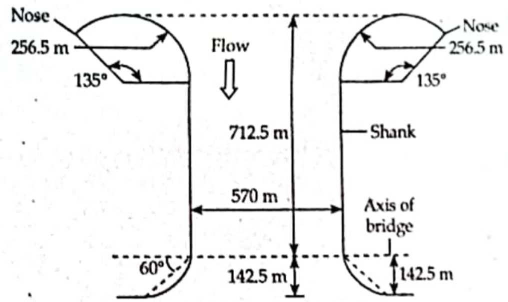


Figure: Plan of guide bank

Example 6.2

The following hydraulic data pertains to a bridge site of a river.
Maximum discharge = 6000 cumecs
Highest flood level = 104.00 m
River bed level = 100 m
Average diameter of river bed material = 0.1 mm
Design and sketch Bell's Bunds including the launching apron to train the river. Assume plentiful availability of boulders near site.

Solution:

I) Waterway

Lacey's regime waterway = Clear waterway = P
$= 4.75\sqrt{Q}$
$= 4.75\sqrt{6000} = 368$ m

Allowing 20% extra for piers etc. the net spacing between the two guide bunds at the bridge site = 1.2 × 368 = 440 m

Hence,

L = 440 m

Length of guide bank u/s = 1.25L = 550 m

Length of guide bank d/s = 0.25L = 110 m
Radius of u/s curve head = 0.45L = 194 m
Radius of d/s curve head = 0.225L = 99 m
Angle of sweep of u/s head = 130 (120 to 140)
Angle of sweep of d/s head = 60

II) Cross section

Assuming a free board of 1.5 m and nil value of afflux, and ignoring velocity head, we have,

Top level of the guide bank = 104.00 +1.5 = 104.5 m (adopt 106 m)

Let us assume;

Top width = 5 m

Height of guide bank above river bed = 106 – 100 = 6 m

Let the side slopes be 2 : 1

We obtain the required section as shown in the figure.

$$\text{Thickness of stone pitching (t)} = 0.06\,Q^{\frac{1}{3}}$$
$$= 0.06 \times (6000)^{\frac{1}{3}}$$
$$= 1.09 \text{ m (say 1.1 m)}$$

$$\text{Volume of stone pitching per meter length (v)} = \sqrt{(2)^2 + 1} \times 6 \times 1.1$$
$$= 14.75 \text{ m}^3$$

III) Launching apron

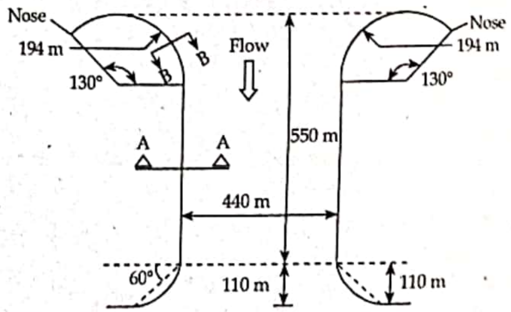


Figure: Plan of designed guide bank

$$\text{Lacey's silt factor (f)} = 1.76\sqrt{0.1} = 0.556$$

$$\text{Lacey's regime scour depth (R)} = 0.47\left(\frac{Q}{f}\right)^{\frac{1}{3}} = 0.47\left(\frac{6000}{0.556}\right)^{\frac{1}{3}}$$
$$= 10.36 \text{ m}$$

At nose of guide bank;

$$\text{Depth of scour below river bed (D)} = xR - y$$
$$= 2.25 \times 10.36 - (104 - 100)$$
$$= 19.36 \text{ m}$$

Length of launching apron = 1.5D = 1.5 × 1936 ≈ 30 m

Thickness of apron at the inner edge = 1.5 t = 1.5 × 1.1 = 1.65 m

Thickness of apron at the outer edge = 2.25 t = 2.25 × 1.1 = 2.475 m

At transition of guide bank;

Depth of scour below river bed (D) = xR – y = 1.5 × 10.36 – 4
= 11.54 m (say 12 m)

Length of apron = 1.5D = 1.5 × 12 = 18 m

Thickness of apron is kept the same as that of the nose.

At shank of guide bank;

Depth of scour below river bed = 1.25R – y = 1.25 × 10.36 – 4 ≈ 9 m

Length of apron = 1.5 D = 13.5 m

Thickness of apron is kept same as that of nose.

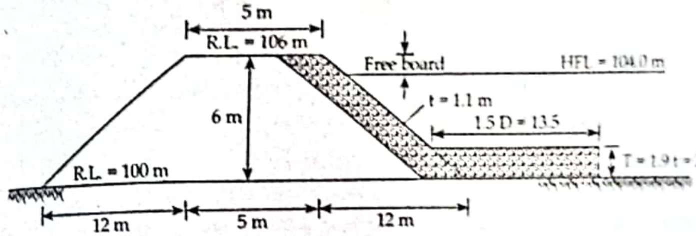


Figure: Cross-section of A-A

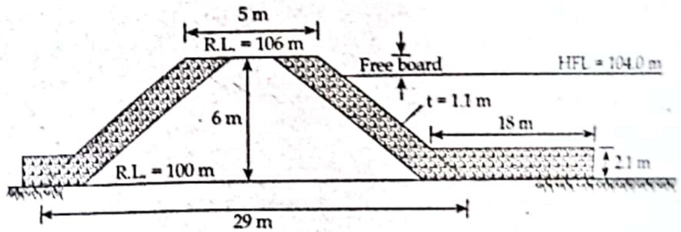


Figure: Cross-section of B-B

III) Groynes or spurs

A spur, spur dyke, or groyne is a structure made to project flow from a river bank into a stream or river with the aim of deflecting the flow away from the side of the river on which the groyne is built. Two to five structures are typically placed in series along straight or convex bank lines where the flow lines are roughly parallel to the bank. Spurs help train a river to flow along a desired course by preventing erosion of the bank and encouraging flow along a channel with a more desirable width and alignment. They are used to control natural meandering at a river bend, to channel wide rivers, and to convert poorly defined streams into well defined channels. The spurs create a zone of slack flow which encourages silting up in the region of the spur to create a natural bank. They generally protect the riparian environment and often improve the pool habitat and physical diversity.

Objectives of groynes

- To train the river along the desired course by attracting of deflecting or repelling the flow in the desired direction.

- To reduce the concentration of flow at a particular point of attack.
- To create a slack zone for silting up the area.
- To protect a bank by keeping the flow away from it.
- To contract a wide river channel for the purpose of increasing its depth for navigation.

Classification of groynes

Classification based on the methods and materials of construction

1. Impermeable groynes

Impermeable groynes do not permit appreciable flow of water through them. They consist of either rock fill, or a core of sand, or sand and gravel, or soil as available in the river bed. Their top and sides are protected by pitching of stone or concrete blocks. They have side slope varying from 2 : 1 to 3 : 1 and head slope varying from 3 : 1 and 5 : 1. They can be designed as both attracting and repelling groynes.

2. Permeable groynes

This type of groyne permit restricted flow of water through them. They obstruct the flow and allow the deposition of sediment carried by the river. They are also called sedimenting groynes. The accumulation of sediment being more or less permanent act as the protection for groynes and no other materials are required for protection. However, they are not strong enough to resist shocks and pressures from debris, floating ice and logs. They are therefore unsuitable for upper reaches of river.

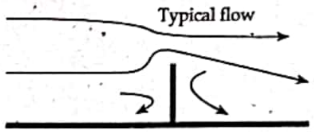


Figure 6.4: (a) Impermeable groynes

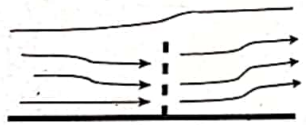


Figure 6.4: (b) Permeable groynes

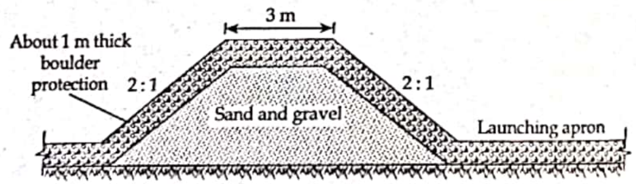


Figure 6.5: Normal section of impermeable groyne

Classification based on the height of groynes

1. Submerged groynes

In case of deep rivers where depths are considerable submerged groynes are used. They can be either solid or permeable groynes. However, submerged groynes are preferable to submerged solid groynes since the former do not create turbulent and eddy conditions as strong as with later.

2. Un-submerged groynes

Generally these types of groynes are used for training of rivers and protection of banks.

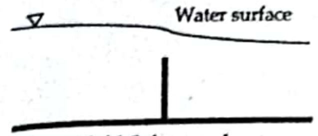


Figure 6.6: (a) Submerged groynes

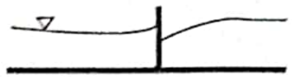


Figure 6.6: (b) Non-submerged groynes

Classification based on the function served

1. Attracting groynes

These are the groynes pointing d/s of the river. They tend to attract the river flow towards the bank on which it is provided and hence called attracting groynes. Such groynes causes scour hole to form closer to the bank than the groyne inclined at right angles to the bank or that inclined slightly u/s and therefore it tends to maintain the deep current close to the bank. They make an angle of 30° to 60° with the bank.

2. Deflecting groynes

A groyne either perpendicular to the bank or pointing slightly u/s and having a relatively short length tends to only deflect the flow without repelling it and hence it is called deflecting groyne. It gives only local protection.

3. Repelling groynes

They are the groynes pointing u/s and tending to repel the flow away from the bank on which they are provided. The angle of inclination varies from 60° to 80°. On its u/s side still water pocket is formed and suspended sediment carried by river water gets deposited in the pocket. Compared to attracting groynes repelling groynes are more effective and do not cause any trouble.

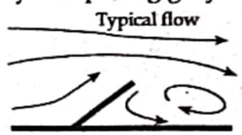


Figure 6.7: (a) Attracting groynes


Figure 6.7: (b) Deflecting groynes


Figure 6.7: (c) Repelling groynes

4. Sedimenting (permeable) groynes

Classification based on the special shape of the groynes

1. T- headed groynes, Hockey type groynes, circular head, rectangular

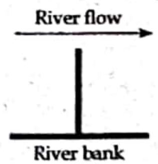


Figure 6.8: (a) Rectangular

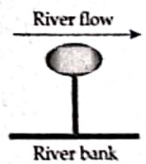


Figure 6.8: (b) Circular head

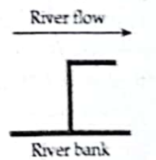


Figure 6.8: (c) L-headed

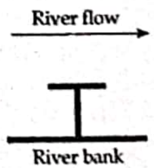


Figure 6.8: (d) T-headed

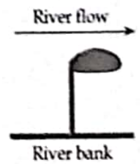


Figure 6.8: (e) Hockey type

General guidelines for spur design and construction

Spurs should not be used where the river is already narrow or where the alignment of the river banks cannot be modified or reduced. It is also not advisable to use spurs where the opposite bank is exposed to transverse flows, which create unacceptable erosion. In such cases continuous longitudinal protection is required. The effectiveness of a spur depends on its design and location, and the resources available. The location of the upstream starting point and the downstream termination point also influence the success of spur installation. The main characteristics to be considered are summarized in the following.

Permeability

Spurs can be permeable or impermeable. Impermeable spurs are built of local soil, stones, gravel, rocks, and gabions, while permeable spurs usually consist of one or several rows of timber, bamboo, or similar. An impermeable spur blocks and deflects the river flow, while a permeable spur allows water to pass through but reduces the water velocity.

Spur height

Spurs can be designed to be higher than the water level at all times (non-submerged), or submerged during the time offloads, emerging only when the flood recedes. In general, submerged spurs are designed to be permeable, whereas non-submerged spurs are impermeable. *The height of non-submerged spurs should not exceed the bank height* because erosion at the end of the spur in the overbank area could increase the probability of outflanking when the water level (stream stage) is high. If stream stages can be greater than or equal to the bank height, the spurs should be equal to the bank height. If flood stages are always lower than the bank height, the spurs should be designed so that overtopping will not occur at the bank. *Submerged spurs should have a height between $\frac{1}{3}$ and $\frac{1}{2}$ of the water depth.*

Spur orientation relative to the river axis

Spurs can be attracting, deflecting, or repelling according to their inclination. An attracting spur points downstream and attracts the flow towards its head and thus to the bank, maintaining a deep current close to the bank. A deflecting spur changes the direction of the flow without repelling it and creates a wake zone behind. A repelling spur points upstream and diverts the flow away from itself. *The first spur in a bend should always be attracting to minimize the impact of the flow.*

Spur shape

Spurs are basically bar shaped, but the end protruding into the water flow can be shaped differently. *An oval or elliptical spur, with the wider portion towards the bank, can change the hydraulic efficiency and reduce the direct impact of the flood water on the spur body.* Investigations have shown that the shape of the spur can affect the bed stress distribution and the scour depth around a spur. For example, the extension of the high shear stress zone is smaller in T-shaped spurs, whereas the maximum scour depth is less around L-shaped spurs.

Spur length

When choosing the length of a spur, it is important to consider the safety of the opposite bank. If a spur is too long, it may guide the river current during a flash flood to the opposite bank which will cause damage; if it is too short, it may cause erosion of the near bank. *As a general rule, the length of a spur should be no more than $\frac{1}{5}$ the river width and no less than 2.5 times the scour depth.* Sometimes a spur is made long and strong with the aim of changing the river course by repelling it towards the opposite bank, in which case the opposite bank should also be protected. Both the river width and the width of the main flow channel to be deflected should be considered when designing the length of a spur. The following two relations are commonly used for determining the length of the spur,

$$\frac{L}{B} = 0.369\left(\frac{A^{0.55}}{F^{0.33}}\right)$$

$$\frac{L}{B} = 0.374\left(\frac{A^{0.29}}{F^{0.145}}\right)$$

where, 'L' is the length of groyne, 'B' is the width of river, 'F' is the Froude number and 'A' is the arc–chord ratio.

Spacing

The effect of a group of spurs depends on their length and spacing. The spacing between two spurs depends on the length of the spurs. The effect on flow is best fulfilled if one strong eddy is created between each pair of spurs. If the spacing is too wide, the effect of the spurs will be insufficient as parts of the bank will remain unaffected. A spacing less than the optimum is wasteful as it does not increase the effect. *The length of bank protected by a spur is generally at least twice the length of the spur projecting perpendicular to the river water current; thus spurs do not need to be closer than twice their projecting length.* More exact calculations can be made using the formulae for eddy stability and energy loss of river flow. *In general, the spacing between two spurs should be 2–2.5 times the spur length along a concave bank and 2.5–3 times the spur length along a convex bank.* In the case of a revetment with spurs, the spacing can be increased without causing harm to the bank. Number of spurs along the stream bank. *The number of spurs depends on the length of stream bank to be protected and the calculated space between spurs.*

Cross section

The cross section of guide bank is similar to that of guide bank with an apron. The following dimensions of an impermeable groyne are generally adopted.

Embankment type

- Free board = 1 m
- Side slope = 2 : 1
- Top width = 3 m
- End slopes at head = 5 : 1
- Length of launching apron and thickness of pitching as per standard practice for the guide banks.

Rock fill type (Low groynes)

- Free board = 0.75 m
- Side slopes = 1.5 : 1
- To width = 3 m
- Mattress = 6 m wide, 0.3 m thick on the u/s.

iv) Artificial cutoff

An artificial cutoff is developed in a meandering river to divert the flow along a straight course. An artificial cutoff is used for the purpose of training of river when its meander goes on increasing and endangers some valuable land or property on its banks. It is also used for straightening the river approach to a structure. Pickles recommendations for artificial cutoff are as follows;

- The pilot channel should be tangential to the main direction of flow in the river both at entry and at exit.
- The pilot channel should be excavated on a mild curve of the river where the curvature is less than the dominant curvature of the river.
- When the cut cannot develop naturally by scouring action because the bank material is not easily erodible or because the velocity is low, it should be excavated to the main river cross section.
- When a series of cut is required to be made the cut at the d/s end should be developed first and the work should then progress u/s to excavate more cuts.

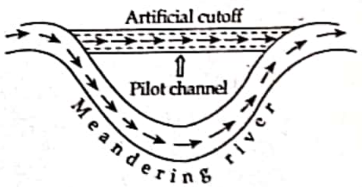


Figure 6.9: Artificial cutoff

6.4 WORKED OUT PROBLEMS

PROBLEM 1

Neatly sketch a guide bund and design the following components of guide bund for river discharge of 4000 m^3/sec. and silt factor 1.1. High flood depth = 5.0 m.
i) Length of guide bund
ii) Thickness of pitching
iii) Width of launching apron
iv) Depth of launching apron

Solution:

Sketch of guide bund

See the definition part 6.3

Design of guide bund

$Q = 4000$ cumec

$f = 1.1$

i) Length of guide bund

Waterway from Lacey's formula,

$$P = 4.75\sqrt{Q} = 4.75\sqrt{4000} = 300.416 \text{ m}$$

Allowing 20% extra for pier thickness, end contractions etc.

Overall waterway (L) = $1.2 \times 300.416 = 360.499 = 360.499 \approx 361$ m

∴ Length of guide bund u/s = $1.25L = 1.25 \times 361 = 451.25$ m

Length of guide bund d/s = $0.25L = 0.25 \times 361 = 90.25$ m

Radius of u/s curve head = $0.45L = 0.45 \times 361 = 162.45$ m

Radius of d/s curve head = $0.225L = 0.225 \times 361 = 81.225$ m

Angle of sweep of u/s head = 135°

Angle of sweep of d/s head = 60°

ii) Thickness of pitching

$$t = 0.06Q^{\frac{1}{3}} = 0.06 \times (4000)^{\frac{1}{3}} = 0.952 \text{ m}$$

iii) Width and thickness of launching apron

$f = 1.1$ m

Lacey's regime scours depth,

$$R = 0.47\left(\frac{Q}{f}\right)^{\frac{1}{3}} = 0.47\left(\frac{4000}{1.1}\right)^{\frac{1}{3}} = 7.227 \text{ m}$$

Now, at nose of guide bank,

$$\begin{aligned}\text{Depth of scour below river bed (D)} &= XR - y = 2.25R - y \\ &= 2.25 \times 7.277 - 5 \\ &= 11.261 \text{ m}\end{aligned}$$

$$\begin{aligned}\text{Length/width of launching apron} &= 1.5D = 1.5 \times 11.261 \\ &= 16.892 \text{ m} \approx 17 \text{ m}\end{aligned}$$

Thickness of launching apron,

At inner edge = $1.5t = 1.5 \times 0.952 = 1.428$ m

At outer edge = $2.25t = 2.25 \times 0.952 = 2.142$ m

At transition of guide bank (curve portion)

Depth of scour below river bed (D) $= XR - y$

$= 1.5R - y$

$= 1.5 \times 7.227 - 5$

$= 5.841$ m

Length/width of launching apron $= 1.5D$

$= 1.5 \times 5.841$ m

$= 8.762 \text{ m} \approx 8.8$ m

Thickness is same as that of nose.

At shank of guide bank

$D = XR - y$

$= 1.25R - y = 1.25 \times 7.227 - 5$

$= 4.034$ m

Length/width of apron $= 1.5D = 1.5 \times 4.034 = 6.051 \approx 6.1$ m

Thickness is same as that of nose.

PROBLEM 2

Enumerate different methods of flood control measures. Explain the function of spur and their types with the help of neat sketches.

Solution:

Different methods of flood control measures are;

i) Marginal embankment or levees
ii) Guide bank
iii) Groynes or spurs
iv) Artificial cutoff
v) Pitched island
vi) Sills (Submerged dikes)
vii) Bandalling
viii) Closing Dikes

Functions of spur

i) To train the river along the desired course by attracting or deflection or repelling the flow in the desired direction.
ii) To protect the bank by keeping the flow away from it.
iii) To create a slack zone for sitting up the area.
iv) To reduce the concentration of the flow at a particular point of attack.
v) To contract a wide river channel for the purpose of increasing its depth for navigation.

Types of spur

See the definition part 6.3

PROBLEM 3

Design a guide bund for a flood discharge of 7000 cumecs, the high flood depth is 5 m and silt factor is 1.1.

Solution: See the solution of Q. no. 1

PROBLEM 4

Design the following components of a guide bund for a river discharge of 6000 m^3/sec. and silt factor 1.1. Take HFL = 150, Bed level = 145 m.

i) Length of guide bund
ii) Thickness of pitching
iii) Width of launching apron
iv) Depth of launching apron

Using the design data draw the following;

A. Plan of guide bank
B. Section at shank and
C. Section at u/s curved head

Solution:

$Q = 6000 \text{ m}^3/\text{sec.}$

$f = 1.1$

High flood depth $= 150 - 145 = 5$ m

Waterway from Lacey's formula,

$P = 4.75\sqrt{Q} = 4.75\sqrt{6000} = 367.933$ m

Increasing by 20%,

Overall waterway (L) $= 1.2 \times 367.933$

$= 441.52 \approx 442$ m

i) Length of guide bank

Length of guide bank u/s $= 1.25L = 1.25 \times 442 = 552.5$ m

Length of guide bank d/s $= 0.25L = 0.25 \times 442 = 110.5$ m

ii) Thickness of pitching.

$t = 0.06Q^{\frac{1}{3}} = 0.06 \times (6000)^{\frac{1}{3}} = 1.09$ m

iii) Width of launching apron

$f = 1.1$ m

$$R = 0.47\left(\frac{Q}{f}\right)^{\frac{1}{3}} = 0.47\left(\frac{6000}{1.1}\right)^{\frac{1}{3}} = 8.273 \text{ m}$$

At nose of guide bank

Depth of scour below river bed (D) $= 2.25R - y$

$= 2.25 \times 8.273 - 5$

$= 13.614$ m

Length/width of launching apron $= 1.5D = 1.5 \times 13.614$

$= 20.421$ m

At transition of guide bank (curve portion)

$D = 1.5R - y = 1.5 \times 8.273 - 5 = 7.41$ m

Length/width of launching apron $= 1.5D = 11.114$ m

At shank of guide bank

$D = 1.25R - y = 1.25 \times 8.273 - 5 = 5.341$ m

Length/width of launching apron $= 1.5D = 8.012$ m

iv) Thickness of launching apron

At inner edge = $1.5t = 1.5 \times 1.09 = 1.635$ m

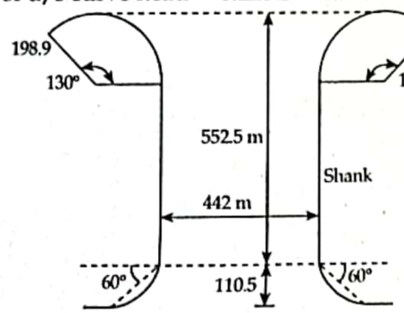
At outer edge = $2.25t = 2.25 \times 1.09 = 2.453$ m

∴ Use 1.7 m at inner edge and 2.5 m at outer edge.

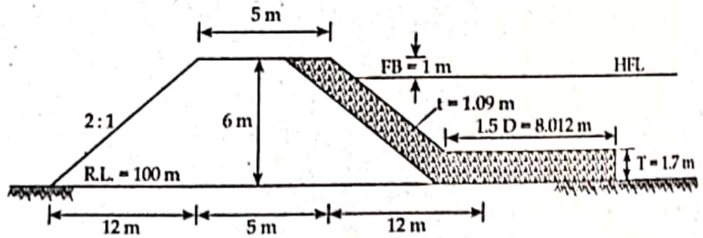
A. Plan of guide bank

Radius of u/s curve head = $0.45L = 0.45 \times 442 = 198.9$ m

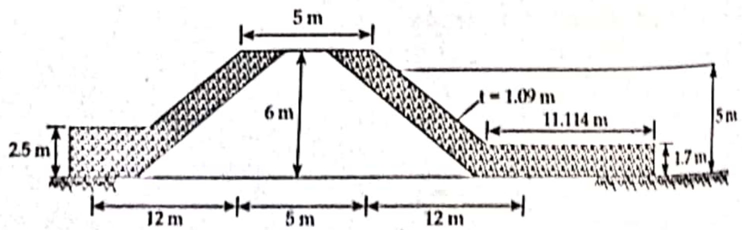
Radius of d/s curve head = $0.225L = 0.225 \times 442 = 99.45$ m



B. Section at shank



C. Section at u/s curve head



PROBLEM 5

A guide bank with stone pitching is required for a bridge on a river having the following particulars.
Design flood discharge = 50,000 cumecs
Silt factor = 1.1
Bed level of river = 130.00 m
High flood level = 140.00 m
Thickness of launching apron = 2.5 times thickness of stone pitching
What length of launching apron is necessary to protect the u/s impregnable head of the guide bank?

Solution:

$Q = 50000$ m^3/sec.

$f = 1.1$

High flood depth $(y) = 140 - 130 = 10$ m

Waterway (P) $= 4.75\sqrt{Q} = 4.75\sqrt{50000} = 1062.132$ m

Increasing 20%,

$L = 1.2 \times 1062.132 = 1274.56 \approx 1275$ m

i) Length of guide bank u/s = $1.25L = 1.25 \times 1275 = 1912.5$ m

Length of guide bank d/s = $0.25L = 0.25 \times 1275 = 318.75$ m

ii) Thickness of pitching

$t = 0.06Q^{\frac{1}{3}} = 0.06 \times (50000)^{\frac{1}{3}} = 2.21$ m

iii) Length of launching apron

$f = 1.1$

$R = 0.47\left(\frac{Q}{f}\right)^{\frac{1}{3}} = 0.47\left(\frac{50000}{1.1}\right)^{\frac{1}{3}} = 16.77$ m

At nose of guide bank

$D = XR - y = 2.25R - y = 2.25 \times 16.77 - 5 = 32.73$ m

Length of launching apron = $1.5D = 1.5 \times 32.73 = 49.095$ m ≈ 49.1 m

At transition of guide bank

$D = 1.5R - y = 1.5 \times 16.77 - 5 = 20.16$ m

Length = $1.5D = 1.5 \times 20.16$ m $= 30.24$ m

At shank,

$D = 1.25R - y = 1.25 \times 16.77 - 5 = 15.96$ m

Length = $1.5D = 1.5 \times 15.96 = 23.94$ m

Thickness of launching apron = $2.5t = 2.5 \times 2.21 = 5.53$ m

NOTE

Plan and sections can be drawn in the same was as that of Q. no. 4

PROBLEM 6

Following hydraulic data near a proposed bridge site are obtained.
Maximum discharge = 4000 m^3/sec.
HFL = 205.0 m
River bed level = 200.00 m
Average diameter of river bed material = 0.1 mm
Design the following components of guide bank and neatly sketch it.
i) Length of guide bank
ii) Thickness of pitching of slope
iii) Length of launching apron
iv) Thickness of launching apron

Solution:

$f = 1.76\sqrt{d_{mm}} = 1.76\sqrt{0.1} = 0.56$

NOTE

Proceed as the solution of Q. no. 1

PROBLEM 7

Describe various methods of river training with neat sketches.[2069 Poush]

Solution:

The various methods of river training are as follows;

i) Marginal embankments
ii) Guide bank
iii) Spurs
iv) Artificial cutoff
v) Miscellaneous methods
   - Pitched islands
   - Sills (Submerged dykes)
   - Bandalling
   - Closing dykes

For detail description and figure

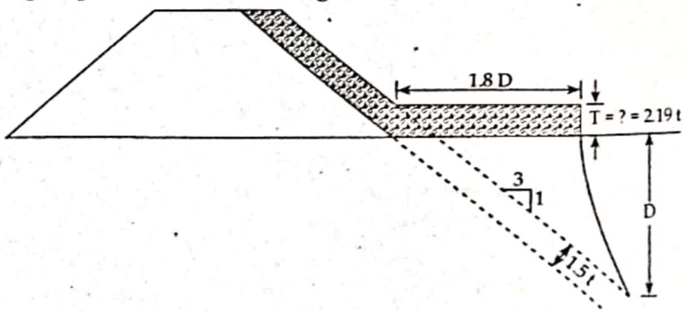
See the definition part

PROBLEM 8

The launching apron of a guide bank is laid in a width equal to 1.8 times the depth of scour below original bed. If a scour slope of 3 : 1 is to be maintained with thickness 1.5t, find the thickness of apron before it get launched. Draw neat sketch of designed structure.

Solution:

According to question the section of guide bank can be drawn as;



i.e., Volume of launching apron before launching/unit length = $1.8D \times T$

Volume of launching apron after launching/unit length = $\sqrt{3^2 + 1^2} \times D \times 1.25t$

$= 3.95\ Dt$

Volume of apron before and after launching remains same, hence,

$1.8D \times T = 3.95\ Dt$

$\therefore\ T = 2.19t$

PROBLEM 9

What is spur? How do you fix the spacing of spur in bend? Draw neat sketches of plan and section of gabion spur. [2070 Magh]

Solution:

Spur

A spur is a structure made to project flow from a river bank into a stream or river with the aim of deflecting the flow away from the side of the river on which the spur is built. Two or five structures are typically placed in series along straight or convex bank lines where the flow lines are roughly parallel to bank.

Spacing of spur in bend

- The spurs do not need to be closer than twice their projecting length.
- Spacing should be 2–2.5 times the spur length along a concave bank.
- Spacing should be 2.5–3 times the spur length along convex bank.

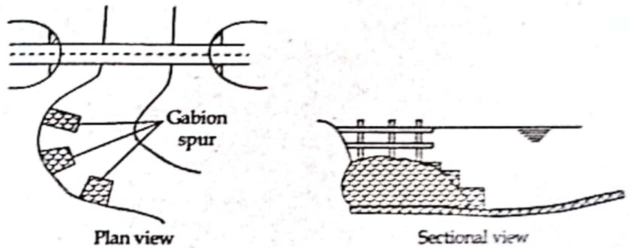


Figure 6.10: Plan and sectional view of gabion spur

PROBLEM 10

What is meant by river training works and what are the different objectives served by it. What are the underlying principles behind the determination of spur spacing? Draw L and X-section of a typical spur.[207

Solution:

River training works

River training works implies various measured adopted on a river to stabilize the river channel along a certain alignment with a certain cross-section. These measures are required to be adopted because rivers in alluvial plains frequently alter their courses and cause damage to land and property adjacent to their banks.

Objectives of river training works

The main objectives of river training works are as follows:

i) To provide a safe passage to flood discharges without overflowing to the banks for protection of cultivated area.
ii) To prevent outflanking of a work like a bridge, weir or aqueduct constructed across the river and to bring the river on the work in a straight non-tortuous approach.
iii) To protect the banks form erosion and improve the alignment by stabilizing the river channel.

iv) To deflect the river away from the bank this is attacking.
v) To provide minimum depth of flow and a good course for navigation purposes.
vi) To transport efficiently the bed and suspended sediment load.

Principles for the determination of spur spacing

i) The spur does not need to be closer than twice their projecting length.
ii) Spacing should be 2–2.5 times the spur length along a concave bank.
iii) Spacing should be 2.5–3 times the spur length along convex bank.

Other factors affecting the spacing of groynes

i) Width of the river

For rivers of equal flood discharges, a larger ratio of spacing to length of groynes may be used for a wide river than for a narrow river.

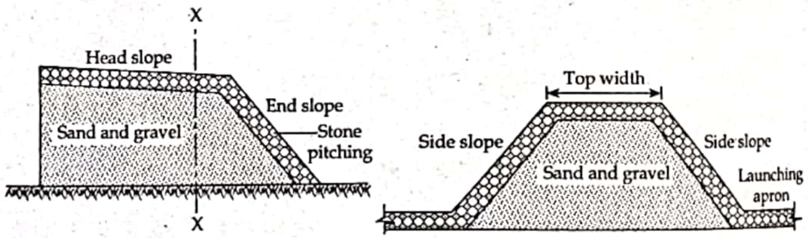
ii) Location of the groynes

Larger spacing may be used for convex banks than for concave banks. The spacing for straight reaches or crossing being kept in between the two.

iii) Type of construction

The permeable groynes may be spaced further apart than solid or impermeable groynes.

Typical L–section and X–section of spur



(a) L–section of spur (b) Cross–section at X-X

Figure 6.11: Typical L–section and X–section view of spur

PROBLEM 11

Design the length, radius of curved head, length and thickness of gabion slope pitching and gabion launching apron of a guide bund to train a river with the following data. [2071 Bhadra: T.U.]
Design floor discharge = 3000 cumecs
River bed level = 240 m
HFL = 245 m
Average diameter of river bed material = 0.1 mm

Solution:
Given that;

$Q = 3000\ m^3/sec.$

$d_{mm} = 0.1$ mm

$f = 1.76\sqrt{d_{mm}} = 0.56$

Now,

High flood depth $= 245 - 240 = 5$ m(say y)

Waterway from Lacey's formula;

$P = 4.75\sqrt{Q} = 4.75\sqrt{3000} = 260.163$ m

Increasing by 20%; we have,

Overall waterway (L) $= 1.2 \times 260.163 = 312.202 \approx 313$ m

i) Length of guide bank

Length of guide bank u/s $= 1.25L = 1.25 \times 313 = 391.25$ m

Length of guide bank d/s $= 0.25L = 0.25 \times 313 = 78.25$ m

ii) Radius of curved head

Radius of u/s curve head $= 0.45L = 0.45 \times 313 = 140.85$ m

Radius of d/s curve head $= 0.225L = 0.225 \times 313 = 70.425$ m

iii) Length of slope pitching

Providing 1 m FB, height of guide bank $= 5 + 1 = 6$ m

Providing side slope 2 : 1,

Length of slope pitching $= 2 \times 6 = 12$ m

iv) Thickness of slope pitching

$$t = 0.06Q^{\frac{1}{3}} = 0.06 \times (3000)^{\frac{1}{3}} = 0.865 \text{ m}$$

v) Length of launching apron

$f = 0.56$

$$R = 0.47\left(\frac{Q}{f}\right)^{\frac{1}{3}} = 0.47\left(\frac{3000}{0.56}\right)^{\frac{1}{3}} = 8.224 \text{ m}$$

At nose of guide bank,

Depth of scour below river bed (D) $= 2.25R - y$
$= 2.25 \times 8.224 - 5$
$= 13.504$ m

Length/width of launching apron $= 1.5D$
$= 1.5 \times 13.504$
$= 20.256$ m

At transition of guide bank (curve portion)

$D = 1.5R - y = 1.5 \times 8.224 - 5 = 7.336$ m

Length/width of launching apron $= 1.5D = 1.5 \times 7.336 = 11$ m

At shank of guide bank

$D = 1.25R - y = 1.25 \times 8.224 - 5 = 5.28$ m

Length/width of launching apron $= 1.5D = 1.5 \times 5.28 = 7.92$ m

vi) Thickness of launching apron

At inner edge $= 1.5t = 1.5 \times 0.865 = 1.298$ m

At outer edge $= 2.25t = 2.25 \times 0.865 = 1.946$ m

Hence, use 1.3 m at inner edge and 2 m at outer edge.

PROBLEM 12

Design and sketch Bell's Bund to train a river at its bridge site having the following data: **[2014 Fall: P.U.]**
Maximum discharge = 5500 cumecs
River bed level = 100 m
HFL = 105 m
Average diameter of bed material = 0.15 mm

Solution:
Given that;

$$Q = 5500 \text{ cumecs}$$
$$f = 1.76\sqrt{d_{mm}} = 1.76\sqrt{0.15} = 0.682$$
$$\text{High flood depth } (y) = 105 - 100 = 5 \text{ m}$$

Waterway from Lacey's formula; we have,

$$P = 4.75\sqrt{Q} = 4.75\sqrt{5500} = 352.269 \text{ m}$$

Increasing by 20%; we have,

$$\text{Overall waterway } (L) = 1.2 \times 352.269 = 422.723 \approx 423 \text{ m}$$

i) Length of Bell's bund

$$u/s = 1.25L = 1.25 \times 423 = 528.75 \text{ m}$$
$$d/s = 0.25L = 0.25 \times 423 = 105.75 \text{ m}$$

ii) Radius of curve head

$$u/s \text{ curve head} = 0.45L = 0.45 \times 423 = 190.35 \text{ m}$$
$$d/s \text{ curve head} = 0.225L = 0.225 \times 423 = 95.175 \text{ m}$$

iii) Thickness of stone pitching

$$t = 0.06Q^{\frac{1}{3}} = 0.06 \times (5500)^{\frac{1}{3}} = 1.059 \text{ m}$$

iv) Length/width of launching apron

$$f = 0.682$$
$$R = 0.47\left(\frac{Q}{f}\right)^{\frac{1}{3}} = 0.47\left(\frac{5500}{0.682}\right)^{\frac{1}{3}} = 9.425 \text{ m}$$

At nose of guide bank

$$D = 2.25R - y = 2.25 \times 9.425 - 5 = 16.206 \text{ m}$$
$$\text{Length/width of launching apron} = 1.5D = 1.5 \times 16.206 = 24.306 \text{ m}$$

At curve portion

$$D = 1.5R - y = 1.5 \times 9.425 - 5 = 9.138 \text{ m}$$
$$\text{Length/width of launching apron} = 1.5D = 1.5 \times 9.138 = 13.707 \text{ m}$$

At shank of guide bank

$$D = 1.25R - y = 1.25 \times 9.425 - 5 = 6.781 \text{ m}$$

$$\text{Length/width of launching apron} = 1.5D = 1.5 \times 6.781 = 10.172 \text{ m}$$

v) Thickness of launching apron

$$\text{At inner edge} = 1.5t = 1.5 \times 1.059 = 1.589 \text{ m}$$
$$\text{At outer edge} = 2.25t = 2.25 \times 1.059 = 2.383 \text{ m}$$

Hence, use 1.6 m at inner edge and 2.4 m at outer edge.
Provide top width = 5 m and side slope 2 : 1

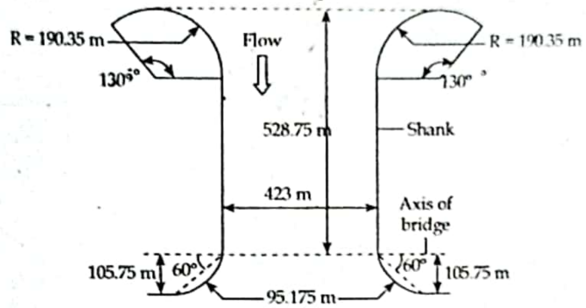


Figure: Plan View

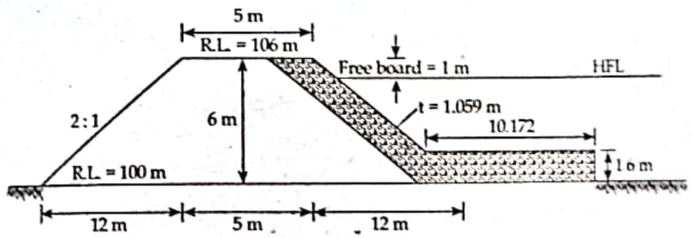


Figure: Section at shank

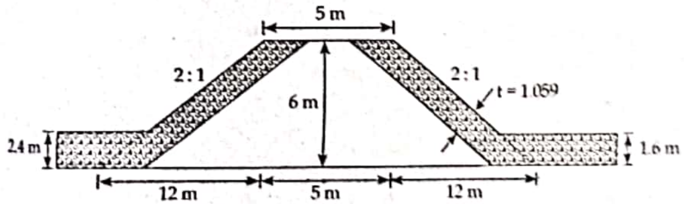


Figure: Section at u/s curved portion

PROBLEM 13

Design and sketch Bell's Bund to train a river at the bridge site having the following hydraulic data:
Maximum discharge = 6000 cumecs
River bed level = 100 m
HFL = 104 m
Average diameter of bed material = 0.1 mm

Solution: Proceed as the solution of problem 12

PROBLEM 14

Hydraulic data pertaining to a bridge site and river is given as follows:
Maximum discharge = 4000 cumecs
River bed level = 119 m
HFL = 125 m
Average diameter of sediment of river = 1.35 mm
Design and sketch Bell's Bund and also find the stone required per metre length in shank portion. [2014 Fall: P.U.]

Solution:

Given that;

$Q = 4000$ cumecs

$HFL = 125$ m

Bed level = 119 m

High flood depth $(y) = 125 - 119 = 6$ m

$f = 1.76\sqrt{d} = 1.76\sqrt{1.35} = 2.045$

From Lacey's formula; we have,

Waterway $(P) = 4.75\sqrt{Q} = 4.75\sqrt{4000} = 300.416$ m

Increasing by 20%; we have,

Overall waterway $(L) = 1.2 \times 300.416 = 360.5 \approx 361$ m

i) Length of Bell's Bund

$u/s = 1.25L = 1.25 \times 361 = 451.25$ m

$d/s = 0.25L = 0.25 \times 361 = 90.25$ m

ii) Radius of curve head

u/s curve head $= 0.45L = 0.45 \times 361 = 162.45$ m

d/s curve head $= 0.225L = 0.225 \times 361 = 81.225$ m

iii) Thickness of stone pitching

$t = 0.06Q^{\frac{1}{3}} = 0.06 \times (4000)^{\frac{1}{3}} = 0.952$ m

iv) Length/width of launching apron

$f = 2.045$

$$R = 0.47\left(\frac{Q}{f}\right)^{\frac{1}{3}} = 0.47\left(\frac{4000}{2.045}\right)^{\frac{1}{3}} = 5.878 \text{ m}$$

At nose of guide bank

$D = 2.25R - y = 2.25 \times 5.878 - 6 = 7.226$ m

Length/width of launching apron $= 1.5D$

$= 1.5 \times 7.226 = 10.838$ m

At transition of guide bank

$D = 1.5R - y = 1.5 \times 5.878 - 6 = 2.817$ m

Length/width of launching apron $= 1.5D$

$= 1.5 \times 2.817$

$= 4.226$ m

At shank of guide bank

$D = 1.25R - y = 1.25 \times 5.878 - 6 = 1.348$ m

Length/width of launching apron $= 1.5D$

$= 1.5 \times 1.348$

$= 2.022$ m

v) Thickness of launching apron

At inner edge $= 1.5t = 1.5 \times 0.952 = 1.428 \text{ m} \approx 1.43$ m

At outer edge $= 2.25t = 2.25 \times 0.952$

$= 2.142 \text{ m} \approx 2.45$ m

Sketch the designed bund in same way as problem 12.

Length of shank portion = Length of Bell's Bund u/s – Radius of u/s curve head

$= 451.25 - 162.45$

$= 288.8$ m

X-section area of stone pitching and launching apron of shank portion is;

$= \sqrt{(7)^2 + (12)^2} \times 0.952 + 2.022 \times 1.43$

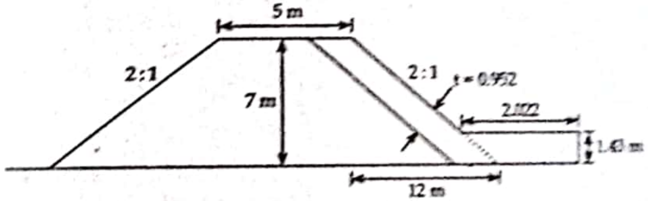
$= 16.117 \text{ m}^2$

Volume of stone $= 16.117 \times 288.8 = 4654.609 \text{ m}^3$

Volume of stone/metre length $= \frac{4654.609}{288.8}$

$= 16.117$

$= 16.12 \text{ m}^3/\text{m}$



PROBLEM 15

Explain with sketch how spur assist in river control work. [2072 Ashwin]

Solution: See the definition part 6.3

PROBLEM 16

Design the length, radius of curved head, length and thickness of launching apron of a guide bund to train a river with the following data.
Design flood discharge = 4500 cumecs
Bed level of river = 150.00 m
HFL = 154.00 m
Average diameter of river bed material = 0.1 mm [2072 Ashwin]

Solution: Proceed same as the solution of example 6.2

PROBLEM 17

Write various methods of river trainings. Discuss with necessary sketch, the type of spurs used for river training works. [2072 Magh]

Solution: See the definition part 6.3

PROBLEM 18

Explain with sketch four different methods of river training works.[2073 B]

Solution: See the definition part 6.3

PROBLEM 19

What is river training works? Explain with sketch three methods of training works normally adopted in Nepalese rivers. [2073 Magh]

Solution: See the definition part 6.1, 6.2 and 6.3

PROBLEM 20

Why river training works are required? Explain with sketch the layout of spurs to train the river in bend. [2074 Bhadra]

Solution: See the definition part 6.1.1

PROBLEM 21

A bridge is to be constructed across a river having the following hydraulic data:
Maximum flood discharge = 5000 m^3/sec.
Highest flood level = 254.0 m
River level = 250 m
Average dia of river sand = 0.25 m
Design and sketch a guide bank including launching apron to train the river. [2074 Bhadra]

Solution: See the solution of example 6.2

PROBLEM 22

Neatly sketch a guide bund and design the following components of a guide bund for a river discharge of 4000 m^3/sec. and silt factor 1.1.
i) Length of guide bund
ii) Thickness of pitching
iii) Width of launching apron
iv) Depth of launching apron [2075 Baishakh]

Solution: *Proceed same as the solution of problem 12*

Hint

Assume,

High flood depth = 5 m

PROBLEM 23

Write procedural steps in spur design. [P.U. 2014]

Solution: *See the definition part 6.3*

PROBLEM 24

Write the design steps of Guide bank with suitable sketch. [2076 Baishakh]

Solution: See the definition part 6.3 (ii)

PROBLEM 25

Write the different stages of river and their characteristics along flow path. [2076 Bhadra]

Solution: See the definition part 6.1.1

PROBLEM 26

Determine the length and thickness of launching apron for the straight portion of guide bund in a river for the data given below. Maximum discharge = 6000 m^3/sec., Average diameter of bed material = 1.0 mm, Highest flood level (HFL) = 330 m, River bed level = 326 m [2076 Bhadra]

Solution: Proceed same as the solution of example 6.1

PROBLEM 27

Design a guide bank for the weir site from the following data provided.
Bed level of river = 105.00 m
Depth of water during high flood = 5 m
Discharge of river = 6500 m^3/sec.
The value of lacey's silt factor may be taken as 1. [2077 Chaitra]

Solution: Proceed same as solution of problem no. 1

PROBLEM 28

Design guide bunds and launching apron required to be provided for a bridge across a river whose total water way is 658.88 m. The design flood discharge is 13,100 m^3/sec. which may be increased by 20% of the design of launching apron. The mean size of river bed material is 0.3 mm. [2078 Baishakh]

Solution:

Given that;

Design flood discharge (Q) = 13100 m^3/sec.

Discharge for the design of launching apron.

$Q_d = 13100 \times 1.2$

$= 15720\ m^3/sec.$

We have,

$$\text{Velocity (V)} = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}}$$

$$\text{So,}\quad V = \left\{\frac{15720 \times (0.96)^2}{140}\right\}^{\frac{1}{6}} = 2.16\ m/sec.$$

$$f = 1.76\sqrt{d} = 1.76\sqrt{0.3} = 0.96$$

Total waterway = 658.88 m

$$\text{High flood depth} = \frac{Q}{V \times B}$$

$$= \frac{15720}{2.16 \times 658.88}$$

$$= 11.04 \text{ m}$$

Now, precede same as solution of problem no. 1.

6.5 OBJECTIVE QUESTION

1. For a meandering alluvial river in flood plain, the meander length is about ....
   a) 6 W b) 18 W c) 17 W d) W
   where, W is normal river width.
2. The ratio of dominant discharge for a river to the peak discharge for that river is of the order of ....
   a) $\frac{3}{16}$ b) $\frac{9}{16}$ c) $\frac{13}{16}$ d) $\frac{3}{2}$
3. The meander length for an alluvial river is ....
   a) the total channel length along its looped course
   b) the total channel length minus the direct straight length
   c) the axial length of one meander
   d) the looped length of one meander
4. Aggrading rivers are ....
   a) silting rivers b) scouring rivers
   c) rivers in regime d) meandering rivers
5. The repelling groynes, which are largely, constructed projecting from river embankments, an anti-erosion works are ....
   a) pointing upstream b) pointing downstream
   c) perpendicular to bank d) none of the above
6. The u/s angle of inclination of a repelling groyne with normal to the bank line, is of the order of ....
   a) 5° to 10° b) 10° to 30°
   c) 30° to 50° d) 70° to 90°
7. An alluvial river increases its length by meandering due to ....
   a) variation of discharge b) variation of land topography
   c) both (a) and (b) d) none of the above
8. Out of the following choices given below, choose impermeable spur ....
   a) an earthen spur protected by stone apron
   b) an earthen spur unprotected by stone apron
   c) a balli spur d) a tree groyne
   e) both (a) and (b)
9. Permeable spurs are best suitable for rivers, which ....
   a) carry heavy suspended load
   b) carry large bed load, but light suspended load
   c) need permanent protection to dikes
   d) need attracting the river current, for providing deeper channel
   e) flow in upper hilly reaches
10. Meander ratio in an alluvial meandering river is given by ....
    a) $\frac{\text{Meander length}}{\text{Meander width}}$ b) $\frac{\text{Meander width}}{\text{Meander length}}$
    c) $\frac{\text{Meander width}}{\text{Meander length}} \times 100$ d) none of the above
11. Denehy's spur is ....
    a) a hockey-shaped earthen spur b) a T-shaped stone spur
    c) a T-shaped earthen spur d) none of the above

12. Which one of the following effects cannot be attributed to have been caused by the construction of dykes along a river course?
    a) Faster travel of a flood wave
    b) Higher flood levels along the river
    c) Increased peak discharges all along d/s points
    d) Decrease in surface slope of the river above the leveed reach
    e) Increased bed levels in the river
13. Which of the following effects produced by a cut off in an alluvial river, is not an advantage to navigation?
    a) Shortened route and elimination of sharp bench
    b) Shortened travel time, particularly at low and moderate river stages
    c) Increased water depth at low river stages
    d) Lowering of the flood stages and flood periods
14. In a meandering river, the ratio of 'actual channel length' to direct axial length is called ....
    a) tortuosity b) inverse of tortuosity
    c) cut off ratio d) none of the above
15. Tortuosity in a meandering river is ....
    a) 1 b) < 1
    c) > 1 d) none of the above
16. A river reach having tortuosity of 1.2 can be said to have ....
    a) 20% tortuosity of meander b) 80% tortuosity of meander
    c) 120% tortuosity of meander d) none of the above

Answer sheet

1	2	3	4	5	6	7	8	9	10
a	b	c	a	a	b	c	e	a	b
11	12	12	14	15	16				
c	e	d	a	c	a				

CHAPTER 7
REGULATING STRUCTURES

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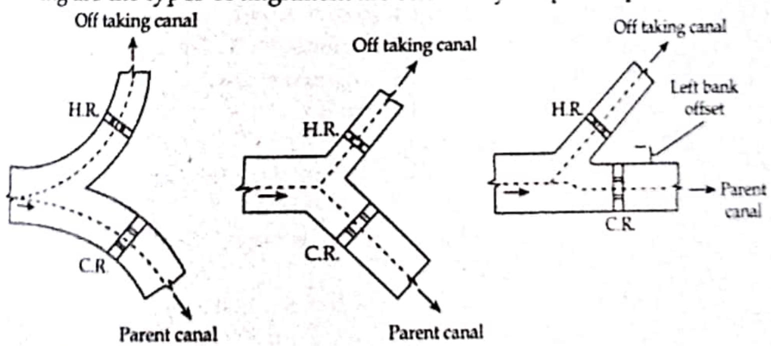
7.1 ALIGNMENT OF OFF-TAKING CHANNELS

When a branch channel takes off from the main channel (called parent channel), the off-take channel must be carefully designed so that an off-taking channel is able to draw its supply without any undesirable effect. Following are the types of alignment are commonly adopted in practice.



(i) Smooth off take (ii) Both inclined to original flow (iii) Parent canal flows straight with reduced width

Legend
C.R. = Cross regulator
H.R. = Head regulator

Figure 7.1: Alignment types for off taking canal from a parent canal.

The best ideal alignment is when the off taking channel makes zero angles with the parent channel initially and then separates out in a transition, as shown in figure (i).

The transition will have to be properly designed, so as to avoid accumulation of silt jetty. As an alternative to the transitive to the transitions, both the channels should make an angle with the off take, as shown in figure (ii).

When the parent channel has to be carried straight, the edge of the canal rather than the center line should be consider in deciding the angle of the off taking as shown in figure (iii).

A distributary head regulator controls the supply of the off taking channel, while a cross regulator controls the supply of the parent channel

7.2 FUNCTION OF HEAD REGULATOR, CROSS REGULATOR, OUTLET, DROP AND ESCAPES

A distributary head regulator controls the supply of the off taking canals while a cross head regulators controls the supply of parent channel.
A head regulator provided at the head of the off-taking channel controls the flow of the water entering in this new channel while a cross regulator may be required in the main parent channel in the downstream.

Function of head regulator

i) It regulates the supply of water from parent channel to off-taking channel.
ii) It controls the entry of silt into the off-taking channel.
iii) It can serve as the meter for the measurement of discharge.
iv) It used for the shutting of the supply of the off-taking channel when water is not needed.

Function of cross-regulator

i) The main function off is to raise the water level in the parent channel on the upstream so that off taking channel can takes its full supply even when the water level in the parent channel is lower than F.S.L.
ii) To effectively control the canal irrigation system.
iii) It is also used to close to supply in the parent channel on its downstream.
iv) It helps to absorb fluctuation in the various sections of the canal system.
v) There is usually a bridge on the cross regulator, which provides a means of communication.
vi) It can be used to control the drawdown when the subsoil water levels are high to ensure safety.
vii) It can provide measures for drop if required, also called fall regulator.

Outlets (modules)

A canal outlet or modules 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

i) Main function of outlet is to take water from minor or distributary channel.
ii) It distributes water in required proportions in fields.
iii) It draws water safely from distributary channel.

Escapes

An escape is a side channel constructed to wasting of some water from an irrigation channel into a natural drain.

Function of escapes

i) It helps to overflow the extra surplus water from the canal safely.
ii) It prevents the damage and de-function of irrigation canal.
iii) It prevents the damage of the farming land.
iv) It helps scouring of excess bed silt deposited.

Fall (drop)

A canal fall is a structure constructed on a channel to lower down the water level and the bed level of the channel when the available natural ground slope is steeper than the designed bed slope of the channel.

Function of fall

i) It lowers the bed level and surface level of the canal.
ii) It dissipates the surplus energy.
iii) It provides safety against erosion in bed, bank and d/s due to excess energy of flow.

7.3 DESIGN OF REGULATOR AND ESCAPES (CREST, LENGTH AND THICKNESS OF IMPERVIOUS FLOOR)

Crest levels

The crest of a cross regulator is generally kept at the u/s bed level of the channel while the crest level of the distributary head regulator is generally kept 0.3 to 1.0 m higher that the crest level of the cross regulator.

Water-way

Water-way can be worked out by using the drowned weir formula, given as;

$$Q = \frac{2}{3} \cdot C_{d_1} \cdot \sqrt{2g} \cdot B \cdot \left[(h + h_v)^{\frac{3}{2}} - h_v^{\frac{3}{2}}\right] + C_{d_2} \cdot B \cdot h_1 \sqrt{2g(h + h_v)}$$

where, $C_{d_1} = 0.577$

$C_{d_2} = 0.80$

B is the clear water-way required.

h is the Difference of water level u/s and d/s of the crest, as shown in the figure.

h_1 is the depth of the d/s water level in the channel above the crest.

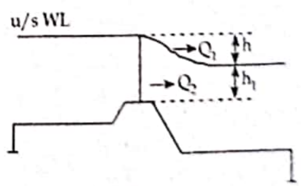


Figure 7.2

h_v is the head due to velocity of approach, which is very small and is generally ignored. The discharge formula then, becomes;

$$Q = \frac{2}{3} \cdot 0.577 \cdot \sqrt{2 \times 9.81} \cdot B \cdot (h)^{\frac{3}{2}} + 0.80 \cdot B \cdot h_1 \sqrt{2 \times 9.81\, h}$$

$$= 1.69 \cdot B \cdot (h)^{\frac{3}{2}} + 3.54 \cdot B \cdot h_1 \sqrt{h}$$

or, $$Q = B \cdot \sqrt{h}[1.69\, h + 3.54 h_1]$$

Conditions of flow for design

The design is done for the worst of the following conditions.

i) Full supply discharge is passing down both the channels with all gates of cross regulator and head regulator fully open.

ii) The discharge in the parent channel is low but the of-take channel is running full, and its F.S.L. is maintained by the partial opening of the gates of cross-regulator.

In the first condition, q and H_L are fixed; while in the second case q reduced and H_L increases, depending upon the flow of the channel. The first condition may become more critical.

If the low flow record of the parent channel is not given, the design can be done for the first flow condition.

Downstream floor level or cistern level

For the above two flow conditions, q and H_L are worked out. Then, E_{f_2} is found from figure 5.27. The levels at which jump would form, *i.e.*, the level of d/s floor, is then given by;

d/s T.E.L. – E_{f_2}

Neglecting the velocity head;

d/s T.E.L. = d/s F.S.L.

∴ Level of d/s floor = d/s F.S.L. – E_{f_2}

If the d/s floor for the worst condition works out to be higher than the d/s bed level of the channel, the floor is provided at the bed level itself.

Length of d/s floor

It is worked out by calculating $5(y_2 - y_1)$. If by chance, this length comes out to be small in comparison to $\frac{2}{3}$ of total floor length (worked out by exit gradient consideration, *i.e.*, $b = \alpha \cdot d$), then, the length of the d/s floor is kept equal to $\frac{2}{3}$ of the total floor length.

Cut-offs is provided as given below.

Upstream cut-off

The minimum depth of u/s cut floor level is kept as;

$$\left(\frac{y_u}{3} + 0.6\text{ m}\right)$$

where, y_u is the depth of water in the channel u/s.

Downstream cut-off

The minimum depth d/s cut-off below the d/s floor level is kept as;

$$\left(\frac{y_d}{2} + 0.6\text{ m}\right)$$

where, y_d is the depth of water in the channel d/s.

Total floor length

The total floor length 'b' is worked out form the safe exit gradient consideration, as explained earlier. This total floor length is then suitably distributed upstream and downstream.

Uplift pressures and floor thickness

The thickness of the d/s floor required, are worked out by the uplift pressures by Khosla's theory. The maximum unbalanced heads at key points are worked out for the maximum static head. The pressure at toe of glacis is also worked out for dynamic condition. The floor thicknesses are designed for the worst case, as explained earlier in the design of weirs and falls. A nominal thickness of 0.3 to 0.5 m is provided on u/s side.

Protection works

The protection works are designed for a scour depth (D) equal to $\left(\frac{y_u}{3} + 0.6\text{ m}\right)$ on the u/s and $\left(\frac{y_d}{2} + 0.6\text{ m}\right)$ in the d/s. The 'C.C. blocks' and 'inverted filter' are provided in a length approximately equal to 1.5 D. The quantity of stone in launching apron is kept as 2.25 D cu. m/meter.

Example 7.1

Design a cross regulator and a head regulator for a channel which takes off from the parent channel with the following data.
Discharge of parent channel = 150 cumecs
Discharge of distributary = 16 cumecs
F.S.L. of the parent channel, u/s = 210.0 m
F.S.L. of the parent channel, u/s = 209.8 m
Bed width of parent channel, u/s = 52 m
Bed width of parent channel, d/s = 46 m
Depth of water in the parent channel
d/s and u/s = 2.5 m
F.S.L. of distributary = 109.1 m
Silt factor = 0.8 m
Assume safe exit-gradient = $\frac{1}{5}$

Solution:

A. **Design of cross regulator**

Crest levels

Crest level of cross is kept same as u/s bed level of parent channel is;

= 210.0 – 2.5 = 207.5 m

Provide crest level at R.L. 207.5 m.

Waterway, neglecting velocity head

$$Q = B \cdot \sqrt{h}[1.69\, h + 3.54 h_1]$$

where, B is the clear waterway required.

In this case,

$h = u/s\ F.S.L. - d/s\ F.S.L. = 210.0 - 209.8 = 0.2\ m$

$h_1 = d/s\ F.S.L. - \text{crest level} = 209.8 - 207.5 = 2.3\ m$

$\therefore \quad 150 = B \cdot \sqrt{0.20}(1.69 \times 0.2 + 3.54 \times 2.3)$

or, $\quad B = \frac{150}{0.447}\left(\frac{1}{8.488}\right) = 39.53$

Provide 5 bays of 8.0 m each with a clear waterway = 5 × 8.0 = 40.0 m.

Provide 4 piers of 1.5 m width each = 6.0 m.

Thus,

Overall waterway = 40 + 6 = 46 m

Downstream floor level or cistern level

$q = \frac{140}{40} = 3.75$ cumecs/meters

$H_L = 0.2\ m$

From figure 5.27;

$E_{f_2} = 1.85\ m$

d/s floor level $= d/s\ F.S.L. - E_{f_2} = 209.8 - 1.85 = 207.95\ m$

d/s bed level $= 209.8 - 2.5 = 207.3\ m$

This is lower than the calculated d/s floor level.

Hence, provide the cistern or d/s at R.L. 207.3 m

Length of d/s floor

Length of d/s floor required $= 5\,(y_2 - y_1)$

From the figure 5.27;

For $E_{f_2} = 1.85$ m;

$y_2 = 1.8\ m$

For $E_{f_1} = E_{f_2} - H_1 = 1.85 + 0.2 = 2.05$ m;

$y_1 = 0.8\ m$

$\therefore \quad 5\,(y_2 - y_1) = 5\,(1.8 - 0.8) = 5.0\ m$

(Subjected to the limitation of $\left(\frac{2}{3}\right)^{rd}$ of total floor length 'b')

Vertical cut-offs

Provide upstream cut-off for a depth $= \frac{y_u}{3} + 0.6\ m = \frac{2.5}{3} + 0.6\ m$

$= 1.43\ m$

Below the floor,

i.e., Level of its bottom = 207.5 − 1.43 = 206.07 m

Downstream cutoff

Depth of d/s cutoff below floor level $= \frac{y_d}{3} + 0.6\ m$

$= \frac{2.5}{3} + 0.6\ m = 1.85\ m$

Hence,

Bottom level of d/s cutoff = 207.30 − 1.85 = 205.45 m

Total floor length from exit gradient considerations;

$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$

where, H = Maximum static head which is caused when there is full water on u/s and there is no water on u/s and there is no water in d/s.

$H = u/s\ F.S.L. - d/s$ bed level $= 210.0 - 207.3 = 2.7\ m$

d = Depth of d/s cutoff = 1.85 m

$G_E = \frac{1}{5}$ (Given)

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

or, $\quad \frac{1}{\pi\sqrt{\lambda}} = \frac{1}{5} \times \frac{1.85}{2.7} = 0.137$

From plate 11.2;

For $\frac{1}{\pi\sqrt{\lambda}} = 0.137$;

$\alpha = 9$

$\therefore \quad b = \alpha\, d = 9 \times 1.85 = 16.65$ m (say 17 m; for more safe)

Minimum d/s floor length required $= \frac{2}{3} \cdot b = \frac{2}{3} \times 17 = 11.3$ m;

which is greater than 5.0 m, *i.e.,* $5\,(y_2 - y_1)$.

Hence, provide 11.3 m as the d/s floor length.

Glacis length = 2 × 0.2 = 0.4 m

The balance *i.e.,* 17 − 11.3 − 0.3 = 5.3 m is provided as u/s floor length, as shown in the figure.

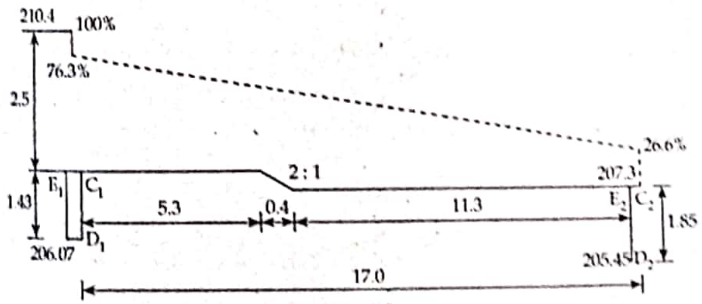


Figure 7.3

Calculation for uplift pressure

u/s cutoff

d = 1.43 m

b = 17.0 m

$\frac{1}{\alpha} = \frac{d}{b} = \frac{1.43}{17.0} = 0.085$

From plate 11.1 (a);

$\phi E_1 = 100\ \%$

$\phi D_1 = 100 - \phi D = 100 - 18.5 = 81.5\ \%$

$\phi C_1 = 100 - \phi E = 100 - 26.5 = 73.5\ \%$

(Assume u/s floor thickness = 0.5 m)

$$\text{Correction to } \phi C_1 \text{ for depth of floor} = \frac{81.5\% - 73.5\%}{1.43} \times 0.5 = 2.8\%\ (+\text{ve})$$

Correction duet to interference is small is very small and neglected.

$\therefore \quad \phi C_1$ (Corrected) $= 73.5\ \% + 2.8\ \% = 76.3\ \%$

Downstream cutoff

$d = 1.85$ m

$b = 17.0$ m

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{1.85}{17.0} = 0.109$$

From plate 11.1(a);

$\phi E_2 = \phi E = 30\ \%$

$\phi D_2 = 21\ \%$

$\phi C_2 = 0\ \%$

(Assume d/s floor thickness near the d/s cut-off = 0.7 m)

$$\text{Correction to } \phi E_2 \text{ for the depth of floor} = \frac{30\% - 21\%}{1.85} \times 0.7 = 3.4\%\ (-\text{ve})$$

Correction duet to interference is small is very small and neglected.

$\therefore \quad \phi E_2$(Corrected) $= 30\ \% - 3.4\% = 26.6\%$

Floor thickness: d/s floor

At toe of glacis;

$$\%\ \text{Pressure at toe of glacis} = 26.6\% + \frac{76.3\% - 26.6\%}{17} \times 11.3 = 26.6\% + 33.1\% = 59.7\ \%$$

Maximum unbalanced head at toe of glacis due to maximum static head (2.7 m) = 59.7 % × 2.7 = 1.61 m

$$\text{Head due to dynamic head action} = 50\ \% \left(y_2 - y_1\right) + \phi \times H_1$$
$$= 50\ \% \,(1.8 - 0.8) + 59.7\% \times 0.2$$
$$= 0.5 + 0.12 = 0.62 < 1.61 \text{ m}$$

Hence, static head governs the thickness.

$$\therefore \quad \text{Thickness required at toe of glacis} = \frac{1.61}{1.24} = 1.3 \text{ m; provide } 1.4 \text{ m}$$

At 3 m beyond the toe of glacis;

$$\%\ \text{pressure} = 26.6\% + \frac{76.3\% - 26.6\%}{17} \times 8.3 = 26.6\% + 24.3\% = 50.9\%$$

Maximum unbalanced head = 59.7 % × 2.7 =1.32 m

$$\text{Thickness required} = \frac{1.32}{1.24} = 1.07 \text{ m; provide } 1.1 \text{ m}$$

At 8 m beyond toe of glacis;

$$\%\ \text{pressure} = 26.6\% + \frac{76.3\% - 26.6\%}{17} \times 3.3$$

$$= 26.6\% + 9.65\ \%$$
$$= 36.25\ \%$$

Maximum unbalanced head = 36.25 % × 2.7 = 0.98 m

$$\text{Thickness required} = \frac{0.98}{1.24} = 0.79 \text{ m}$$

Provide 0.8 m from this point to the end, as shown in the figure.

u/s floor

Theoretically no floor thickness is required under the upstream floor, since the uplift is more than counterbalanced by the weight of the water standing over it. But a nominal thickness of 0.5 m is provided. The floor is thickened to 1.0 m under the crest in a length equal to 2.0 m as shown in figure.

$$\text{Upstream protection shall be provided for scour depth} = \frac{y_u}{3} + 0.6 \text{ m} = \frac{2.5}{3} + 0.6 \text{ m} = 1.43 \text{ m}$$

Launching apron

Provide a launching apron of thickness 1.2 m in length $= \dfrac{2.25 \times 1.43}{1.2} = 2.63$ m

$\therefore$ Volume required = 2.25 cu. m/ metre

Use launching apron = 1.2 m thick laid in a length of 2.8 m.

C.C. blocks

Let us provide C.C. blocks of size 0.8 m × 1.43 = 2.2 m. Hence, 3 rows of C.C. blocks of size 0.8 m × 0.8 m × 0.6 m with 10 cm jhories filled with bajri, laid over packed stone apron of 0.6 m thickness, in a total length of 2.6 m, shall be provided as shown in figure.

Downstream protection

It shall be provided for a scour depth;

$$D = \frac{y_d}{2} + 0.6 \text{ m} = \frac{2.5}{2} + 0.6 \text{ m} = 1.85 \text{ m}$$

Launching apron

Provide a launching apron of thickness equal to 1.2 m.

$$\text{Length of apron required} = \frac{2.25\,D}{t} = \frac{2.25 \times 1.85}{3} = 3.84 \text{ m (say 3.6 m)}$$

Provide a launching apron of 1.2 m thickness laid in a length of 3.6 m.

Inverted filter

$$\text{Length of filter required} = 1.5 \times 1.85 = 2.77 \text{ m}$$

Provide 4 rows of C.C. blocks of size 0.8 m × 0.8 m × 0.6 m with 10 cm gaps in between, laid over an inverted filter of 0.6 m thickness, in a total length of 3.5 m as shown in the figure.

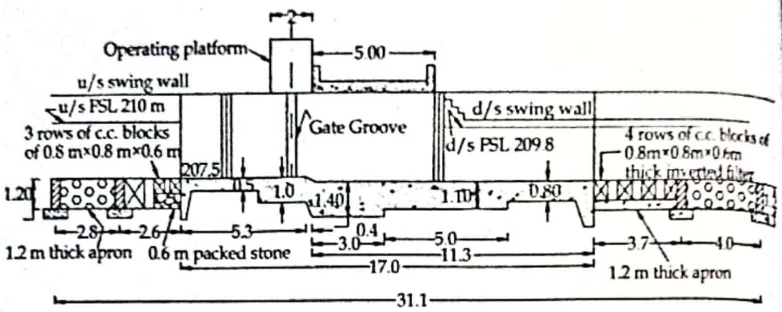


Figure 7.4: Details of cross regulator

Provide 0.4 m thick and 1.2 m deep toe walls between blocks and apron in u/s as well as on d/s.

B. Design of distributary head regulator

We shall first of all determine a suitable regime section for the distributary, so as to carry the given discharge of 16 cumecs with the given silt factor = 0.8

Lacey's regime perimeter (P) = $4.75\sqrt{14} = 19$ m

For small flow width of waterway (Q) = 0.8P = 0.8 × 19 = 15.2 ≃ 16 m

$$V = \frac{Qf^2}{140} = \frac{16 \times (0.8)^2}{140} = 0.65 \text{ m/sec.}$$

and, Depth of water $= \frac{Q}{VB} = \frac{16}{0.65 \times 16} = 1.54$ m

Hence, provide 1.5 m depth.

Bed width of such a channel = 16 m

Depth of water in this channel = 1.5 m

Bed level of Distributary = F.S.L. – Depth = 209.1 – 1.5 = 207.6 m

Crest level

Crest level of distributary head is generally kept 0.3 to 1.0 m higher than the bed level of parent channel. Let the parent channel. Let keep it 0.6 m higher.

∴ Crest level = 207.5 + 0.6 = 208.1 m

Hence, deep the crest of head regulator at R.L. 208.1 m.

Waterway

The discharge is given by;

$$Q = B \cdot \sqrt{h}[1.69\,h + 3.54h_1]$$

where, B is the clear waterway required.

In this case,

h = F.S.L. of parent channel – F.S.L. of Distributary

= 210.0 – 209.1 = 0.9 m.

h_1 = F.S.L. of distributary – crest level = 209.1 – 208.1 = 1.0 m

Q = 16 cumecs

∴ $16 = B\sqrt{0.9}\,[1.69 \times 0.9 + 3.54 \times 1]$

or, $16 = 0.949\,[1.69 \times 0.9 + 3.54]$

$$B = \frac{16}{0.949 \times 5.061} = 3.33 \text{ m}$$

This is very small as compared to bed width as distributary, i.e., 16 m.

Hence, provide 2 bays of 3.0 m each with 1.0 m thick pier in between.

Overall waterway provided = 6 + 1 = 7.0 m

Clear waterway provided = 6.0 m

The wing walls shall be explained with proper divergences, so as to provide the normal width of the channel.

Cistern or d/s floor

Discharge intensity $q = \frac{16}{6} = 2.67$ cumecs/metre

H_L = u/s F.S.L. – d/s F.S.L.

= F.S.L. of parent channel – F.S.L. of distributary

= 210.0 – 209.1 = 0.9 m

From plate 10.1;

$E_{f_2} = 1.8$ m

$E_{f_1} = E_{f_2} + H_L = 1.8 + 0.9 = 2.7$ m

From plate 10.2;

For $E_{f_1} = 2.7$ m;

$y_1 = 0.4$ m

For $E_{f_2} = 1.8$ m;

$y_2 = 1.7$ m

R.L. of cistern (d/s floor) = d/s F.S.L. $-E_{f_2}$ = 209.1 – 1.8 = 207.3 m

Length of cistern required $= 5(y_2 - y_1) = 5(1.7 - 0.4)$

$= 8.0$ m $\left(\text{subjected to minimum of } \frac{2}{3}b\right)$

Vertical cutoff = Depth of u/s cutoff below floor

$$= \frac{y_u}{3} + 0.6 \text{ m}$$

$$= \frac{2.5}{3} + 0.6 \text{ m} = 1.43 \text{ m}$$

Bottom level of u/s cutoff = 207.5 – 1.43 = 206.07 m

Depth of d/s cutoff below floor $= \frac{y_d}{2} + 0.6 \text{ m} = \frac{1.5}{2} + 0.6 \text{ m} = 1.35$ m

Provide d/s cutoff depth = 1.6 m

Bottom of d/s cut-off = 207.3 – 1.6 = 205.7 m

Total floor length from exit gradient considerations

Maximum static head (H) is caused when there is full water in u/s and no water in d/s

H = 210.0 – 207.3 = 2.7 m

d = depth of d/s cutoff = 1.6 m

$$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$$

$G_E = \frac{1}{5}$ (given)

$\therefore \quad \frac{1}{5} = \frac{2.7}{1.6} \times \frac{1}{\pi\sqrt{\lambda}}$

or, $\quad \frac{1}{\pi\sqrt{\lambda}} = \frac{1}{5} \times \frac{1.6}{2.7} = 0.119$

From plate 11.2;

For $\frac{1}{\pi\sqrt{\lambda}} = 0.119$;

$\alpha = 13$

$\therefore \quad b = \alpha d = 13 \times 1.6 = 20.8$ m (say 21 m)

Minimum d/s floor length required $= \frac{2}{3} \times b = \frac{2}{3} \times 21 = 14$ m

Hence, provide 14 m as the d/s floor length.

Length of d/s glacis = 1.6 m

Length of crest = 1.0 m

Length of u/s glacis = 0.6 m

Total = 17.2 m

The remaining 3.8 m is provided on the u/s side, as shown in the figure.

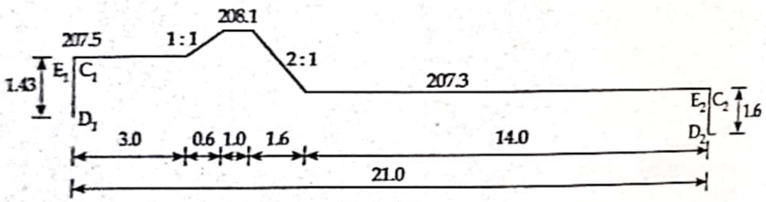


Figure 7.5

Calculation for uplift pressure

i) **u/s cutoff**

$d = 1.43$ m

$b = 21.0$ m

$\frac{1}{\alpha} = \frac{d}{b} = \frac{1.43}{21.0} = 0.068$

From plate 11.1 (a);

$\phi E_1 = 100\%$

$\phi D_1 = 100 - \phi D = 100 - 16.5 = 83.5\%$

$\phi C_1 = 100 - \phi E = 100 - 24 = 76\%$

Assume u/s floor thickness = 0.5 m.

Correction to ϕC_1 for depth of floor thickness $= \frac{83.5\% - 76\%}{1.43} \times 0.5$

$= 2.62\%$ (+ ve)

Correction due to interference is small is very small and neglected; hence,

ϕC_1 (corrected) $= 76\% + 2.62\%$

$= 78.62\%$ (say 78.6%)

ii) **Downstream cutoff**

$d = 1.6$ m

$b = 21.0$ m

$\frac{1}{\alpha} = \frac{d}{b} = \frac{1.6}{21.0} = 0.076$

From plate 11.1 (a);

$\phi E_2 = \phi E = 25.5\%$

$\phi D_2 = \phi D = 17.5\%$

$\phi C_2 = 0\%$

Assume u/s floor thickness = 0.8 m.

Correction to ϕE_2 for depth of floor thickness $= \frac{25.5\% - 17.5\%}{1.6} \times 0.8$

$= 4\%$ (−ve)

Correction duet to interference is small is very small and neglected; hence,

ϕE_2 (corrected) $= 25.5\% - 4\% = 21.5\%$

Floor thickness: d/s floor

At toe of glacis;

% Pressure $= 21.5\% + \frac{78.6\% - 21.5\%}{21} \times 14 = 21.5\% + 38.1\% = 59.6\%$

Maximum unbalanced head at toe of glacis due to the maximum static head (2.7) is;

$= 59.6\% \times 2.7 = 1.58$ m

Head due to dynamic head action $= 50\%(y_2 - y_1) + \phi \times H_L$

$= 50\%(1.7 - 0.4) + 59.6\% \times 0.9$

$= 0.65 + 0.54 = 1.19 \text{ m} < 1.58$ m

Hence, static head governs the thickness.

$\therefore$ Thickness required at toe of glacis $= \frac{1.58}{1.24} = 1.28$ m; provide 1.4 m.

At 3 m beyond the toe of glacis;

% Pressure $= 21.5\% + \frac{78.6\% - 21.5\%}{21} \times 11 = 21.5\% + 29.9\% = 51.4\%$

Maximum unbalanced head $= 51.4\% \times 2.7 = 1.39$ m

Thickness required $= \frac{1.39}{1.24} = 1.12$ m; provide 1.2 m

At 6 m beyond toe of glacis;

% Pressure $= 21.5\% + \frac{78.6\% - 21.5\%}{21} \times 8 = 21.5\% + 21.8\% = 43.3\%$

Maximum unbalanced head $= 43.3\% \times 2.7 = 1.17$ m

Thickness required $= \frac{1.17}{1.24} = 0.945$ m; provide 1 m

At 10 m beyond toe of glacis;

% Pressure $= 21.5\% + \frac{78.6\% - 21.5\%}{21} \times 4 = 21.5\% + 10.9\% = 32.4\%$

Maximum unbalanced head $= 32.4\% \times 2.7 = 0.875$ m

$$\text{Thickness required} = \frac{0.875}{1.24} = 0.705 \text{ m}$$

Provide 0.8 m up to the end, as shown in the figure.

U/s floor

Provide a nominal thickness of 0.5 m under the floor and extend it under the crest and then join it to the bottom of d/s glacis, as shown in figure.

$$\text{Upstream protection shall be provided for a scour depth} = \frac{y_u}{3} + 0.6 \text{ m}$$

$$= \frac{2.5}{3} + 0.6 \text{ m}$$

$$= 1.43 \text{ m}$$

Downstream protection

It shall be provided for a scour depth.

$$D = \frac{y_d}{2} + 0.6 \text{ m} = \frac{1.5}{2} + 0.6 \text{ m} = 1.35 \text{ m}$$

Launching apron

Provide a launching apron of thickness equal to 1.2 m.

$$\text{Length of apron required} = \frac{2.25\,D}{t} = \frac{2.25 \times 1.35}{1.2} = 2.54 \text{ m (say 2.6 m)}$$

Provide a launching apron of 1.2 m thickness laid in a length of 2.6 m.

Inverted filter

$$\text{Length of filter required} = 1.5 \times 1.35 = 2.03 \text{ m}$$

Provide 3 rows of C.C. blocks of size 0.8 m × 0.6 m with 10 cm gaps in between, laid over an inverted filter of 0.6 m thickness.

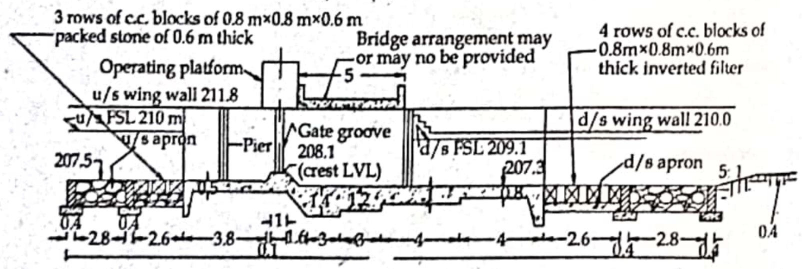


Figure 7.6: Details of distributory head regulator

Types of canal escapes

Canal escapes may be of following two types depending on the structure.

I) Weir types

In this type, the crest of the weir wall is kept at R.L. equal to canal F.S.L., as shown in the figure when water level rises above F.S.L., it gets escaped.

II) Regulator types (sluice type)

In this type, the sill of escape is kept at canal bed level and the flow is controlled by gates as shown in figure. These types of escapes are preferred these days, as they give better control and can be used for complexly emptying the canal.

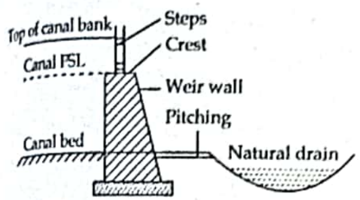


Figure 7.7: Weir type escapes

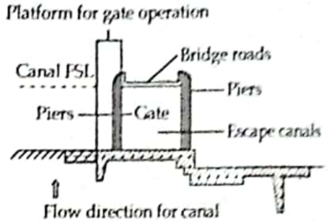


Figure 7.8: Regulator type escapes

This type may be constructed for the purpose of scouring of excess bed silt deposited in head reaches, from time to time and called scouring escape also.

Depending upon the purpose, there are three types of escapes:

i) Canal scouring escape

It is constructed for the purpose of scouring of excess bed silt deposited in the head reaches from time to time.

ii) Surplus escape

It is used to wasting excess of water.

iii) Tail escape

Provided at the tail end of the anal and used for maintaining required F.S.L. in the tail reaches of canal.

Design

The sill of the regulator is generally kept at about 0.3 m below the canal bed level at the escape site. The regulator type escape can be design like a head regulator without any raised crest.

7.4 TYPES OF OUTLET, DESIGN OF OUTLET (FREE AND SUBMERGED)

Types of outlets

The various types of outlets can be classified into the three classes.

i) Non-modular outlets

These are such types of outlets in which discharge depends upon the difference of head between the distributary and the water courses. Thus the discharge through the outlet varies by the variation of the level of the distributaries of water-courses. *For example;* open sluice, drowned pipe outlets

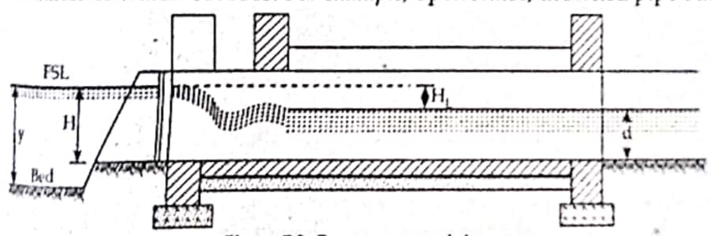


Figure 7.9: Open masonry sluice

Working principles of non-modular outlet

i) The discharge passing through outlet depends on the level difference of F.S.L. of distributory and water courses and cross-section area of the outlet.

ii) Required cross-section of outlet for required discharge is designed according to working head H_L with formula.

a) For open sluice,

$$Q = C_d\sqrt{2gH_L} \cdot B\left(H - \frac{1}{3}H_L\right)$$

Here, B is calculated.

b) For submerged pipe outlet

$$Q = C_d \cdot A\sqrt{2gH_L}$$

Here, A is calculated.

iii) Discharge is regulated by providing a shutter on u/s end. Arrangement is provided to lock the shutter in any required position to required discharge.

Advantage

i) Can work in low available working head.
ii) Minimum head loss.
iii) Silt conduction is also quite good.
iv) Easy and simple to construct and regulate.
v) Cheaper.

Disadvantage

i) Unequal distribution of water.
ii) Increase in bed level and vegetation in water courses decreases discharge.

II) Semi modular or flexible modules

These are such types of outlets in which discharge is independent of the water level of water-courses but depends only upon the water level of the distributary so long as the minimum working head is available. The common examples of these types of outlets are pipe outlets, venture flume, open flume, and orifice semi flume.

Advantage

a) Equal distribution of water level irrespective of FSL of water course FSL.
b) Free pipe outlet is simple and easy to construct.
c) Silt conduction is quite good and efficiency is high.

Disadvantage

a) Available level difference between distributary and watercourse may be insufficient.
b) Except free pipe outlet other types; semi-modular outlet is costly.
c) Silt conduction is quite good and efficiency is high.

III) Rigid modules or modular outlets

These are such types of outlets through which discharge is constant or fixed within limits, irrespective of the fluctuations of the water levels of either of distributary or of the water courses or both. *For example;* Gibb's module

Advantage

a) Equal distribution of water level irrespective of distributary and water course FSL within certain working range.

Disadvantage

a) It is costly and requires skill manpower to construct.

Requirement of good module

a) It should fit well according to the principle of water distribution *i.e.*, if supply is to fix then discharge is fix and if supply is to be changed capable of changing discharge time to time.
b) It is simple so that easy to construct and fabricate by local material and technicians.
c) It should be cheaper since required in large number.
d) It should work efficiently with small working head.
e) It should be strong and durable.
f) Any interference by local people should be made difficult and easily detectable.

Flexibility

Flexibility is defined as the ratio of the rate of the change of the discharge of the outlets to the rate of the change of the distributary channel. *i.e.*, $F = \dfrac{\frac{dq}{q}}{\frac{dQ}{Q}}$

$$\text{Flexibility} = F = \frac{m}{n} \times \frac{y}{H}$$

where, m is the constant depending upon the type of outlet such that; $\dfrac{dq}{q} = m\dfrac{dW}{W}$.

n is the constant depending upon distributary; $\dfrac{dQ}{Q} = n\dfrac{dy}{y}$.

y is the depth of water in distributary.

H is the head acting on the outlet.

Proportionality

The outlet is said to proportional when rate of the change of outlet discharge equals the rate of change of the channel discharge *i.e.*, when flexibility is equals to 1. Hence for a proportional outlet,

$$F = \frac{m}{n} \times \frac{y}{H} = 1$$

or, $$\frac{H}{y} = \frac{m}{n} = \frac{\text{Outlet index}}{\text{Channel index}}$$

Sensitivity

Sensitivity is defined as the ratio of the change of the discharge through the outlet to the rate of change of the water level of the distributary, referred to the normal depth of the channel. *i.e.*, $F = \dfrac{\frac{dq}{q}}{\frac{dy}{y}}$

For rigid modules discharge is fixed and sensitivity is zero.

Relation between flexibility and sensitivity

We have,

$$\text{Flexibility (F)} = \frac{\frac{dq}{q}}{\frac{dQ}{Q}}$$

and, $\frac{dQ}{Q} = n\frac{dy}{y}$

i.e., $F = \frac{\frac{dq}{q}}{\left(\frac{ny}{y}\right)dy} = \frac{1}{n}\left(\frac{\frac{dq}{q}}{\frac{dy}{y}}\right)$

or, $F = \frac{S}{n}$

$\therefore \quad S = n \cdot F$

Setting

The ratio $\frac{H}{y}$ *i.e.*, the ratio of the depth of the sill level of the outlet below the F.S.L. of the distributary, to the full supply depth of the distributary, is known as setting. Thus, for a proportional outlet;

$$\frac{\text{Outlet index}}{\text{Channel index}} = \frac{H}{y},$$ *i.e.*, outlet setting should be made.

Minimum modular head

The minimum difference between the upstream and downstream water levels, which is required to be maintained so as to enable the module to pass the designed discharge, is known as minimum modular head.

Efficiency of outlet

It may be defined as the ratio of head recovered to the head put in. Lesser is the head required for functioning of the outlet; more efficient the outlet will be. It is the measure of the conservation of head by the outlet.

Drowning of ratio

It is the ratio of the depth of water level over crest on the d/s of the module to the depth of the water level over crest on the u/s of the module. In case of weir type outlet, the efficiency is the same as the drowning ratio.

Modular limit and modular range

The modular limits are the extreme values of any one or more variables, beyond which and outlet becomes incapable of acting as a module or semi module. The range between the lowest and highest limiting values of various such factors is known as modular range.

Design of pipe outlets

I) Design of submerged pipe outlets

The two types of pipe outlets are shown in figure 7.10 and 7.11. 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 figure (ii) so as to increase silt conductivity. The pipes are generally laid about 21 cm. below the water surface level of the distributary channel.

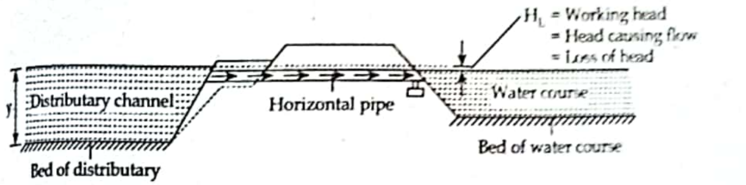


Figure 7.10: Horizontal pipe outlet (submerged)

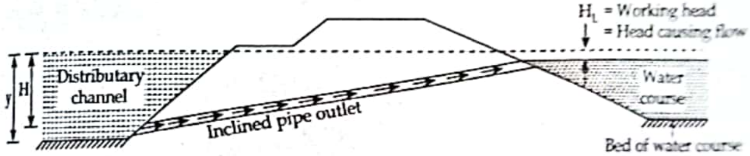


Figure 7.11: Inclined pipe outlet (submerged)

The velocity through pipe can be precisely computed by using the relation;

H_L = Total loss of head

= Entry loss + Frictional loss + Velocity head at exit

or, $$H_L = 0.5\frac{V^2}{2g} + \frac{f'lV^2}{2gd} + \frac{V^2}{2g}$$

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 water course.

l is the length of the pipe.

d is the diameter of the pipe.

f' is the coefficient of friction of the material of the pipe.

After computing the velocity from above equation discharge 'q' can be computed as;

Q = v x Area of outlet pipe

The discharge, however for all practical purposes, may be easily computed by using the simple relation;

$$Q = C_d A\sqrt{2gH_L}$$

where, Q is the discharge through the outlet.

C_d is the coefficient of discharge for the pipe outlet. Its average value is 0.73. Although it varies as the length, size and material of the pipe.

A is the cross-section area of pipe.

H_L is the Difference of head between the F.S.L. of the distributary and F.S.L. of the water course.

Example 7.2

Design an irrigation outlet for the following data.
F.S.Q. of outlet = 60 lit./sec.
F.S.L. of distributary on u/s side of outlet = 200.00 m
F.S.L. in water course on d/s side of outlet = 199.92 m
Depth in distributary on u/s side of outlet = 1.05 m

Solution:

Available head across outlets = F.S.L. of distributary − F.S.L. of water course

$= 200.00 - 199.92$
$= 0.08$ m

Since the available head is very small, a. non-modular outlet (such as submerged pipe outlet) will have to be provided.

$$Q = 60 \text{ lit./sec.} = \frac{60}{1000} = 0.06 \text{ m}^3/\text{sec.}$$

and, $C_d = 0.73$

Now,

Discharge in such submerged pipe outlets, $Q = C_d A\sqrt{2gH}$

or, $0.06 = 0.73\,A\,\sqrt{2 \times 9.81 \times 0.08}$

or, $\left(\frac{\pi d^2}{4}\right) = 0.063 \text{ m}^2$

or, $d = 0.029$ m

Here, use a pipe of diameter 30 cm, say

R.L. of the bed of the distributary = 200.00 − 1.05
= 198.95 m

Let, top of the pipe be at the level 22 cm below the F.S.L. of the distributary.

Lower level of the pipe = 200.00 − 0.22 − 0.03
= 199.484 m

II) Design of the free pipe outlet

- Pipe outlet discharge freely into the atmosphere.
- Simplest and oldest type of outlets.
- Discharge through free pipe outlet only depends upon the water level of the distributary.
- Necessity of sufficient level difference.
  The discharge can be computed by the equation,

$$Q = C_d A\sqrt{2gH_0}$$

where, C_d is coefficient of discharge = 0.62 for general condition of free out fall.
H_0 is the head on u/s since measured from F.S.L. of distributary up to the centre of the pipe outlet.
A is the area of cross section of pipe.

Example 7.3

Design a pipe outlet for the following data F.S.L.
Full supply discharge at the head of water course = 100 lit./sec.
F.S.L. in distributary = 205.00 m
F.S.L. in water course = 204.00 m

Solution:

Available head across the outlet = 205.00 − 204.00 = 1 m

Here,

$C_d = 0.62$

and, $Q = 0.10 \text{ m}^3/\text{sec.}$

Now, the discharge through such an outlet is given as;

$Q = C_d A\sqrt{2gH_0}$

Assuming the diameter of the pipe as 25 cm; we have,

$$0.10 = 0.62\left\{\frac{3.14 \times 0.25 \times 0.25}{4}\right\}\sqrt{2gH_0} = 0.62 \times 0.049 \times 4.43\sqrt{H_0}$$

or, $H_0 = 0.55$ m

R.L. of centre of pipe outlet = 205.00 − 0.55 = 204.45 m

R.L. of invert of pipe outlet (sill level) $= 204.45 - \frac{0.25}{2} = 204.325$

> FSL of Water course *i.e.*, 204 m

Hence, pipe of 25 cm dia. can be laid horizontally with its bottom or sill level 204.325 m discharging freely as semi-module.

7.5 TYPES OF FALLS (DROPS)

Some important types of falls are as follows:

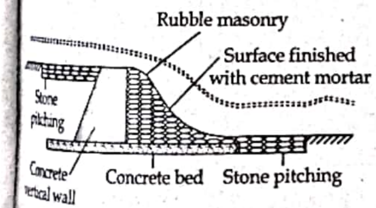


Figure 7.12: Ogee fall

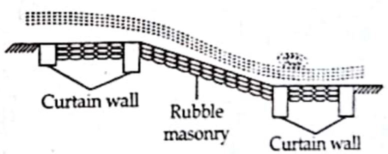


Figure 7.13: Rapid fall

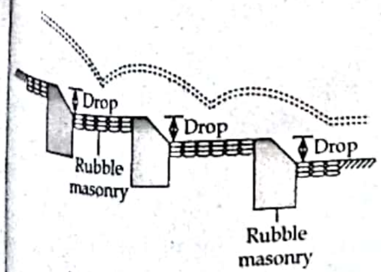


Figure 7.14: Steeped fall

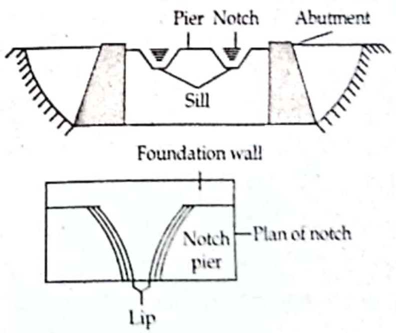


Figure 7.15: Trapezoidal fall

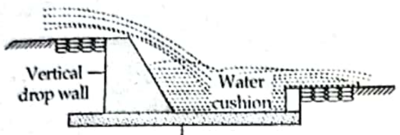


Figure 7.16: Vertical drop fall

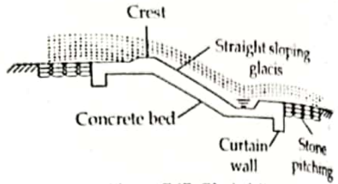


Figure 7.17: Glacis fall

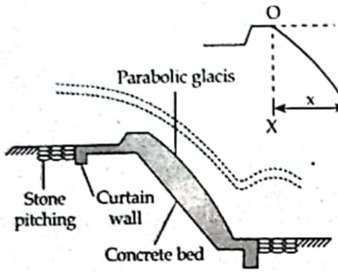


Figure 7.18: Montague type fall

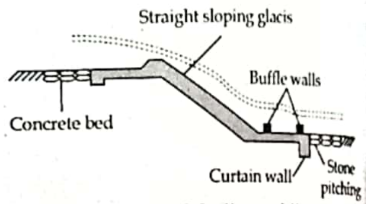


Figure 7.19: Inglis type fall

i) Ogee falls

The ogee type fall is constructed in olden days on project like Ganga canal. This was gradually led down by providing convex and concave curves, as shown in the figure.

ii) Rapids

Rapids were provided with long sloping floors (called glacis) having gentle slopes in the range of 1in 10 to 1 in 20.

iii) Stepped falls

The stepped falls, in which long glacis of rapid is replaced by long, steeped floor.

iv) Trapezoidal notch falls

It consists of a number of trapezoidal notches constructed in a high crested wall across the channel with a smooth entrance and a flat circular lip projecting downstream from each notch to spread out the falling jet.

v) Vertical drops fall

In a vertical drop 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.

vi) Straight glacis fall

This is the modern type fall. A straight glacis (generally sloping 2 : 1) is provided after a raised crest. The hydraulic jump is made to occur on the glacis, causing sufficient energy dissipation.

vii) Montague type fall

This is the improvement over straight glacis fall by replacing the straight glacis by a parabolic glacis.

viii) Inglis falls or baffle falls

Inglis fall is also a modified form of glacis fall by providing a baffle wall of a certain height at some distance d/s of the toe of the glacis.

ix) Sarda fall

This is the raised crest type fall and a type of vertical drop fall. Neppe leaving the crest provides the water in the cistern resulting in the destruction of energy.

Design of vertical drop

Design of Sarda type of fall

The design criteria of fall are based on the recommendations of Bahadarbad research station.

i) **Length of the crest**

- Bed width of the canal.
- For future expansion (bed width + depth of water).

ii) **Shape of the crest**

Top width of the crest (B_t) = $0.55\sqrt{d}$ (maximum)

Thickness at base (B_t) = $\frac{(h+d)}{G}$ (minimum)

where, d = Height if the crest above d/s bed level.

h = Head over the crest.

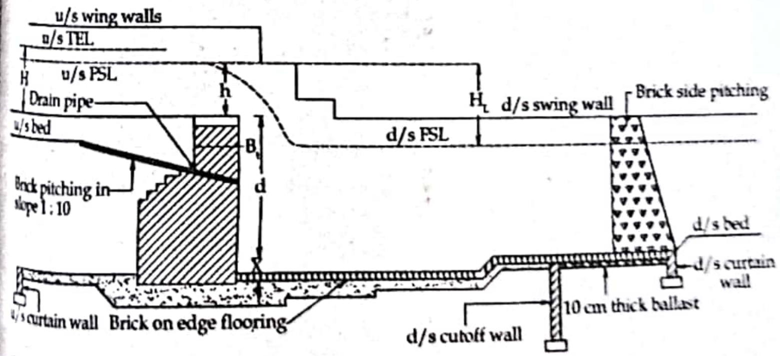
Take G = 2 for masonry.

For discharge (Q) > 14 cumecs

Trapezoidal crest with u/s side slope of 1 : 3 and d/s side slope of 1 : 8

Top width of crest (B_t) = $0.55\sqrt{H+d}$

iii) **Crest level**



Q = Up to a maximum of 14 cumecs

B_t = Top width of crest = $0.55\sqrt{d}$

Base width = $\frac{H+d}{2}$

$Q = 1.84L(H)^{\frac{3}{2}}\left(\frac{H}{B_t}\right)^{\frac{1}{6}}$

Figure7.20: Rectangular crest for Sarda type fall

The following discharge formula is used to determine height of water level above the crest.

$$Q = C_d\sqrt{2g}\cdot L\cdot H^{\frac{3}{2}}\cdot\left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

where, $C_d = 0.415$ for rectangular crest.

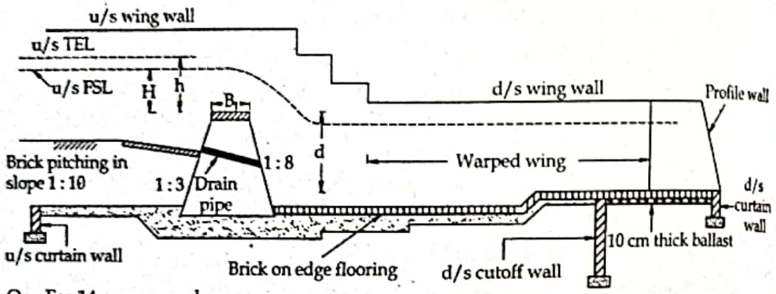
0. 45 for trapezoidal crest.

L is the length of crest

B_t is the top width of the crest.

Height of crest above u/s bed level = y – h; where, y is the normal depth of channel u/s.

= y – H (neglecting velocity head)



Q = For 14 cumecs and over

B_t = Top width of crest = $0.55\sqrt{H+d}$

Base width = $\frac{h+d}{2}$

$$Q = 1.99L(H)^{\frac{3}{2}}\left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

Figure 7.21: Trapezoidal crest for Sarda type fall

iv) Upstream wing wall

For rectangular crest, the approach wings may be splayed straight at 45° and for trapezoidal crest as shown in the figure below.

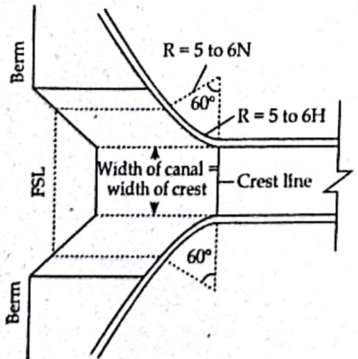


Figure 7.22: u/s wing walls for trapezoidal crest of sarda type fall

v) Upstream protection

Brick pitching in length equal to upstream water depth, at sloping towards crest at a slope of 1 : 10 with drain pipes.

vi) Upstream curtain wall

1.5 brick thick upstream of depth $\frac{1}{3}$ of water depth.

vii) Impervious concrete floor

- Total length of impervious floor can be determined by Bligh's theory floor small works and Khosla's theory for large works.
- Minimum length of d/s floor of the toe crest wall = 2(Water depth + 1.2 m) + Drop and remaining provide in u/s
- Thickness of d/s floor shouldn't be less than 0.4 m and nominal thickness of 0.3 m is provided in u/s side.

viii) Cistern

Length of cistern in meters is given by;

$$L_c = 5\sqrt{H\cdot H_L}$$

Cistern depression below downstream bed in meters is given by;

$$X = \frac{1}{4}(H\cdot H_L)^{\frac{2}{3}}$$

where, H is the head of water over the crest.

Including velocity head (m) = (u/s T.E.L. – Crest level)

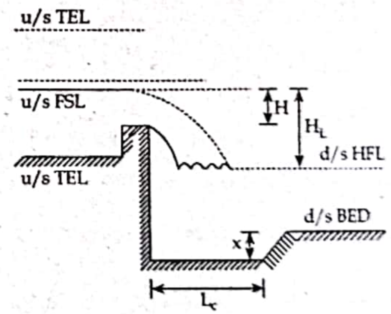


Figure 7.23

ix) Downstream protection

- The d/s bed may be protected with dry brick pitching about 20 cm thick resting on 10 cm ballast.
- The length of the d/s pitching is given by the values of table; or 3 × depth of d/s water whichever is more.
- The pitching may be provided between two or three curtain walls.
- The curtain wall may be 1.5 brick thick and of depth = 0.5 × d/s depth or as given in table.

Head over crest	Total length of d/s pitching	Remarks	Curtain walls	
			No.	Depth in metres
Up to 0.3 m	3.0		1	0.30
0.3 to 0.45	$3.0 + 2\times H_L$	All sloping down at 1 in 10 horizontal up to end of masonry wings and then sloping down at 1 : 10	1	0.30
0.45 to 0.60	$4.5 + 2\times H_L$		1	0.45
0.60 to0.75	$6.0 + 2\times H_L$		1	0.60
0.75 to 0.90	$9.0 + 2\times H_L$		1	0.75
0.90 to 1.05	$13.5 + 2\times H_L$		2	0.90
1.05 to 1.2	$18.0 + 2\times H_L$		2	1.05
1.2 to 1.5	$22.5 + 2\times H_L$		3	1.35

Table 7.1

$$\text{Depth of d/s cut of wall} = \frac{y_d}{2} + 0.6 \text{ m}$$

Design of vertical drop weir on Bligh's theory

Even this theory is replaced by modern Khosla's theory; it is still used especially for minor works owing to its simplicity. Some empirical formulas for the length of u/s and d/s given by Bligh's are given below.

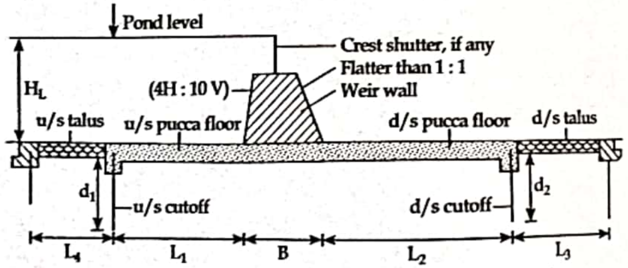


Figure 7.24: Dimensions of vertical drop weir based on Bligh's theory

i) Length of d/s pucca floor from toe of weir

$$L_2 = 2.21C\sqrt{\frac{H_L}{13}};\ \text{for weir having crest shutters.}$$

$$L_2 = 2.21C\sqrt{\frac{H_L}{10}};\ \text{for weir having no crest shutters.}$$

ii) $L_2 + L_3 = 18C\sqrt{\frac{H_L}{13}\frac{q}{75}}$; for weir having crest shutters.

$$L_2 + L_3 = 18C\sqrt{\frac{H_L}{10}\frac{q}{75}};\ \text{for weir having no crest shutters.}$$

where, q is the discharge intensity in cumecs/meter.

L_3 is the length of d/s loose stone talus.

Length of u/s talus (L_4) may be kept half of d/s talus.

Design of weir wall

By Bligh's empirical formulas

$$\text{Top width of weir wall } (B') = \sqrt{\frac{H}{G-1}}$$

where, H is the head of water over the weir wall at time of maximum flood.

G is the specific gravity of floor material.

and, $B' = 0.6 +$ Height of crest shutters (for having crest shutters)

$$\text{Bottom width } (B') = \sqrt{\frac{H + \text{Height of weir}}{G-1}}$$

Example 7.4

Design a 1.5 meters Sarda type fall a canal having a discharge of 10 cumecs, with the following data.
Bed level of channel = 103.0 m
Side slopes of channel = 1 : 1
Bed level downstream = 101.5 m
Full supply level upstream = 104.5 m
Bed width u/s and d/s = 8.5 m
Soil good loam = Good loam
Assume Bligh's coefficient = 6

Solution:

Length of crest

Same as d/s bed width = 8.5 m

Crest level

A rectangular crest is provided, since the discharge is less than 14 cumecs.

Using discharge formula; we have,

$$Q = 1.84 \cdot L \cdot H^{\frac{3}{2}} \cdot \left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

Assume, top width of the crest as 0.85 m.

$$10 = 1.84 \times 8.5 \times H^{\frac{3}{2}} \times \frac{H^{\frac{1}{6}}}{(0.85)^{\frac{1}{6}}}$$

or, $H^{\frac{5}{3}} = \frac{10 \times 0.974}{1.84 \times 8.5} = 0.623$

or, $H = (0.623)^{\frac{3}{5}} = 0.753$ m (say H =0.76 m)

$$\text{Velocity of approach } (V_a) = \frac{\text{Discharge}}{\text{Area}} = \frac{10}{(8.5 + 1.5)1.5} = 0.67 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V_a^2}{2g} = 0.023 \text{ m}$$

U/s T.E.L. = u/s F.S.L. + Velocity head = 104.5 + 0.023 = 104.523 m

R.L. of the crest = u/s T.E.L. − H = 104.523 − 0.753 m = 103.77 m

Height of crest above u/s floor = 103.77 − 103.0 = 0.77 m

Shape of the crest,

$$\text{Width of the crest } (B_t) = 0.55\sqrt{d}$$

d = Height of the crest above d/s bed

$= 103.77 - 101.5$

$= 2.27\text{m}$

$B_t = 0.55\sqrt{d} = 0.55\sqrt{2.27} = 0.825 \text{ m} < 0.85 \text{ m}$ (O.K.)

Keep 0.85 m width of crest.

$$\text{Thickness at base} = \frac{(h+d)}{2}$$

$$= \frac{(0.755 - 0.025) + 2.27}{2} = 1.5 \text{ m}$$

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° from the u/s edge of the crest and shall be embedded by the berm. On the d/s side, wing wall are kept straight and parallel up to the end of the floor and joined to return walls, as shown in the figure.

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.

$$\text{Upstream curtain wall} = \frac{y_u}{3} = \frac{1.5}{3} = 0.5 \text{ m}$$

Provide 0.4 m × 0.5 m deep curtain wall over 0.3 m thick cement concrete on the wall on the u/s.

Cistern

$$\text{Depth of cistern (X)} = \frac{1}{4}(H \cdot H_L)^{\frac{2}{3}}$$

$$= \frac{1}{4} \times (0.76 \times 1.5)^{\frac{2}{3}} = 0.273 \text{ (say 0.3 m)}$$

∴ R.L. of cistern $= 101.5 - 0.3 = 101.2$ m

$$\text{Length of the cistern} = 5\sqrt{H \cdot H_L}$$

$$= 5\sqrt{0.76 \times 1.5} = 5.34 \text{ m (say 5.5 m)}$$

Provide 5.5 m long cistern at R.L. 101.2 m.

Impervious floor

Maximum static head = (Crest level – d/s bed level)

$= 103.77 - 101.50 = 2.27$ m

Total length of required = C.H = $6 \times 2.27 = 13.62$ m (say 13.7 m)

where, C is Bligh's coefficient and H is the maximum static head.

Minimum d/s floor length required = $[2(\text{water depth} + 1.2) + H_L]$

$= 2(1.5 + 1.2) + 1.5$

$= 2(2.7) + 1.5$

$= 5.4 + 1.5 = 6.9$ m

Provide 7 m d/s floor and the balance 6.7 m under and u/s of the crest, as shown in the figure.

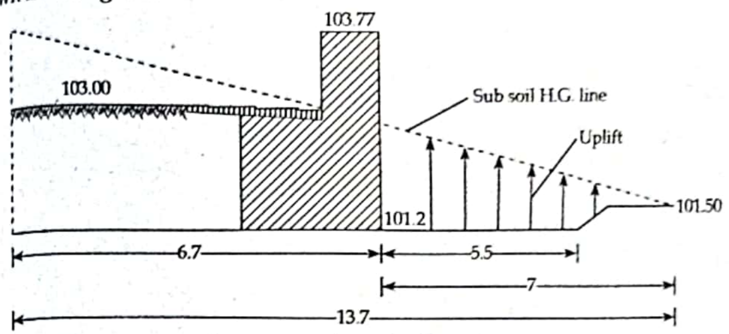


Figure 7.25

Floor thickness H.G. line for the maximum static head is shown in the figure.

$$\text{Max. unbalanced uplift at d/s toe of crest} = 0.3 + \frac{(103.77 - 101.5)}{13.7} \times 7$$

$= 0.3 + 1.16$

$= 1.46$ m

$$\text{Thickness required} = \frac{1.46}{1.24} = 1.29 \text{ m (say 1.3 m)}$$

Provide 1.1 m thick concrete overlain with 0.2 m thick brick pitching.

$$\text{Unbalanced head at 3 m form crest} = 0.3 + \frac{(2.27)}{13.7} \times 4$$

$= 0.3 + 0.67$

$= 0.97$ m

$$\text{Thickness required} = \frac{0.97}{1.24} = 0.789 \text{ m (say 0.8 m)}$$

Use 0.6 m thick concrete with 0.2 m brick layer.

$$\text{Unbalanced head at 5 m from toe} = 0.3 + \frac{(2.27)}{13.7} \times 2$$

$= 0.3 + 0.33$

$= 0.55$ m

Use 0.35 m thick concrete with 20 cm thick brick layer, as shown in the figure.

D/s curtain wall

The curtain wall at the d/s end of the floor should be 0.75 deep (H = 0.76 m in Table 7.1)

or, Depth of curtain wall = 0.5 × d/s depth

$= 0.5 \times 1.5 = 0.75$ m

Cut-off wall

$$\text{Depth of cut off wall} = \frac{1.5}{2} + 0.6 = 1.35 \text{ m}$$

Provide 0.4 m × 1.35 m deep cut of walls over 0.3 m thick cement concrete at the end of floor *i.e.*, up to a level of $101.5 - 1.65 = 99.58$ m *i.e.*, deepest foundation level.

D/s pitching length = 9 + 2 × 1.5

= 12 m (according table 7.1)

Pitching is kept sloped at 1 : 10. A curtain wall of 0.4 m × 0.75 m over 0.3 m thick cement concrete shall be provided at the end of the pitching, as shown in the figure.

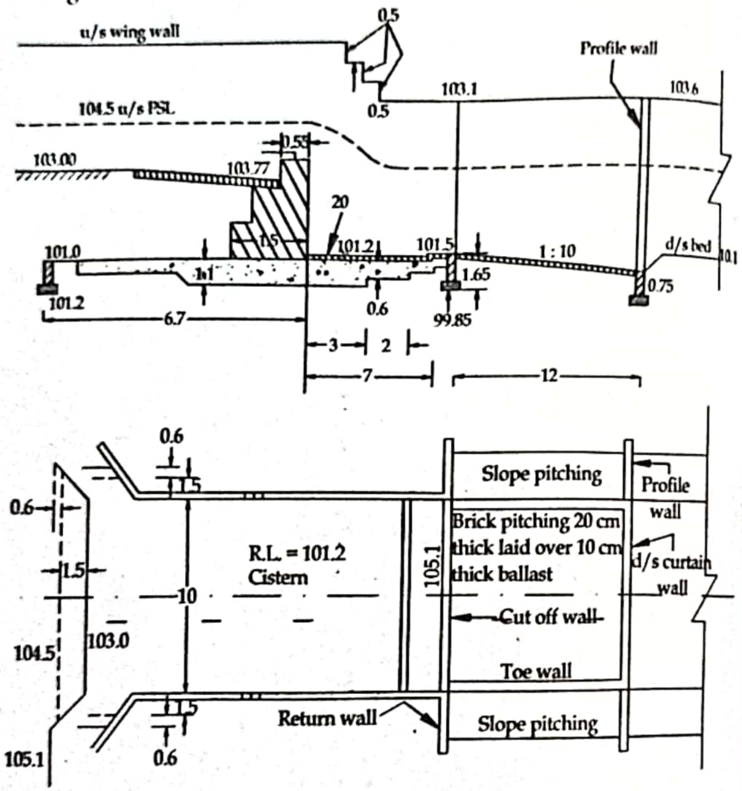


Figure 7.26: Details of Sarda type fall (rectangular crest)

Example 7.5

Design a 1.5 meters Sarda Type fall for a canal carrying a discharge of 70 cumecs with the following data.
Bed level u/s = 105.0 m
Bed level d/s = 103.5 m
Side slopes of channel = 1 : 1
Full supply level u/s = 106.8 m
Full supply level d/s = 105.3 m
Berm level u/s = 107.4 m
Bed width u/s and d/s = 45 m
Safe exit gradient for Khosla's theory = $\frac{1}{5}$

Solution:

Length of crest

It is kept equal to the bed width = 45 meters.

Crest level

A trapezoidal crest is provided, since the discharge is more than 14 cumecs. The discharge formula is given by;

$$Q = 1.99 \cdot L \cdot H^{\frac{3}{2}} \cdot \left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

Assume top width of the crest as 1.0 m.

$$\therefore \quad 60 = 1.99 \times 45 \times H^{\frac{3}{2}} \times \frac{H^{\frac{1}{6}}}{(1)^{\frac{1}{6}}}$$

$$\text{or,} \quad H^{\frac{5}{3}} = \frac{70}{1.99 \times 45} = 0.782$$

$$\text{or,} \quad H = (0.782)^{\frac{3}{5}} = 0.863 \text{ m; Say } H = 0.865 \text{ m.}$$

$$\text{Velocity of approach } (V_a) = \frac{\text{Discharge}}{\text{Area}} = \frac{70}{(45 + 1.8)1.8}$$

$$= 0.83 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V_a^2}{2g} = 0.035 \text{ m}$$

U/s T.E.L. = u/s F.S.L. + Velocity head = 106.8 + 0.035 = 106.835 m

R.L. of crest = u/s T.E.L. − H = 106.835 − 0.865 m = 105.97 m

Adopt a crest level of 105.97 m.

Shape of the crest

Width of the crest $(B_t) = 0.55\sqrt{H + d}$

d = Height of the crest above d/s bed

= 105.97 − 103.5

= 2.47m

$B_t = 0.55\sqrt{0.865 + 2.47} = 0.55\sqrt{3.335} = 1.0$ m

Adopt a trapezoidal crest with top width of 1.0 m and u/s slope 1 : 3 and d/s 1 : 8

Upstream wings walls

Radius of the wings should be 5 to 6 times the head over the crest = 5 × 0.865 = 4.325 to 6 × 0.865 = 5.19 m. Use 5 m radius for the wings. U/s wing wall shall be kept segmental with 5 m radius subtending an angle of 60° at centre and then carried tangentially into the berm.

Downstream wing wall

The d/s wings shall be kept straight up to a distance say $6\sqrt{H + H_L}$, i.e., $6\sqrt{0.865 + 1.5} = 6.8$ m; say 7 m, and then warped in a slope of 1 : 1 and shall be taken up to 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 = $\left(\frac{1}{3}\right)^{rd}$ 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 the curtain wall of 0.4 m × 1 m on the u/s.

Downstream curtain wall

$$\text{Minimum thickness} = \frac{\text{Depth}}{2} = \frac{1.8}{2}$$

$$= 0.9 \text{ m or from table 7.1, it is equal to } 0.75 \text{ m.}$$

Provide a d/s curtain wall 0.4 m × 1 m over 0.3 m cement concrete. Thus, total depth of d/s curtain wall shall be 1.3 m.

Cutoff wall

$$\text{Depth of cutoff wall} = \frac{y_d}{2} + 0.6 = \frac{1.8}{2} + 0.6 = 1.5 \text{ m}$$

Provide cutoff wall of 0.4 × 1.5 m over 0.3 m thick cement concrete.

Total depth of cutoff wall = 1.8 m with bottom level at 101.7 m

Cistern

$$\text{Depth of cistern (X)} = \frac{1}{4}(H \cdot H_L)^{\frac{2}{3}} = \frac{1}{4} \times (0.865 \times 1.5)^{\frac{2}{3}} = \frac{1}{4} \times 1.19$$

$$= 0.3 \text{ m (say)}$$

$$\text{R.L. cistern} = 103.5 - 0.3 = 103.2 \text{ m}$$

$$\text{Length of cistern} = 5\sqrt{H \cdot H_L}$$

$$= 5\sqrt{0.865 \times 1.5}$$

$$= 5 \times 1.14 = 5.7 \text{ m}$$

Provide 5.7 m long cistern.

Total floor length and exit gradient

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$$

where, H is maximum static head.

Maximum head (H) is caused when water is stored up to the crest level and there is no water in d/s.

$$H' = 105.97 - 103.5 = 2.47 \text{ m}$$

d = 1.8 m (*i.e.*, Depth of d/s cut-off wall)

$$G_E = \frac{1}{5} \text{ (given)}$$

$$\therefore \quad \frac{1}{5} = \frac{2.47}{1.8} \times \frac{1}{\pi\sqrt{\lambda}}$$

or, $b = 10 \times 1.7 = 18$ m; Use 18 m.

$$\text{Minimum floor length required on the d/s} = 2(\text{water depth} + 1.2) + H_L$$

$$= 2(1.8 + 1.2) + 0.865$$

$$= 6.865 \text{ m (say 7 m)}$$

Provide balance length of 11 under and u/s of crest, as shown in the figure.

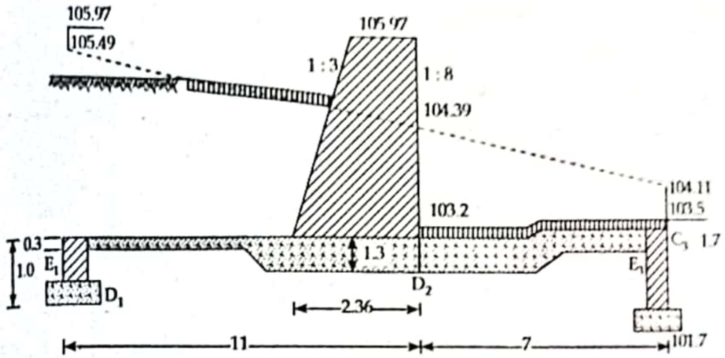


Figure 7.27

Uplift pressure calculation

Assume u/s floor thickness as 0.5 m, and d/s floor thickness as 0.8 m, and floor thickness at toe of the crest as 1.3 m.

I) **Upstream wall**

$b = 18$

and, $d = 1.0$ m

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{1.0}{18.0} = 0.056$$

From plate 11.1 (a);

$$\phi E_1 = 0\%$$

$$\phi D_1 = 100 - \phi D = 100 - 16 = 84\%$$

$$\phi C_1 = 100 - \phi E = 100 - 23 = 77\%$$

$$\text{Correction to } \phi C_1 \text{ for the depth of floor} = \frac{84\% - 77\%}{103.0 - 120.0} \times 5$$

$$= \frac{7}{1} \times 0.5 = 3.5\% \text{ (+ ve)}$$

Correction due to influence of other wall is very small and neglected.

$$\therefore \quad \phi C_1 = \text{(Corrected)} = 77\% + 3.5\% = 80.5\%$$

I) **Toe of crest**

$b_1 = 11$ m

and, $b = 18$ 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);

$$\phi D_1 = 36\%$$

iii) Downstream curtain wall

$d = 1.7$ m

and, $b = 18$ m

$\frac{d}{b} = \frac{1.7}{18} = 0.094$

From plate 11.1 (a);

$\phi E_3 = \phi E = 29\%$

$\phi D_3 = \phi D = 20\%$

$\phi C_3 = 0\,\%$

$$\text{Correction for depth to } \phi E_3 = \frac{29\% - 20\%}{1.7}$$

$$= \frac{9}{1.7} \times 0.8$$

$$= 4.2\,\%\ (-\text{ ve})$$

ϕE_3 = (corrected) = 29 − 4.2 = 24.8 %

The levels of H.G. line for maximum static head are worked out in table and plotted below:

Conditions of flow	u/s W.L. in meters	d/s W.L. in meters	Height/Elevation of H.G. line above datum						
			ϕE_1 100%	ϕD_1 84%	ϕC_1 80.5%	ϕD_2 36%	ϕE_3 24.8%	ϕD_3 20%	ϕC_3 0%
Maximum static head i.e., water up to crest level on u/s and no water in d/s	105.97	103.50	2.47 105.97	2.08 105.58	1.99 105.49	0.89 104.39	0.61 104.11	0.50 104.00	0.90 103.5

Floor thickness

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

$$\text{Thickness required} = \frac{1.19}{1.24} = 0.97 = \text{m; Use } 1.2 \text{ m}$$

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

$$\text{Thickness required} = \frac{0.61}{1.24} = 0.5 \text{ m; Use } 0.7 \text{ m}$$

Use 0.5 m thick C.C. overlain by 0.2 m thick bricks.

$$\text{Unbalanced head at 3 m from d/s toe of crest} = 0.91 + \frac{0.28}{7} \times 3$$

$$= 0.91 + 0.12$$

$$= 1.03 \text{ m}$$

$$\text{Thickness required} = \frac{1.03}{1.24} = 0.83 \text{ m}$$

Provide 0.8 m thick C.C. laid over by 0.2 m thick bricks.

Thicknesses provided are shown in the figure.

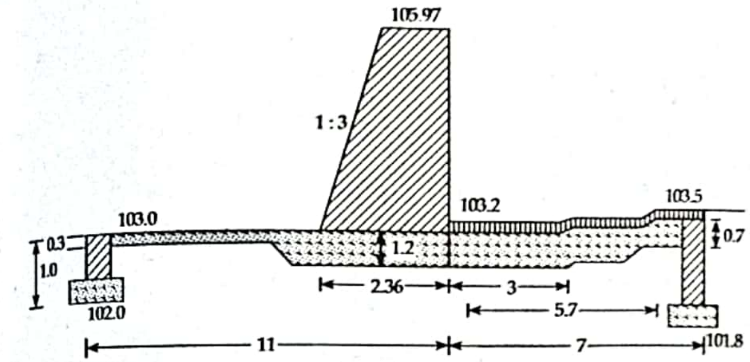


Figure 7.28

Downstream pitching

From the table 7.1;

$$\text{Length of d/s pitching required} = 9 + 2H_L$$

$$= 9 + 2 \times 1.5$$

$$= 12 \text{ m}$$

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.

7.6 WORKED OUT PROBLEMS

PROBLEM 1

Design a canal drop structure for the data given below.
$Q = 5\ m^3/sec.$
FSL u/s = 110.5 m
FSL d/s = 109.5
Normal depth u/s and d/s = 1.5 m
Bed width = 3.0 m
Bligh's coefficient = 7

Solution:

Length of crest = d/s bed width = 3.0 m = L

Crest level

Q < 14 cumecs so a rectangular crest is provided.

Using discharge formula; we have,

$$Q = 1.84 \cdot L \cdot H^{\frac{3}{2}}\left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

Assume,

Top width = B_t = 0.8 m

or, $5 = 1.84 \times 3.0 \times H^{\frac{3}{2}}\left(\frac{H}{0.8}\right)^{\frac{1}{6}}$

or, $H^{\frac{5}{3}} = 0.872$

or, $H = (0.872)^{\frac{3}{5}} = 0.92$ m

Velocity of approach $(V_a) = \dfrac{\text{Discharge}}{\text{Area}} = \dfrac{5}{(3+1.5)1.5}$

[Depth of water = 1.5, let side slope = 1:1]

= 0.74 m/sec.

Velocity head $= \dfrac{V_a^2}{2g} = \dfrac{(0.74)^2}{19.6} = 0.028$ m

Now,

u/s TEL = u/s FSL + Velocity head = 110.5 + 0.028 = 110.528 m

R.L. of crest = u/s TEL – H = 110.528 – 0.92 = 109.608

Height of crest above u/s floor = 109.608 – u/s Bed level
= 109.608 – (110.5 – 1.5)
= 0.608 m

Shape of crest

$B_t = 0.55\sqrt{d}$

d = Height of crest above d/s bed = 109.608 – 108 = 1.608

$B_t = 0.55\sqrt{1.608} = 0.7$ m < 0.80 m (O.K.)

Thickness at base $= \dfrac{h+d}{2} = \dfrac{(0.92 - H_V) + 1.608}{2} = 1.25$ keep 1.3 m

u/s curtain wall

Depth of curtain wall $= \dfrac{y_u}{3} = \dfrac{1.5}{3} = 0.5$ m

Provide 0.4 × 0.8 m deep curtain wall on u/s. [0.5 + 0.3 = 0.8 m]

Cistern

Depth of cistern (X) $= \dfrac{1}{4}[H \times H_L]^{\frac{1}{3}}$ $\quad [H_L = 110.5 - 109.5 = 1\ m]$

$= \dfrac{1}{4}[0.92 \times 1]^{\frac{1}{3}}$

= 0.236 Provide 0.25 m depth

R.L. of cistern = 108.0 – 0.25 = 107.75 m

Length of cistern $= 5\sqrt{H \cdot H_L} = 5\sqrt{0.92 \times 1} = 4.8$ m

Impervious floor

Maximum static head = R.L. of crest – d/s bed level
= 109.608 – 108.0

$H_S = 1.608$

Total floor length required $= C \cdot H_S = 7 \times 1.608 = 11.256$ m

Provide 12 m length impervious floor.

Minimum d/s floor length required $= 2(\text{Water depth} + 1.2) + H_L$
= 2(1.5 + 1.2) + 1
= 6.4 (say 6.5 m)

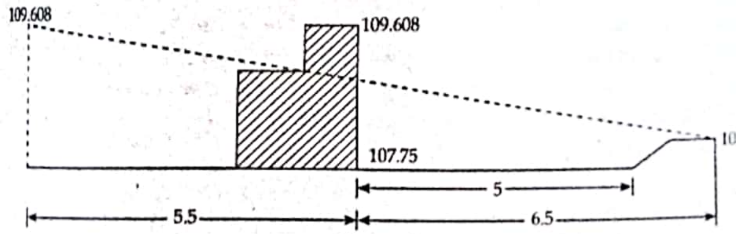


Figure 7.29

Maximum unbalanced uplift at d/s toe of crest $= 0.25 + \dfrac{H_S}{12} \times 6.5$

$= 0.25 + \dfrac{1.608}{12} \times 6.5$

= 1.121 m

Thickness required $= \dfrac{1.12}{G-1} = \dfrac{1.12}{1.24} = 0.91$ m

Provide 0.8 m thick concrete with 20 cm thick brick layer

At 3 m from the toe of crest $= 0.25 + \dfrac{1.608}{12} \times (6.5 - 3) = 0.72$ m

Thickness required $= \dfrac{0.72}{1.24} = 0.58$ m

Provide 0.5 m thick concrete with 0.2 m thick brick layer

5 m from the toe of crest $= 0.25 + \dfrac{1.608}{12} \times (6.5 - 5)$

= 0.45 m

$$\text{Thickness required} = \frac{0.45}{1.24} = 0.36 \text{ m}$$

Provide 0.3 m thick concrete with 20 cm thick brick layer and a nominal thickness of 0.3 m concrete on u/s.

d/s curtain wall

$$\text{Depth of wall} = \frac{y_u}{2} = \frac{1.5}{2} = 0.75 \text{ m}$$

d/s cutoff wall

$$\text{Depth of wall} = \frac{y_u}{2} + 0.6 = \frac{1.5}{2} + 0.6 = 1.35 \text{ m}$$

Provide 0.4 × 1.35 m deep cut off wall over 0.3 m cement concrete on the d/s end of floor.

d/s pitching

$$\begin{aligned}\text{Total length of d/s pitching} &= 13.5 + 2 \times H_L \qquad [\text{for } H = 0.92]\\ &= 13.5 + 2\\ &= 15.5 \text{ m}\end{aligned}$$

Provide pitching at slope 1 : 10.

PROBLEM 2

Design crest, length and thickness of impervious floor of a vertical drop structure for the data given below.	
Discharge = 1.8 m^3/sec.	**Bed level u/s = 205.05 m**
Side slope of channel = 1:1	**Bed level d/s = 204.35 m**
FSL u/s = 205.95 m	**Bed width u/s and d/s = 1.5 m**
Bligh's coefficient (C) = 6.0	**Specific gravity of masonry drop = 2.2**
Top width of crest = 0.8 (for initial assumption)	
C_d = 0.415	

Solution:

Length of crest = d/s bed width = 1.5 m

Since, Q < 14 cumecs, rectangular crest is provided.

Using discharge formula; we have,

$$Q = 1.84 \cdot L \cdot H^{\frac{3}{2}} \left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

Let assume 0.8 m.

$$1.8 = 1.84 \times 1.5 \times H^{\frac{3}{2}} \left(\frac{H}{0.8}\right)^{\frac{1}{6}}$$

or, $H^{\frac{5}{3}} = 0.628$

or, $H = (0.628)^{\frac{3}{5}} = 0.756$ m; say 0.76 m

$$\text{Velocity of approach } (V_a) = \frac{\text{Discharge}}{\text{Area}} = \frac{Q}{(B + y)y} = \frac{1.8}{(1.5 + 0.9)0.9}$$

(y = FSL u/s – Bed level u/s)

$V_a = 0.834$ m/sec.

$$\text{Velocity head } (H_V) = \frac{V_a^2}{2g} = \frac{(0.834)^2}{19.6} = 0.035 \text{ m}$$

R.L. of TEL = FSL u/s + Velocity head = 205.95 + 0.035 = 205.99 m

R.L. of crest = 205.99 – 0.76 = 205.23 m

Height of crest above bed = 205.23 – 205.05 = 0.18 m

Shape of crest

$B_t = 0.55\sqrt{d}$

d = Height of crest above d/s bed = 205.23 – 204.35 = 0.88 m

$B_t = 0.55\sqrt{0.88} = 0.52 < 0.8$ (O.K.)

$$\text{Thickness at base} = \frac{h + d}{2} = \frac{(0.76 - 0.025) + 0.88}{2} = 0.81 \text{ m}$$

Let provide 1 m.

$$\text{Upstream curtain wall} = \frac{y_u}{3} = \frac{0.9}{3} = 0.3 \text{ m}$$

Provide 0.4 × 0.5 m deep curtain wall on u/s.

Cistern

$$\text{Depth of cistern } (X) = \frac{1}{4}[H \times H_L]^{\frac{2}{3}} = \frac{1}{4}[0.76 \times H_L]^{\frac{2}{3}}$$

$$\left[\begin{aligned}\text{d/s FSL} &= \text{d/s Bed level} + \text{Water depth}\\ &= 204.35 + 0.9\\ &= 205.25 \text{ m}\end{aligned}\right]$$

or, H_L = u/s FSL – d/s FSL

or, H_L = 205.95 – 205.25 = 0.7 m

$(X) = \frac{1}{4}[0.76 \times 0.7]^{\frac{2}{3}} = 0.164$ use 0.2 m

R.L. of crest = 204.35 – 0.2 = 204.15 m

Length of cistern = $5\sqrt{H \cdot H_L}$ = 3.65 m, Say 3.7 m

Provide 3.7 m long cistern at 204.15 m R.L.

Impervious floor

Maximum static head = Crest level – d/s bed level

or, H_S = 205.23 – 204.35 = 0.88 m

Total length of floor required = $C \cdot H_S$ = 6 × 0.88 = 5.28 m

Let provide 6.0 m impervious floor.

$$\begin{aligned}\text{Minimum d/s floor length required} &= 2\,[\text{Water depth} + 1.2] + H_L\\ &= 2(0.9 + 1.2) + 0.7\\ &= 4.9 \text{ m}\end{aligned}$$

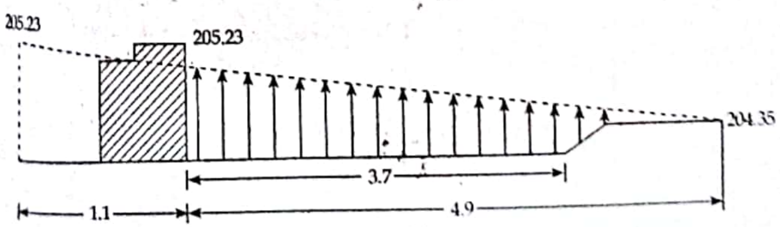


Figure 7.30

Provide 4.9 m d/s floor and the remaining 1.1 on u/s.

Maximum unbalanced uplift at d/s toe of crest $= 0.2 + \frac{H_S}{6.0} \times 4.9$

$= 0.2 + \frac{0.88}{6.0} \times 4.9$

$= 0.92$ m

Thickness required $= \frac{0.92}{G-1} = \frac{0.92}{2.2-1} = \frac{0.92}{1.2} = 0.76$ m

Provide 0.6 m thick concrete with 20 cm thick brick layer.

At 3 m from toe of crest at d/s side $= 0.2 + \frac{0.88}{6} \times (4.9 - 3) = 0.48$ m

Thickness required $= \frac{0.48}{1.2} = 0.4$ m

Provide 0.3 m concrete with 20 cm thick brick layer and a nominal thickness of 0.3 m concrete at u/s side.

PROBLEM 3

A diversion weir with a vertical drop to be designed for an irrigation system has the following data:
Design flood = 560 m^3/sec.
Natural width of the source river = 200 m
Bed material = coarse sand
Bligh's C = 12
Lacey's f = 1.2
Height of weir above low water = 3.0 m
Top width of crest = 2.0
Fix the length of floor and depth of cutoffs using suitable seepage theory. Compute the thickness of the floor at key points. Make suitable assumptions if necessary. Draw neat sketches of designed weir.

Solution:

Total head loss (H_L) = Height of weir above low water = 3 m

Total length of creep required including creep along cut off (L) $= CH_L$

$= 12 \times 3$

$= 36$ m

Total length of d/s floor (L_2) $= 2.21C\sqrt{\frac{H_L}{10}} = 14.52$ m

For head over the weir with high flood discharge is passing.

$$Q = C[L - 0.1nH]^{\frac{3}{2}}$$

Let assume broad crested weir with high flood, $C = 1.7$ and $L \gg H$ so neglecting 0.1nH.

$\therefore \quad Q = 1.7LH^{\frac{3}{2}}$

or, $\quad 560 = 1.7 \times 200 \times H^{\frac{3}{2}}$

or, $\quad H^{\frac{3}{2}} = 1.647$

or, $\quad H = 1.394 \approx 1.4$ m

Top width (B) = 2 m (given)

$2B > H$

Hence, our assumption of broad crested weir is correct.

Head over the weir crest = 1.4 m

u/s HFL (assuming d/s bed level as 100 mm) $= 100 + 3 + 1.4$

$= 104.4$ m

Lacey's regime scour depth (R) $= 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35\left[\frac{\left(\frac{560}{200}\right)^2}{1.2}\right]^{\frac{1}{3}}$

$= 1.35 \times (6.533)^{\frac{1}{3}}$

$= 2.52$ m

Depth of u/s sheet pile below u/s HFL $= 1.5R = 1.5 \times 2.52 = 3.78$ m

R.L. of bottom u/s pile $= 104.4 - 3.78 = 100.62$ m

Depth of u/s street pile $= 100 - 100.62 = -0.62$ m (which is negative)

so, provide u/s cut off sheet pile of depth $= \frac{y_4}{3} + 0.6 \qquad \begin{bmatrix} y_4 = 104.4 - 100 \\ = 4.4 \text{ m} \end{bmatrix}$

$= \frac{4.4}{3} + 0.6$

$= 2.067 \simeq 2.1$ m

Since, d/s HFL is not known, provide similar cut off of 2.1 m below weir floor.

Bottom width of weir $= \frac{H + \text{Height of weir}}{G-1} = \frac{1.4+3}{\sqrt{1.4}} = 3.72$ m

Total creep length $= 2 \times 2.1 + 3.72 + 14.52 + 2 \times 2.1 = 26.64$ m

Balance length $= 36 - 26.64 = 9.36$ m

Say 10 m is provided on u/s floor.

Total creep length $= 26.64 + 10 = 36.64$ m

Now,

$L_2 + L_3 = 18C\sqrt{\frac{H_L}{10}\frac{q}{75}} = 18 \times 12\sqrt{\frac{3}{10} \times \frac{2.8}{75}} = 22.86$ m

$L_2 = 14.52$

$L_3 = 22.86 - 14.52 = 8.34 \simeq 9$ m

Hence, provide 1 m thick d/s loose talus of 9 m in length and same as in u/s also. (i.e., $L_3 = L_4$)

H.G. line is plotted as shown in the figure.

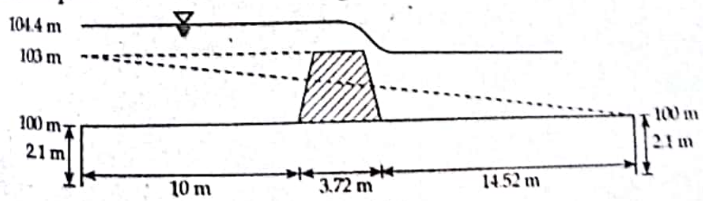


Figure 7.31

Max. ordinate of H.G. line at toe of weir (h) $= \frac{3}{36.64} \times (2.1 \times 2 + 14.52)$
$= 1.53$ m

Thickness of d/s floor at this point (t) $= 1.33 \times \frac{h}{G-1}$
$= 1.33 \times \frac{1.53}{1.4}$ [$\because$ 1.33 is factor of safety and G = 2.4 m]
$= 1.455 \simeq 1.5$ m

Thickness required at half way of d/s floor length (t) $= \frac{1.33h}{1.4}$

$\left[\because h = \frac{3}{36.64} \times (2.1 \times 2 + 7.26) = 0.94 \text{ m}\right]$

$\therefore \quad t = \frac{1.33h}{1.4} = \frac{1.33 \times 0.94}{1.4} = 0.9$ m (Say 1 m)

Further provide a nominal thickness of 0.8 m below the u/s floor and 1 m below weir wall.

NOTE

In other problem of simple vertical drop, FOS is not used because there we did not count creep length due to cut off or curtain wall.

PROBLEM 4

Design a vertical drop for the data given below.
Full supply discharge u/s and d/s = 1.5 m^3/sec.
Drop height = 1.0 m
FSL u/s and d/s = 106.0 m and 105.0 m respectively
Full supply depth u/s and d/s = 0.8 m
Bed level of u/s and d/s = 105.2 and 104.2 respectively
Bed width u/s and d/s = 1.1 m
Top width of crest = 0.50 m [2070 Magh]

Solution:

Length of crest = d/s bed width = 1.1 m

Crest level

$Q < 14$ m^3/sec.

So, a rectangular crest is provided.

Using discharge formula; we have,

$$Q = 1.84 \cdot L \cdot H^{\frac{3}{2}} \left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

or, $1.5 = 1.84 \times 1.1 \times H^{\frac{3}{2}} \left(\frac{H}{0.5}\right)^{\frac{1}{6}}$

or, $H^{\frac{5}{3}} = 0.66$

or, $H = (0.66)^{\frac{3}{5}} = 0.78$ m

Velocity of approach $= \frac{\text{Discharge}}{\text{Area}}$
$= \frac{1.5}{(1.1 + 0.8)0.8}$ [Let, side slope = 1:1]
$= 0.99$ m/sec.

Velocity head $= \frac{V_a^2}{2g} = \frac{(0.99)^2}{19.62} = 0.05$ m

R.L. of TEL = 106.0 + 0.05 = 106.05 m

R. of crest = R.L. of TEL − H = 106.05 − 0.78 = 105.27 m

Height of crest above u/s bed = 105.27 − 105.2 = 0.07 m

Shape of crest

$B_t = 0.55\sqrt{d}$

d = Height of crest above d/s bed = 105.27 − 104.2 = 1.07 m

$B_t = 0.55\sqrt{1.07} = 0.57$ m

Let provide $B_t = 0.6$ m

Thickness at base $= \frac{h+d}{2}$
$= \frac{(H - \text{Velocity head}) + d}{2} = \frac{(0.78 - 0.05) + 1.07}{2}$
$= 0.9$ m; Provide 1.0 m

u/s curtain wall

Depth $= \frac{y_u}{3} = \frac{0.8}{3} = 0.27$ m

Let provide 0.4 m × 0.4 m curtain wall at u/s.

Cistern

Depth of cistern (X) $= \frac{1}{4}[H \times H_L]^{\frac{2}{3}} = \frac{1}{4}[0.78 \times 1]^{\frac{2}{3}}$
$= 0.218$; Say 0.22 m

R.L. of cistern = 104.2 − X = 104.2 − 0.22
= 103.98; Say 4.42 m

Length of cistern $= 5\sqrt{H \cdot H_L} = 5\sqrt{0.78 \times 1}$
$= 4.416$ m; Say 4.42 m

A cistern of 4.42 m at R.L. of 103.98 is provided.

Impervious floor

Maximum static head (H_S) = R.L. of crest − d/s bed level
= 105.27 − 104.2
= 1.07 m

Total length of floor required $= C \cdot H_S = 6 \times 1.07$
= 6.42 m [Let, C = 6]

Let 7 m be provided.

Min. length of impervious floor at d/s = 2[Water depth + 1.2] + H_L
= 2(0.8 + 1.2) + 1
= 5 m

Remaining 2 m is provided at u/s.

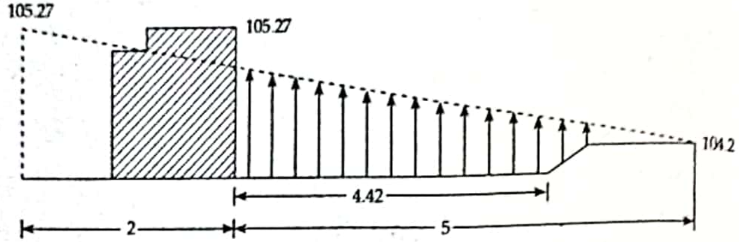


Figure 7.32

Uplift at toe of crest $= 0.22 + \frac{1.07}{7} \times 5 = 0.985$ m

Thickness required $= \frac{0.985}{G-1} = \frac{0.985}{1.24} = 0.8$ m

Provide 0.6 m thick concrete with 20 cm thick brick layer.

At 3 m from the toe of crest uplift $= 0.22 + \frac{1.07}{7} \times (5-3) = 0.525$ m

Thickness required $= \frac{0.525}{1.24} = 0.43$ m

Provide 0.4 thick concrete with 20 cm thick brick layer.

A nominal thickness of 0.3 m thick concrete is provided at u/s side.

d/s curtain wall

Depth $= \frac{y_d}{2} = \frac{0.8}{2} = 0.4$

Provide 0.4×0.7 m deep curtain wall.

d/s cut off wall

Depth $= \frac{y_d}{2} + 0.6 = 0.4 + 0.6 = 1$ m

Provide 0.4×1 m cut off wall over 0.3 m concrete.

Length pitching $= 9 + 2H_L = 9 + 2 = 11$ m

Provided at slope of 1 : 10 slope.

PROBLEM 5

Design the crest cistern of drop structure (Sarda type) for a discharge of 9 cumecs and drop height of 1.2 m: FSL u/s and d/s = 105.7 and 104.5 m; bed level u/s and d/s = 104.2 and 103 m; bed width u/s d/s = 8 m; side slope of channel = 1 : 1 **[2071 Bhadra T.U.]**

Solution:

Given that;

Length of crest = d/s bed width = 8 m

Crest level

$Q < 14\ m^3/sec.$

Hence, a rectangular crest is provided.

Using discharge formula; we have,

$$Q = 1.84L \cdot H^{\frac{3}{2}} \left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

Let, $B_t = 0.8$ m

or, $9 = 1.84 \times 8 \cdot H^{\frac{3}{2}} \left(\frac{H}{0.8}\right)^{\frac{1}{6}}$

or, $H^{\frac{5}{3}} = 0.59$

∴ $H = 0.728\ m \simeq 0.73$ m

Now,

Velocity of approach $(V_a) = \frac{\text{Discharge}}{\text{Area}} = \frac{9}{(8+1.5) \times 1.5} = 0.63$ m/sec.

[∵ Depth of water = 105.7 − 104.2 = 1.5 m]

Velocity head $= \frac{V_a^2}{2g} = \frac{(0.63)^2}{19.62} = 0.02$ m

R.L. of TEL on u/s = R.L. of u/s FSL + Velocity head
= 105.7 + 0.02
= 105.72 m

R.L. of crest = R.L. of TEL − H = 105.72 − 0.72 = 105 m

R.L. of crest above u/s bed = 105 − 104.2 = 0.8 m

Shape of crest

Here,

$B_t = 0.55\sqrt{d}$

where, d is the height of crest above d/s bed = 105 − 103 = 2 m.

∴, $B_t = 0.55\sqrt{2} = 0.78\ m < 0.8$ m (OK)

Provide 0.8 m wide crest on top with 0.2 m thick cement concrete.

Cistern

Depth of cistern (X) $= \frac{1}{4}[H \cdot H_L]^{\frac{2}{3}} = \frac{1}{4}[0.73 \times 1.2]^{\frac{2}{3}} = 0.2$ m

R.L. of cistern = R.L. of d/s bed − X = 103 − 0.2 = 102.8 m

Length of cistern $= 5\sqrt{H \cdot H_L} = 5\sqrt{0.73 \times 1.2} = 4.68$ m

A cistern of 4.7 m length at R.L. of 102.8 is provided.

PROBLEM 6

Find the thickness of downstream impervious floor for a fall having following data:

a) **Discharge $\frac{u/s}{d/s} = \frac{10 \text{ cumecs}}{10 \text{ cumecs}}$**

b) **Full supply level $\frac{u/s}{d/s} = \frac{201.50}{200.25}$**

c) **Drop = 1.25 m**

d) **Bed level $\frac{u/s}{d/s} = \frac{200.00}{198.75}$**

e) **Bed width $\frac{u/s}{d/s} = \frac{9.0 \text{ m}}{9.0 \text{ m}}$**

f) **Full supply depth $\frac{u/s}{d/s} = \frac{1.50 \text{ m}}{1.50 \text{ m}}$**

g) **Bligh's creep coefficient = 8**

Solution:

Here,

Length of crest = d/s bed width = 9 m

Crest level

$Q < 14 \text{ m}^3/\text{sec.}$

Hence, a rectangular crest is provided.

Using discharge formula; we have,

$$Q = 1.84L \cdot H^{\frac{3}{2}}\left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

Let, $B_t = 0.8$ m

or, $10 = 1.84 \times 9 \times H^{\frac{3}{2}}\left(\frac{H}{0.8}\right)^{\frac{1}{6}}$

or, $H^{\frac{5}{3}} = 0.582$

∴ $H = 0.72$ m

Now,

$$\text{Velocity of approach } (V_a) = \frac{\text{Discharge}}{\text{Area}} = \frac{10}{(9+1.5)\times 1.5} = 0.635 \text{ m/sec.}$$

[Assuming side slope of 1 : 1]

$$\text{Velocity head} = \frac{V_a^2}{2g} = \frac{(0.635)^2}{19.62} = 0.02 \text{ m}$$

R.L. of TEL on u/s = R.L. of u/s FSL + Velocity head
= 201.5 + 0.02 = 201.52 m

R.L. of crest = R.L. of TEL on u/s − H
= 201.52 − 0.72
= 200.8 m

Height of crest above u/s bed = 200.8 − 200 = 0.8 m

Cistern

$$\text{Depth of cistern } (X) = \frac{1}{4}[H \cdot H_L]^{\frac{2}{3}} = \frac{1}{4}[0.72 \times 1.25]^{\frac{2}{3}} = 0.234 \text{ m}$$

R.L. of cistern = R.L. of d/s bed − X = 198.75 − 0.234 = 198.516 m

$$\text{Length of cistern} = 5\sqrt{H \cdot H_L} = 5\sqrt{0.72 \times 1.25} = 4.75 \text{ m}$$

Provide 5 m length cistern at R.L. of 198.516 m.

Impervious floor

Maximum static head (H_S) = R.L. of crest − d/s bed level
= 200.80 − 198.75 = 2.05 m

Total length of floor required = $C \cdot H_S = 8 \times 2.05 = 16.4$ m

Let, 17 m long is provided.

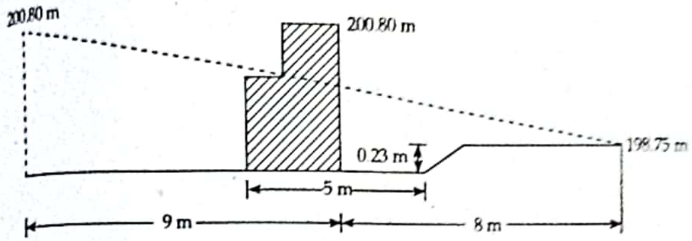


Figure 7.33

Min. length of impervious floor at d/s = 2(Water depth + 1.2) + H_L
= 2(1.5 + 1.2) + 1.25
= 6.65 m

Let provide 8 m in d/s and remaining (17 − 8 = 9) on u/s.

$$\text{Uplift at toe of crest} = 0.234 + \frac{2.05}{17} \times 8 = 1.2 \text{ m}$$

$$\text{Thickness required} = \frac{1.32}{G-1} = \frac{1.32}{1.24} = 0.97 \text{ m}$$

Provide 0.9 m thick concrete over 20 cm thick brick layer.

$$\text{Uplift at 3 m from the toe of crest} = 0.234 + \frac{2.05}{17} \times (8-3) = 0.84 \text{ m}$$

$$\text{Thickness required} = \frac{0.84}{1.24} = 0.68 \text{ m}$$

Provide 0.6 m thick concrete with 20 cm brick layer.

$$\text{Uplift at 5 m from toe of crest} = 0.234 + \frac{2.05}{17} \times (8-5) = 0.6 \text{ m}$$

$$\text{Thickness required} = \frac{0.6}{1.24} = 0.48 \text{ m}$$

Provide nominal thickness of 0.5 m over 0.2 m thick brick layer from that point.

PROBLEM 7

Design a 1.5 m Sarda type of fall for a canal having a discharge 12 cumec with the following data:
Bed level u/s = 103.0
Bed level d/s = 101.5 m
Side slope of the channel = 1 : 1
FSL u/s = 104.5 m
Bed width u/s and d/s = 10 m
Soil good loam
Assume Bligh's coefficient (C) = 6

Solution:

Here,

Length of crest = d/s bed width = 10 m

Crest level

Q < 14 cumecs

Hence, a rectangular crest is provided.

Using discharge formula; we have,

$$Q = 1.84L \cdot H\left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

Let, $B_t = 0.8$ m

or, $12 = 1.84 \times 10 \cdot H\left(\frac{H}{0.8}\right)^{\frac{1}{6}}$

or, $H^{\frac{5}{3}} = 0.623$

∴ $H = 0.755 \text{ m} \simeq 0.76 \text{ m}$

Now,

$$\text{Velocity of approach } (V_a) = \frac{\text{Discharge}}{\text{Area}} = \frac{12}{(10+1.5)\times 1.5} = 0.696 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V_a^2}{2g} = \frac{(0.696)^2}{19.62} = 0.025 \text{ m}$$

u/s TEL = R.L. of u/s FSL + Velocity head = 104.5 + 0.025
= 104.525 m

R.L. of crest = R.L. of TEL on u/s − H = 104.525 − 0.755 = 103.77 m

Height of crest above u/s floor = 103.77 − 103 = 0.77 m

Shape of the crest

Here,

d is the height of crest above d/s bed = 103.77 − 101.5 = 2.27 m

∴ Width of crest (B_t) = $0.55\sqrt{d} = 0.55\sqrt{2.27} = 0.825$ m

Keep 0.85 m width of crest.

$$\text{Thickness at base} = \frac{h+d}{2} = \frac{(0.755+0.0.025)+2.27}{2} = 1.5 \text{ m}$$

Remaining part of design

Refer to the solution of example 7.4

PROBLEM 8

Explain the working principle of non-modular and semi modular outlet. What are the requirements of good module? **[2072 Ashwin]**

Solution: See the definition part 7.4

PROBLEM 9

Design the crest and cistern of a vertical drop structure for data given below.
Discharge = 4.5 cumec
Bed level u/s = 105.00
Side slope of channel = 1 : 1
Bed level at d/s = 103.5
FSL at u/s = 106.5
Bed width u/s and d/s = 3.0 m
Top width of crest = 0.75 m (For initial assumption)
C_d = 0.41 **[2072 Ashwin]**

Solution:

Length of crest = d/s bed width = 3 m = L

Crest level

Q < 14 cumecs, so a rectangular crest is provided using discharge formula

$$Q = C_d\sqrt{2g}LH^{\frac{3}{2}}\left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

or, $Q = 0.41\sqrt{2g}LH^{\frac{3}{2}}\left(\frac{H}{B_t}\right)^{\frac{1}{6}}$

or, $Q = 1.82\, LH^{\frac{3}{2}}\left(\frac{H}{B_t}\right)^{\frac{1}{6}}$

$B_t = 0.75$ for initial assumption

so, $4.5 = 1.82 \times 3 \times H^{\frac{3}{2}} \times \left(\frac{H}{0.75}\right)^{\frac{1}{6}}$

or, $H^{\frac{5}{3}} = 0.786$

or, $H = 865$ m

Now, depth of water is u/s = FSL in u/s − u/s bed level = 106.5 − 105
= 1.5 m

Due to equal width depth of water at d/s is also 1.5 m.

FSL of d/s = d/s bed level + 1.5 = 103.5 + 1.5 = 105 m

$$\text{Velocity of approach } (V_a) = \frac{\text{Discharge}}{\text{Area}} = \frac{4.5}{(3+1.5)\,1.5} = 0.667 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V_a^2}{2g} = \frac{(0.667)^2}{19.6} = 0.0227 \text{ m}$$

Now,

u/s TEL = u/s FSL + Velocity head
= 106.5 + 0.0227
= 106.5263 m

R.L. of the crest = u/s TEL − H = 106.523 − 0.865 = 105.658 m

Height of the crest above u/s floor = 105.658 − u/s bed level
= 105.658 − 105
= 0.658 m

Shape of the crest

$B_t = 0.55\sqrt{d}$

d = Height of crest above d/s bed = 105.658 − 103.5 = 2.158 m

$B_t = 0.55\sqrt{d} = 0.55\sqrt{2.158} = 0.808 \text{ m} > 0.75 \text{ m}$

so, take, $B_t = 0.81$ m

$$\text{Thickness at base} = \frac{h+d}{G} = \frac{(0.865 - H_V) + 2.158}{2}$$

$$= \frac{0.865 - 0.227 + 2.158}{2}$$

$$= 1.5 \text{ m}$$

Cistern

Depth of cistern (X) $= \frac{1}{4}[H \times H_L]^{\frac{2}{5}}$

where, H_L = [u/s FSL − d/s FSL] = 106.5 − 105 = 1.5 m

$X = \frac{1}{4}[0.865 \times 1.5]^{\frac{2}{3}} = 0.298 \text{ m} = 0.3 \text{ m}$

Provide 0.3 m depth cistern.

R.L. of cistern = d/s bed level − X = 103.5 − 0.3 = 103.2 m

Length of cistern $= 5\sqrt{H \times H_L} = 5\sqrt{0.865 \times 1.5} = 5.7$ m

Provide 6 m long cistern.

PROBLEM 10

Making a suitable sketch compute the water level required in the distributary to convey a flow as 50 lps through a 10 m long free discharging pipe outlet (n = 0.016) of 20 cm diameter to a water course with FSL at 100 m. [2072 Magh]

Solution:

Given that;

Discharge = 50 *l*ps = 0.05 m³/sec.

Length of pipe = 10 m

Manning's coefficient (n) = 0.016

Diameter of pipe = 20 cm = 0.2 m

FSL at water course = 100 m

Now,

R.L. of centre of pipe outlet = 100 + 0.1 = 100.01 m

Cross section area of pipe $= \frac{\pi D^2}{4} = \frac{\pi \times (0.2)^2}{4}$

$= 0.031416 \text{ m}^2$

Using Manning's formula

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

or, $0.05 = \frac{1}{0.016} \times \left(\frac{0.2}{4}\right)^{\frac{2}{3}} \times S^{\frac{1}{2}} \times 0.031416$ $\quad \left[\because R = \frac{D}{4}\right]$

or, $S^{\frac{1}{2}} = 0.187$

or, $S = 0.0352$

Head required = S × Length of pipe

= 0.035 × 10 = 0.35 m

R.L. of FSL at distributary = 200.1 + 0.35 = 200.45 m

PROBLEM 11

Drawing a definition sketch, design a vertical drop in 10 m wide canal (side slope 1 : 1) discharging a flow as 20 m³/s. The canal bed level u/s and d/s are 102 m and 100 m respectively, whereas the FSL u/s and d/s is 105 m and 103 m respectively. Determine design crest level, length of cistern and u/s floor length using Bligh's safe hydraulic gradient as $\frac{1}{8}$. [2072 Magh]

Solution:

Given that;

Width of canal = 10 m

Side slope = 1 : 1

Discharge = 20 m³/sec.

u/s and d/s bed levels = 102 m and 100 m

u/s and d/s FSL = 105 m and 103 m

Now,

Length of crest = Width of canal = 10 m

Crest level

A trapezoidal crest is provided since discharge is > 14 cumec.

Using discharge formula;

$$Q = 1.99 \, LH^{\frac{3}{2}} \left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

Assume top width of crest (B_t) = 1 m

or, $20 = 1.99 \times 10 \times H^{\frac{3}{2}} \left(\frac{H}{1}\right)^{\frac{1}{6}}$

or, $H^{\frac{5}{3}} = \frac{20}{1.99 \times 10} = 1.005$ m

or, H = 1 m

Velocity of approach $(V_a) = \frac{\text{Discharge}}{\text{Area}} = \frac{20}{(10+3)3} = 0.51$ m/s

Velocity head $= \frac{V_a^2}{2g} = \frac{(0.51)^2}{19.6} = 0.013$ m

u/s TEL = u/s FSL + velocity head = 105 + 0.013 = 105.013 m

R.L. of crest = u/s TEL − H = 105.013 − 1 = 104.013 m

Adopt crest level of 104.013 m.

Shape of the crest

Width of the crest $(B_t) = 0.55\sqrt{H + d}$

d = Height of the crest above d/s bed = 104.013 − 100 = 4.013 m

$B_t = 0.55\sqrt{1 + 4.013} = 1.25$

Adopt B_t = 1.25

Adopt a trapezoidal crest with top width of 1.25 m and u/s slope 1 : 3 and d/s slope 1 : 8.

Cistern

Depth of cistern (X) $= \frac{1}{4}(H \times H_L)^{\frac{2}{3}}$

$H_L = 105 - 103 = 2$ m

$X = \frac{1}{4}(1 \times 2)^{\frac{2}{3}} = 0.4$ m

R.L. of cistern = R.L. of d/s − 0.4 m = 100 − 0.4 = 99.6 m

Length of cistern $= 5\sqrt{H \times H_L} = 5\sqrt{1 \times 2} = 7$ m

Provide 7 m long cistern.

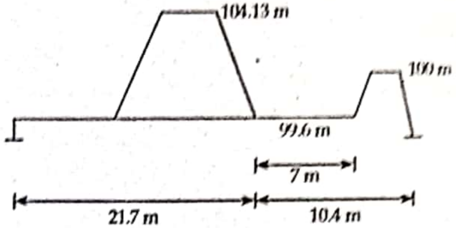
Impervious floor

Maximum static head (H_s) = R.L. of crest − d/s bed level
= 104.013 − 100 = 4.013 m

Total length required $= CH_s = 8 \times 4.013 = 32.104$ m

Min. length of d/s floor from toe of crest wall = 2(Water depth + 1.2 m) + H_L
= 2(3 + 1.2) + 2
= 10.4 m

u/s floor length = 32.104 − 10.4 = 21.7 m



PROBLEM 12

Why drop structures are required in a canal irrigation system? Explain the types of drop structures with neat sketches. [2073 Bhadra]

Solution: See the definition part 7.2 and 7.5

PROBLEM 13

Design a crest width, cistern length and its level of a vertical drop structure for the data given below.
Full supply discharge u/s and d/s = 1.55 cumecs
Drop height = 0.75 m
FSL u/s and d/s = 105.997 and 105.247
Full supply depth u/s and d/s = 0.929 m
Bed levels u/s and d/s = 1.05.068 and 104.318
Bed width u/s and d/s = 1.1 m
Top width of crest = 0.5 for initial assumption c_d = 0.415 for rectangular crest. The drop structure is masonry with specific gravity 2.0 side slope of the canal is 1 : 1. The Bligh's coefficient as 7.0 for sandy loam soil at foundation. [2073 Bhadra]

Solution: Proceed same as the solution of Q. no. 9

PROBLEM 14

What are the functions of head regulator and cross regulator? Sketch the section of cross regulator. [2074 Magh]

Solution: See the definition part 7.2

PROBLEM 15

Design a vertical drop structure for the data given below.
Full supply discharge u/s and d/s = 1.2 cumecs
Drop height = 0.75 m
FSL u/s and d/s = 1.06.997 and 106.247
Full supply depth u/s and d/s = 0.929 m
Bed levels u/s and d/s = 1.06.068 and 105.318
Bed width u/s and d/s = 1.2 m
Top width of crest = 0.5 for initial assumption c_d = 0.415 for rectangular crest. The drop structure is masonry with specific gravity 2.0 side slope of the canal is 1 : 1. The Bligh's coefficient as 6.0 for sandy loam soil at foundation. [2073 Magh]

Solution: Proceed same as the solution of Q. no. 9

PROBLEM 16

Describe briefly the different types of canal outlet. What is flexibility outlet? [2074 Bhadra]

Solution: See the definition part 7.4

PROBLEM 17

Write the stepwise design procedure of cross regulator and distributing head regulator with supporting sketches. [2074 Bhadra]

Solution: See the definition part 7.3

PROBLEM 18

Explain different types of outlets used in irrigation projects. [2075 Baishakh]

Solution: See the definition part 7.4

PROBLEM 19

Write short notes on: Different types of fall structures [2075 Baishakh]

Solution: See the definition part 7.5

PROBLEM 20

Design a 1.5 m sarda type fall for a canal carrying a discharge of 10 cumec with the following given data:

Bed level u/s : 105.00 m | **Bed level d/s : 103.50 m**
Side slope of channel : 1 : 1 | **FSL u/s : 106.50 m**
Bed width u/s and d/s : 3 m | **Soil type : Good loam**
Bligh's coefficient : 6
Assume any other suitable data, if needed. [P.U. 2016]

Solution: See the definition part 7.4

PROBLEM 21

Design a suitable Sarada type full for a canal carrying a discharge of 12 m³/sec. with the following data:

u/s bed level = 103 m	**d/s bed level = 101.5 m**
Side slope of canal = 1.5 : 1 (H : V)	**Full supply level u/s = 104.5**
Bed width of canal = 10 m	**Bligh's coefficient = 6**

[2076 Baishakh]

Solution:

See the problem 7 of this chapter and use side slope of channel 1.5:1 instead of 1 : 1

PROBLEM 22

Design a canal head regulator for the data given below.

Crest level of under sluice = 300 m	**Pond level = 304 m**
Silt factor (f) = 1	**u/s HFL = 307 m**
Full supply discharge = 200 m³/sec.	**Full supply level = 303 m**
Bed level of canal = 299.50 m	

A silt excluder is provided in the under sluice; take $G_E = \frac{1}{6}$ [2076 Baishakh]

Solution:

Here;

Depth of water in canal = F.S.L. of canal – Bed level of canal

= 303 – 299.5 = 3.5 m

As silt excluder is provided crest level of distributor head regulator is kept 1 m higher than crest level of under sluice.

∴ Crest level = 300 + 1 = 301 m

According to Lacey's theory; we have,

$$V = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}} = \left(\frac{200 \times 1}{140}\right)^{\frac{1}{6}} = 1.06 \text{ m/sec.}$$

$$Q = B \times V \times y$$

or, $$B = \frac{200}{1.06 \times 3.5} = 53.84 \text{ m}$$

Here for [Lacey's theory clear water way = 53.84 m]

For water way;

$$Q = B\sqrt{h}\,[1.69\,h + 3.54\,h_1]$$

Here;

h = F.S.L. of parent or pond level channel – F.S.L. of distributory

= 304 – 303

= 1 m

h_1 = F.S.L. of distributor crest level = 303 – 301 = 2 m

or, $200 = B\sqrt{1}\,[1.69 + 3.54 \times 2]$

or, $200 = B \times 8.77$

or, B = 22.8 m

which is very small comparison to normal bed width of distributory 54 m.

Hence, provide 10 bays of 3 m each with 1 m thick pier in between.

Overall water way provided = 30 + 9 = 39 m

The wing wall shall be in proper divergence so as to provide the normal width of channel.

From the question,

H_L = Pond level – d/s F.S.L. = 304 – 303 = 1 m

Depth of water in canal = y_2 = 3.5 m

At pre-jump = y_1

We have,

$$H_L = \frac{(y_2 - y_1)^3}{4y_1 - y_1}$$

or, $$1 = \frac{(3.5 - y_1)^3}{4y_1 \times 3.5}$$

or, $$(3.5 - y_1)^3 - 14y_1 = 0$$

Solving the question; we have,

y_1 = 1.05 m

Length of cistern required $= 5\,(y_2 - y_1) = 5\,(3.5 - 1.05)$

$= 12.25$ m (subjected to $\frac{1}{3}$ of f.b.)

RL of cistern = d/s F.S.L. – E_{f_2} = 303 – E_{f_2} = 303 – 3.55 = 299.45 m

Let's keep 299.5 m.

Here,

$$E_{f_2} = y_2 + \frac{v_2^2}{2g} = 3.5 + \frac{(1.05)^2}{2g} = 3.55$$

$$\text{u/s vertical cut off} = \frac{y_4}{3} + 0.6 = \frac{6}{3} + 0.6 = 2.6 \text{ m}$$

$$\text{d/s vertical cut off} = \frac{y_d}{2} + 0.6 = \frac{3.5}{2} + 0.6 = 2.35 \text{ m}$$

Total floor length from exit gradient considerations

Maximum static head = u/s H.F.L. – 299.5 = 7.5 m

Let keep depth of d/s cut off 4 m considering exit gradient because of high static head.

F = 4 m

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$$

or, $$\frac{1}{6} = \frac{7.5}{4} \times \frac{1}{\pi\sqrt{\lambda}}$$

or, $\lambda = 12.8$

Since, $\lambda = \dfrac{1+\sqrt{1+\alpha^2}}{2}$

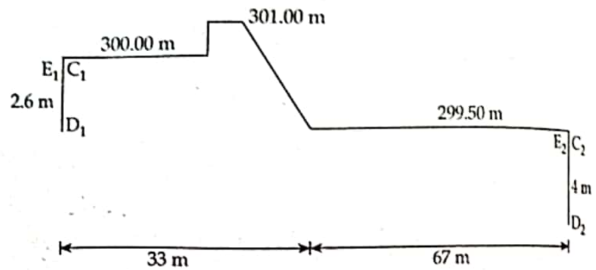
or, $12.8 = \dfrac{1+\sqrt{1+\alpha^2}}{2}$

or, $\alpha = 24.6$

so, $b = \alpha \times d = 24.6 \times 4 = 98.4$ m

Provide b = 100 m

Minimum length of d/s floor $= \dfrac{2}{3} \times 100 = 67 \text{ m} > 12.25 \text{ m (ok)}$



Calculation of uplift pressure

i) u/s cut off

b = 100 m

d = 2.6 m

$\dfrac{1}{\alpha} = \dfrac{d}{b} = 0.026$

From Khosla pressure curves;

$\phi E_1 = 100\%, \phi_D = 10\%, \phi E = 17\%$

$\phi D_1 = 100 - \phi_D = 100 - 10 = 90\%$

$\phi C_1 = 100 - \phi_E = 100 - 17 = 83\%$

Assuming u/s floor thickness = 0.5 m

Correction of ϕC_1 due to floor thickness $= \dfrac{90-83}{2.6} \times 0.5$

$= 1.34\%$ (+ve)

ϕC_1 corrected $= 83 + 1.34 = 84.34\%$

Here, correction due to d/s pile is neglected.

ii) d/s cut off

b = 100, d = 4 m

$\dfrac{1}{\alpha} = \dfrac{4}{100} = 0.04$

From Khosla's pressure curves.

$\phi E_2 = \phi_E = 18\%$

$\phi D_2 = \phi_D = 12\%$

$\phi C_2 = 0$

Assuming d/s floor thickness near cut off = 0.8

Correction to ϕE_2 due to floor thickness $= \dfrac{18-12}{4} \times 0.8 = 1.2\%$ (-ve)

Neglecting correction due to u/s pile interference; we have,

ϕE_2 corrected $= 18 - 1.2 = 16.8\%$

For floor thickness;

% of pressure at toe of glacis $= 16.8 + \dfrac{(84.3-16.8)}{100} \times 67 = 62\%$

Maximum unbalanced head due to static head $= 7.5 \times 62\%$

$= 4.65$ m

Floor thickness required $= \dfrac{4.65}{1.24} = 3.75$ m

At 20 m beyond toe of d/s glacis

% of pressure $= 16.8 + \dfrac{(84.3-16.8)}{100} \times 47 = 48.52\%$

Maximum unbalanced head $= 7.5 \times 48.52\% = 3.64$ m

Thickness required $= \dfrac{3.64}{1.24} = 2.93$ m

Similarly, at 40 m beyond toe of d/s glacis

% of pressure $= (16.8 + 0.675 \times 27) = 35.025\%$

Unbalanced head $= 7.5 \times 35.02\% = 2.62$ m

Thickness required $= \dfrac{2.62}{1.64} = 2.11$ m, provide up to end.

For u/s floor provide nominal thickness of 0.5 m and extend it under the crest and join it to bottom of d/s glacis.

PROBLEM 23

Design crest, length and thickness of impervious floor of a vertical drop structure for the following data.

Discharge = 2.1 m³/sec. **Side slope of canal = 1 : 1**

FSL U/S = 276.10 m

Top width of crest = 0.55 m (for initial assumption)

Specific gravity of masonry drop structure = 2.25

Bed level u/s = 275.15 m **Bed level d/s = 274.45 m**

Bed width u/s and d/s = 1.5 m **For rectangular crest $C_4 = 0.415$**

Bligh's coefficient = 0.6 **[2076 Bhadra]**

Solution: Proceed same as the solution of Q. no. 2

PROBLEM 24

Describe escape structures with net sketches. **[2076 Bhadra]**

Solution: See the last portion of the definition part of 7.3

PROBLEM 25

Design crest, length and thickness of impervious floor of a vertical drop structure for the following data:
Discharge = 5.2 m^3/sec
Bed level u/s = 205.15 m
Side slope of the channel = 1 : 1
Bed level d/s = 204.45 m
F.S.L. u/s = 206.10 m
Bed level u/s and d/s = 1.5 m
Top width of crest = 0.55 m (for initial assumption)
C_d = 0.415
Specific gravity of masonry drop structure = 2.25
Bligh's coefficient = 6.0 [2077 Chaitra]

Solution: Proceed same as the solution of Q. no. 2

PROBLEM 26

What do you understand by head regulator? State functions of outlets and escape structures in irrigation canal. [2077 Chaitra]

Solution: See the definition part 7.2

PROBLEM 27

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

i) **Discharge of parent channel = 125 cumecs**
ii) **Discharge of distributary channel = 20 cumecs**
iii) **FSL of parent channel $\frac{u/s}{d/s} = \frac{120\ m}{119.7\ m}$**
iv) **Bed width of parent channel $\frac{u/s}{d/s} = \frac{50\ m}{45\ m}$**
v) **Depth of water in the parent channel $\frac{u/s}{d/s} = \frac{3\ m}{3\ m}$**
vi) **F.S.L. of distributary = 119 m**
vii) **Bed width of distributary = 20 m**
viii) **Full supply depth of distributary = 1.8 m**
ix) **Silt factor = 0.8**
x) **Assume safe exit gradient = $\frac{1}{6}$** [2078 Baishakh]

Solution:

Proceed same as solution of the "crest, length and thickness of impervious floor part" of example 7.1

7.7 OBJECTIVE QUESTION

1. Canal drops are required to ....
   a) dissipate excess energy b) dissipate inadequate landscape
   c) dissipate excess land slope d) none of the above
2. The depth discharge relationship of the upstream canal remains practically unaffected by the introduction of a fall of the type ....
   a) Ogee fall b) Sarda type vertical fall
   c) Trapezoidal notch fall d) none of the above
3. The type of fall, which you may recommend for very high drops and very low discharge is ....
   a) Sarda type fall b) Siphon well drop
   c) Straight glacis fall d) Inglis fall
4. Canal fall, involving parabolic glacis is called ....
   a) Straight glacis fall b) Glacis fall
   c) Inglis fall d) Montague fall
5. Energy dissipation in a sarda type fall is caused by ....
   a) hydraulic jump b) friction blocks
   c) water pool d) baffle wall
6. The best energy dissipation on the down-stream side of a canal drop, is caused in ....
   a) Sarda type fall b) Glasis fall
   c) Ogee fall d) Montague fall
7. The length of the water cistern to the provided in sarda type fall is ....
   a) $5(y_2 - y_1)$ b) $5\sqrt{H \times H_L}$
   c) $\frac{1}{4}(H \times H_L)^{\frac{2}{3}}$ d) $1.84\ H^{\frac{3}{2}}$
8. The canal regulator, which is constructed at a diversion head works, is called a ....
   a) cross regulator b) distributary head regulator
   c) canal module d) none of the above
9. The gated regulator, which is constructed in the parent canal near the site of an off taking canal is called ....
   a) canal head regulator b) distributary head regulator
   c) cross regulator d) none of the above
10. The arrangement made in canal network, which acts as safety valve is ....
    a) cattle crossing b) canal crossing
    c) canal escape d) canal module
11. The arrangement provided in a canal to help the cattle safely cross the canal is called a ....
    a) canal ladder b) canal crossing
    c) cattle crossing d) all of the above
12. Canal outlets are also called ....
    a) canal escapes b) canal modules
    c) canal offtakes d) canal opening

13. The type of irrigation module, which makes equitable distribution of water more difficult is a ....
    a) non-modular outlet b) semi-modular
    c) rigid module d) none of the above
14. A good irrigation module is the one, which ....
    a) drows heavy silt from the canal
    b) drows clear water from the canal
    c) drows fair share of silt from the canal
    d) none of above
15. If the rate of change of discharge from an irrigation outlet is equal to the rate of change of discharge in distributary, then the outlet is called ....
    a) flexible b) proportional
    c) sensitive d) sensitivity is zero
16. The rate of change of discharge through an irrigation outlet becomes equal to the rate of change of water depth in the channel, when it is ....
    a) flexibility 1 b) sensitivity 1
    c) setting 1 d) sensitivity zero
17. If the sensitivity of an irrigation module is $\frac{1}{3}$, then 50% variation in canal water depth will cause X% variation in outlet discharge, where X is ....
    a) 100% b) 50%
    c) 25% d) 12.5%
18. A fully modular canal outlet has its ....
    a) sensitivity = 1, and flexibility = 1
    b) sensitivity = 1, and flexibility = 0
    c) sensitivity = 0, and flexibility = 1
    d) sensitivity = 0, and flexibility = 0
19. An irrigation outlet is said to be proportional, when it's ....
    a) sensitivity = 1 b) flexibility = 1
    c) setting = 1 d) all of above
20. Free pipe outlet is a ....
    a) rigid module b) flexible outlet
    c) non-modular module d) all of above

Answer sheet

1	2	3	4	5	6	7	8	9	10
c	b	b	d	c	d	b	d	c	c
11	12	13	14	15	16	17	18	19	20
c	b	a	c	b	b	c	d	b	b

CHAPTER 8

CROSS DRAINAGE STRUCTURE

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*****

8.1 TYPES (DRAWING AND SELECTION)

A cross drainage structure is a structure which is constructed at the crossing of a canal and natural drain, so as dispose of drainage water without interrupting the continuous canal supplies. Cross drainage structure may be of following three types.

Types of cross drainage works

i) **Irrigation canal passes over the drainage**
   a) Aqueduct
   b) Siphon aqueduct

ii) **Drainage passes over the irrigation canal**
   a) Super passage
   b) Siphon super passage (canal siphon)

iii) **Drainage and canal intersection each other of the same level**
   a) Level grossing
   b) Inlet and outlet

Selection of type of cross-drainage works

However, in actual field, ideal conditions may not be available and the choice would then depend upon the many factors, such as;

- Relative bed levels
- Availability of suitable foundation
- Economical consideration

- Discharge of the drainage
- Construction problems
- Suitable canal alignment

The following considerations are more important.

- When the bed level of the canal is much above the H.F.L. of the drainage, an aqueduct is the obvious choice. (Figure 8.1)
- When the bed level of the drain is well above F.S.L. of canal, super passage is provided. (Figure 8.3)
- The necessary headway between the canal bed level and the drainage H.F.L. can be increased by shifting the crossing to the downstream of drainage. If, however, it is not possible to change canal alignment, a siphon aqueduct may be provided. (Figure 8.2)
- When canal bed level is much lower, but F.S.L. of canal is higher than bed level of drainage, canal siphon or siphon super-passage is preferred. (Figure 8.4)
- When drainage and canal cross each other practically at same level, a level crossing may be preferred (Figure 8.5). This type of work is avoided as far as possible.
- Suitable canal alignment
- Position of water table and availability of dewatering equipment

I) For passing the canal over the drainage

a) Aqueduct

When the H.F.L. of the drain is sufficiently below the bottom of the canal such that the drainage water flows freely under gravity, the structure is known as aqueduct. In this, canal water is carried across drainage in a trough supported on piers. This is the ridge carrying water.

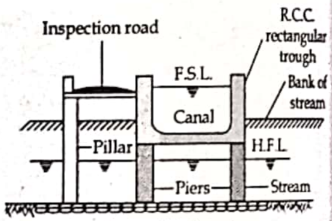


Figure 8.1: Aqueduct

Provided when sufficient level difference is available between the canal and natural and canal bed is sufficiently higher than H.F.L.

b) Siphon aqueduct

In case of the siphon Aqueduct, the H.F.L. of the drain is much higher above the canal bed, and water runs under siphonic action through aqueduct barrel. The drain bed is generally depressed and provided with pucca floors, on the upstream side, the drainage bed may be joined to the pucca floor either by a vertical drop or by glacis of 3 : 1.

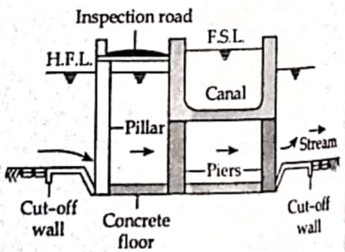


Figure 8.2: Siphon aqueduct

The downstream rising slope should not be steeper than 5 : 1.

When the canal is passed over the drain, the canal remains open for inspection throughout and the damage caused by flood is rare.

However, during heavy floods, the foundations are susceptible to scour or the waterway of drain may get choked due to debris, tress etc.

II) For passing the canal below the drainage

a) Super passage

The hydraulic structure in which the drainage is passing over the irrigation canal is known as super passage. This structure is suitable when the bed level of drainage is above the flood surface level of the canal. The water of the canal passes clearly below the drainage.

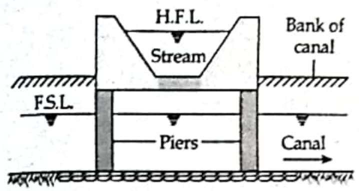


Figure 8.3: Super passage

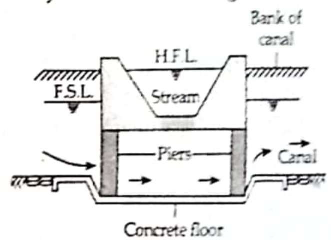


Figure 8.4: Siphon super passage

b) Canal siphon

If two canals cross each other and one of the canals are siphoned under the other, then the hydraulic structure at crossing is called "canal siphon". In case of siphon the F.S.L. of the canal is much above the bed level of the drainage trough, so that canal runs under siphonic action.

III) For passing drain through the canal

a) Level crossing

The level crossing is an arrangement provided to regulate the flow of water through the drainage and the canal when they cross each other approximately at the same bed level. The level crossing consists of the following components.

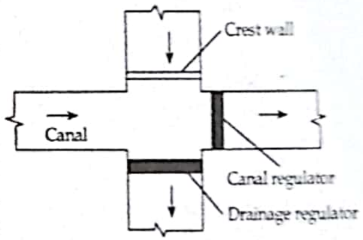


Figure 8.5: Level crossing

Crest wall

It is provided across the drainage just at upstream side of the crossing point. The top level of crest wall is kept at full supply level of the canal.

Drainage regulator

It is provided across the drainage just at downstream side of crossing point. The regulator consists of adjustable shutters at different tiers.

Canal regulator

It is provided across the canal just at the downstream side of the crossing point. This regulator also consists of adjustable shutters at different tiers.

b) Inlet and outlet

In the crossing of small drainage with small channel no hydraulic structure is constructed. Simple openings are provided for the flow of water in their respective directions. This arrangement is known as inlet and outlet.

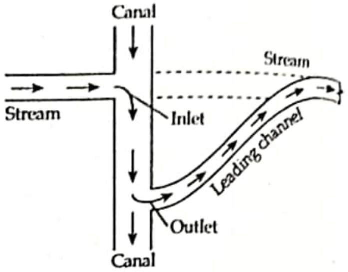


Figure 8.6: Inlet and outlet

In this system, an inlet is provided in the channel bank simply by open cut and the drainage water is allowed to join the channel. At the points of inlet and outlet, the bed and banks of the drainage are protected by stone pitching.

Types of aqueducts and siphon–aqueducts

Depending upon the nature of the sides of the aqueduct or siphon aqueduct it may be classified under three headings.

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.

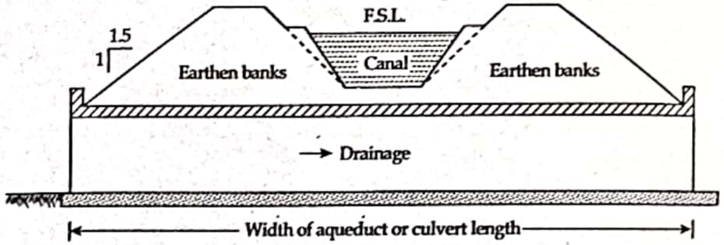


Figure 8.7: Aqueduct I

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.

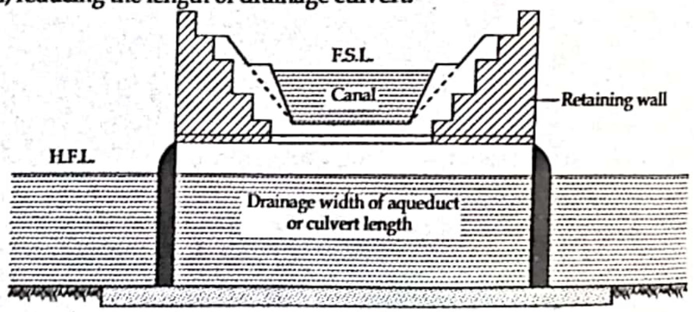


Figure 8.8: Aqueduct II

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.

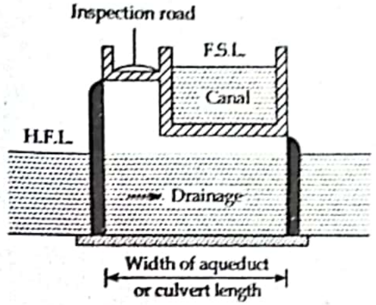


Figure 8.9: Aqueduct III

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.

So, generally in very small drain type I is most economical and in very large drain type III is most economical.

8.12 DESIGN OF SIPHON AQUEDUCT (DETAIL DRAWING, DRAINAGE WATERWAY AND BARREL, CANAL WATERWAY AND TRANSITION, LENGTH AND THICKNESS OF IMPERVIOUS FLOOR AND THICKNESS OF IMPERVIOUS FLOOR AND PROTECTION WORKS)

i) Design of drainage waterway

An approximate value of waterway for the drain may be obtained by using the Lacey's equation, given by;

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

where, P is the wetted perimeter in metres.

Q is the total discharge in cumecs.

For wide drains width of water way is $B \simeq P$.

No extra provision is generally made for the space occupied by piers.

In siphon aqueducts, the required area of the drainage waterway can be obtained by dividing the drainage discharge by the permissible velocity through the barrels. This velocity is generally limited to 2 to 3 m/sec. The waterway area is the divided by the decided waterway width of the drain opening, to compute the height of opening and extend of depressed floor.

For small drain, length of drain water way is taken as 0.8P.

ii) Afflux and head loss through siphon barrels

It was stated earlier that the velocity through siphon barrels is limited to the scouring value of about 2 m/sec. to 3 m/sec. A higher velocity may cause quick abrasion of the barrel surface by rolling girt, etc. and shall definitely result in higher amount of afflux on the upstream side of the siphon aqueduct, and thus requiring higher and longer marginal banks.

The head loss (h) trough siphon barrels and the velocity (V) through them are generally related by Unwin's formula given as;

$$h = \left[1 + f_1 + f_2 \times \frac{L}{R}\right]\frac{V^2}{2g} - \frac{V_a^2}{2g}$$

where, V is the velocity through the barrels.

V_a is the velocity approach which is generally neglected.

f_1 is the 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;

$$f_2 \times \frac{L}{R} \times \frac{V^2}{2g}$$

where, f_2 is given as;

$$f_2 = \alpha\left[1 + \frac{b}{R}\right]$$

where, values of a and b for different material may be taken as given in table.

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

iii) Fluming of canal

Fluming is done in all 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/s), because, if supercritical velocities are generated, then transition back to the normal section on the d/s side of the work may involve the possibility of the formation of the hydraulic jump, this would lead to undue loss of head and large stress in the work.

After deciding normal canal section and 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 26.5° (i.e., 2 : 1 splay) and the d/s or departure wings should not be steeper than 18.5° (i.e., 3 : 1 splay). Generally, normal depth is adopted in flumed section unless special condition.

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

a) Mitra's method of design of transitions (when water depth remains constant)

Shri A.C. Mitra 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

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{B_n \cdot L_f - x(B_n - B_f)2}$$

where, B_n is the bed width of the normal section.

B_f is the bed width of the flumed section.

B_x is the bed width at any distance 'x' form the flumed section.

L_f is the length of transition which is different for expansion and contraction transition.

b) Chaturvedi's semi-cubical parabolic transition when water depth remains constant

Prof. R.S. Chaturvedi, Head of civil engineering department in Roorkee University, on the basis of his own experiments, had in 1993, proposed the following equation for channel transitions when water depth remains constant.

$$x = \frac{L B_n^{\frac{3}{2}}}{B_n^{\frac{3}{2}} - B_f^{\frac{3}{2}}}\left[1 - \left[\frac{B_f}{B_x}\right]^{\frac{3}{2}}\right]$$

Choosing various convenient values of B_x; the corresponding distance x can be computed easily form the above equation.

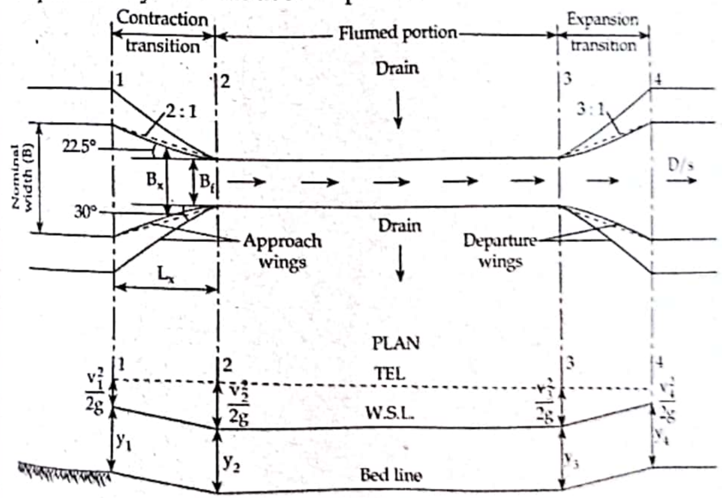


Figure 8.10: L-section

h) Design of bottom of floor of aqueduct and siphon aqueduct

Floor of aqueduct or siphon-aqueduct is subjected to uplift due to two causes.

a) Uplift due to water table

This force acts where the bottom floor is depressed below the drainage bed, especially in siphon aqueducts.

The maximum uplift under the worst condition would occur when there is no water flowing in the drain and the water table has risen up to the drainage bed. The maximum net uplift in such case would be equal to the difference in level between drainage bed and bottom of the floor, as shown in the figure.

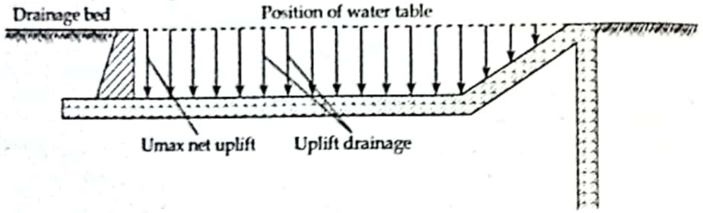


Figure 8.11

b) Uplift due to seepage of water from the canal to the drainage.

The maximum uplift due to this seepage occurs when the canal is running full and there is no water in the drain. The seepage pressures can be evaluated by Bligh's theory as explained below.

The seepage flow occurs from the beginning of pucca canal through (point a) and reappears in the drainage bed on either side of the impervious floor along the centre of the floor the first culvert bay(say point c or d) point b is the point under the centre of the first culvert bay.

The seepage path form 'a' to 'b' and from 'b' to 'c' can be known. The total creep length will be equal to be = ab + bc. If H is total seepage head (*i.e.*, H = F.S.L. of canal – d/s bed level of drain), the residual head at the point b (*i.e.*, H_b) is given by Bligh's theory equal to;

$$H_b = H - \left[\frac{H}{ab+bc} \times ab\right]$$

$$H_b = \left[\frac{H}{ab+bc} \times bc\right]$$

The floor of the siphon aqueduct must be designed for the total of lift which is equal to the sum of the uplift due to seepage plus the uplift due to static head. The total uplift may be partly be resisted by the weight of the floor and partly by bending in reinforcement.

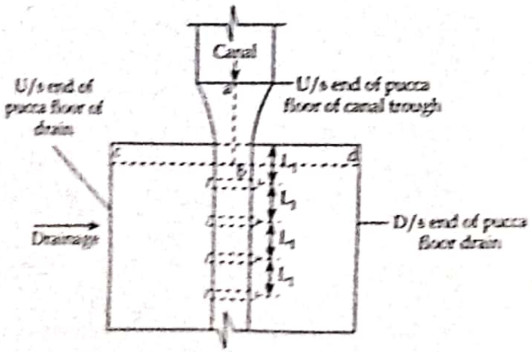


Figure 8.12

Example 8.1

Design a siphon aqueduct if the following data at the crossing of canal and drainage are given.

Discharge of canal = 50 cumecs
Bed width of canal = 30 m
Full supply depth of canal = 1.5 m
Bed level = 106 m
Side slope of canal = 1.5 H : 1 V
High flood discharge of drainage = 500 m
High flood level of drainage = 107 m
Bed level of drainage = 104 m
General ground level = 106 m

Solution:

Since the drainage is of large size and required length of aqueduct is also large so type III aqueduct is adopted.

Bed level of canal is slightly below drainage H.F.L. siphon aqueduct is adopted.

The earthen banks of the canal will be discontinued and canal water is taken in a concrete through.

Step 1: Design of drainage waterway

Lacey's regime perimeter $= P = 4.75\sqrt{Q} = 4.75\sqrt{500} = 106.21$ m

Provide 12 clear spans of 8 m each and let the width of each pier be 1.5 m

Length of water way $= 12 \times 8 + 11 \times 1.5 = 112.5$ m

Let the limiting velocity of through the siphon barrels is 2 m/sec.

$$\text{Height of barrels required} = \frac{\text{Discharge}}{\text{Velocity} \times \text{Width of waterway}}$$

$$= \frac{500}{2 \times 8 \times 12} = 2.6 \text{ m/sec.}$$

Hence, providing 12 rectangular barrels 8 m wide and 2.5 m height.

$$\text{Actual velocity through the barrels} = \frac{500}{2.5 \times 12 \times 8} = 2.08 \text{ m/sec.}$$

Step 2: Design of canal waterway

Normal bed width of canal = 30 m

Let the width be reduced to 15 m.

$$\text{Velocity through trough } \frac{50}{1.5 \times 15} = 2.22 \text{ m/sec.} < 3 \text{ m/sec. (O.K.)}$$

Providing a splay of 2 : 1 in contraction,

$$\text{Length of contraction transition} = \frac{30-15}{2} \times 2 = 15 \text{ m}$$

Providing a splay of 3 : 1 in expansion,

$$\text{Length of expansion transition} = \frac{30-15}{2} \times 3 = 15 \text{ m}$$

Length of flumed rectangular portion of the canal between abutments = 103 (provided). In transitions, the side slopes of the canal section shall be warped in plan from the original slope of 1.5 H : 1 V to vertical.

Step 3: Design of bed level at different sections

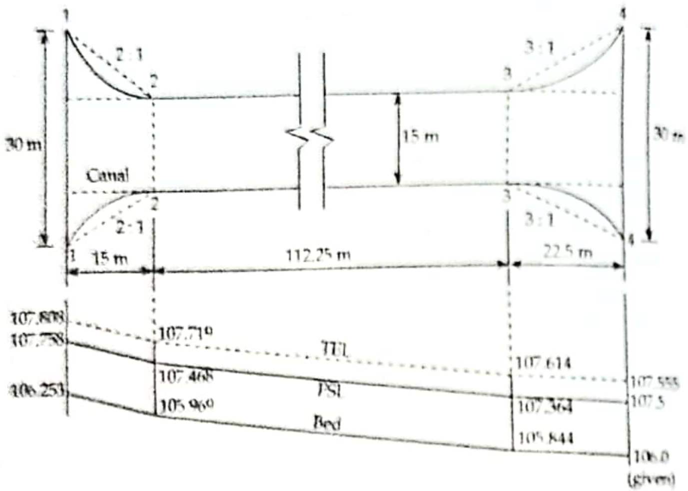


Figure 8.13

At section 4–4

When canal returns to its normal sections we have known conditions as follows.

Area of trapezoidal section $= (B + 1.5\,y)\,y$

$= (30 + 1.5 \times 1.5)1.5$

$= 48.37\ m^2$

where, B = bed width = 30 m

Y = depth = 1.5 m

Velocity of $V_4 = \frac{Q}{A} = \frac{50}{48.37} = 1.03\ m/sec.$

Velocity of head $= \frac{V_4^2}{2g} = \frac{(1.03)^2}{2g} = 0.055\ m$

R.L. of canal bed at 4–4 = 106 m (given)

Water depth = 1.5 m (given)

R.L. of water surface at 4–4 = 106 + 1.5 = 107.5 m

R.L. of T.E.L. at 4–4 = 107.5 + 0.055 = 107.555 m

At section 3–3

Assuming the constant depth of 1.5 m throughout the channel, we have at section 3-3, a rectangular channel, as follows:

Bed width = 15 m

Depth = 1.5 m (assumed constant)

Area = $15 \times 1.5 = 22.5\ m^2$

Velocity = $V_3 = \frac{50}{22.5} = 2.22\ m/sec.$

Velocity head $= \frac{V_3^2}{2g} = \frac{(2.22)^2}{2g} = 0.25\ m$

Assuming that the loss of head in expansion from section 3-3 to section 4-4 is taken as;

$$= 0.3\left[\frac{V_3^2 - V_4^2}{2g}\right] = 0.3\,[0.25 - 0.055] = 0.059\ m$$

R.L. of T.E.L. at 3-3 = R.L. of T.E.L. at 4-4 + Loss in expansion

= 107.555 + 0.059

= 107.614 m

R.L. of water surface at 3-3 = 107.614 − 0.0.25 = 107.364 m

R.L. of bed at 3-3 = 107.364 − 1.5 = 105.844 m

At section 2–2

From section 2-2 to 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 computed by Manning's formula as equal to:

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

where, n is rugosity coefficient, whose value in a concrete trough may be taken as 0.016 and L is the length of channel = 112.5 m.

Hydraulic mean depth (R) $= \frac{A}{P} = \frac{15 \times 1.5}{15 + 2 \times 1.5}$

$= \frac{22.5}{18} = 1.25\ m$

Velocity in trough (V) $= \frac{Q}{A} = \frac{50}{22.5} = 2.22\ m/sec.$

Head loss $(H_L) = \frac{n^2 \cdot V^2 \cdot L}{R^{\frac{4}{3}}} = \frac{(0.016)^2 \times (2.22)^2 \times 112.5}{(1.25)^{\frac{4}{3}}} = 0.105\ m$

R.L of T.E. L. at 2-2 = R.L. of T.E. L. at 3-3 + Head loss in trough

= 107.614 + 0.105 = 107.719 m

R.L of water level at 2-2 = 107.719 − Velocity head

= 107.719 − 0.25

= 107.469 m

R.L of bed at 2-2 = 107.469 −1.5 = 105.969 m

At section 1–1

Loss of head in contraction transition from section 1-1 to section 2-2 may be taken as;

$$= 0.2\left[\frac{V_2^2 - V_1^2}{2g}\right] = 0.2\left[\frac{(2.22)^2 - (1.05)^2}{2 \times 9.81}\right] = 0.089\ m$$

R.L. of T.E. L. at 1-1 = R.L. of T.E. L. at 2-2 + Loss in contraction

= 107.719 + 0.089

= 107.808 m

R.L. of water level at 1-1 = 107.808 − 0.055 = 107.758 m

R.L. of bed at 1-1 required to maintain constant depth = 107.758 − 1.5

= 106.253 m

All these levels are plotted and shown in above figure.

Step 4: Design of transition

i) Contraction transitions

Since depth is constant, the transition shall be designed on the basis of Mitra's hyperbolic transition equation 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 the above values; 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 o to 15 m, various values of B_x are worked out as shown in table.

The distance 'x' is measured form the flumed section 2–2'

X (m)	0	2	4	6	8	10	12	14	15
$B_x = \frac{450}{30-x}$ (in m)	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,

$B_f = 15$ m

$B_n = 30$ m

$L_f = 22.5$ m

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{B_n \cdot L_f - x(B_n - B_f)} = \frac{30 \times 15 \times 15}{30 \times 15 - x(30-15)} = \frac{675}{45 - x}$$

For various values of 'x' lying between 0 to 22.5 m, corresponding values of B_x are worked out, as shown in the table.

The distance 'x' is measured from the flumed section 3-3.

X (m)	0	2	4	6	8	10	12	14	16	18	20	22.5
$B_x = \frac{675}{45-x}$ (m)	15	15.7	16.46	17.3	18.25	19.3	20.4	21.75	23.3	25	27	30

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 rod (5 m wide) shall be carried on the extreme left compartments, as shown in the figure. A free board of 0.6 m above the normal water depth of 1.5 m is sufficient, and hence the bottom level of bridge slab may be kept at 1.5 + 0.6 = 2.1 m above the bed level of the trough. The height of the trough will also be kept equal to 2.1 m. The entire trough section can be designed as monolithic reinforced concrete structure by the usual structural methods. The tentative thickness may be used as follows.

Outer wall = 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,

Overall outer width of trough (including wall) $= 15 + 2 \times 0.3 + 2 \times 0.4$
$= 16.4$ m

Hence, the length of siphon barrel = 16.4 m.

Step 6: Head loss through the siphon barrels

The head loss through the siphon barrels is given by Unwin's formula as equal to (neglecting velocity of approach)

$$h = \left[1 + f_1 + f_2 \times \frac{L}{R}\right]\frac{V^2}{2g}$$

where, V is the velocity through the barrels = 2.08 m/sec.

f_1 is the coefficient of head loss at entry = 0.505

$$f_2 = \alpha\left[1 + \frac{b}{R}\right]$$

where, the values of α and bare taken from table 7.1.

forcement plastered barrel

$\alpha = 0.00316$

$b = 0.030$

R is the hydraulic mean depth for barrel $= \frac{A}{P} = \frac{8 \times 2.5}{2(8 + 2.5)} = \frac{20}{21} = 0.953$

L is the length of barrel = 16.4 m

Substituting these values; we get,

$$f_2 = 0.00316\left[1 + \frac{0.030}{0.953}\right] = 0.00326$$

$$\therefore \quad h = \left[1 + 0.505 + 0.00326 \times \left(\frac{16.4}{0.953}\right)\right]\frac{(2.08)^2}{2 \times 9.81} = 0.344 \text{ m}$$

High flood level of drainage = 107.0 m (Given)

d/s H.F.L. = 107.0 m

Afflux (h) = 0.344 m

u/s H.F. L. = d/s H.F.L. + Afflux (or loss of head)

= 107.0 + 0.344

= 107.344 m

Step 7: Uplift pressure on roof barrels

R.L. of bottom of trough = R. L. of canal bed – Slab thickness

= 106 – 0.4

= 105.6 m

Loss of head at entry of barrel $= 0.505 \times \frac{V^2}{2g}$

$$= 0.505 \times \frac{(2.08)^2}{2 \times 9.81}$$

$= 0.111$ m

Uplift on roof = u/s H.F.L. – Loss at entry – Level of underside of roof slab

= 107.344 – 0.111 – 106.0

= 1.223 m of water

= 12.23 kN /m² (1.223 t/m²)

Assuming unit weight of water = 10 kN/m² or 1 t/m²

The concrete trough slab is 0.4 thick and thus exert a downward load of 0.4 × 24 = 0.96 kN/m².

Assuming unit weight of concrete = 24 kN/m²

The balance of the uplift pressure *i.e.,* 12.23 – 9.6 = 2.63 kN/m² 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 (1.5 m of water) plus self-weight, when the drainage water is low and not exerting any uplift. Suitable reinforcement at bottom of slab may be provided for this downward force.

Step 8: Uplift in the bottom floor of barrel

i) Static head

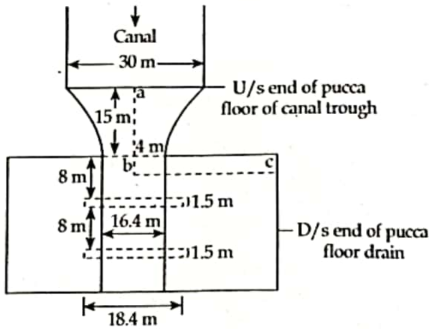


Figure 8.14

R.L. of barrel floor = R.L. of trough bottom – Height of barrel

= 105.6 – 2.5 = 103.1 m

Let us assume that the thickness of 0.8 m is provided.

∴ R.L. of bottom of floor = 103.1 – 0.8 = 102.3 m

Bed level of the drain = 104 m

Assuming that the water-table has gone up to bed level of drain,

Static uplift on the floor (refer figure) = 104 – 102.3 = 1.7 m of water

ii) Seepage head

The seepage head will be maximum when the canal is running full and the drain is dry. Thus,

Total seepage head = F.S.L. of canal – Bed level of drain

= (106 + 1.5) – 104

= 3.5 m

The residual head at a point 'a' in the centre of the first barrel (figure B) has been calculated by Bligh's theory as follows.

Assuming that;

Total length of drainage floor = 30 m

The seepage line abc will traverse creep lengths as follows;

ab = Length of u/s transition + half the barrel span = 15 + 4 = 19 m

bc = 15 m (Half of the total length of 30 m (assumed)

Total creep length = 19 + 15 = 34 m

Residual seepage head at b = $3.5\left[1-\frac{19}{34}\right] = 3.5\times\frac{15}{34} = 1.55$ m

Total uplift = State head + Seepage head

= 1.7 + 1.55 = 3.25 m of water

= 32.5 kN/m²

The provided 0.8 m thickness of slab will resist due to its own weight,

Uplift = 0.8 × 24 = 19.2 kN/m²

Therefore, balance to be resisted by reinforcement due 't' bending action = 32.5 – 19.2 = 13.3 kN/m².

Suitable reinforcement for the uplift (13.3 kN/m²) has been provided at the top of the culvert floor so as to counteract the bending action.

NOTE

The length of the floor as been provided equal to 32 m, as shown in the figure below on the following considerations;

Length of floor required under barrel = 16.4 m

Extra floor length required to accommodate pier;

Noses on both sides = 2.0 m

Horizontal length of d/s ramp jointing to bed level at slope of 5 : 1 is;

= 5 (104 – 103.1) = 4.5 m

Width of d/s cut-off beyond ramp = 0.6 m

Length of extra floor provided in u/s side = 6 m

Total length = 29.5 m

Step 9: Design of cutoff and protection works for the drainage floor

Depth of scour (R) = $0.47\left[\frac{Q}{f}\right]^{\frac{1}{3}}$

Assuming f = 1.0;

$R = 0.47\left[\frac{500}{1}\right]^{\frac{1}{3}} = 0.47 \times 7.94 = 3.73$ m

Provide depth of cut-off for scour hole of 1.5 R on both sides; we have,

Depth of u/s cut off below H.F.L. = 1.5R = 1.5 × 3.73 = 5.6 m

R.L. of bottom of u/s cut-off = u/s H.F. L. – 5.6

= 107.344 – 5.6 = 101.744 m

R.L. of bottom of d/s cut-off = d/s H.F. L. – 5.6

$= 107.0 - 5.6 = 101.4$ m

Length of u/s protection (*i.e.*, 40 cm thick brick pitching) is;

= 2[R.L. of u/s bed − R.L. of bottom of u/s cut-off]

= 2[103.1 − 101.744]

$= 2 \times 1.356$

= 2.71 m (provide 3 m)

Similarly,

Length of d/s brick pitching = 2[R.L. of d/s bed − R.L. of bottom of d/s cut-off]

= 2[104 − 101.6]

$= 2 \times 2.6$

= 5.2 m

The pitching may be supported by 0.4 m wide and 1 m deep toe walls, as shown in the figure below.

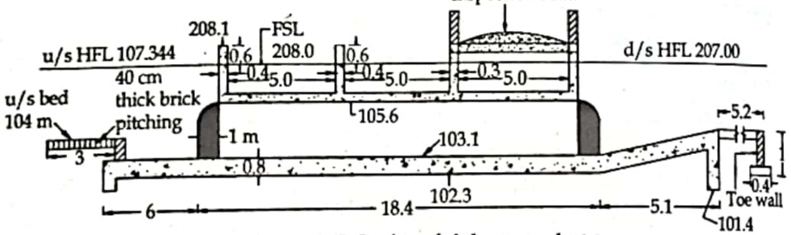


Figure 8.15: Section of siphon aqueduct

8.3 WORKED OUT PROBLEMS

PROBLEM 1

Canal data	Drainage data
Discharge = 20 m³/sec.	**Discharge = 200 m³/sec.**
Depth of water 1.5 m	**HFL = 250.7 m**
FSL = 251.5 m	**Bed level = 248.5 m**
Ground level = 250.0 m	

From above data design following components of siphon aqueduct.

i) Canal waterway

ii) Drainage waterway

iii) Afflux and head loss through siphon barrel

iv) Uplift on drainage slab

Solution:

i) Design of drainage waterway

Here,

Lacey's regime perimeter (P) $= 4.75\sqrt{Q} = 4.75\sqrt{200} = 67.18$ m

Provide 8 clear span of 8 m each and width of pier = 1.5 m.

Total length of waterway $= 8 \times 8 + 1.5 \times 7 = 74.5$

Let, limiting velocity through siphon barrels is 2 m/ sec.

$$\text{Height of barrels required} = \frac{\text{Discharge}}{\text{Velocity} \times \text{Clear width of waterway}}$$

$$= \frac{200}{2 \times 64}$$

$= 1.563$ m

Provide 8 rectangular barrels, each 8 m wide and 1.6 m high.

$$\text{Actual velocity through barrels} = \frac{200}{8 \times 8 \times 1.6}$$

$= 1.95$ m/sec.

ii) Design of canal waterway

Let, Bed width of canal = 30 m

Width be reduced to 15 m

Providing a splay of 2 : 1 in contraction;

$$\text{Length of contraction transition} = \frac{30 - 15}{2} \times 2 = 15 \text{ m}$$

Providing 3 : 1 splay in expansion transition;

$$\text{Length of expansion transition} = \frac{30 - 15}{2} \times 3 = 22.5 \text{ m}$$

Length of flumed rectangular portion = 74.5 m

iii) Afflux or head loss through siphon barrels

$$\text{Head loss through the siphon barrels (h)} = \left[1 + f_1 + f_2 \cdot \frac{L}{R}\right]\frac{V^2}{2g}$$

$V = 1.95$ m/sec.

$f_1 = 0.505$ = Coefficient of head loss at entry

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

For cement plastered barrels

$a = 0.00316$

$b = 0.030$

$$R = \frac{A}{P} = \frac{8 \times 1.6}{2(8+1.6)} = 0.67$$

$L =$ Length of barrel $= 15 + 0.3 \times 2 + 0.4 \times 2 = 16.4$ m

Width of inner wall $= 0.3$ m

Width of outer wall $= 0.4$ m

$$f_2 = 0.00316\left(1 + \frac{0.030}{0.67}\right) = 0.0033$$

$$h = \left[1 + 0.505 + 0.0033 \times \frac{16.4}{0.67}\right] \times \frac{(1.95)^2}{19.62} = 0.306$$

d/s HFL $= 250.7$ m

u/s HFL $= 250.7 +$ Afflux(h) $= 250.7 + 0.306 = 251.00$ m

iv) Uplift on drainage slab

a) Static head

R.L. of trough bottom = FSL of canal − Water depth − Thickness of trough

$= 251.5 - 1.5 - 0.4$

$= 249.6$

R.L. of barrel floor = R.L. of trough bottom − Height of barrel

$= 249.6 - 1.6$

$= 248$ m

Let, thickness of floor $= 0.8$ m

R.L. of bottom of barrel floor $= 248.0 -$ Thickness of floor

$= 247.2$ m

Bed level of drain $= 248.5$ m (Given)

Assuming that the water table has gone to bed level of drain,

Static uplift on the floor $= 248.5 - 247.2 = 1.3$ m of water

b) Seepage head

The seepage head will be maximum when the canal is running full and the drain is dry. Thus,

Total seepage head = F.S.L. of canal − bed level of drain

$= 251.5 - 248.5$ m $= 3$ m

The residual seepage head at a point 'b' in the centre of first barrel is calculated by Bligh's theory.

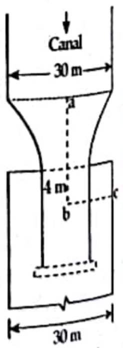


Figure 8.16

Assuming;

Total length of drainage floor $= 30$ m

ab $= 15 + 4 = 19$ m

bc $= 15$ m

Total creep length $= 34$ m

Residual seepage head $= 3 \times \frac{15}{34} = 1.32$ m

Total uplift = Static head + Seepage head

$= 1.3 + 1.32 = 2.62$ m

$= 26.2$ kN/m^2

Balanced to be resisted by reinforcement due to bending action is;

$= 26.2 - 0.7 \times 24$

$= 7$ kN/m^2

PROBLEM 2

Find out the waterway, bed level and FSL of a suitable cross drainage structure for the following data given below. The structure should be flumed to achieve economy.

Parameters	Canal	Drainage
Q_{max} (m^3/sec.)	30	200
B (m)	20	80
Bed level at d/s	200	198
FSL/HFL	202	201

Assuming Manning's rugosity coefficient as 0.014 for concrete.

Solution:

Here, bed level of canal is below HFL of drainage so, siphon aqueduct is provided.

i) Drainage waterway

Lacey's Regime parameter

$$P = 4.75\sqrt{Q} = 4.75\sqrt{200} = 67.18 \text{ m}$$

Provide 8 clear span of 8 m each and 1.5 m width pier.

Total length of waterway $= 8 \times 8 + 7 \times 1.5 = 74.5$ m

Let, Limiting velocity through siphon barrels $= 2.0$ m/sec.

$$\text{Height of barrel required} = \frac{\text{Discharge}}{\text{Velocity} \times \text{Clear width of waterway}}$$

$$= \frac{200}{2 \times 8 \times 8}$$

$= 1.563$ m

Provide 8 rectangular barrels each 8 m wide and 1.6 m high

$$\text{Actual velocity through siphon barrels} = \frac{200}{8 \times 8 \times 1.6}$$

$= 1.95$ m/sec.

ii) Design of canal waterway

Normal bed width $= 30$ m

Let, the width be reduced to 15 m.

Providing a splay of 2 : 1 in contraction, Length of contraction transition $= \frac{30-15}{2} \times 2 = 15$ m

Providing a splay of 3 : 1 expansion,

Length of expansion transition $= \frac{30-15}{2} \times 3 = 22.5$ m

Length of flumed rectangular portion of canal between abutments = 74.5 m

Let, Side slope of canal = 1.5 : 1

In transitions, the side slope of canal section shall be warped in plan from the original slope of $\frac{1}{2}$H : 1V to vertical.

iii) Design of bed levels at different section

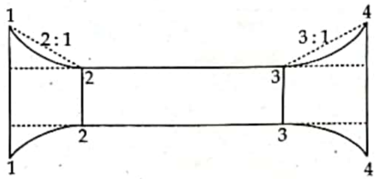


Figure 8.17

At section 4 – 4

Area of trapezoidal canal section $= (B + 1.5y)y = (30 + 1.5 \times 2)2$
$= 66 \text{ m}^2$

$$V_4 = \frac{\text{Discharge}}{\text{Area}} = \frac{30}{66} = 0.455 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V_4^2}{2g} = \frac{(0.455)^2}{19.62} = 0.01 \text{ m}$$

R.L. of canal bed at 4 – 4 = 200 m

R.L. of water surface at 4 – 4 = 202 m

R.L. of TEL at 4 – 4 = 202.0 + 0.01 = 202.01 m

At section 3 – 3

Assuming constant depth of 2.0 m throughout channel, we have at 3 - 3, a rectangular channel,

Bed width = 15 m

$$\text{Velocity} = \frac{30}{2 \times 15} = 1 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V_3^2}{2g} = \frac{(1)^2}{19.62} = 0.05 \text{ m}$$

$$\text{Loss of head in expansion from } 3-3 \text{ to } 4-4 = 0.3\left(\frac{V_3^2 - V_4^2}{2g}\right)$$
$$= 0.3(0.05 - 0.01)$$
$$= 0.012 \text{ m}$$

R.L. of TEL at 3 – 3 = R.L. of TEL at 4– 4 + Loss in expansion
= 202.01 + 0.012 = 202.022 m

R.L. of water surface at 3 – 3 = 202.022 – 0.05 = 201.972 m

R.L. of bed at 3 – 3 = 201.972 – Water depth
= 201.972 – 2 = 199.972 m

At section 2 – 2

Loss in friction between section 2 – 2 and 3 – 3

Given that;

n = 0.014

$$R = \frac{A}{P} = \frac{15 \times 2}{(15 + 2 \times 2)} = 1.58$$

$$\text{Head loss } (H_L) = \frac{n^2V^2L}{R^{\frac{4}{3}}} = \frac{(0.014)^2 \times (1)^2 \times 74.5}{(1.58)^{\frac{4}{3}}} = 0.008 \text{ m}$$

R.L. of TEL at 2 – 2 = R.L. of TEL at 3-3 + Head loss in trough
= 202.022 + 0.008
= 202.03 m

R.L. of water level at 2 – 2 = 202.03 – Velocity head at 2-2
= 202.03 – 0.05
= 201.98 m

R.L. of bed at section 2 – 2 = 201.98 – 2 = 199.98 m

At section 1 – 1

$$\text{Head loss in contraction from } 1-1 \text{ to } 2-2 = 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$$
$$= 0.2(0.05 - 0.01)$$
$$= 0.008 \text{ m}$$

R.L. of TEL at 1 – 1 = R.L. of TEL at 2-2 + Loss in contraction
= 202.03 + 0.008
= 202.038

R.L. of water surface at section 1 – 1 = 202.038 – Velocity head
= 202.038 – 0.01
= 202.028 m

R.L. of bed at 1 – 1 = 202.028 – 2 = 200.008 m

PROBLEM 3

Compute the waterway of canal and drainage of a suitable cross drainage work and draw a schematic neat sketch of the designed structure

$Q_{canal} = 20 \text{ m}^3/\text{sec.}$

$Q_{drain} = 200 \text{ m}^3/\text{sec.}$

FSL depth = 1.40 m

HFL of drain = 101.5 m

Bed level of canal = 100.00 m

Bed level (drain) = 98.00 m

Bed width = 16 m

Solution:

Here,

$Q_{drain} \gg Q_{canal}$

and, FSL of drain = 101.5 m is higher than bed level of canal = 100.00 m.

So, a siphon aqueduct is provided.

i) Design of drainage waterway

Using Lacey's perimeter formula; we have,

$$P = 4.75\sqrt{Q} = 4.75\sqrt{200} = 67.18 \text{ m}$$

Provide 8 clear span of 8 m each and 1.5 m width pier.

$$\text{Total length of waterway } = 8 \times 8 + 1.5 \times 7 = 74.5$$

Let limiting velocity through siphon barrels is 2 m/ sec.

$$\text{Height of barrels required} = \frac{\text{Discharge}}{\text{Velocity} \times \text{Clear width of waterway}} = \frac{200}{2 \times 64} = 1.563 \text{ m}$$

Provide 8 rectangular barrels, each 8 m wide and 1.6 m high.

$$\text{Actual velocity through barrels} = \frac{200}{8 \times 8 \times 1.6} = 1.95 \text{ m/sec.}$$

ii) Design of canal waterway

Normal bed width of canal = 16 m

Let, the width be reduced to 10 m.

Providing a splay of 2 : 1 on contraction,

$$\text{Length of contraction transition} = \frac{16-10}{2} \times 2 = 6 \text{ m}$$

Providing a splay of 3 : 1 in expansion,

$$\text{Length of expansion transition} = \frac{16-10}{2} \times 3 = 9 \text{ m}$$

Length of flumed rectangular portion of canal between abutments = 74.5 m

In transitions, side slope of canal section shall be warped in plan from the original slope.

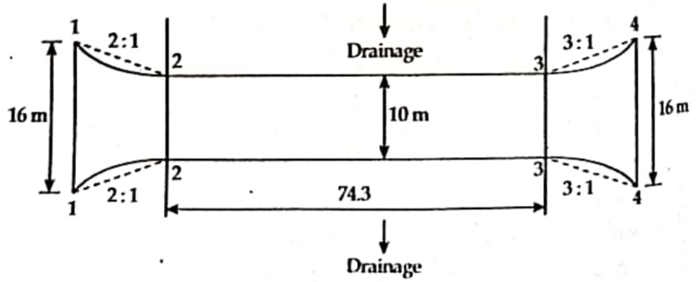


Figure 8.18

PROBLEM 4

Design siphon aqueduct (Drainage waterway, canal waterway, Bed levels and transitions) if the following data at crossing of canal and drainage are given:

Discharge of canal = 50 cumecs
Bed width of canal = 32 m
Full supply depth of canal = 1.80 m
Can bed level = 200.0 m
Side slopes of canal = 1.5H : 1V
High flood discharge of drainage = 400 cumecs
High flood level of drainage = 200.6 m
Bed level of drainage = 198.0 m
General ground level = 200.20 m

Solution:

i) Design of drainage waterway

$$\text{Lacey's regime perimeter} = P = 4.75\sqrt{Q} = 4.75\sqrt{400} = 95 \text{ m}$$

Let provide 11 clear spans of 8 m each and width of each pier is 1.5 m.

$$\text{Total length of waterway} = 8 \times 11 + 1.5 \times 10 = 103 \text{ m}$$

Let, Limiting velocity through siphon barrel = 2 m/sec.

$$\text{Height of barrel required} = \frac{400}{8 \times 11 \times 2.0} = 2.27 \text{ m}$$

Provide 11 rectangular barrels, each 8 m wide and height 2.3 m.

$$\text{Actual velocity through barrels} = \frac{400}{8 \times 11 \times 2.3} = 1.98 \text{ m/sec.}$$

ii) Design of canal waterway

Normal bed width = 32 m

Let the width be reduced to 15 m.

$$\text{Velocity through trough} = \frac{50}{15 \times 1.8} = 1.85 \text{ m/sec.} < 3 \text{ m/sec. (O.K.)}$$

Providing a splay of 2 : 1 on the contraction,

$$\text{Length of contraction transition} = \frac{32-15}{2} \times 2 = 17 \text{ m}$$

Providing a splay of 3 : 1 in expansion,

$$\text{Expansion transition length} = \frac{32-15}{2} \times 3 = 25.5 \text{ m}$$

Length of flumed rectangular portion = 103 m

In transitions side slopes of the canal section shall be warped in plan from the original slope of 1.5 W : 1 V to vertical.

iii) Design of bed levels at different sections

At section 4 – 4

$$V_4 = \frac{Q}{A} = \frac{Q}{(B + 1.5y)y} = \frac{50}{(32 + 1.5 \times 1.8)1.8} = 0.8 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V_4^2}{2g} = \frac{(0.8)^2}{19.62} = 0.033 \text{ m}$$

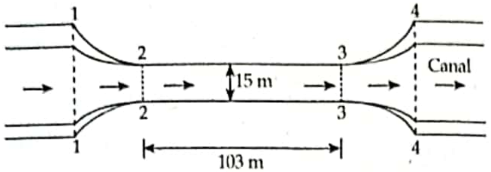


Figure 8.19

R.L. of canal bed at 4 - 4 = 200.0 m

R.L. of water level at 4 - 4 = 200.0 + 1.8 = 201.8 m

R.L. of TEL at 4 - 4 = 201.8 + Velocity head

= 201.8 + 0.033 = 201.833 m

At section 3 – 3

Assuming water depth is constant = 1.8 m

At section 3 - 3 section is rectangular

Velocity at 3 - 3 = $V_3 = \frac{Q}{A} = \frac{50}{15 \times 1.8} = 1.85$ m/sec.

Velocity head = $\frac{(1.85)^2}{2g} = 0.175$ m

Loss of head in expansion from 3 - 3 to 4 - 4 $= 0.3\left(\frac{V_3^2 - V_4^2}{2g}\right)$

$= 0.3(0.175 - 0.033)$

$= 0.042$ m

R.L. of TEL at 3 - 3 = R.L. of TEL at 4- 4 + Loss in expansion

= 201.833 + 0.042

= 201.876 m

R.L. of water surface at 3 - 3 = 201.876 – 0.175 = 201.7 m

R.L. of bed at 3 - 3 = 201.7 – 1.8 = 199.9 m

At section 2 – 2

Friction loss between 2 - 2 and 3 - 3;

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

n = 0.016; for concrete

$$R = \frac{A}{P} = \frac{15 \times 1.8}{(15 + 2 \times 1.8)} = 1.45$$

$$\therefore \quad H_L = \frac{(0.016)^2 \times (1.85)^2 \times 103}{(1.45)^{\frac{4}{3}}} = 0.055 \text{ m}$$

R.L. of TEL at 2 - 2 = R.L. of TEL at 3-3 + H_L

= 201.876 + 0.055 = 201.93 m

R.L. of water level at 2 - 2 = 201.93 – Velocity head

= 201.93 – 0.175 = 201.76 m

R.L. of bed level at 2 - 2 = 201.76 – 1.8 = 199.96 m

At section 1 – 1

Head loss in contraction from 1 - 1 to 2 - 2 $= 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$

$= 0.2(0.175 - 0.033)$

$= 0.028$ m

R.L. of TEL at 1 - 1 = R.L. of TEL at 2-2 + Head loss

= 201.93 + 0.028

= 201.96 m

R.L. of water surface = 201.96 – 0.033

= 201.925

R.L. of bed at 1 - 1 = 201.925 – 1.8

= 200.125 m

iv) Design of transition

a) Contraction transition

Applying Mitra's hyperbolic transition equation; we have,

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{B_n \cdot L_f - x(B_n - B_f)}$$

$B_n = 32$ m

$B_f = 15$ m

$L_f = 17$ m

$$\therefore \quad B_x = \frac{32 \times 15 \times 17}{32 \times 17 - x(32 - 15)}$$

$$= \frac{8160}{544 - 17x}$$

$$= \frac{480}{32 - x}$$

x (m)	0	2	4	6	8	10	12	14	16	17
$B_x = \frac{480}{32 - x}$	15	16	17.14	18.46	20	21.82	24	26.67	30	32

x is measured from the flumed section.

b) Expansion transition

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{B_n \cdot L_f - x(B_n - B_f)}$$

where, $L_f = 25.5$ m.

$$\therefore \quad B_x = \frac{32 \times 15 \times 25.5}{32 \times 25.5 - x(32 - 15)}$$

$$= \frac{12240}{816 - 17x}$$

$$= \frac{720}{48 - x}$$

x (m)	0	2	4	6	8	10	12	14	16	18	20	22	24	25.5
$B_x = \frac{720}{48 - x}$	15	15.65	16.36	17.14	18	18.94	20	21.176	22.5	24	25.71	27.7	30	32

PROBLEM 5

Following data are obtained at the crossing of a canal and a drainage

Canal data	**Drainage data**
$Q = 20\ m^3/sec.$	**$Q = 200\ m^3/sec.$**
Depth of water 1.5 m	**HFL = 150.7 m**
FSL = 151.5 m	**Bed level = 148.5 m**
Bed width = 12 m	**Ground level = 150.0 m**
Side slope (H : V) = (1.5 : 1)	

Design the following component of siphon aqueduct

i) Drainage waterway
ii) Canal waterway
iii) Transition
iv) Uplift **[2070 Bhadra]**

Solution:

i) Design of drainage waterway

Lacey's regime perimeter $(P) = 4.75\sqrt{Q} = 4.75\sqrt{200} = 67.18\ m$

Provide 8 clear span of 8 m each and pier 1.5 m width.

Total length of waterway $= 8 \times 8 + 1.5 \times 7 = 74.5$

Let, limiting velocity through siphon barrels is 2.0 m/sec.

$$\text{Height of barrel} = \frac{Q}{8 \times 8 \times \text{Velocity}} = \frac{200}{2 \times 64} = 1.563\ m$$

Provide 8 rectangular barrels each 8 m wide and 1.6 m high.

$$\text{Actual velocity through barrels} = \frac{200}{8 \times 8 \times 1.6} = 1.95\ m/sec.$$

ii) Design of canal water way

Normal bed width of canal = 12 m

$$\text{Velocity in trough} = \frac{Q}{A} = \frac{20}{6 \times 1.5} = 2.22\ m/sec. < 3\ m/sec.\ (O.K.)$$

Let, the width be reduced to 6 m.

Providing a splay of 2 : 1 in contraction,

$$\text{Length of contraction transition} = \frac{12-6}{2} \times 2 = 6\ m$$

Providing a splay of 3 : 1 in expansion transition,

$$\text{Length of expansion transition} = \frac{12-6}{2} \times 3 = 9\ m$$

Length of flumed rectangular portion = 74.5 m

In transitions side slopes of the canal section is warped in plan from the original slope of 1.5 H : 1 V to vertical.

iii) Transition

a) Contraction transition

Using Mitra's hyperbolic transition equation; we have,

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{B_n \cdot L_f - x(B_n - B_f)}$$

$B_n = 12\ m$

$B_f = 6\ m$

$L_f = 6\ m$

$$\therefore\ B_x = \frac{12 \times 6 \times 6}{12 \times 6 - x(12-6)} = \frac{432}{72 - 6x} = \frac{72}{12 - x}$$

x in m	0	2	4	6
$B_x = \frac{72}{12-x}$	6	7.2	9	12

b) Expansion transition

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{B_n \cdot L_f - x(B_n - B_f)}$$

$B_n = 12\ m$

$B_f = 6\ m$

$L_f = 9\ m$

$$\therefore\ B_x = \frac{12 \times 6 \times 9}{12 \times 9 - x(12-6)} = \frac{648}{108 - 6x} = \frac{108}{18 - x}$$

x in m	0	2	4	6	8	9
$B_x = \frac{108}{18-x}$	6	6.75	7.14	9	10.8	12

iv) Uplift

a) Static head

R.L. of canal bed = R.L. of FSL − Water depth
= 151.5 − 1.5 = 150 m

Let, Thickness of trough = 0.4 m

R.L. of through bottom = 150 − 0.4 = 149.6 m

R.L. of barrel floor = R.L. of trough bottom − Height of barrel
= 149.6 − 1.6 = 148 m

Let, Thickness of barrel floor = 0.8 m

R.L. of bottom of barrel floor = 148 − 0.8 = 147.2 m

Bed level of drain = 148.5 m (given)

Maximum static uplift = 148.5 − 147.2 = 1.3 m of water

b) Seepage head

Maximum seepage head = F.S.L. of canal
−Bed level of drain
= 151.5 − 148.5
= 3 m

ab = 6 + 4 = 10 m

$$bc = \frac{12}{2} = 6\ m$$

Residual seepage head at point 'b' according to Bligh's theory is;

$$= \frac{3}{16} \times 6 = 1.125\ m$$

Total uplift = 1.3 + 1.125 = 2.425 m

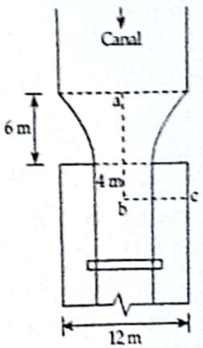


Figure 8.20

$= 24.25\ kN/m^2$

Balanced to be resisted by reinforcement $= 24.25 - 0.8 \times 24$

$= 5.05\ kN/m^2$

PROBLEM 6

Design a siphon aqueduct across stream for the following data.

Canal	**Stream**
Full supply discharge = 56 m^3/sec.	**High flood discharge = 425 m^3/sec.**
Bed width = 32 m	**High flood level = 268.20 m**
Depth of flow = 2.0 m	**Bed level = 265.50 m**
	General ground level = 267.2 m

Make suitable assumption, if required. Draw plan and sectional elevation of the designed siphon aqueduct. **[2070 Magh]**

Solution:

i) Design of drainage waterway

Lacey's regime perimeter $= 4.75\sqrt{Q}$

$= 4.75\sqrt{425} = 97.923\ m$

Provide 11 spans of 8 m each and piers width of 1.5 m.

Total length of waterway $= 11 \times 8 + 10 \times 1.5 = 103\ m$

Let, limiting velocity = 2.0 m/sec.

$$\text{Height of barrel required} = \frac{Q}{\text{Velocity} \times 8 \times 11}$$

$$= \frac{425}{2 \times 8 \times 11} = 2.41\ m$$

Let provide 11 rectangular barrels each 8 m wide and 2.5 m high.

$$\text{Velocity through barrels} = \frac{425}{2.5 \times 8 \times 11}$$

$= 1.932$ m/sec.

ii) Design of canal waterway

Normal bed width = 32 m

Let, the width is reduced to 15 m.

Providing a splay of 2 : 1 in contraction,

$$\text{Contraction transition length} = \frac{32 - 15}{2} \times 2 = 17\ m$$

Providing a splay of 3 : 1 in expansion,

$$\text{Expansion transition length} = \frac{32 - 15}{2} \times 3 = 25.5\ m$$

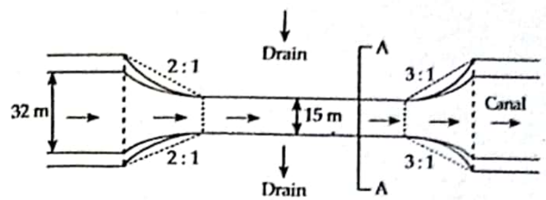
Length of flumed rectangular portion of canal between abutments = 103 m

In transitions, the side slopes of canal shall be warped in plan from the original side slope of canal.

Assume,

Bed level of canal = 267.0 m

Plan and sectional elevation of designed siphon aqueduct is given below.



PLAN

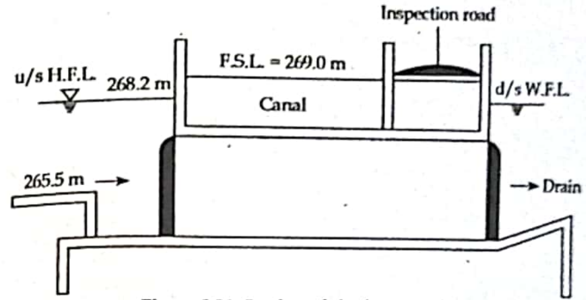


Figure 8.21: Section of sipohon aqueduct

PROBLEM 7

Determine bed and water levels at four critical locations of the canal waterway at transition of siphon aqueduct designed with the following data. **[2071 Bhadra]**

Canal	**Drainage**
Full supply discharge = 40 m^3/sec.	**Max. flood discharge = 520 m^3/sec.**
Full supply level = 151.8 m	**High flood level = 150.6 m**
Side slope = 1.5 : 1	**Bed level = 148.2 m**
Depth of water = 1.5 m	**Normal ground level = 150.00 m**
Bed level = 150.00 m	
Bed width = 32 m	

Solution:

Here,

Bed width of canal = 32 m (given)

Let, width be reduced to 15 m. Then,

$$\text{Velocity through canal trough} = \frac{Q}{A} = \frac{40}{15 \times 1.5}$$

$= 1.77$ m/sec. < 3 m/sec. (Ok)

and, Lacey's regime perimeter (P) $= 4.75\sqrt{Q} = 4.75\sqrt{520} = 108.31\ m$

Provide 12 clear span of 8 m each with 1.5 m width pier.

Total length of flumed section $= 12 \times 8 + 11 \times 1.5 = 112.5\ m$

Design of bed levels at different section

At section 4–4

Area of trapezoidal canal section $= (B + 1.5y)y$

$= (30 + 1.5 \times 1.5) \times 1.5$

$= 48.375\ m^2$

$V_4 = \frac{\text{Discharge}}{\text{Area}} = \frac{40}{48.375} = 0.827\ m/sec.$

Velocity head $= \frac{V_4^2}{2g} = \frac{(0.827)^2}{19.62}$

$= 0.035\ m$

R.L. of canal bed at 4–4 = 150 m

R.L. of water level at 4–4 = 150 + 1.5

$= 151.5\ m$

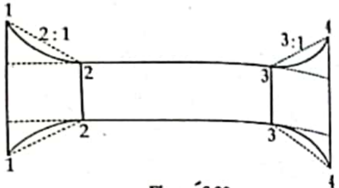


Figure 8.22

R.L. of TEL at 4–4 = 151.5 + 0.035

= 151.535 m

At section 3–3

Assuming constant depth of 1.5 m throughout the channel, we have, at 3-3 a rectangular channel.

Velocity $(V_3) = \frac{Q}{A} = \frac{40}{15 \times 1.5} = 1.77\ m/sec.$

Velocity head $= \frac{V_3^2}{2g} = \frac{(1.77)^2}{19.62} = 0.16\ m$

Loss of head in expansion from 3-3 to 4-4 $= 0.3\left(\frac{V_3^2 - V_4^2}{2g}\right)$

$= 0.3(0.16 - 0.035)$

$= 0.0375\ m$

R.L. of TEL at 3–3 = R.L. of TEL at 4–4 + Loss in expansion

= 151.535 + 0.0375

= 151.57 m

R.L. of water level at 3–3 = 151.57 – Velocity head

= 151.57 – 0.16 = 151.41 m

R.L. of bed level at 3–3 = 151.41 – Water depth

= 151.41 – 1.5 = 149.91 m

At section 2–2

Here,

n = 0.016 [For concrete]

$R = \frac{A}{P} = \frac{15 \times 1.5}{(15 + 2 \times 1.5)} = 1.25\ m$

Head loss $(H_L) = \frac{n^2V^2L}{R^{\frac{4}{3}}} = \frac{(0.016)^2 \times (1.77)^2 \times 112.5}{(1.25)^{\frac{4}{3}}} = 0.09\ m$

R.L. of TEL at 2–2 = R.L. of TEL at 3–3 + Head loss in trough

= 151.57 + 0.09

= 151.66 m

R.L. of water surface at 2–2 = 151.66 – Velocity head at 2–2

= 151.66 – 0.16

= 151.5 m

R.L. of bed level at 2–2 = 201.98 – Water depth

= 151.5 – 1.5 = 150 m

At section 1–1

Here,

Head loss in contraction from 1-1 to 2-2 $= 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$

$= 0.2\left(\frac{0.16 - 0.035}{19.62}\right)$

$= 0.025\ m$

R.L. of TEL at 1–1= R.L. of TEL at 2–2 + Loss in contraction

= 151.66 + 0.025

= 151.685 m

R.L. of water surface = R.L. of TEL at 1–1 – Velocity head

= 151.685 – 0.035

= 151.65 m

R.L. of bed level = 151.65 – 1.5 = 150.15 m

PROBLEM 8

Following data are obtained at the crossing of a canal and drainage:

Canal	**Drainage**
Discharge = 36 cumecs	**Discharge = 400 cumecs**
Full supply depth = 1.5 m	**H.F.L. = 211 m**
Bed width = 28 m	**Bed width = 14 m**
Bed level = 210.4 m	**Bed level = 208.6 m**
Side slope = 1.5 H : 1 V	**General ground level = 210.5 m**

Determine bed and water level at four critical locations of canal waterway at transitions of syphonic aqueduct. [2071 Magh: T.U.]

Solution:

Here,

Lacey's regime perimeter (P) $= 4.75\sqrt{Q}$

$= 4.75\sqrt{400} = 95\ m$

Let's provide 11 clear span of 8 m each and width of each pier is 1.5 m.

Total length of waterway $= 8 \times 11 + 1.5 \times 10$

= 103 m

Normal bed width = 28 m

Let, width be reduced to 14 m. Then,

Velocity through trough $= \frac{Q}{A} = \frac{36}{14 \times 1.5}$

$= 1.71\ m/sec. < 3\ m/sec.$ (OK)

Design of bed and water level at four critical location

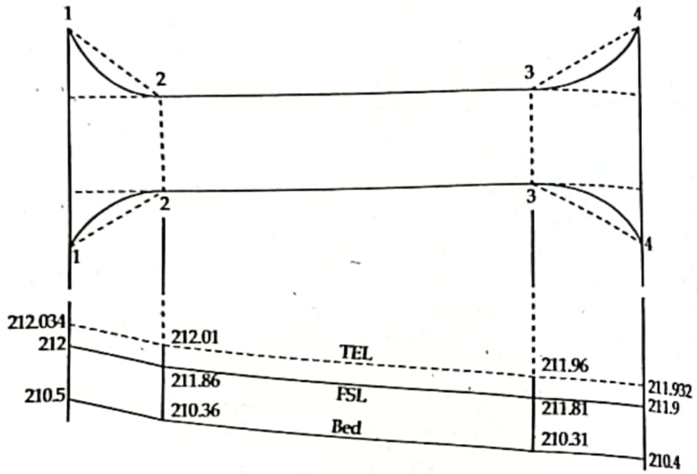


Figure 8.23

At section 4–4

$$V_4 = \frac{\text{Discharge}}{\text{Area}} = \frac{Q}{A} = \frac{36}{(B+1.5y)y}$$

$$= \frac{36}{(28+1.5\times 1.5)\times 1.5} = 0.793\ \text{m/sec.}$$

$$\text{Velocity head} = \frac{V_4^2}{2g} = \frac{(0.793)^2}{19.62} = 0.032\ \text{m}$$

R.L. of canal bed at 4–4 = 210.4 m

R.L. of water level at 3–3 = 210.4 + 1.5 = 211.9 m

R.L. of TEL at 3–3 = 211.9 + 0.032 = 211.932 m

At section 3–3

Assuming constant depth of 1.5 m; we have,

$$\text{Velocity at 3–3} = \frac{Q}{A} = \frac{36}{36\times 1.5} = 1.71\ \text{m/sec.}$$

$$\text{Velocity head at 3–3} = \frac{V_3^2}{2g} = \frac{(1.71)^2}{19.62} = 0.15\ \text{m}$$

$$\text{Loss of head in expansion from 3–3 to 4–4} = 0.3\left(\frac{V_3^2 - V_4^2}{2g}\right)$$

$$= 0.3(0.05 - 0.032)$$

$$= 0.035\ \text{m}$$

R.L. of TEL at 3–3 = R.L. of TEL at 4–4 + Head loss

= 211.932 + 0.035

= 211.96 m

R.L. of water surface = 211.96 – Velocity head

= 211.96 – 0.15

= 211.81 m

R.L. of bed level = 211.81 – 1.5 = 210.31 m

At section 2–2

Here, friction loss between 2–2 to 3–3 is given by;

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

where, n = 0.016; for concrete.

$$R = \frac{A}{P} = \frac{14\times 1.5}{(14+2\times 1.5)} = 1.24\ \text{m}$$

$$\therefore\ H_L = \frac{n^2V^2L}{R^{\frac{4}{3}}} = \frac{(0.016)^2\times(1.71)^2\times 103}{(1.24)^{\frac{4}{3}}} = 0.058\ \text{m}$$

Now,

R.L. of TEL at 2–2 = R.L. of TEL at 3–3 + Head loss in trough

= 211.96 + 0.058

= 212.01 m

R.L. of water level at 2–2 = 212.01 – Velocity head at 2–2

= 212.01 – 0.15

= 211.86 m

R.L. of bed at 2–2 = 211.86 – Water depth

= 211.86 – 1.5

= 210.36 m

At section 1–1

Here,

$$\text{Head loss in contraction from 1–1 to 2–2} = 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$$

$$= 0.2(0.15 - 0.032)$$

$$= 0.024\ \text{m}$$

R.L. of TEL at 1–1 = R.L. of TEL at 2–2 + Head loss

= 212.01 + 0.024

= 212.034 m

R.L. of water surface at 1–1 = 212.034 – Velocity head

= 212.034 – 0.032

= 212.002

R.L. of bed at 1–1 = 212.002 – Water depth

= 212.002 – 1.5

= 210.5 m

PROBLEM 9

What do you mean by cross drainage structures? What are the factors to be considered when selecting cross drainage works at a point?

Solution: See the definition part 8.1

PROBLEM 10

Design a suitable cross drainage structures from the following data:

Canal	**Drainage**
Discharge (Q) = 32 m³/sec.	**Discharge (Q) = 200 m³/sec.**
Full supply level (FSL) = 213.5	**High flood level (HFL) = 210 m**
Canal bed level = 212 m	**High flood depth = 2.5 m**
Canal bed width = 20 m	
Canal side slope (H : V) = 1.5 : 1	**[2013 Po.U.]**

Solution:

Here, the bed level of canal (212 m) is in higher level than the high flood level of drainage (210 m) so aqueduct should be constructed.

and, $Q_{drain}(200\ m^3/sec.) \gg Q_{canal}(32\ m^3/sec.)$

Hence, length of the canal waterway will be large so type III will be adopted.

i) Design of drainage waterway

Lacey's regime perimeter (P) $= 4.75\sqrt{Q} = 4.75\sqrt{200} = 67.18$ m

Provide 8 clear span of 8 m each with 1.5 m width pier.

Total length of waterway $= 8 \times 8 + 1.5 \times 7 = 74.5$ m

ii) Design of canal waterway

Bed width of canal = 20 m

Let, width be reduced to 10 m; then,

$$\text{Velocity through trough} = \frac{32}{10 \times 1.5}$$
$$= 2.133\ m/sec. < 3\ m/sec.\ (O.K.)$$

Providing a splay of 2 : 1 in contraction; we have,

$$\text{Length of contraction transition} = \frac{20 - 10}{2} \times 2 = 10\ m$$

Providing 3 : 1 splay in expansion transition; we have,

$$\text{Length of expansion transition} = \frac{20 - 10}{2} \times 3 = 15\ m$$

In transitions, the side slope of the canal section will be warped in plan from the original slope of 1.5 : 1 to vertical.

iii) Design of bed levels at different sections

At section 4–4

Depth of water $= 113.5 - 112 = 1.5$ m

$$\text{Area of trapezoidal section} = (B + 1.5y)y$$
$$= (20 + 1.5 \times 1.5) \times 1.5 = 33.375\ m^2$$

$$V_4 = \frac{\text{Discharge}}{\text{Area}} = \frac{32}{33.375} = 0.96\ m/sec.$$

$$\text{Velocity head} = \frac{V_4^2}{2g} = \frac{(0.96)^2}{19.62} = 0.047\ m$$

R.L. of canal bed at 4–4 = 212 m

R.L. of water surface 4–4 = 212 + 1.5 = 213.5 m

R.L. of TEL at 4–4 = 213.5 + 0.047 = 213.547 m

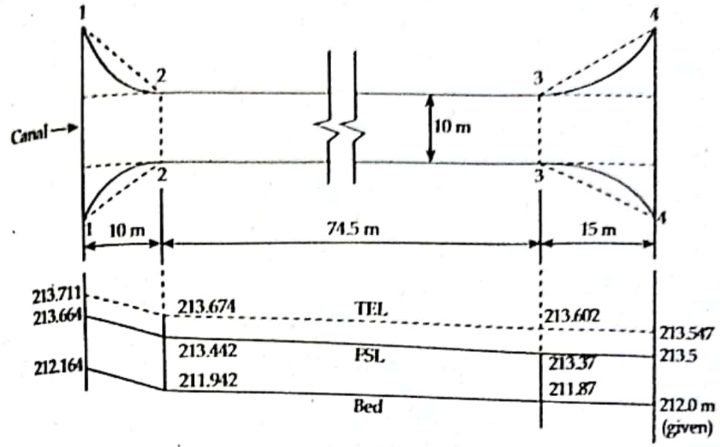


Figure 8.24

At section 3–3

Assuming the constant depth of 1.5 m throughout the channel, we have at 3-3, a rectangular channel,

Bed width = 10 m

$$V_3 = \frac{Q}{A} = \frac{32}{10 \times 1.5} = 2.133\ m/sec.$$

$$\text{Velocity head} = \frac{V_3^2}{2g} = \frac{(2.133)^2}{19.62} = 0.232\ m$$

$$\text{Loss of head in expansion} = 0.3\left(\frac{V_3^2 - V_4^2}{2g}\right) = 0.3(0.232 - 0.047)$$
$$= 0.055\ m$$

R.L. of TEL at 3–3 = R.L. of TEL at 4–4 + Loss in expansion
= 231.547 + 0.055
= 213.602 m

R.L. of water surface at 3–3 = 213.602 – Velocity head
= 213.602 – 0.232
= 213.37 m

R.L. of bed at 3–3 = 213.37 – Water depth
= 213.37 – 1.5 = 211.87 m

At section 2–2

Here, friction loss between 2-2 to 3-3 is given by;

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

where, $n = 0.016$; for concrete.

$$R = \frac{A}{P} = \frac{10 \times 1.5}{(10 + 1.5 \times 2)} = 1.154\ m$$

$$\therefore \quad H_L = \frac{n^2V^2L}{R^{\frac{4}{3}}} = \frac{(0.016)^2 \times (2.133)^2 \times 74.5}{(1.154)^{\frac{4}{3}}} = 0.072 \text{ m}$$

R.L. of TEL at 2-2 = R.L. of TEL at 3-3 + Head loss in trough
= 213.602 + 0.072 = 213.674 m

R.L. of water surface at 2-2 = 213.674 − Velocity head at 2-2
= 213.674 − 0.232
= 213.442 m

R.L. of bed level at 2-2 = 213.442 − 1.5 = 211.942 m

At section 1-1

Here,

Loss of head in contraction transition from 1-1 to 2-2 $= 0.2(V_2^2 - V_1^2)$
$= 0.2 \times (0.232 - 0.047)$
$= 0.037$ m

R.L. of TEL at 1-1 = R.L. of TEL at 2-2 + Loss in contraction
= 213.674 + 0.037
= 213.711 m

∴ R.L. of water surface at 1-1 = 213.711 − Velocity head
= 213.711 − 0.047 = 213.664 m

R.L. of bed level at 1-1 = 213.664 − 1.5 = 212.164 m

iv) Design of transition

a) Contraction transition

Applying Mitra's hyperbolic transition equation; we have,

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{B_n \cdot L_f - x(B_n - B_f)}$$

where, $B_n = 20$ m
$B_f = 10$ m
$L_f = 10$ m

$$\therefore \quad B_x = \frac{20 \times 10 \times 10}{20 \times 10 - x(20-10)} = \frac{2000}{200 - 10x} = \frac{200}{20 - x}$$

x (m)	0	2	4	6	8	10
$B_x = \frac{200}{20-x}$	10	11.11	12.5	14.29	16.67	20

where, x is measured from the flumed section.

b) Expansion transition

Here,

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{B_n \cdot L_f - x(B_n - B_f)}$$

where, $L_f = 15$ m.

$$\therefore \quad B_x = \frac{20 \times 10 \times 15}{20 \times 15 - x(20-10)} = \frac{3000}{300 - 10x} = \frac{300}{30 - x}$$

x (m)	0	2	4	6	8	10	12	14	15
$B_x = \frac{300}{30-x}$	10	11.71	11.54	12.5	13.64	15	16.67	18.75	20

v) Design of trough

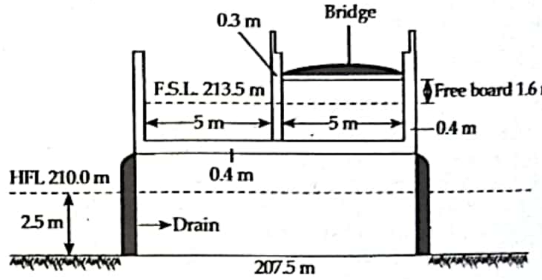


Figure 8.25

Here,

Depth of water in canal = 1.5 m

Provide free board of 0.6 m; then,

Weight of trough = 1.5 + 0.6 = 2.1 m

Divide trough in two equal compartments of 5 m each with following:

Intermediate wall = 0.3 m

Outer walls = 0.4 m

Bottom slab of trough = 0.4 m

PROBLEM 11

Design a suitable cross drainage structure having following data:

Canal	Drainage
Full supply discharge = 32 Cumecs	**High flood discharge = 300 Cumecs**
Fully supply level = 213.5 m	**High flood level = 210.0 m**
Canal bed level = 212.0 m	**Drainage bed level = 207.5 m**
Canal bed width = 20 m	
Side slope of canal = 1.5 H : V	**[2014 P.U.]**

Solution:

Here, the bed level of canal (212 m) is in higher level than the high flood level of drainage (210 m) and $Q_{drain} \gg Q_{canal}$.

Hence, type III aqueduct is provided.

i) Design of drainage waterway

Here,

Lacey's regime perimeter (P) $= 4.75\sqrt{Q} = 4.75\sqrt{300}$
$= 82.3$ m

Provide 9 clear span of 8 m each with pier of 1.5 m width.

Length of waterway = 9 × 8 + 1.5 × 8 = 84 m

ii) Design of canal waterway

See the solution of Q. no. 10 (Design of waterway)

iii) Design of bed levels at different section

At section 4–4 and section 3–3

See the solution of Q. no. 10

At section 2–2

Friction loss between 2-2 and 3-3 is given by;

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

n = 0.016; for concrete

and, $R = \frac{A}{P} = \frac{15 \times 1.5}{(10 + 1.5 \times 2)} = 1.154$ m

$$\therefore \quad H_L = \frac{(0.016)^2 \times (2.133)^2 \times 84}{(1.154)^{\frac{4}{3}}} = 0.08 \text{ m}$$

R.L of TEL at 2-2 = R.L. of TEL at 3-3 + H_L
= 213.602 + 0.08 = 213.682 m

R.L. of water surface 2-2 = 213.682 – Velocity head
= 213.682 – 0.232 = 213.45 m

R.L. of bed level at 2-2 = 213.45 – Water depth
= 213.45 – 1.5 = 211.95 m

At section 1–1

Loss of head from 1-1 to 2-2 = $0.2(V_2^2 - V_1^2) = 0.2(0.232 - 0.047)$
= 0.037 m

R.L. of TEL at 1-1 = R.L. of TEL at 2-2 + Head loss
= 213.682 + 0.037 = 213.719 m

R.L. of water surface at 1-1 = 213.719 – Velocity head
= 213.719 – 0.047 = 213.672 m

R.L. of bed level at 1-1 = 213.672 – 1.5 = 212.172 m

iv) Design of transition

See the solution of Q. no. 10

v) Design of trough

See the solution of Q. no. 10

PROBLEM 12

Following are the data obtained at the crossing of a canal and drainage

Canal data	**Drainage data**
Discharge = 50 cumec	**Discharge = 400 cumec**
Full supply depth = 1.6 m	**HFL = 211.0 m**
Bed width = 35 m	**Bed level = 208.5 m**
Bed level = 210.3 m	**General ground level = 210.5 m**
Side slope = 1.5 H : 1 V	

Design the drainage waterway, canal waterway and find the bed levels and FSL at four different sections of the canal trough. **[2072 Ashwin]**

Solution:

i) Design of drainage waterway

Lacey's regime perimeter (P) = $4.75\sqrt{Q} = 4.75\sqrt{400} = 95$ m

Let's provide 11 clear span of 8 m each and width of each pier is 1.5 m.

Total length of waterway = 8 × 11 + 1.5 × 10 = 103 m

Let limiting velocity of water in drainage = 2.0 m/s

$$\text{Height of barrel required} = \frac{Q}{\text{Velocity} \times 8 \times 11} = \frac{400}{2 \times 8 \times 11} = 2.27 \text{ m}$$

Provide 11 rectangular barrels each 8 m wide and 2.5 m height

$$\text{Velocity through barrels} = \frac{400}{2.5 \times 8 \times 11} = 1.82 \text{ m/s (OK)}$$

ii) Design of canal waterway

Normal bed width = 35 m

Let the width is reduced to 15 m.

Providing a splay of 2 : 1 contraction; we have,

$$\text{Contraction transition} = \frac{35 - 15}{2} \times 2 = 20 \text{ m}$$

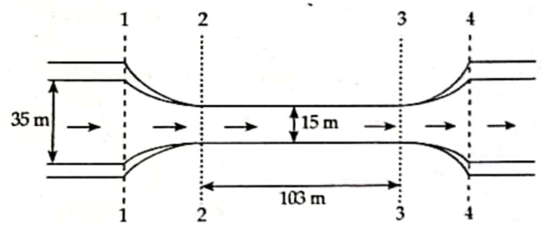
Providing a splay of 3 : 1 expansion transition,

$$\text{Length of expansion transition,} = \frac{35 - 15}{2} \times 3 = 30 \text{ m}$$

Length of flumed rectangular portion is = 103 m

In transitions side slope of canal section is warped in plan from original slope of 1.5 H : 1 V to vertical.

iii) Bed levels and FSL at different sections of canal



At section 4–4

$$V_4 = \frac{Q}{A} = \frac{Q}{(B + 1.5y)\,y} = \frac{50}{(35 + 1.5 \times 1.6)\,1.6} \qquad [y = 1.6 \text{ given}]$$

= 0.83 m/sec.

$$\text{Velocity head} = \frac{V_4^2}{2g} = \frac{(0.83)^2}{19.6} = 0.035 \text{ m}$$

R.L. of canal bed = 210.3 m

R.L. of water level at 4-4 = 210.3 + 1.6 = 211.9 m

R.L. of TEL at 4-4 = 211.9 + 0.035 = 211.935 m

At section 3–3

Assuming depth of water is constant = 1.6 m

At section 3-3 section is rectangular

Velocity at 3-3 $(V_3) = \frac{Q}{A} = \frac{50}{15 \times 1.6} = 2.08$ m/sec.

Velocity head $= \frac{V_3^2}{2g} = \frac{(2.08)^2}{19.6} = 0.221$ m

Loss of head in expansion form 3-3 to 4-4 $= 0.3\left(\frac{V_3^2 - V_4^2}{2g}\right)$

$= 0.3(0.221 - 0.035)$

$= 0.0558$ m

R.L. of TEL at 3-3 = R.L. of TEL at 4-4 + Loss in expansion

= 211.935 + 0.0558

= 211.99 m

R.L. of water surface at 3-3 = 211.99 – Velocity head

= 211.99 – 0.221

= 211.77 m

R.L. of bed at 3-3 = 211.77 – 1.6 = 210.17 m

At section 2–2

Friction loss between 2-2 and 3-3

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

where, n = 0.016 for concrete

$$R = \frac{A}{P} = \frac{15 \times 1.6}{(15 + 2 \times 1.6)} = 1.32 \text{ m}$$

$$H_L = \frac{(0.016)^2 \times (2.08)^2 \times 103}{(1.32)^{\frac{4}{3}}} = 0.078 \text{ m}$$

R.L. of TEL at 2-2 = R.L. of TEL at 3-3 + H_L

= 211.99 + 0.078 m

= 212.068 m

R.L. of water surface at 2-2 = 212.068 – velocity head

= 212.068 – 0.221

= 211.847 m

R.L. of bed at 2-2 = 211.847 – 1.6 = 210.247 m

At section 1–1

Head loss in contraction from 1-1 to 2-2 $= 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$

$= 0.2(0.221 - 0.035)$

$= 0.037$ m

R.L. of TEL at 1-1 = R.L. of TEL at 2-2 + Head loss

= 212.068 + 0.037 = 212.105 m

R.L. of Water surface at 1-1 = 212.105 – Velocity head

= 212.105 – 0.035 = 212.07 m

R.L. of bed at 1-1 = 212.07 – 1.6 = 210.47 m

PROBLEM 13

Design a suitable cross drainage (waterway, bed levels at different sections and design of transitions) works if the following data at crossing of canal and drainage are given.

Canal	**Drainage**
Q = 40 m³/s	**Q = 45 m³/s**
Bed width = 30 m	**HFL = 207 m**
FSD of canal = 1.6 m	**Bed level = 204.5 m**
Bed level = 206.4 m	**General ground level = 206.5 m**
Side slope = $1\frac{1}{2}$ H : 1 V	**[2072 Magh]**

Solution:

Here, form the question, bed level of canal is below the HFL of drain and FSL of canal (206.4 + 1.6 = 208 m) is higher than HFL of drain 207 m so siphon aqueduct is provided as cross drainage works.

i) Design of canal waterway

Lacey's regime perimeter (P) $= 4.75\sqrt{Q}$

$= 4.75\sqrt{45} = 31.86$ m

Let provide 4 clear spans of 8 m each and width of each pier is 1.5 m.

Total length of waterway $= 8 \times 4 + 3 \times 1.5$

$= 36.5$ m

Let, Limiting velocity through siphon barrel = 2 m/sec.

Height of barrel required $= \frac{45}{8 \times 4 \times 2} = 0.703$ m

Provide 4 rectangular barrels, each 8 m wide and height 0.75 m.

Actual velocity through barrels $= \frac{45}{8 \times 4 \times 0.75}$

$= 1.875$ m/sec.

ii) Design of canal waterway

Normal bed width = 30 m

Let the width be reduced to 15 m

Velocity through trough $= \frac{40}{15 \times 1.6} = 1.67$ m/sec. < 3 m/sec. (OK)

Providing splay of 2 : 1 on the contraction,

Length of contraction transition $= \frac{30 - 15}{2} \times 2 = 15$ m

Providing splay of 3 : 1 in expansion,

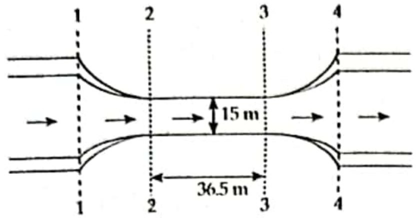
Length of expansion transition $= \frac{30 - 15}{2} \times 3 = 22.5$ m

Length of flumed rectangular portion = 36.5 m

iii) Design of bed levels at different sections

At section 4–4

$$V_4 = \frac{Q}{A} = \frac{Q}{(B + 1.5y)\,y} = \frac{40}{(30 + 1.5 \times 1.6)\,1.6} = 0.772 \text{ m/sec.}$$



Velocity head $= \frac{V_4^2}{2g} = \frac{(0.772)^2}{19.6} = 0.03$ m

R.L. of canal bed = 206.4 m

R.L. of water level = 206.4 + 1.6 = 208 m

R.L. of TEL at 4–4 = 208 + 0.03 = 208.03 m

At section 3–3

Assuming depth of water is constant = 1.8 m

At section 3-3 section is rectangular

Velocity at 3-3, $(V_3) = \frac{Q}{A} = \frac{40}{15 \times 1.6}$

$= 1.667$ m/sec.

Velocity head $= \frac{V_3^2}{2g} = \frac{(1.667)^2}{19.6} = 0.142$ m

Loss of head in expansion form 3-3 to 4-4 $= 0.3\left(\frac{V_3^2 - V_4^2}{2g}\right)$

$= 0.3(0.142 - 0.03)$

$= 0.336$ m

R.L. of TEL at 3-3 = R.L. of TEL at 4-4 + Loss in expansion

= 208.03 + 0.0336

= 208.064 m

R.L. of water surface at 3-3 = 208.064 – velocity head

= 208.064 – 0.142

= 207.922 m

R.L. of bed at 3-3 = 207.922 – 1.6

= 206.322 m

At section 2–2

Friction loss between 2-2 and 3-3

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

n = 0.016 for concrete

$$R = \frac{A}{P} = \frac{15 \times 1.6}{(15 + 2 \times 1.6)} = 1.32 \text{ m}$$

$$H_L = \frac{(0.016)^2 \times (1.67)^2 \times 36.5}{(1.32)^{\frac{4}{3}}} = 0.018 \text{ m}$$

R.L. of TEL at 2-2 = R.L. of TEL at 3-3 + H_L

= 208.064 + 0.018 m

= 208.082 m

R.L. of water surface at 2-2 = 208.082 – Velocity head

= 208.082 – 0.142

= 207.94 m

R.L. of bed at 2-2 = 207.94 – 1.6 = 206.34 m

At section 1–1

Head loss in contraction from 1-1 to 2-2 $= 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$

$= 0.2(0.142 - 0.03)$

$= 0.0224$ m

R.L. of TEL at 1-1 = R.L. of TEL at 2-2 + Head loss

= 208.082 + 0.0224

= 208.105 m

R.L. of Water surface at 1-1 = 208.105 – Velocity head

= 208.105 – 0.03

= 208.075 m

R.L. of bed at 1-1 = 208.075 – 1.6 = 206.475 m

iv) Design of transition

a) Contraction transition

Since depth of water is constant, the transition shall be designed on the basis of Mitra's hyperbolic transition equation given by,

$$B_x = \frac{B_n \times B_f \times L_f}{B_n \times L_f - x(B_n - B_f)}$$

where, $B_f = 15$ m

$B_n = 30$ m

$L_f = 15$ m

Substituting the above values, 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 the various of x lying between 0 to 15 m, values of B_x are worked out as shown in table.

The distance 'x' measured from the flumed section 2-2,

x (m)	0	2	4	6	8	10	12	14	15
$B_x = \frac{450}{30 - x}$	15	16.04	17.27	18.72	20.42	22.5	25	28.10	30

b) Expansion transition

In this case, we have,

$B_f = 15$ m

$B_n = 30$ m

$L_f = 22.5$ m

$$B_x = \frac{30 \times 15 \times 22.5}{30 \times 15 - x(30 - 15)} = \frac{675}{45 - x}$$

x (m)	0	2	4	6	8	10	12	14	16	18	20	22.5
$B_x = \frac{675}{45 - x}$	15	15.7	16.46	17.5	18.25	19.3	20.4	21.75	23.3	25	27	30

PROBLEM 14

Following data are obtained at the crossing of a canal and drainage

Canal data	**Discharge data**
Discharge: 25 cumec	**Discharge: 360 cumec**
Full supply depth: 2.0 m	**HFL: 211.0 m**
Bed width: 30 m	**Bed level: 208.5 m**
Bed level: 210.3 m	**General ground level: 210.5 m**
Side slope: 1.5H : 1V	

Design the drainage waterway, canal waterway and find the bed levels and FSL at four different sections of the canal through. **[2073 Bhadra]**

Solution: Proceed same as the solution of Q. no. 2

PROBLEM 15

Design the following components of a suitable C/D work for the following data:

Discharge of canal = 50 m^3/sec. **Bed width of canal = 30 m**

Depth of water in canal = 1.5 m **Bed level of canal = 100.0 m**

High flood discharge of drain = 450 m^3/sec.

High flood level of drainage = 100.50 m

Bed level of drainage = 98.8 m **General ground level = 100 m**

i) **Design of drainage water-ways**

ii) **Design of canal water-way**

iii) **Design of transition and**

iv) **Uplift pressure on the roof** **[2073 Magh]**

Solution: Proceed same as example 8.1 up to the step 7

PROBLEM 16

Design a siphon adequate with the data given below:

FSL of canal = 30m^3/sec.

Bed width of canal = 24 m

Full supply depth = 1.25 m

Side slope of canal section = 1.5 : 1 (H : V)

Bed level of canal = 100.00 m

Maximum flood discharge of drain = 500m^3/sec.

High flood level = 100.50 m

Bed level of drainage = 98.00 m

Normal ground level = 100.00 m

Lacey's silt factor = 1.0

Rugosity coefficient (w) = 0.016

Make suitable data where necessary. **[2074 Bhadra]**

Solution: See the solution of example 8.1

PROBLEM 17

Write short notes on different types of cross drainage works. **[2075 Baishakh]**

Solution: See the definition part 8.1

PROBLEM 18

Define cross drainage structures. Enlist the different types of cross drainage structures. Which types of cross drainage structure is favourable in hilly area of Nepal? **[2076 Bhadra]**

Solution:

For the first and second part

See the definition part 8.1

In case of third question, aqueduct or super passage is favourable cross drainage structures in hilly areas of Nepal for irrigation canal. However, the suitability of cross drainage structure may depend on particular site condition and design requirement.

PROBLEM 19

From the following data, select and sketch the suitable type of cross drainage structures and determine the drainage water way and canal water way. **[2076 Bhadra]**

Canal data	**Drainage data**
Full supply discharge = 32 cumecs	**High flood discharge = 303 cumecs**
Full supply level = 213.5 m	**High flood level = 210.0 m**
Canal bed level = 212.0 m	**High flood depth = 2.5 m**
Canal bed width = 20 m	**General ground level = 212.5 m**

Solution: Proceed same as the solution of problem no. 10 or 11

PROBLEM 20

Design siphon aqueduct (Drainage water way, canal water way, Bed levels and transitions) if the following data at crossing of canal and drainage are given:

a) **Discharge of canal = 60 cumecs**

b) **Bed width of canal = 35 m**

c) **Full supply depth of canal = 2 m**

d) **Canal bed level = 300 m**

e) **Side slopes of canal = 1.5 H : 1 V**

f) **High flood discharge of drainage = 500 cumecs**

g) **High flood level of drainage = 300.8 m**

h) **Bed level of drainage = 298.2 m**

i) **General ground level = 300.2 m** **[2077 Chaitra]**

Solution: Proceed same as the solution of problem 4

PROBLEM 21

Design drainage waterway, canal waterway and canal transitions of a cross drainage works for a canal having discharge 56 m³/sec., bed width 24 m, full supply depth 1.98 m and bed level 267.00 m. The drainage has a high flood discharge of 425 m³/sec., high flood level of 268.20 m, general bed level of low water cross-section of 265.50 m, and general ground level of 267.2 m. **[2078 Baishakh]**

Solution: Proceed same as the solution of problem no. 4

OBJECTIVE QUESTION

The type of canal alignment, which involves maximum cross drainage works, is a ....

a) Ridge canal b) Contour canal
c) Canal regulator d) Canal crossing

When an irrigation canal is taken over a drainage channel, the crossing is called ....

a) an aqueduct b) a super passage
c) a level crossing d) none of the above

An irrigation canal, freely flowing under a drainage channel is specifically called a ....

a) Canal junction b) Canal crossing
c) Canal siphon d) Super passage

An irrigation canal flowing freely above the drainage, which in turn is following under pressure, is specifically, called a ....

a) Canal siphon b) Canal aqueduct
c) Siphon aqueduct d) Super passage

The crossing arrangement, preferably made at the junction of a huge canal and a river stream carrying short lived–high flood discharge at almost equal bed levels is a ....

a) Super passage b) Aqueduct
c) Level crossing d) Canal siphon

The drainage water is sometimes allowed to join the canal water to augment canal supplies through a hydraulic structure is called a ....

a) Canal outlet b) Canal inlet
c) Module d) Level crossing

A cross–drainage work is called siphon, when it carries the canal water ....

a) below the drainage under pressure
b) below the drainage at atmospheric pressure
c) above the drainage at atmospheric pressure

In a siphon aqueduct, the worst condition of uplift on the roof occurs, when ....

a) the canal and drainage are flowing full
b) the canal is flowing fuel and there is no drainage discharge
c) the canal is empty and drainage flowing full
d) none of the above

In a siphon aqueduct, the worst condition of uplift on the floor occurs, when ....

a) the canal is full and the drainage empty, with water table at drainage bed
b) the canal is empty and the drainage full, with water table at drainage bed
c) the canal and drainage both full
d) the canal is full and drainage empty, with water table below the floor

The dead loss through the siphon barrel is usually given by Unwin's formula, by neglecting velocity as ....

$N_1 = \left(1 + f_1 + f_2 \times \frac{L}{R}\right)\frac{V^2}{2g}$, where, f_1 and f_2 respectively represent the coefficient of head losses due to ....
a) barrel friction and entry
b) entry and barrel function
c) barrel friction and exit
d) exit and barrel friction

Answer sheet

1	2	3	4	5	6	7	8	9	10
b	a	d	c	c	b	a	c	a	b

CHAPTER 9

WATER LOGGING AND DRAINAGE

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WATER LOGGING

A cultivated land is called water logged if its productivity decreases due to rise in water table. In fact productivity decreases when root zoon of plant get flooded with water and thus become ill-aerated.

Plant requires different nutrients like nitrates, phosphate for their growth and development. Nitrates are provided by bacteria under a process called nitrification. These bacteria need oxygen for their survival. The supply of oxygen gets cut-off when land becomes ill-aerated, resulting death of bacteria and fall in the production of nitrates. As a result, growth of plant reduces which reduces the crop yield.

SALINITY

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

9.1 CAUSES, EFFECTS AND PREVENTIVE MEASURE OF WATER LOGGING

9.1.1 Causes of water logging

Water logging is rise of water table which may occur due to following factors.

i) Over and Intensive Irrigation

Maximum irrigation over small area is called intensive irrigation. When policy of intensive irrigation is adopted, then maximum irrigable area of a small region is irrigated. This leads to much irrigation in that region. As a result, heavy percolation and subsequent rise of water table occurs.

ii) Seepage of water through adjoining high lands

Water from the adjoining high lands may seep into the sub-soil of the affected land and may rises water table.

iii) Seepage of water through the canals

Water may seep through the beds and sides of adjoining canal, reservoir, etc. situated at higher level to lower level land as a result water table of affected area increases. When the soil at the site of canals, reservoirs, etc. is very pervious, the seepage is excessive.

iv) Excessive rains

Excessive rainfall may create temporary water logging and in the absence of good drainage, it may lead to continue water logging.

v) Impervious obstruction

Water seeping below the soil moves horizontally *i.e.*, laterally but may find an impervious obstruction, causing the rise of water table on the upstream side of obstruction. Similarly, an impervious stratum may occur below the top layers of pervious soil. In such cases, water seeping through the pervious will not able to go deep, and hence, quickly resulting in high water table.

vi) Inadequate natural drainage

Soils having less permeable below the top layers of pervious soils, will not be able to drain the water deep into the ground and hence, resulting in high water level in affected soil.

vii) Submergence of floods

If a land continuously remains submerged by floods, water loving plants like grasses, weeds, etc. may grow, which obstruct the natural surface drainage of the soil thus increasing the chances of water logging.

viii) Irregular or flat topography

In steep terrain, the water is drained quickly. On flat or irregular terrain having depressions etc. the drainage is very poor. All these factors lead to greater detention of water on the land causing more percolation and raised water table.

9.1.2 Effect of water logging

The water logged soils are rendered infertile because of ill effects of water logging to the soil, to the crop and to the environment.

I) Effects of soil

a) Lack of aeration

Due to the presence of excess water, soil pores within the root zone of plants are saturated to cut-off the normal circulation of air. The capacity of soil bacteria is affected resulting in retardation in the formation of organic acids to dissolve plant food materials.

b) Reduced in soil temperature

Water logged soil is slow to warm up. Low soil temperature restricts root development, depresses biotic activity in the soil resulting in lowered rate of production of available nitrogen hampering seed germination and seed growth.

c) Creation of salinization

Capillary water coming to the surface brings up in the solution harmful salts from the soil or these present in the ground water. Thus, high salinization and deposits of sodium salts in the soil at or near the ground surface are created which may be toxic or lead to the formation of alkaline condition.

d) Inhibiting activity of soil bacteria

When soil structure is affected and tillage and cultivation of wet soil is resorted to, which tends to reduce normal biotic activity and root development.

e) Denitrification

Denitrification occurs because of the competition for nitrogen by the soil micro-organisms that thrive in saturated soil and reduction in numbers of nitrifying organisms due to lack of aeration.

f) Retards cultivation

Difficulty in carrying out normal cultivation in water logged soil.

II) Effects of crops

a) Delayed cultivation operation

Normal cultivation operations of tilling and ploughing are adversely effected due to presence of excess water in the soil.

b) Growth of weeds

Water loving wild plants grow profusely and have competition with the crops, thereby affecting the growth of useful crops. Weed removal entails extra investment on the part of cultivator. In extreme water logged condition, only wild growth is there.

c) Diseased crops

Water logged condition causes physiological diseases to crops. Decay of roots, external symptoms on the foliage, fruits, etc. are common.

d) Loss of cash crop

Cash crops desired to be grown in the land cannot be cultivated. High sub soil restricts the use of land to crop like paddy.

e) Low yield

Maturity period of crop is reduced resulting in low yield.

f) Oxygen depletion

In saturated soil, plant roots are denied normal circulation of air, the level of oxygen declines and that of carbon dioxide increases resulting wilting and ultimately death of plants.

iii) Effects on environment

Water logging impairs sanitary conditions, promotes malaria and there by result in unhealthy environment for human population, animals and plants in the area.

9.1.3 Preventive measure of water logging

Water logging can be controlled only if the quantity of water into the soil below checked and reduced. To achieve this, the inflow of water into the underground reservoir should be reduced and the outflow from this reservoir should be increased as to keep the highest position of water table at least about 3 m below ground surface. The various measures adopted for controlling water logging are enumerated below:

i) Preventive measures

Preventive measures envisage reduction of inflow into the sub-soil water. The various measures in this context are as follows:

a) Reducing intensity of irrigation

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

b) Adopting crop rotation

Sowing of wet crops (requiring more water) in every season should be discouraged. A wet crop should be followed by a dry crop *i.e.*, crop rotation should be adopted.

c) Optimum use of water

Farmers should be educated and trained in using just the required quantity of water, which will give optimum yield.

d) Providing intercepting

To collect water seeping through the sides of canal intercepting drains shall be provided in the banks of canal.

e) Improving natural drainage of the area

The storm water and excess irrigation water should be effectively drained off to reduce their percolation into ground water reservoir. This is achieved by improving the natural drainage of the area by measures like clearing the obstruction in path of flow, desilting, etc.

f) Provision of artificial drainage system

An efficient drainage system in the form of surface drains shall be provided to drain off storm water and surplus irrigation water.

g) Lining of canals and water courses

To control the seepage through bed and sides of canal passing through pervious soils, canals should be lined with impervious materials.

h) Introduction of lift irrigation

Lift irrigation should be introduced in the place of canal irrigation, in places where there is a possibility of water logging. Lift irrigation utilizes underground water and hence maintain the water of a lower level.

9.1.4 Remedial measures of water logging

i) Controlling the loss of water by seepage from the canal.
ii) Providing adequate surface drainage.
iii) Providing efficient under drainage.
iv) Charging irrigation tax on volumetric basis.

Conjunctive use

The combined use of sub surface water (ground water) and surface water (canal water) in a judicious manner, as to deceive maximum benefits, is called conjunctive use of water.

9.2 WATER LOGGING AND DRAINAGE OF IRRIGATED LAND

9.2.1 Water logging

A cultivated land called water logged if its productivity decreases due to rise in water table. In fact productivity decreases when root zoon of plant get flooded with water.

9.2.2 Drainage

Process of removing and controlling excess water either on the surface soil or in the root zone beneath the soil called drainage.

Necessity of effective drainage system

- To remove excess water
- For drain out storm water effectively and to prevent great percolation

Types of drainage system

- Surface drainage system
  - Shallow drainage system
  - Deep drainage system
- Sub-surface drainage system

9.3 SURFACE DRAINAGE SYSTEM AND THEIR DESIGN

9.3.1 Surface drainage or Open drainage

The removal of excess rain water falling on the field or the excess irrigation water applied to the field, by constructing open ditches fields drain, and other related structure is called surface drainage. There are two types of surface drainage system.

I) Shallow surface drain

- To remove the excess irrigation water
- To remove storm water
- Trapezoidal in shape
- Manning and Kutter's formula used in design of shallow surface drainage

II) Deep surface drainage

They are relatively deeper than shallow surface drains and are quite effective for draining out subsoil water. These drains are commonly used as outlet drains for a close drainage system.

- Dug up to level of ground water table
- In draining out water logged area

Layout planning for drainage

As far as possible open drains should be located so as to follow the path of natural drainage of the area. All the open drains should discharge into an outfall drain which is either a large open drain or a natural stream. The location of outfall drain can be adjusted to give the required bed slope to the open drain.

9.3.2 Internal drainage of Bunded field

Design discharge of excess rainfall from bunded area (field) is called internal drainage.

9.3.3 External drainage

It is used for design of culverts and hydraulic structures.

9.3.4 Design of shallow surface drainage

There are different assumption to design the drainage system in Terai and Hilly region of Nepal.

Assumption for Terai region

- Rain fall 5 years to 10 years return period should be considered.
- Initial water level in field should be considered as 40 mm.
- Maximum water level is 300 mm which persist for one day.
- Depth in excess of 200 mm may persist for 3 days.
- No rain fall follows the design rainfall for several days (more than 3 days)
- Evaporation and transpiration is neglected.
- Irrigation inflows are neglected.

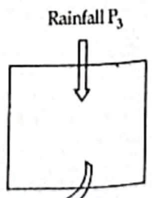

Figure 9.1

In the given figure;

P_3 = Yearly maximum rainfall for consecutive 3 days

Then, from water balance (W.B.) equation 'Q';

$$h = 40 + \frac{P_3 \times t}{3} - Q \times t$$

where, Q is the outflow in mm/day.

Example 9.1

Design the surface drainage for the field having rainfall intensity of P_3 = 450 mm, for the area of 30 ha.

Solution:

First trail Q = 50 mm/day (drainage discharge)

$$h = 40 + \frac{P_3 \times t}{3} - Q \times t$$

Day	Initial depth(mm)	$\frac{P_3}{3}$	Q	Final water depth
1	40	150	50	$140 = 40 + 150 \times 1 - 50 \times 1$
2	140	150	50	$240 = 140 + 150 \times 1 - 50 \times 1$
3	240	150	50	340

Here, water depth > 300 mm, second trail is required.
Here, 'h' is greater than 300 mm, but theoretically 'h' should not greater than 300 mm. So, next trail required. To meet our requirement 'Q' should be increased and h may tend to 300 mm.

Second trail, Q = 75 mm/day

Day	Initial depth	$\frac{P_3}{3}$	Q	Final water depth
1	40	150	75	$115 = 40 + 150 - 75$
2	115	150	75	$190 = 115 + 150 - 75$
3	190	150	75	$265 = 190 + 150 - 75$
4	265	–	75	$190 = 265 - 75$
5	115	–	75	$115 = 190 - 75$

Here, no water depth greater than 300 mm.
Depth excess of 200 mm occur in one day so, theoretically this design is ok but uneconomical so, take next trail Q = 65 mm/day.

Day	Initial depth	$\frac{P_3}{3}$	Q	Final water depth
1	40	150	65	$125 = 40 + 150 - 65$
2	125	150	65	$210 = 125 + 150 - 65$
3	210	150	65	$295 = 210 + 150 - 65$
4	295	–	65	230
5	230	–	65	165

Here, water level greater than 200 mm occur for 3 days.
No water depth greater than 300 mm.
Hence, design is ok, and also economical.

Hence, design discharge is 65 mm/day.

If area, A = 30 Ha

$$Q = A \times V$$

or, $$Q = \frac{30 \times 10^4 \times 65 \times 10^{-3}}{(24 \times 3600)} = 0.226 \text{ m}^3/\text{sec.}$$

Assumptions for hilly region

- Rain fall for 5 or 10 year return is considered.
- Initial water level in the field should be considered as 40 mm for design.
- No rain fall follows the design rain fall for several days.
- Evaporation and transpiration losses are neglected.
- Irrigation flows are neglected.

If P_3 = maximum yearly rainfall

Q = Drainage discharge in mm/day

Now, using W.B. equation; we have,

$$40 + P - Q = 100$$

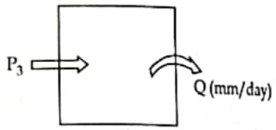

Figure 9.2

Design criteria of surface drainage channel

- Bed slope should be 1 : 300 to 1 : 500
- Velocity $v \le 0.85$ m/sec.
- B/D ratio 4 : 1 (irrigation)
- B/D ratio 3 : 1 (drainage)
- Capacity of drain should be designed for storm water crop sustainability
- The drain alignment should follow natural drain as far as possible

9.4 SUB–SURFACE DRAINAGE SYSTEM/TILE DRAINAGE SYSTEM AND THEIR DESIGN

9.4.1 Layout of subsurface drainage system

The tile drain may be aligned in different fashions depending upon the topography of the area. Generally, laterals runs through most of the drainage area and join the mains which in turn into some deep open drain. The depth of deepest tile drain shall be kept within the range of 3 m from the surface.

Various system of laying surface drainage is as follows (or lay out planning);

i) Natural system

- It is preferred in rolling terrain.
- Topography where drainage of isolated area required.
- This system is suitable when the land is not to be completely drained.
- This system is quite flexible and permits location of drains where they are most needed.
- In this system main and laterals are provided in natural course.

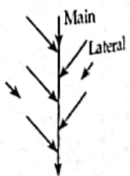

Figure 9.3

ii) Grid iron system

- It is useful in leveled land.

- It consists of lateral and sub main.
- The laterals are provided in only one side of main.
- This system is adopted when land is practically level.

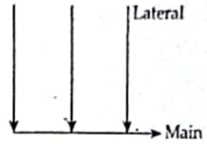

Figure 9.4

iii) Herring bone system

- Lateral join the sub main each side alternatively.
- This layout is adopted when the main (or sub main) is laid in depression.
- The land along the main is double drained, since it exists in depression.
- It requires more drainage than land on the adjacent slopes.

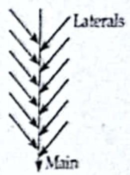

Figure 9.5

iv) Double main system

- Having two main with separate lateral for each.
- This layout is adopted when bottom of depression is wide.
- This arrangement reduces the length of the laterals and eliminates the break in slope of lateral at the depression.

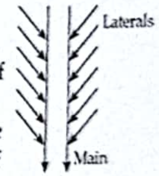

Figure 9.6

v) Intercepting drainage system

- There is no lateral drain.
- A sub-main is provided at the toe of slope.
- Adopted main source of drainage in hilly region.

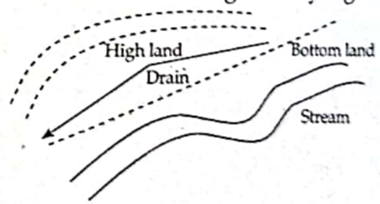

Figure 9.7

9.4.2 Flow of ground water to the tile drain and spacing of tile drain

Depth and spacing of drain pipes

The depth and spacing of the sub-surface drain pipes should be sufficient to lower the water table from the root zone of the plants. For most plant the top point of the water table must be at level 1.0 to 1.5 m below the ground level. The distance may vary from 0.7 to 2.5 m depending upon soil type and type of crop.

The tile drain may be placed at about 0.3 below the designed highest level of the water table. The flow of ground water is as shown in the figure.

For spacing

Let, 's' be spacing of drain pipe, 'a' be depth of impervious stratum. Let 'y' be the height of impervious stratum at 'x' distance from center of drain, 'b' be the maximum height of water table from stratum.

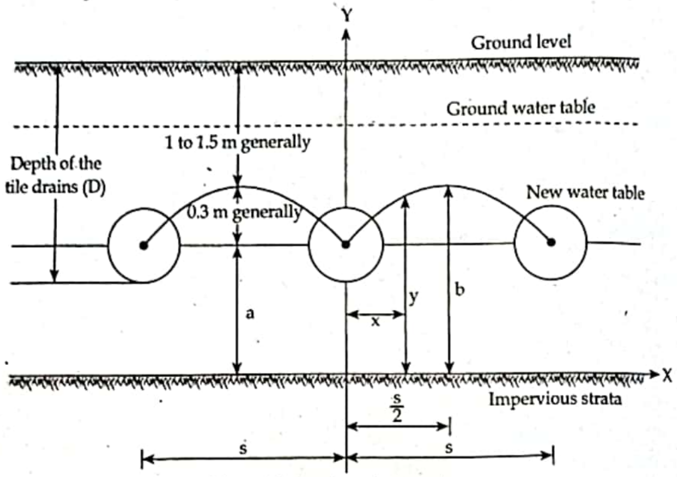

Figure 9.8: Spacing of tile drains

From Darcy law,

$$Q = K \times I \times A$$

Discharge per unit length of the drain passing section at 'y' (qy) is given by;

$$q_y = k\frac{dy}{ds}y$$

Assuming inclination of water surface to be small $\frac{dy}{dx} = \frac{dy}{ds}$.

$$q = k\frac{dy}{dx}y \qquad (1)$$

When $x = \frac{s}{2}$

$$q_y = 0$$

When $x = 0$;

$$q_y = \frac{q}{2}$$

or, $$q_y = \frac{q}{2} - \frac{q}{2} \times \frac{x}{\frac{s}{2}}$$

or, $$q_y = q \times \frac{(s-2x)}{2s} \qquad (2)$$

From equation (1) and (2); we have,

$$q_y \times \frac{(s-2x)}{2s} = k \times y \times \frac{dy}{dx}$$

On integration; we get,

$$\int \frac{q(s-2x)}{2ks}dx = \int y\,dy$$

or, $$\frac{\frac{q(sx-2x^2)}{2}}{2ks} = \frac{y^2}{2} + C$$

When $x = 0$;

$$y = a$$

$$C = \frac{-a^2}{2}$$

or, $$\frac{q(sx-2x^2)}{2ks} = \frac{y^2}{2} - \frac{a^2}{2}$$

When $x = \frac{s}{2}$

$$y = b$$

or, $$q \times s \times \frac{(s - \frac{s}{2})}{2 \times k \times s \times 2} = \frac{(b^2 - a^2)}{2}$$

or, $$\frac{q \times s}{4k} = (b^2 - a^2)$$

or, $$s = \frac{4k(b^2 - a^2)}{q}$$

Again;

$$q = s \times \text{Drainage recharge (rain fall)}$$

$$S^2 = 4k(b^2 - a^2) \times \frac{1}{\text{Recharge }(R_e)}$$

R_e, is annual average precipitation (P_{AA}); then,

$$S^2 = 8.64 \times 10^6 \times \frac{4k(b^2 - a^2)}{P_{AA}}$$

Formula for steady flow

Smooth flow (Q) = $50d^{2.57}i^{0.57}$

Corrugated pipe (Q) = $22d^{2.67}i^{0.50}$

Formula for unsteady flow

Smooth pipe (Q) = $89d^{2.71}i^{0.57}$

Corrugated pipe (Q) = $88d^{2.67}i^{0.5}$

Example 9.2

Design a suitable subsurface drainage 250 m × 900 m plot .The depth of impervious layer is 10 m. The root zone depth is 1 m.
The drainage recharge is 5 mm/day.
Saturated conductivity = 2.5 m/day and slope of drainage is 1 : 100.

Solution:

Here,

$$a = 8.7\text{ m}$$

$$b = 9.0\text{ m}$$

$$R_e = 5\text{ mm/day} = 0.005\text{ m/day}$$

$$K = 2.5\ \text{m/day}$$

But, $S^2 = \dfrac{4k\left(b^2 - a^2\right)}{R_e} = \dfrac{4 \times 2.5 \times \{(9)^2 - (8.72)^2\}}{0.005} = 103$

Adopting s = 100 m

$$Q = 100 \times 250 \times 5\ \text{mm/day} = 0.00144\ \text{m}^3/\text{sec.}$$

For unsteady flow;

$$\text{Smooth pipe (Q)} = 89d^{2.71}i^{0.57}$$

or, $0.00144 = 89d^{2.71}i^{0.57} = 89d^{2.71} \times (2.5)^{0.57}$

or, $d = 0.045\ \text{m} = 45\ \text{mm}$

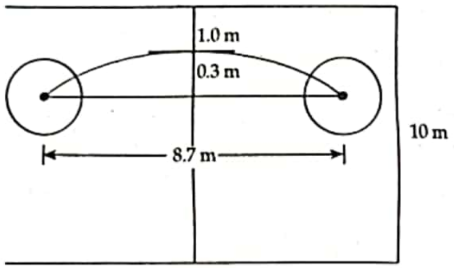

Figure 9.9

Example 9.3

Determine the size of a tile 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 coefficient of discharge means that 1cm of water from an area of 6 hectares entering the tiles per day.

$$\text{Volume of water passing drain in 1 day} = \left(\frac{1 \times 6 \times 104}{100}\right) = 600\ \text{m}^3/\text{day}$$

$\therefore$ $Q = \dfrac{1}{144}\ \text{m}^3/\text{sec.}$

Now,

$$Q = \frac{1 \times A \times R^{\frac{2}{3}} \times S^{0.5}}{n}$$

For circular drain pipe of diameter 'D'; we have,

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

$$P = \pi D$$

$$R = \frac{D}{4}$$

or, $\dfrac{1}{144} = \left(\dfrac{1}{0.011}\right) \times \left(\dfrac{\pi D^2}{4}\right) \times \left(\dfrac{D}{4}\right) \times \left(\dfrac{0.3}{100}\right)^{0.5}$

On solving; we get,

$$D = (0.00447)^{\frac{3}{8}} = 0.132\ \text{meter} = 13.2\ \text{cm}$$

Adopt 15 cm diameter pipe which is available in market.

Example 9.4

A tile drainage system the drainage is with their centers 1.5 m below the ground level. The impervious level is 0.9 m below the ground level and the average annual rainfall in the area is 80 cm. If 1% rainfall is to be drained in 24 hrs to keep the highest position of the water table to 1 meter below the ground level, determine the spacing of the drain pipe. Coefficient of permeability may be taken as 0.001 cm/sec.

Solution:

We have,

$$q = \frac{\%P_{AA}}{(24 \times 3600)} \times (S \times 1)\ \text{cum/m length of drain}$$

$$q = \left(\frac{1}{100}\right) \times \left(\frac{80}{100}\right) \times \frac{(S \times 1)}{86400}\ \text{m}^3/\text{s/m length of drain pipe}$$

$$q = \frac{0.8\,S}{\left(8.64 \times 10^6\right)}\ \text{m}^3/\text{s/m length of pipe} \qquad (1)$$

Again; we have,

$$S = \frac{4k\left(b^2 - a^2\right)}{q} \qquad (2)$$

where, b is the height of water table above impervious layer = 9 – 1 = 8 m.

a is depth of impervious stratum below center of drains = 9 – 1.5 = 7.5 m

$$k = 0.001\ \text{cm/s} = \frac{0.001}{100}\ \text{m/sec.}$$

From equation **(1)** and **(2)**; we get,

$$S \times q = 4 \times 0.001 \times \frac{\left[(8)^2 - (7.5)^2\right]}{100}$$

or, $S \times \dfrac{0.8\,S}{\left(8.64 \times 10^6\right)} = 4 \times 0.001 \times \dfrac{\left[(8)^2 - (7.5)^2\right]}{100}$

or, $S^2 = 3348$

or, $S = 57.86\ \text{m}$

9.5 PLANNING AND DESIGN OF GROUND WATER IRRIGATION SCHEMES

9.5.1 Exploration and development of ground water

When rainfall occurs, some portion of rainfall is lost in the form of evaporation, interception, depression on the ground, etc. The remaining rainfall water enters into the ground surface directly or after passing a certain distance. The entrance of rain water into the ground is known as infiltration. The movement of water after entrance is known as percolation. The portion of rainfall percolates in the ground and is stored as ground water at hard stratum. This ground water percolates through the soil and appears again at the surface in the form of spring or dry-weather flow of streams. The water table of underground water generally corresponds to water level of stream, rivers and sea depending upon their position.

9.5.2 Types of well

Wells are the vertical holes made below the ground surface to the top. There are two types of well in common which are as follows:

i) Shallow wells

These are the well dug in the uppermost layer of the earth and obtain their water supply from the sub-soil water table. These are very suitable for supplying water in rural areas or for only a portion of a town. Not only the discharge of these wells is less but also the quantity of water in this case is poor.

ii) Deep wells

These wells are drilled to an aquifer below an impervious stratum. As the water travel a long distance from the out crop to the site of the well, it gets purified due to natural filter of soil particles. The water is usually hard as it contains dissolved salts in it. Bore holes may be done either by percussion boring or rotary boring depending upon the nature of soil. A strainer is provided for the entire depth of the porous stratum to exclude sand and to admit water.

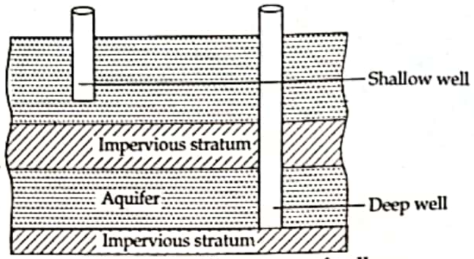

Figure 9.10: Different types of wells

9.6 WORKED OUT PROBLEMS

PROBLEM 1

Derive the expression for subsurface drainage.

Solution: See the definition part 9.4.2

PROBLEM 2

Write down the causes and effects of water logging. Explain methods of reclamation of water logged area.

Solution:

Causes and effects of water logging

See the definition part 9.1

Land reclamation

Restoring the productivity and fertility of the land which has became unculturable or has suffered a reduction in the crop yield because of water logging, salinity, etc.

Before starting the reclamation process, it is necessary to properly level the land surface so that uniform reclamation is achieved. Land reclamation is done by adopting one or more the following methods.

i) Drainage and lowering of water table

In general, the water logged land is considerably improved by adopting various drainage methods and lowering the water table. The source of water which is causing water logged should be located, and suitable measure should be adopted to check the flow from that source. If water logging is caused by underground water, the source should be cut off by constructing intercepting drains. If water logging is caused by surface water suitable surface drainage systems should be adopted.

ii) Leaching

Leaching is the process in which land is flooded with an abundant quantity of water to a depth 15 to 25 cm over the surface. The excess salts are washed down from the land surface to the ground water, provided the water table had already been lowered to a safe limit. This process is continued till salt in the surface layer reduced to a safe limit. Leaching operation can be repeated after 4 to 5 years, if necessary.

iii) Use of chemicals

Chemicals are sometimes added for reclamation of lands. Gypsum is most commonly used chemical for alkali soils, Sodium carbonate in the soil is easily removed by spreading gypsum on the land at rate of 2 tonne/ha before leaching. Calcium carbonate is also sometime added. However it is less effective than gypsum. The low solubility of gypsum results in better activity than that in calcium chloride.

Sometimes sulphuric acid is also applied to the land. The top 20 cm soil layer is usually treated with 1 to 5 percent solution of sulphuric acid to neutralize alkalies in the soil. It improves growth of plant. Acid-forming fertilizers are also effective.

iv) Adopting rice cultivation

Salinity of land is reduced by rice cultivation. In rice cultivation, the large depth of water over land leaches the salts and keeps them at a safe depth below surface. Rice cultivation also causes reduction in the alkalinity of the soil. The roots of rice plants produce carbon dioxide which lowers the p^H value, increases the percolation and brings the exchangeable sodium ions of soil into solution. However, during rice cultivation nitrogen in soil is decreases. To compensate the nitrogen in the soil, a leguminous crop such as gram should cultivate in next Rabi season.

v) Crop rotation

Salinity of soil can be further reduced after leaching by adopting crop rotation in which rice or maize is introduced. The following two crop rotations are commonly adopted.

a) Rice in rotation
   1. Rice-Wheat moong-rice
   2. Rice-Senji-Sugarcane-rice
   3. Rice-berseem-rice

b) Maize in rotation
   1. Wheat-maize-berseem-cotton-wheat
   2. Wheat-maize-Senji-wheat
   3. Wheat-maize-rice-wheat

vi) Green manuring

A crop called Jantar is generally used as green manuring crop. It has rapid growth in saline and poorly drained soil. Other green manuring corps is San, Senji, Gaura, Berseem, etc.

Green manuring also improves the structure of the soil. It releases the organic acids which lower P^H value and add nitrogen to the soil.

vii) Addition of agricultural waste products

The salinity of the soil can be reduced by adding agricultural waste products such as ground nut hull, saw dust molasses with limit sludge, distillery waste, sunflower hull, tamarind seed power, etc. Molasses alone are not effective for the reclamation of alkali soils. However, when combined with lime sludge, they are quite effective. Distillery wastes are acidic and quite effective in reducing alkalinity and in replacing sodium with calcium.

viii) Use of argemona plant

Argemona is a plant which grows on the waste land. The plants are highly acidic and can be effectively used to reduce the alkalinity of the soil.

ix) Electro-dialysis

Electro-dialysis can be used to reduce the alkalinity of the soil. When an electric current is passed through the soil, it renders it porous and permeable. Hence, the infiltration rate is increased and soluble salts are washed out. This method is expensive and can be used where cheap power is available.

x) Use of processed coal

The alkalinity of a soil can be reduced by addition of a small quantity of processed coal to the soil.

PROBLEM 3

What is water logging? Write down its causes, effects and preventive measures of water logging.

Solution: See the definition part 9.1

PROBLEM 4

The annual rainfall in Biratnagar is 2000 mm. Find the spacing of sub-surface drains if 2% of average annual rainfall is to be drained in 2 days. Depth of impervious stratum from the top of soil surface = 12 m Position of drains is 2 m below the top soil surface and the depth of highest position of water table below the top soil surface = 1.5 m Permeability $k = 1 \times 10^{-4}$ m/sec.

Solution:

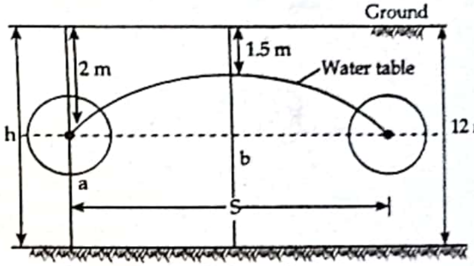

Figure 9.10: Impervious stratum

$$q = \frac{\%\text{ of } P_{AA}}{\text{time}} \times S \times 1$$

where, P_{AA} = average annual rainfall

S = Spacing of drain

q = Total discharge per unit length

$$q = \frac{\frac{2}{100} \times \left(\frac{2000}{1000}\right)}{2 \times 24 \times 3600} \times S \times 1 \text{ m}^3/\text{s}$$

$$= 2.31485 \times 10^{-7} \text{ m}^3/\text{s/m (length of tile drain)} \qquad (1)$$

Again;

$$\text{Spacing (S)} = \frac{4k\left(b^2 - a^2\right)}{q}$$

Here,

b = Depth of water table above the impervious layer

= (12 – 1.5) m = 10.5 m

d = Depth of impervious stratum below the centre of the drains

= 12 – 2 = 10 m

$k = 1 \times 10^{-4}$ m/sec.

Now,

$$S = \frac{4 \times 1 \times 10^{-4} \times \{(10.5)^2 - 10^2\}}{q}$$

or, $S \times q = 4 \times 10^{-3}$

Substituting value of 'q' from equation (1); we get,

$$S \times 2.31485 \times 10^{-7} = 4 \times 10^{-3}$$

or, $S^2 = 17280.11$

or, $S = 131.45$ m

Hence, spacing of drain pipe is 131.45 m.

PROBLEM 5

Design a surface drainage for a field of 40 ha area in Terai with following data. Design maximum yearly precipitation for three consecutive days = 50 mm, longitudinal slope of channel 1 : 400, Manning roughness coefficient 0.025, Maximum water level is 300 mm which may persist per up to one day and depends in excess of 200 mm persists for up to 3 days. Assume other suitable values if necessary.

Solution:

Here,

Maximum yearly precipitation for 3 day is 50 mm, which is not reasonable.

So, assume maximum yearly precipitation (P_3) = 450 mm

Assume initial water depth = 40 mm

Out let discharge (Q) = 50 mm/day

Now, using water balance equation (W.B.); we have,

$$h = 40 + \frac{P_3}{3} \times t - Q \times t$$

Day	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	40	150	50	140
2	140	150	50	240
3	240	150	50	340
4	340	-	50	290

Here,

Water depth (h) > 300 mm, so second trail is needed.

Theoretically 'h' should not greater than 300 mm.

To meet our requirement 'Q' should increased and 'h' may tends to 300 mm

Second trail

Q = 75 mm/day

Day	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	40	150	75	115
2	115	150	75	190
3	190	150	75	265
4	265	-	75	190
5	190	-	75	115

Here,

No water depth > 300 mm

Depth excess of 200 mm occurs only in one day.

Theoretically this design is ok but uneconomical.

So, take next trail.

Q = 65 mm/day

Day	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	40	150	65	125
2	125	150	65	210
3	210	150	65	245
4	245	-	65	230
5	230	-	65	165

Here, water level greater than 200 mm persist for 3 days.

No water depth greater than 300 mm. Hence, design is ok and economical.

∴ Design discharge (Q) = 65 mm/day

Here,

Area (A) = 40 ha

Q = AV

or, $Q = \dfrac{40 \times 10^4 \times 65 \times 10^{-3}}{24 \times 3600}$ [∵ 1 ha = 10^4 m², 1 day = 24×3600 S]

∴ Q = 0.301 m³/sec.

Given that;

Bed slope = 1 : 400

Manning roughness coefficient (N) = 0.025

Assume side slope = 1 : 1

Bed width to depth ratio (B : D) = 3 : 1

Now,

$$Q = \frac{1}{N} \times \frac{A^{\frac{5}{3}} S^{\frac{1}{2}}}{P^{\frac{2}{3}}}$$

where, $A = BD + D^2 = 3D \times D + D^2 = 4D^2$

$P = B + 2\sqrt{2}D = (3D + 2\sqrt{2}D) = 5.828\ D$

Now,

$$0.301 = \frac{1}{0.025} \times \frac{(4D^2)^{\frac{5}{3}} \times \frac{1}{\sqrt{500}}}{(5.828\ D)^{\frac{2}{3}}}$$

On solving; we get,

D = 0.224 m

Take available size D = 0.3 m

and, B = 0.9

PROBLEM 6

Find the spacing of tile drains for an area having average annual rainfall of 1350 mm, if 1% is to be drained in 24 hours. From ground level depth of impervious stratum = 8.75 m; depth of drains = 2.1 m and depth of highest position of water table = 1.2 m. Coefficient of permeability may be taken as 0.001 cm/ sec. **[2069 Poush]**

Solution:

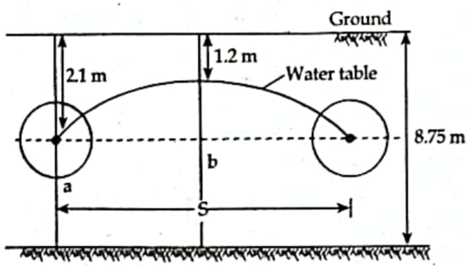

Figure 9.11: Impervious stratum

Given that;

Drained % of annual rainfall = 1%

Time = 24 hrs

Depth of impervious stratum = 8.75 m

Depth of drains = 2.1 m

Depth of highest position of water table = 1.2 m

Coefficient of permeability = 0.001 cm/sec. = $\frac{0.001}{100}$ m/sec.

We have,

$$q = \frac{\text{\% of } P_{AA}}{\text{time}} \times S \times 1 = \frac{\frac{1}{100} \times \left(\frac{1350}{1000}\right)}{24 \times 3600} \times S \times 1 = 1.5625 \times 10^{-7} \times S \times 1 \quad (1)$$

$$= 1.5625 \times 10^{-7} \times S \times 1 \text{ m}^3/\text{s/m length of tile drain}$$

Again,

$b = 8.75 - 1.2 = 7.55$ m

$a = 8.75 - 2.1 = 6.65$ m

Now,

$$\text{Spacing (S)} = \frac{4k\left(b^2 - a^2\right)}{q}$$

or, $S \times q = 4 \times \frac{0.001}{100} \times (7.55^2 - 6.65^2)$ (2)

or, $S \times q = 5.112 \times 10^{-4}$

Substituting value of 'q' from (I) to (II); we get,

$S \times 1.5625 \times 10^{-7} = 5.112 \times 10^{-4}$

or, $S^2 = 3271.68$

or, $S = 57.2$ m

Hence, spacing is 57.2 m.

PROBLEM 7

Estimate the internal drainage discharge in l/s/ha from bunded rice fields at Janakpur area for 10 years 3-day design rainfall 350 mm. Assume necessary data suitably.

Solution:

Given that;

$P_3 = 350$ mm

$\frac{P_3}{3} = \frac{350}{3} = 116.67$ mm

Assume;

Initial water depth = 40 mm

Q = 50 mm/day

Then from water balance equation; we have,

First trail

Day	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	40	116.67	50	106.67
2	106.67	116.67	50	173.34
3	173.34	116.67	50	240.01

This design is not economical.

Second trail

Q = 45 mm/day

Day	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	40	116.67	45	116.67
2	116.67	116.67	45	183.34
3	183.34	116.67	45	255.01
4	255.01	-	45	210.01
5	210.01	-	45	165.01

Here, no water depth greater than 300 mm

Depth excess of 200 mm occurs in 2 day. So, theoretically this design is ok, but for economical design, take; Q = 40 mm/day

Trail third

Day	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	40	116.67	40	116.67
2	116.67	116.67	40	193.34
3	193.34	116.67	40	270.01
4	270.01	-	40	230.01
5	230.01	-	40	190.01

Here, water depth greater than 200 mm occur for 2 days. No water depth greater than 300 mm. Hence, design is ok and also economical.

Hence, design discharge 40 mm/day

$$Q = 40 \text{ mm/day} = 0.04 \text{ m/day}$$

$$Q = \frac{0.04}{86400} \text{m/sec.} \times \frac{1 \text{ ha}}{1 \text{ ha}} = \frac{0.04}{86400} \times 10^4 \frac{m^3}{\text{sec./ha}}$$

$$= 4.63 \text{ l/sec./ha}$$

PROBLEM 8

Determine drainage rate in l/s/ha required to meet following conditions for healthy growth of rice paddies in bonded fielded in Terai of Nepal.
Initial water level in filed = 50 mm
Maximum water level is 400 mm which may persist for up to one day
Depth in excess of 250 mm may persist for up to 2 days
No rain follows the design rainfall for several days
Neglect ET and deep percolation losses
Design 3 day rainfall in 400 mm

Solution:

Given that;

Initial water depth = 50 mm

Maximum yearly rainfall (P_3) = 400 mm

$$\frac{P_3}{3} = \frac{400}{3} = 133.33$$

Now, from water balance equation

$$h = \text{Initial water depth} + \frac{P_3}{3} \times t - Q \times t$$

Assume,

$Q = 50$ mm/day

Trail 1

Days	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	50	133.33	50	133.33
2	133.33	133.33	50	216.66
3	216.66	133.33	50	299.99
4	299.99	-	50	249.99

Here, depth greater than 250 mm exists only for one day. So do next trail.

Trial 2

$Q = 45$ mm/day

Days	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	50	133.33	45	138.33
2	138.33	133.33	45	226.66
3	226.66	133.33	45	314.99
4	314.99	-	45	269.99
5	269.99	-	45	224.99

Here, depth greater than 250 mm exists for only one day.

This design is ok but uneconomical.

Again, take $Q = 40$ mm/day

Trail 3

$Q = 40$ mm/day

Day	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	50	133.33	40	143.33
2	143.33	133.33	40	236.66
3	236.66	133.33	40	329.99
4	329.99	-	40	289.99
5	289.99	-	40	249.99

Here, depth greater than 300 mm exist for only one day and depth greater than 250 mm persists up to two day and no water depth greater than 400 mm. Hence, design is ok and economical.

so, Design discharge (Q) = 40 mm/day

$$Q = \frac{40 \times 10^{-3}}{86400} \text{m/sec.}$$

$$= \frac{40 \times 10^{-3}}{86400} \text{m/sec.} \times \frac{1 \text{ ha}}{1 \text{ ha}}$$

$$= \frac{40 \times 10^{-3}}{86400} \times 10^4 \text{ m}^3\text{/sec./ha} \qquad [\because 1 \text{ ha} = 10^4 \text{ m}^2]$$

$$= \frac{400}{86400} \text{ m}^3\text{/sec./ha}$$

$$= \frac{400}{86400} \times 1000 \text{ l/sec./ha} \qquad [\because 1000 \text{ l} = 1 \text{ m}^3]$$

$$= 4.63 \text{ l/sec./ha}$$

PROBLEM 9

Determine the discharge of drainage channel in l/sec./ha, designed to meet following condition for the growth of rice paddies in bunded fields in plains of Nepal.
Initial water level in field = 30 mm
Maximum water level is 250 mm which may persist for up to one day
Depth in excess of 200 mm may persist for up to 2 days
No rain follows the design rainfall for several days
Design 3 days rainfall in 300 mm **[2070 Magh]**

Solution:

Here,

$$\frac{P_3}{3} = \frac{300}{3} = 100 \text{ mm}$$

Initial water level = 30 mm

Assume Q = 20 mm/day

From water balance equation; we have,

$h = \text{Initial water depth} + \frac{P_3}{3} \times t - Q \times t$

Day	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	30	100	20	110
2	110	100	20	190
3	190	100	20	270
4	270	-	20	250
5	250	-	20	230
6	230	-	20	210

Here, depth is greater than 200 mm exist up to 4 days so this design is not ok so increase 'Q'.

Take Q = 30 mm/day

Day	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	30	100	30	100
2	100	100	30	170
3	170	100	30	240
4	240	-	30	210
5	210	-	30	180

Here, no water depth greater than 250 mm

Depth excess of 200 mm exists for up to 2 days. This design is ok.

Now, take another trail

$Q = 25 \text{ mm/day}$

Trail 3

Day	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	30	100	25	105
2	105	100	25	180
3	180	100	25	255

Here, depth greater than 250 mm exists for one day so this design is not ok.

Here, trail 3 is not required.

Third is only to check economic criteria of design.

Hence, Q = 30 mm/day is design discharge

$$Q = 30 \text{ mm/day}$$

$$Q = \frac{30 \times 10^{-3}}{86400} \times 10^4 \text{ m}^3\text{/sec./ha}$$

$$= \frac{30 \times 10^{-3} \times 10^4 \times 1000}{86400} \text{ } l\text{/sec./ha}$$

$$= 3.47 \text{ } l\text{/sec./ha}$$

PROBLEM 10

List out the main effects and preventive measures of water logging. Estimate the rate of internal drainage discharge in lps/ha from bunded rice fields of Terai area. The 3-day design rainfall of 10 years frequency in that area has been estimated as 400 mm. Make suitable assumptions for removing excess water from the field of Terai. [2071 Bhadra: T.U.]

Solution:

The effects of water logging are as follows:

i) Less crop yields.
ii) Inadequate circulation of air in root zone of crops.
iii) Fall in sail temperature.
iv) Delay in cultivation operation.
   a) Difficult in ploughing and mulching.
   b) Showing and growth of crops are also delayed.
v) Breading of insects and mosquitoes.
vi) Growth of unwanted planted.
vii) Inhabiting activity of bacteria.

The preventive measures of the water logging are as follows:

i) Lining of canal water course.
ii) Reducing the intensity of irrigation.
iii) By reducing crop rotation.
iv) By optimum use of water.
v) By providing intercepting drains.
vi) By provision of effective drainage system.
vii) By adopting consumptive use of surface and sub-surface water.

Here,

Design 3 days rainfall (P_3) = 400 mm

$\therefore \frac{P_3}{3} = \frac{400}{3} = 133.33 \text{ mm}$

Assuming;

Initial water level = 40 mm

Maximum water level 300 mm which persist for 1 day.

Depth in excess of 200 mm may persists for 3 days.

From water balance equation; we have,

$h = \text{Initial water depth} + \frac{P_3}{3} \times t - Q \times t$

For first trail; take Q = 50 mm/day

Day	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	40	133.33	50	123.33
2	123.33	133.33	50	206.67
3	206.67	133.33	50	289.99
4	289.99	-	50	239.99
5	239.99	-	50	189.99

Here, water depth > 300 mm is not available and depth of water more than 200 mm exists 3 days which is in accordance with design consideration. For accuracy, let's do next trial 'Q' should be decreased so 'h' tends to 300 mm.
Take Q = 45 mm/day;

Day	Initial water depth	$\frac{P_3}{3}$	Q	Final water depth
1	40	133.33	45	128.33
2	128.33	133.33	45	216.66
3	216.66	133.33	45	305
4	305	–	45	259.99
5	259.99	–	45	214.99
6	214.99	–	45	170

Here, water depth > 300 mm, so this design is not ok.
Hence, design discharge = 50 mm/day

$$Q = \frac{50 \times 10^{-3}}{86400} \times 10^4 \text{ m}^3/\text{sec./ha}$$
$$= \frac{50 \times 10^{-3} \times 10^4 \times 1000}{86400} \; l/\text{sec./ha}$$
$$= 5.787 \; l/\text{sec./ha}$$

PROBLEM 11

Derive the equation to find the spacing of tile drain. Calculate the spacing of the tile drains for an area having average rainfall of 1600 mm, if 1.5% is to be drained in 24 hrs. From ground level, depth of impervious stratum is 9 m, depth of drains is 2.0 m and depth of highest position of water table is 1.0 m, coefficient of permeability = 0.001 cm/sec. [2071 Magh: T.U.]

Solution:

Derivation of spacing of tile drain

See the definition part 9.4.2

Numerical

We have,

$$q = \left(\frac{\%P_{AA} \times 0.8}{24 \times 3600}\right)(S \times 1) \text{ cu. m/m length of drain}$$

or, $$q = \frac{1}{100} \times \frac{1600}{1000} \times \frac{S \times 1}{86400} \text{ m}^3/\text{sec./m length of drain}$$

$\therefore$ $$q = \frac{1.6S}{8.64 \times 10^6} \text{ m}^3/\text{sec./m length of drain} \quad (1)$$

Again,

$$S = \frac{4K(b^2 - a^2)}{q} \quad (2)$$

where, b is the height of water table above impervious layer = 9 – 1 = 8 m.

a is the depth of impervious stratum below centre of drains = 9 – 2 = 7 m

$$K = 0.001 \text{ cm/sec.} = \frac{0.001}{100} \text{ m/sec.}$$

Then, from equation (1) and (2); we get,

$$S \times q = 4 \times 0.001 \times \frac{(8)^2 - (7)^2}{100}$$

or, $$S \times \frac{1.6S}{8.64 \times 10^6} = 4 \times 0.001 \times \frac{15}{100}$$

or, $$S^2 = 3240$$

$\therefore$ $$S = 56.92 \text{ m}$$

PROBLEM 12

Explain in details the procedures of designing drainage canals in irrigated paddy fields. [2072 Ashwin]

Solution: See the definition part 9.3.4

PROBLEM 13

Explain all steps required to arrive design discharge of a drainage canal in irrigated paddy field. [2072 Magh]

Solution: See the definition part 9.3.4

PROBLEM 14

Write down the effects and preventive measures of water logging. [2074 Bhadra]

Solution: See the definition part 9.1

PROBLEM 15

Determine the drainage late required to meet the following condition. Maximum yearly precipitation for 3 consecutive days = 300 mm. the design rainfall is to be taken as 10 years return periods. Initial water level in field = 40 mm, maximum water level is 300 mm, which may persist for up to 1 day and depth in excess 200 mm may 1 day and depth in excess of 200 mm may persist up to 3 days. Take growth factor for 10 years return period as 1.5. Assume other suitable data if necessary. [2074 Bhadra]

Solution: See the solution of Q. no. 8

NOTE

Use factor 1.5 to multiply the final drainage rate.

PROBLEM 16

Derive expression for the spacing of sub surface drainage. [P.U. 2015]

Solution: See the definition part 9.4.2

PROBLEM 17

Define water logging. Explain its causes, effect and prevention of water logging. [P.U. 2015]

Solution: See the definition part 9.1

PROBLEM 18

Determine drainage rate in mm/day required to meet the following conditions for healthy growth of rice bunded field of plain area of Nepal.
Initial water level in field = 55 mm
Maximum water level is 400 mm which may persist for up to 1 day
Depth of excess of 250 mm may persist for up to 2 days
No rainfall follows the design rainfall for several days
Design 3 days rainfall is 400 mm
Neglect ET and deep percolation losses. [2076 Bhadra]

Solution: Proceed same as the solution of problem no. 8

PROBLEM 19

Derive the equation to find the spacing of tile drain. Calculate the spacing of the tile drains for an area having rainfall of 1700 mm, if 1.5% is to be drained in 24 hrs. From ground level, depth of impervious stratum is 10 m, depth of drain is 2.2 m and depth of highest position of water table is 1.2 m, coefficient of permeability = 0.001 cm/sec. [2077 Chaitra]

Solution: Proceed same as the solution of problem no. 11 or example 9.4

PROBLEM 20

Design the drainage canal in paddy field for 5 ha or land to meet the following conditions. Maximum yearly precipitation for 3 consecutive days = 300 mm, the designed rainfall is to be taken as 10 years return periods. Initial water level in the field = 40 mm. Maximum water level is 300 mm, which may persist for one day and depth in excess of 200 mm may persist up to 3 days. Take growth factor for 10 years return period of 1.5 years. Assume other suitable data if necessary. [2078 Baishakh]

Solution: Proceed same as the solution of problem no. 5

Note

Use factor 1.5 to multiply the design discharge as growth factor is given 1.5 from question.

9.7 OBJECTIVE QUESTION

1. A land is said to be water logged when ....
   a) the land is necessarily submerged under standing water
   b) there is a flowing water over the land
   c) the pH value of the soil becomes as high as 8.5
   d) the soil pores in the root zone gets saturated with water, either by the actual water table or by its capillary fringe
2. Water-logging of cropped land leads to reduced crop yields due to ....
   a) ill-aeration of root zone, causing lack of oxygen to plants
   b) growth of water-logging plant interfering with the sown crop
   c) surrounding of the root zone by the resultant saline water, which extracts the good water from plant roots by osmosis
   d) none of the above
3. Alkaline soils are best reclaimed by ....
   a) leaching b) addition of gypsum to soil
   c) providing good drainage
   d) addition of gypsum to soil and leaching
4. Which one of the following does not contribute water logging?
   a) Inadequate drainage
   b) Seepage from unlined canals
   c) Frequent flooding
   d) Excessive tapping of ground water
5. The soil become practically infertile when its pH value is about ....
   a) 0 b) 7 c) 11 d) none of above
6. Point out incorrect statement, out of the following
   a) Salinity is caused by water logging.
   b) Water logging is not caused by salinity.
   c) Salinity subsides, when once the water logging is remove.
   d) none of the above
7. A richly reclaimed alkaline soil should preferably be shown with a salt resistant crop like ....
   a) wheat b) cotton c) barseem d) none of above
8. Which one of the following is not a remedial measure for water logging?
   a) Good drainage for irrigated land b) Conjuctive use of water in basin
   c) Lining of canals and water course d) Contour bunding
9. Which of the following crops can withstand the highest water table, which normally should be 0.7 and 2.5 m below the level of the cropped land?
   a) Wheat b) Lucerine c) Cotton d) Rice
10. The method, which uses dead furrows on cropped farms for drainage of excess irrigation or rain water is called ....
    a) surface limit b) tile drainage c) bedding d) French drain

Answer sheet

1	2	3	4	5	6	7	8	9	10
d	d	d	d	c	c	c	d	d	c