ATME College of Engineering

DEPARTMENT OF CIVIL ENGINEERING

(ACADEMIC YEAR 2025-26)

SUB: Irrigation Engineering and Hydraulics Structures

SEMESTER: VI

SUB CODE: BCV602

VISION OF THE INSTITUTE

Development of academically excellent, culturally vibrant, socially responsible and globally competent human resources.

MISSION OF THE INSTITUTE

- To keep pace with advancements in knowledge and make the students competitive and

  capable at the global level.

- To create an environment for the students to acquire the right physical, intellectual,

  emotionalandmoralfoundationsandshineastorchbearersoftomorrow'ssociety.

- To strive to attain ever-higher benchmarks of educational excellence.

VISION OF THE DEPARTMENT

To develop globally competent civil engineers who excel in academics, research and are ethically responsible for the development of the society.

MISSION OF THE DEPARTMENT

- To provide quality education through faculty and state of the art infrastructure.
- To identify current problems in the society pertaining to Civil Engineering disciplines and to address them effectively and efficiently.
- To inculcate the habit of research and entrepreneurship in our graduates to address current infrastructure needs of society.

PROGRAM EDUCATIONAL OBJECTIVES (PEO)

Graduates who complete their UG course through our institution will be,

PEO l- Engaged in professional practices, such as construction, environmental, geotechnical, structural, transportation, or water resources engineering by using technical, communication and management skills.

PEO 2- Engaged in higher studies and research activities in various Civil Engineering fields and a life time commitment to learn ever changing technologies to satisfy increasing demand of sustainable infrastructural facilities

PEO3-Serve in a leadership position in any professional or community organization, or local/state engineering board

PEO 4- Registered as a professional engineer or developed a strong ability leading to professional licensure being an entrepreneur.

PROGRAM SPECIFIC OUTCOMES (PSO'S)

PSO1-Provide necessary solutions to build infrastructure for all situations through Competitive plans, maps and designs with the aid of a thorough Engineering Survey and Quantity Estimation.

PSO 2- Assess the impact of anthropogenic activities leading to environmental imbalance on land, in water & in air and provide necessary viable solutions revamping water resources and transportation for a sustainable development

Faculty Name: **AKHILA C G**	Academic Year: 2025-26
Department: Civil Engineering	

Course Code	Course Title	Core/Elective	Prerequisite	Contact Hours			Total Hrs. /Sessions
				L	T	P	
BCV602	**Irrigation Engineering and Hydraulics**	Core	**Knowledge of Engineering Mechanics.**	**03**	**02**	**-**	**50**

Objectives

1. Analyse and design gravity dam
2. Find the cross section of earth dam and estimate the seepage loss
3. Design spillways and apron for diversion work
4. Design CD works and chose appropriate canal regulation works

Topics Covered as per Syllabus

MODULE 1:

Irrigation: Definition. Benefits and ill effects of irrigation. System of irrigation: surface and ground water, flow irrigation, lift irrigation, Bandhara irrigation. Water Requirements of Crops: Duty, delta and base period, relationship between them, factors affecting duty of water crops and crop seasons in India, irrigation efficiency, frequency of irrigation

MODULE 2:

Canals: Types of canals. Alignment of canals. Definition of gross command area, cultural command area, intensity of irrigation, time factor, crop factor. Unlined and lined canals. Standard sections. Design of canals by Lacey's and Kennedy's method. Reservoirs: Definition, investigation for reservoir site, storage zones determination of storage capacity using mass curves, economical height of dam.

MODULE 3:

Gravity dams: Forces acting on a gravity dam, causes of failure of a gravity dam, elementary profile, and practical profile of a gravity dam, limiting height of a low gravity dam, Factors of Safety – Stability Analysis, Foundation for a Gravity Dam, drainage and inspection galleries.

MODULE 4:

Earth dams: Types of Earth dams, causes of failure of earth dam, criteria for safe design of earth dam, seepage through earth dam-graphical method, measures for control of seepage. Spillways: types of spillways, Design principles of Ogee spillways – Spillway gates. Energy Dissipaters and Stilling Basins Significance of Jump Height Curve and Tail Water Rating Curve – USBR and Indian types of Stilling Basins.

MODULE 5:

Diversion Head works: Types of Diversion head works- weirs and barrages, layout of diversion head work – components. Causes and failure of Weirs and Barrages on permeable foundations, -Silt Ejectors and Silt Excluders, Weirs on Permeable Foundations – Creep Theories – Bligh’s, Lane’s and Khosla’s theories, Determination of uplift pressureVarious Correction Factors – Design principles of weirs on permeable foundations using Creep theories – exit gradient, U/s and D/s Sheet Piles – Launching Apron

List of Text Books

Irrigation Engineering and Hydraulic structures by Santhosh kumar Garg, Khanna Publishers
Irrigation engineering by K. R. Arora Standard Publishers.
Irrigation and water power engineering by Punmia & Lal, Laxmi publications Pvt. Ltd., New Delhi
Theory and Design of Hydraulic structures by Varshney, Gupta & Gupta
Irrigation Engineering by R.K. Sharma and T.K. Sharma, S. Chand Publishers 2015.
Irrigation Theory and Practice by A. M. Micheal Vikas Publishing House 2015.
Irrigation and water resources engineering by G.L. Asawa, New Age International Publishers.

Course Syllabus with CO's

List of URLs, Text Books, Notes, Multimedia Content, etc	
https://searchworks.stanford.edu/view/10496310 https://searchworks.stanford.edu/view/13576277 https://searchworks.stanford.edu/view/11842972	
Course Outcomes	1. Demonstrate different methods of irrigation, methods of application of water and irrigation procedure. 2. Design canals and canal network based on the water requirement of various crops and compute the reservoir capacity 3. Analysing and designing of Gravity dam 4. Finding cross section of earth dam and estimate the seepage loss 5. Designing spillways and apron for diversion work
Internal Assessment Marks:25marks (3SessionTests are conducted during the semester and marks allotted based on average of 2bestperformances) + 25 Marks Assignment	

Subject Code:	**BCV602**	**TITLE: Irrigation Engineering and Hydraulics**					**Faculty Name:**			**AKHILA C G**	
List of Course Outcomes	**Program Outcomes**										
	PO1	**PO2**	**PO3**	**PO4**	**PO5**	**PO6**	**PO7**	**PO8**	**PO9**	**PO10**	**PO11**	**Total**
CO-1	3	1	-	-	-	2	1	-	-	-	1	8
CO-2	3	1	-	-	-	2	1	-	-	-	1	8
CO-3	3	1	-	-	-	2	1	-	-	-	1	8
Total	12	3				6	3				3	**27**

Subject Code:	**BCV602**	**TITLE: Irrigation Engineering and Hydraulics**	**Faculty Name: AKHILA C G**
List of Course Outcomes	**Program Specific Outcomes**		
	PSO1	**PSO2**	**Total**
CO-1	1	-	1
CO-2	1	-	1
CO-3	1	-	1

CO-4	1	-	1
CO-5	1	-	1
Total	5	-	5

The Correlation of Course Outcomes (CO's) and Program Specific Outcomes (PSO's)

Module – 01
Irrigation

Structure

4.0 Introduction
4.1 Objectives
4.2 Definition. Benefits
4.3 Surface irrigation
4.4 Sub-surface irrigation
4.5 Sprinkler irrigation
4.6 Flow irrigation
4.7 Lift irrigation
4.8 Bandara irrigation
4.9 Duty, delta and base period
4.10 Factors affecting duty of water crops
4.11 Crop seasons in India
4.12 Irrigation efficiency
4.13 Frequency of irrigation
4.14 Recommended questions
4.15 Outcomes
4.16 Further Reading

4.0 Introduction

Irrigation may be defined as the process of artificially supplying water to the soil for raising crops. It is a science of planning and designing an efficient low cost irrigation system to suite the natural conditions. It is the engineering of controlling and harnessing the various natural sources of water by the construction of dams and reservoirs, canals and head works finally distributing the water to the agricultural fields. Irrigation engineering includes the study and design of works connected with river control, drainage of water logged areas and generations of hydroelectric power.

4.1 Objectives

- Demonstrate different methods of irrigation, methods of application of water and irrigation procedure.

Necessity of Irrigation

India is basically an agricultural country and its resources depend on the agricultural output. Prosperity of our country depends mainly upon proper development of agriculture. Even after 70 years of Independence, we have not succeeded in solving our food problems. The main reason for this miserable state of affair is that we still continue to remain at the mercy of rain and practice old age methods of cultivation.

Plants usually derive water from nature through rainfall. However, the total rainfall in a particular area may be either insufficient or ill timed. In order to get the maximum yield, it is necessary to have a systematic irrigation system for supplying optimum quantity of water at correct timing.

Importance of irrigation can be summarized under the following four aspects:

1. Area of less rainfall: Artificial supply of water is necessary when the total rainfall is less than the water requirement of crops in such cases, irrigation works may be constructed at a place where more water is available and conveyed to water deficit areas.

Eg: The Rajasthan canal supplies water from the river Yamuna to the arid regions of Rajasthan where annual rainfall is less than 100 to 200 mm.

2. Non-Uniform rainfall: The rainfall in a particular area may not be uniform over the entire crop period. Rainfall may be there during the early period of crops and may become scanty or unavailable at the end resulting in lesser yield or total loss of the crop. Collection of water during periods of excess rainfall and supplying the stored water during periods of scarcity may prove beneficial to the farmers. Most irrigation projects in India are based on this aspect.

3. Commercial crops with additional water: The rainfall in a particular area may be sufficient to raise the usual crops but insufficient for raising commercial and cash crops such as sugarcane and cotton. In such situations, utilizing stored water by irrigation facilities is advantageous.

4. Controlled Water Supply: Dams are normally meant for storing water during excess flow periods. But in situations of heavy rainfall, flooding can be controlled by arresting the flow in the river and excess water can be released during low flow conditions.

4.2 Definition: Irrigation is technique of supplying water to crops by artificial methods throughout the crop period for its complete growth.

4.2.1 Benefits of Irrigation:

There are many direct and indirect benefits or advantages of irrigation which can be listed as follows.

- **Increase in food production**: Crops need optimum quantity of water at required intervals assured and timely supply of water helps in achieving good yield and also superior crops can be grown and thus, the value of the crops increases.
- **Protection from famine:** Irrigation works can be constructed during famine (drought). This helps in employment generation and people also get protection from famine. After completion of such works, continuous water supply may be available for crops and people.
- **Cultivation of Cash crops:** With the availability of continuous water supply, cash crops such as sugarcane, indigo, tobacco, cotton etc. can be grown.
- **Increase in prosperity of people:** Due to assured water supply people can get good yield and returned for their crops. Land value increases and this raises the standard of living of the people and hence prosperity takes place.

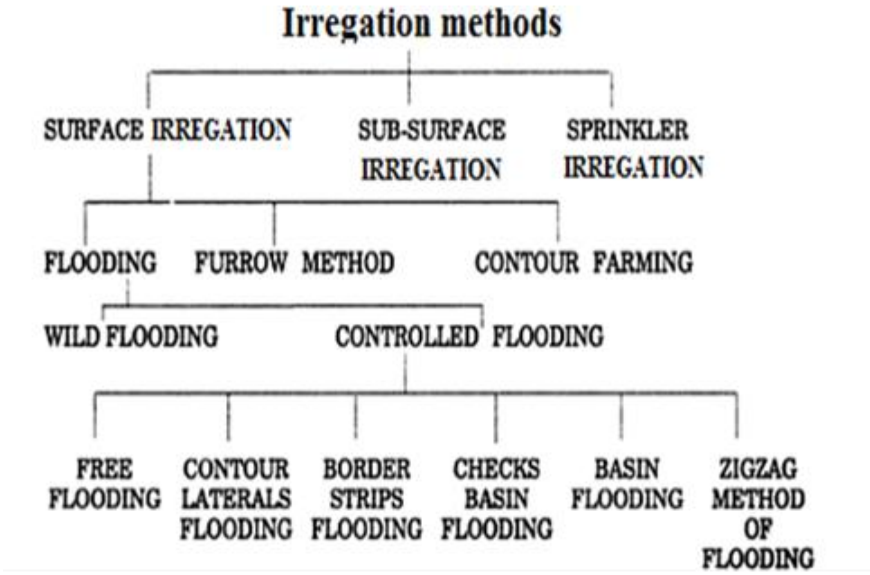
- **Generation of hydroelectric power:** Major River valley projects are designed to provide power generation facilities also apart from irrigation needs.
- **Domestic and Industrial water supply:** Water stored in reservoirs can also be used to serve other purposes like domestic water supply to towns and cities and also for industrial use. Canals can also be effectively used to serve these purposes.
- **Inland Navigation:** In some cases, the canals are very large enough to be used as channels for inland navigation as water ways are the cheapest means of transportation.
- **Improvement in communication:** Main canals in large irrigation projects are provided with inspection roads all along the sides. These roads can be asphalted and used as a means of communication.
- **Canal plantation:** Due to continuous flow of water adjoining areas of a canal are always saturated with water. In such places, trees can be planted which increases the timber wealth of the country.
- **Improvement in ground water storage:** Due to constant percolation and seepage of irrigation water, ground water table rises. The ground water may percolate and may be beneficial to other areas.
- **Aid in civilization:** Due to introduction of river valley projects, tribal people can adopt agriculture as their profession which helps in improving the standards of living.
- **General development of a country**: By assured water supply, farmers can expect good yield. By exporting surplus goods, Government can get revenue. The government can then come forward to improve communications facilities such as roads and railways and also social development by providing schools, hospitals etc.

4.2.2 Ill-Effects of Irrigation

If water is used in a controlled and careful manner, there would be no ill effects of irrigation. Excess and unscientific use of irrigation of water, givers raise to the following ill effects.

- **Water logging:** Excess water applied to the fields allows water to percolate below and ground water table rise. The ground water table may rise saturating the root zone of the crop and cutting of air supply to the roots of the crops. Such a phenomenon is called water logging. Under such conditions fertility of land reduced and also reduction of crop yield.
- **Breeding placed for mosquitoes:** Excess application of water for irrigation leads to water logging and formation of stagnant water fouls, which become breeding places for mosquitoes, thus helping spreading of malaria.
- **Unhealthy Climate:** Due to intense irrigation the climate becomes damp during summer due to humidity, the climate is sultry and in winter it becomes excessively cold. The resistance of the body to diseases is reduced. In addition to the above, careless use of water leads to wastage of useful irrigation water for which any government will have incurred huge amounts.

System of Irrigation:



Irrigation water may be applied to the crops by three basic methods, viz.
a. Surface irrigation methods
b. Subsurface irrigation methods
c. Sprinkler irrigation
Good irrigation methods result in increased yield, conservation of soil productivity and economic utilization of water. Over irrigation results in soil erosion, water logging, salt accumulation, nutrient leeching etc. The overall objective of an irrigation method is to see that the required amount of moisture is available in the root zone of the crops. The objectives or reasons for adopting any irrigation method for applying water to fields are as follows.

- For light irrigation uniform water distribution with a small depth of application, as small as cm should be possible
- For heavy irrigation uniform water depth application of 15 to 20 cm should be possible.
- Large concentrated flow should be possible for reducing conveyance losses and labour costs.
- Mechanical farming should be facilitated.

4.3 Surface irrigation method: In this method the irrigation water is applied by spreading water as a sheet or as a small stream on the land to be irrigated. Various surface irrigation methods that are practiced are listed as follows.

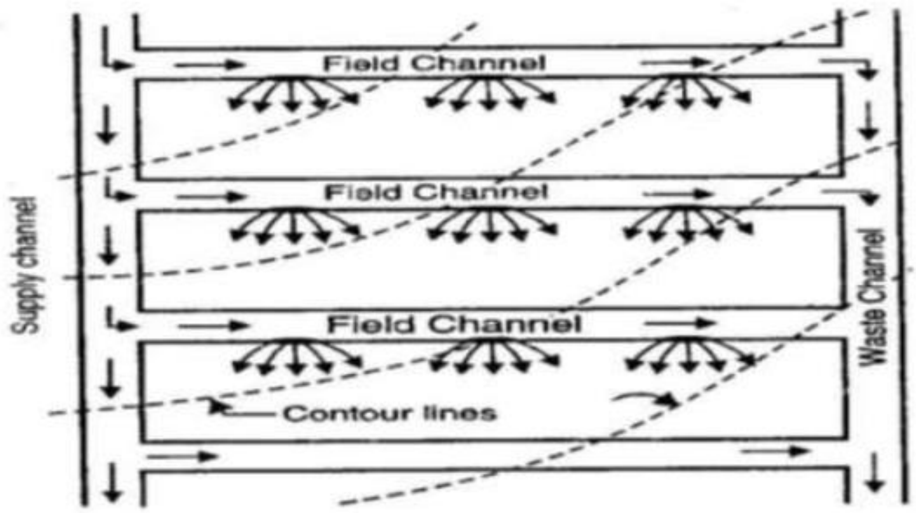
4.3.1 Flooding

4.3.1.1 Wild flooding: In this method water is applied by spreading water over the land to be irrigated without any preparation. There is no restriction for the movement of water. It follows the natural slope of the land. The water may be applied to the land directly from a natural stream during season of high flow as in inundation irrigation. This method is suitable for flat and smooth land but involves wastage of water and hence it can be practiced where water is abundant and inexpensive.

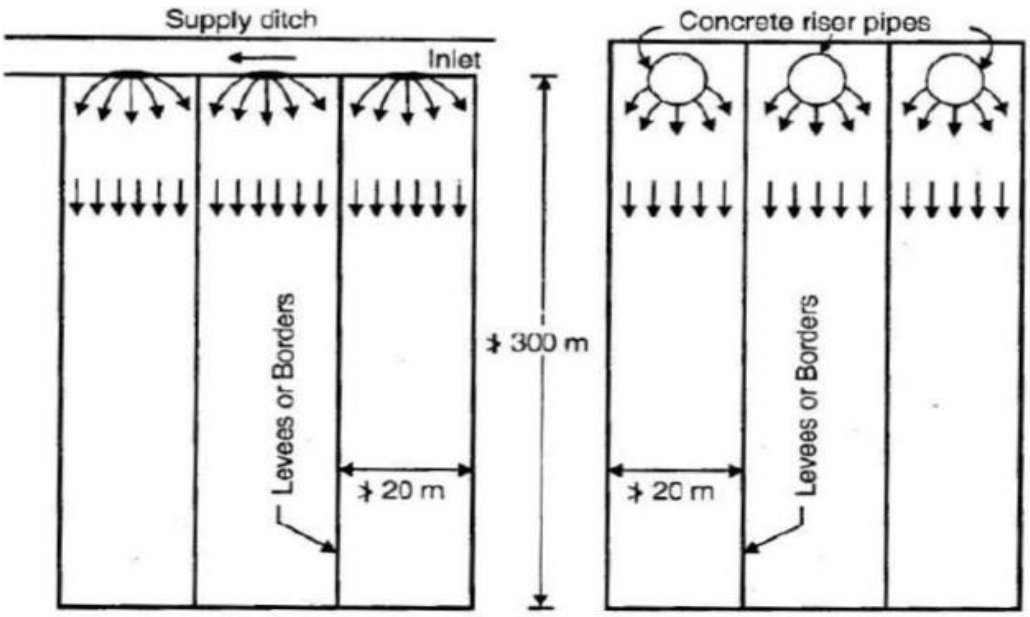
4.3.1.2 Controlled flooding: In this method water is applied by spreading it over the land to be irrigated with proper control over the flow of water and as well as the quantity of water to be applied. In such methods prior land preparation is essential.

Various controlled flooding methods are as follows.

i) Free flooding: This method is also known as irrigation by plots. Here the field is divided into a number of small sized plots which are practically level. Water is admitted at the higher end of the plots and the water supply is cut off as soon as the water reaches the lower end of the plots.



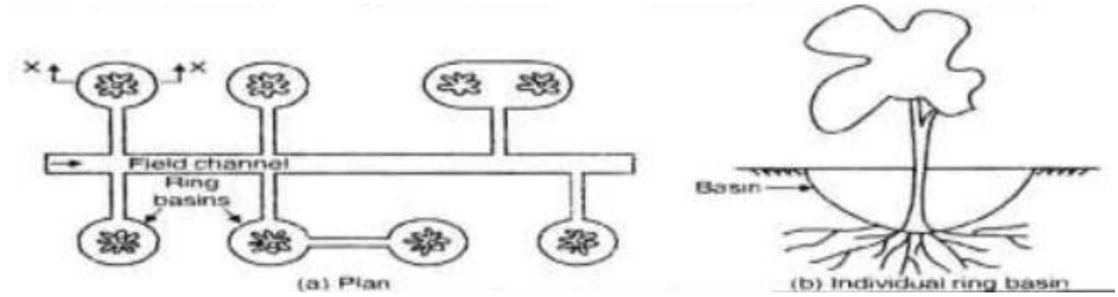
ii) Border strip method: In this method the land to be irrigated is divided into a series of long narrow strips separated from each other by levees (Earthen bunds) or borders. The width of the strips varies between 10 to 20 m and the length of the strip varies between 60 to 300 m depending upon the nature of the soil and rate of water supply.



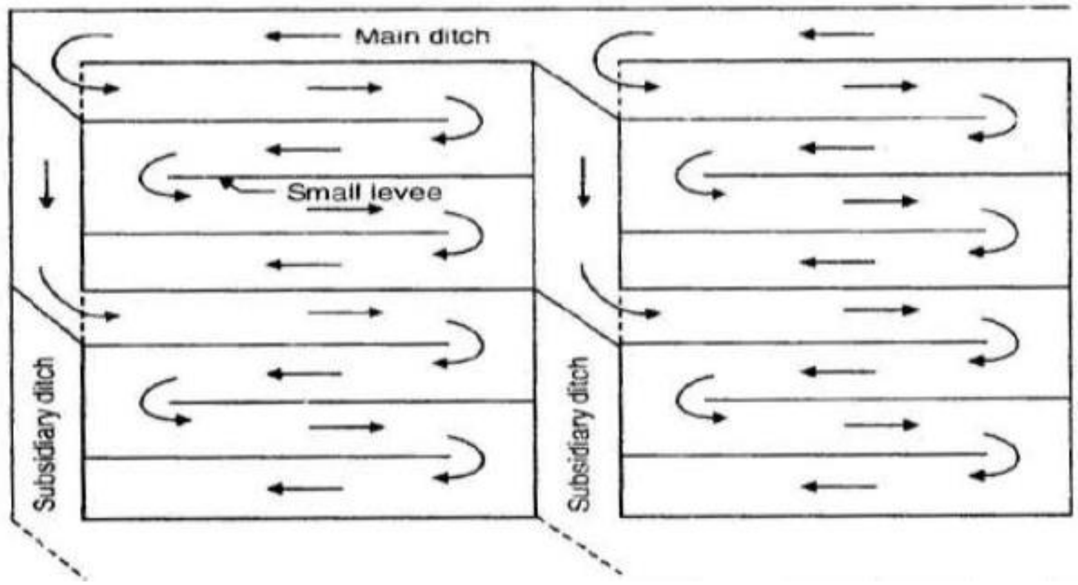
The strip of the land has no cross slope and has uniform gentle slope in the longitudinal direction. This method is suitable for forage crops requiring least labour. Mechanized farming can be adopted in this method.

iii) Checks or Levees: In this method a comparatively large stream of water discharged into a relatively level plot surrounded by check or levees or low rise bunds. The checks are usually 30 cm high. The checks may be temporary for a single crop season or semi permanent for repeated used as in case of paddy fields. The size of the plots depends upon the discharge of water and porosity of the soil. The usual size of the plot varies between 0.04 hectares to 0.05 hectares.

iv) Basin flooding: This method of irrigation is adopted for irrigating orchards (enclosures of fruit trees). For each tree, a separate basin which is circular usually is made. However, in some cases basins are made large to include two or more trees in each basin. Water is supplied through a separate field channel, but in some cases the basins are inter connected.

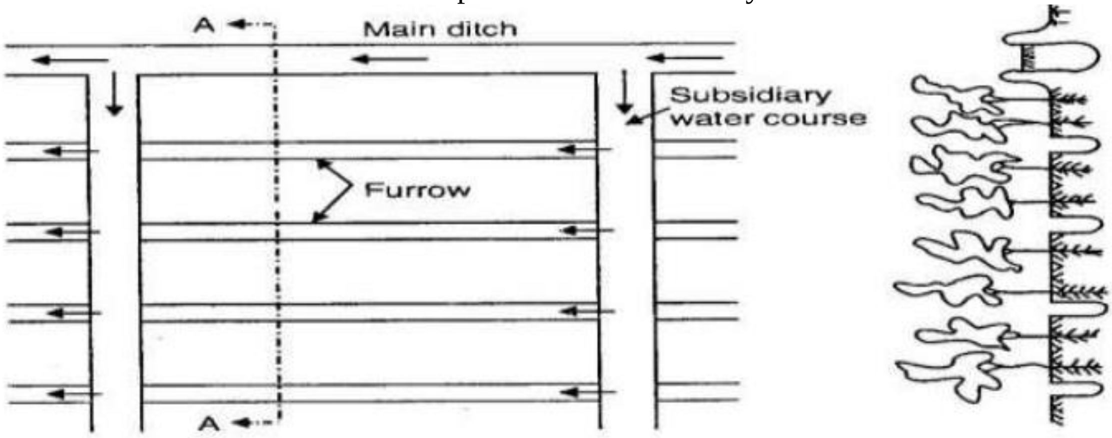


v) Zigzag flooding: This is a special method of flooding where the water takes a circuitous route before reaching the dead end of each plot. Each plot is subdivided with help of low bunds. This method is adopted in loose soils to prevent erosion at the higher ends.



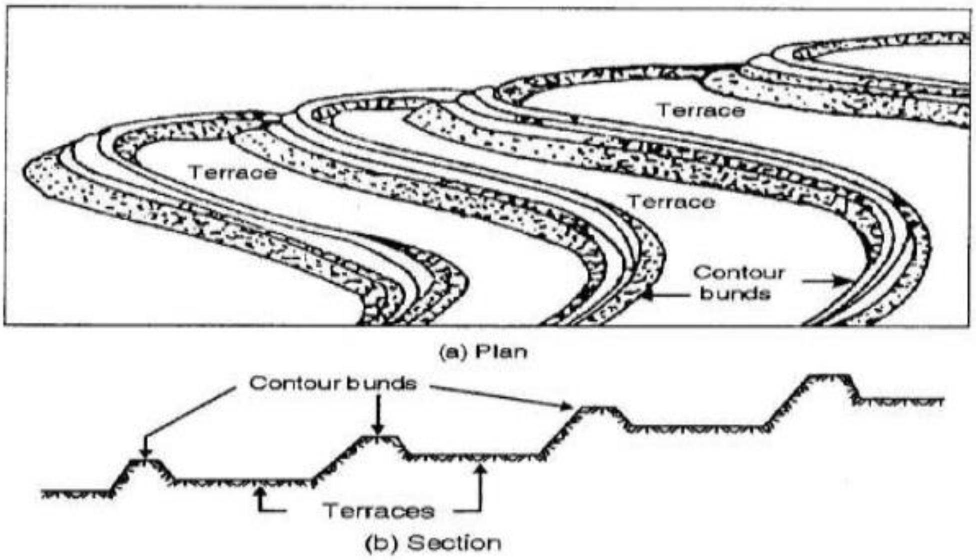
4.3.2 Furrow method:

In this method water is applied to the land to be irrigated by a series of long narrow field channels called furrows. A furrow is a narrow ditch 75 to 125 mm deep excavated between rows of plants to carry irrigation water. The spacing of furrows depends upon the spacing of the plants. The length of a furrow is usually 200metres. In this method only one fifth to one half of the surface is wetted. The evaporation losses are very much reduced.



4.3.3 Contour farming:

Contour farming is practiced in hilly regions where, the land to be irrigated has a steep slope. Here the land is divided into a series of strips usually known as terraces or benches which are aligned to follow the contour of the sloping area. This method also helps in controlling soil erosion.



4.4 Sub Surface Irrigation:

This method consists of supplying water directly to the route zone through ditches at a slow rate which are 0.5 m to 1 m deep and 25 to 50 cm wide. The ditches are spaced 50 to 100 m apart. Water seeps into the ground and is available to the crop in the form of a capillary fringe. Proper drainage of excess water is permitted either naturally or providing suitable drainage works, thereby preventing water logging in fields. The favourable conditions to practice subsurface conditions are,

- Availability of imperious subsoil at a reasonable depth (2 to 3m).
- Water table is present at shallow depth.
- Availability of moderate slope.

- Availability of good quality irrigation water.

With the above favourable conditions and necessary precautions, it is possible to achieve higher yields at low cost.

4.5 Sprinkler Irrigation: This method consists of applying water in the form of a fine spray as similar to rain fall. Stationary overhead perforated pipes or fixed nozzle pipes installations were earlier used. However, with the introduction of light weight pipes and quick couplers, portable sprinkler systems with rotating nozzle have been developed and hence these have become popular. A pump usually lifts water from the source and supplies it through the distribution system and then through the sprinkler nozzle or sprinkler head mounted on the riser pipes. About, 80 % irrigation efficiency is possible with sprinkler irrigation, particularly in semi-arid and humid regions. The efficiency of this system decreases by 5 % for every 7.5 km/hour of increase in wind velocity.
Irrigation efficiency (η) is given as,

$$\boldsymbol{\eta} = \left(\frac{w_s}{w_p} * \mathbf{100} \right)\%$$

Where
w_s - Represents amount of irrigation water stored in root zone
w_p - Amount of water pumped or supplied into the system.

Sprinkler irrigation method is adopted in regions where, surface irrigation methods cannot be used due to the following reasons.

- The soil is too pervious or impervious.
- The nature of the soil is too erosive.
- The topography is not uniform or very steep.
- The land is not suitable for surface irrigation method.

Advantages:

- Soil erosion is well controlled by adjusting the discharge through the nozzle.
- Uniform water application is possible.
- In case of seedlings and young plants, light irrigation is possible easily.
- Much land preparation is not essential and hence labour cost is reduced.
- More land for cropping is available since borders and ditches are not required.
- Small amounts of irrigation water in water scarcity regions can be effectively utilized.

Disadvantages:

- Wind will distort the sprinkling pattern.
- Constant water supply under pressure is required for economic use of equipment.
- Irrigation water must be free from silt, sand and impurities.
- Initial investment is high.
- Energy requirement for pumping water is high.
- Heavy soil with poor infiltration (clayey soil) cannot be irrigated efficiently.

Drip or trickle irrigation:

This is the latest irrigation method, which is becoming popular in water scarcity areas and water with salt problems. In this method, small diameter plastic or PVC pipes with drip nozzles commonly called emitters or drippers are adopted to deliver water to the land surface near the base of the plant. Water can be applied at a rate varying between 2 to 10 litres per hour to keep the soil moisture within the desired range for plant growth.

The main components of a drip irrigation system are a pumping unit, main pipelines, sub main pipe lines, lateral pipelines, emitters, pressure gauges etc.,

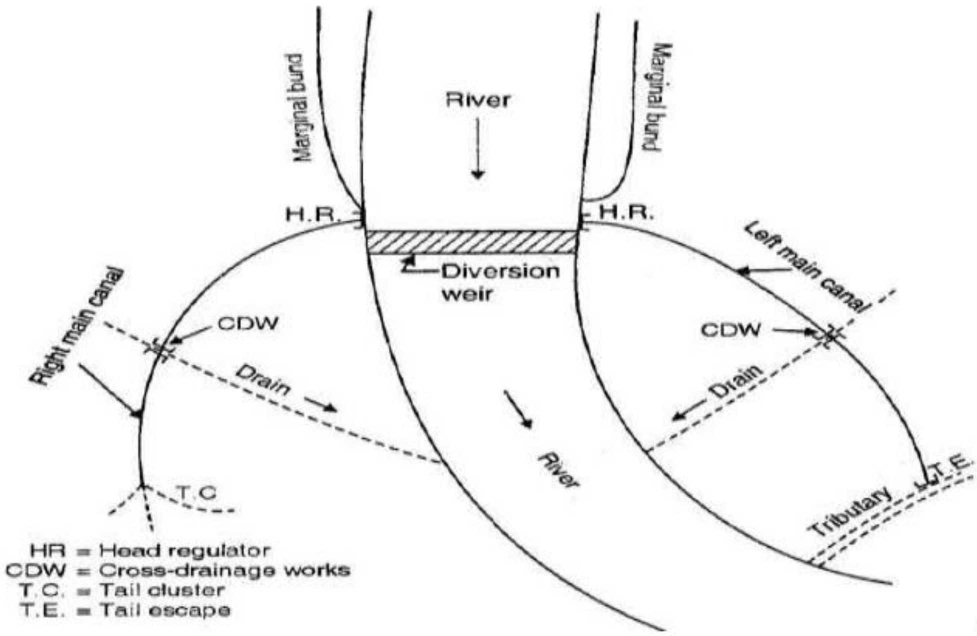
Advantages of this method are,

- Excellent control over water application and efficiency as high as up to 95% can be achieved.
- Evaporation losses from land surface are minimum.
- Losses due to deep percolation can be avoided.
- Saline waters can be applied effectively.
- Water soluble fertilizers can be applied through drip irrigation.

The draw backs of this system are,

- It involves large investment.
- Frequent blocking of nozzles takes place.
- Tilling operation may be obstructed.

4.6 Flow irrigation



Inundation Irrigation: It is that system of irrigation in which large quantity of water flowing in a river is allowed to flood or inundate the fields to be cultivated. The land becomes thoroughly saturated. Excess water is drained off and the land is prepared for cultivation. Moisture stored in the soil is sufficient to bring the crop to maturity. Inundation irrigation is commonly practiced in delta region of rivers. Canals may be also employed to inundate the fields when water is available in plenty.

Perennial Irrigation: It is that system of irrigation in which irrigation water is supplied as per the crop requirements at regular intervals throughout the crop period. The source of irrigation water may be a perennial river, stored water in reservoirs or ground water drawn from open wells or bore wells. This is the most commonly adopted irrigation system.

Direct Irrigation: It is a type of flow irrigation in which water from rivers and streams are conveyed directly to agricultural fields through a network of canals, without making any attempt to store water this is practiced in areas where the rivers and streams are perennial. Small diversion dams or barrages may be constructed areas the rivers to raise the water level and then divert the water into canals.

Storage Irrigation: Dams are constructed across rivers which are non- perennial. The discharge in such rivers may be very high during rainy season and may become less during dry stream. By constructing dams across such rivers water can be stored as reservoir during excess flow and can be utilized or diverted to agriculture fields through canals as and when required. Such a system is known as storage irrigation.

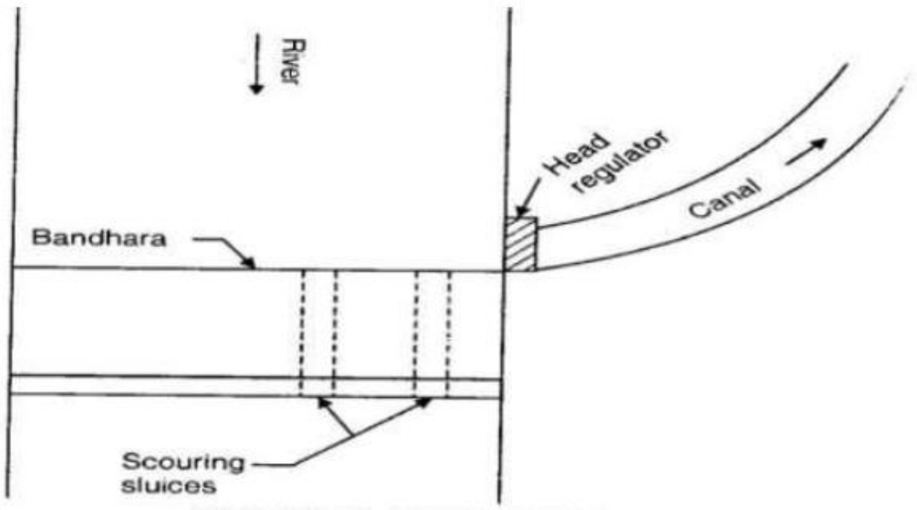
4.7 Lift irrigation

It is that system of irrigation in which irrigation water is available at a level lower than that of the land to be irrigated and hence water is lifted by pumps or other mechanism (Hydraulic ram and siphon action) and then conveyed to agriculture fields by gravity flow. Irrigation through wells is an example of lift irrigation. Water from canals or any other source can also be lifted when the level of water is lower than that of the area to be irrigated.

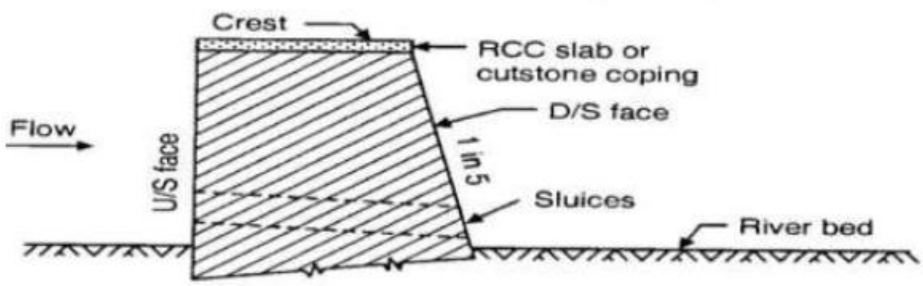
4.8 Bandhara irrigation

It is a special irrigation scheme adopted across small perennial rivers. This system lies somewhere between inundation type and permanent type of irrigation. A Bandhara is a low masonry weir (obstruction) of height 1.2m to 4.5m constructed across the stream to divert water into a small canal. The canal usually takes off from one side and the flow into the canal is controlled by a head regulator.

The length of the main canal is usually restricted to about 8km. A series of Bandharas may be constructed one below the other on the same stream so that water spilling over from one Bandhara is checked by another Bandhara. The irrigation capacity of each Bandhara is may be about 400 hectares. Bandharas are adopted across small streams having isolated catchments which are considered to be non feasible or uneconomical to be included under a large irrigation scheme.



(a) Bandhara irrigation layout



This method of irrigation is followed in Central Maharashtra and is commonly known there as the ‘Phad’ system.

Advantages of Bandharas:

- Small quantity of flow in streams can be fully utilized or otherwise might have gone as a waste.
- As the length of the canal is short, seepage and evaporation losses are less.

- Intensive irrigation with high duty may be achieved and the area to be irrigated is close to the source
- The initial investment and maintenance cost of the system is low.

Disadvantages of Bandharas:

- The supply of water is unreliable when the flow in streams becomes lesser.
- Excess water available cannot be utilized as area for cultivation below each Bandhara is fixed.
- In dry seasons, people living on the downstream side of Bandharas may be deprived of water for domestic made also.

Water Requirements of Crops

Soil groups in India

The Indian soils may be divided into four major groups' viz., (i) Alluvial soils, (ii) Black soils, (iii) Red soils, and (iv) Laterite soils. In addition to these four groups there exist another group of soils which includes forest soils, desert soils, and saline and alkali soils.

Alluvial soil

Alluvial soils are formed by successive deposition of silt transported by rivers during floods, in the flood plains and along the coastal belts. The silt is formed from the weathering of the rocks by river water in the hilly terrain through which it flows. These soils form the largest and the most important group of soil in India. The alluvial soils occur in the Indo-Gangetic plains and Brahmaputra plains in north India and also in the plains of various rivers in other parts of the country. These are in general deep soils that is having more than 1 metre depth above a hard stratum, but the properties of these soils occurring in different parts of the country vary mainly because the parent material from which they have been derived are different. These soils vary from clayey loam to sandy loam. The values of pH for these soils usual range between 7.0 and 9.0, and hence these soils may be neutral or alkaline in character. The water holding capacity of these soils is fairly good and they give good response to irrigation.

Black Soil

Black soils have evolved from the weathering of rocks such as basalts, traps, granites and gneisses. These soils occur chiefly in the states of Andhra Pradesh, Gujarat, Madhya Pradesh, Tamil Nadu, Maharashtra and Karnataka. The colour of these soils ranges from dark brown to black. Further irrespective of the nature of the parent rock from which black soils have developed, they do not differ many in general physical and chemical properties. These soils are highly argillaceous and very fine grained. Thus these are heavy textured soils and their clay content varies from 40 to 60 percent. The values of pH for these soils vary from 8.0 to 9.0 or higher in different states, and hence these soils are alkaline in character. A special feature of the black soils is that they are plastic and sticky when wet, a very hard when dry.

These soils possess a high water holding capacity but poor drainage. Black soils ai sub-divided as (i) shallow black soils which have a depth of 0.3 metre or less, (ii) medium black soil which is 0.3 metre to 1.0 metre in depth, and (iii) deep soils which are over 1 metre deep. Deep black soils are also referred to as black cotton soils since cotton is the most important crop in these soils.

Red Soil

Red soils are formed by the weathering of igneous and metamorphic rocks comprising gneisses and schists. These soils mostly occur in Tamil Nadu, Karnataka, Maharashtra, Andhra Pradesh, Madhya Pradesh and Orissa. They also occur in Bihar, West Bengal and some parts of Uttar Pradesh. These soils are in general light textured loams, but the properties of these soils vary from place to place. These are in general deep soils with values of pH ranging between 5.0 and 8.0, and hence in most of the cases these soils are acidic in character. The red soils have low water holding capacity. These soils react well to the application of irrigation water and on account of low water holding capacity they are well drained.

Laterite Soil

Laterite soils are derived from the weathering of the laterite rocks. These soils occur mostly in Karnataka, Kerala, Madhya Pradesh, the Eastern Ghat region, Orissa, Maharashtra, and Malabar and in some parts of Assam. These soils are reddish or yellowish-red in colour. The laterite soils have values of pH between 5.0 and 6.0 and hence these soils are acidic in character. These soils have low clay content and hence possess good drainage characteristics.

Forest Soil

Forest soils are formed by the deposition of organic matter derived from forest growth.

Desert Soil

These soils are found in the arid areas in the north-western region in the states of Rajasthan, Haryana and Punjab and are lying between the Indus river on the west and the range of Aravali Hills on the east. These soils are blown in from the coastal region and Indus valley, and are also derived from disintegration of rocks in the adjacent areas. These are light textured sandy soils of depth extending beyond 0.5 metre and react well to the application of irrigation water. However, these soils have fairly high values of pH, and some of these soils contain a high percentage of soluble salts.

Saline and Alkali Soil

These are the soils which have appreciable concentration of soluble salts and exchangeable sodium content. These salts usually appear in the form of a white efflorescent crust on the surface of the soil. These soils are formed due to inadequate drainage of the irrigated lands, and hence these are found among the groups of alluvial, black and red soils. These soils are not suitable for cultivation, unless these are reclaimed by adopting suitable methods.

4.9 Duty, delta and base period

Duty of Water: Duty represents the irrigating capacity of a unit of water.
It is usually defined as the area of land in hectares which can be irrigated to grow a crop of one cumec of water is continuously supplied for the entire period of the crop.
Example: If 5100 hectares of land can be irrigated for growing a crop with a available discharge of 3 cumec continuously for the entire crop period, then the duty of water for this crop = 5100/3 = 1700 hectares/cumec.
Different crops require different amounts of water before their harvesting and hence duty of water varies with the crops. Duty of water is said to be high, if the area of land irrigated per cumec is large.

Delta: It is the total depth of water required by a crop during the entire crop period and is denoted as 'Δ'
Example: A crop require 12 watering at an interval of 10 days and depth of water required in each watering is 10cms, the delta for the crop is 12x10cms=120cms=1.2m
If the crop is grown in an area of 'A' hectares, then the total quantity of water required is = 1.2 x A hectares-meter in a period of 120 days.

Base period: It refers to the whole period of cultivation from the time when irrigation water is first applied for preparation of the ground for planting the crop to its last watering before harvesting.

Crop Period: It is the time in days that a crop takes from the instant of its sowing to that of its harvesting.

Gross command area: It is the total area laying between the drainage boundaries which can be commanded or irrigated by a canal system.

Culturable command area: Gross command area may also contain villages, ponds, barrel lands, alkaline lands etc., and such areas are turned as unculturable area. The remaining area on which crops can be grown satisfactory is known as culturable command area.

4.9.1 Relationship between Duty and Delta:

Let D = Duty of water in hectares/cumec
Δ = Total depth of water required during base period in 'm'
B = Base period in days
i. If we take a field of area D hectares, water supplied to the field corresponding to the water depth Δ metres will be = Δ x D hectares-metres = D x Δ x10^4 cubic-metres. .... (1)
ii. Again for the same field of D hectares, one cumec of water is required to flow during the entire base period. Hence, water supplied to this field. = (1) x (B x 24 x 60 x 60) m^3.... (2)
Equating Equations (1) and (2), we get D x Δ x 10^4 = B x 24 x 60 x 60

$$\boldsymbol{\Delta = 8.64 * \frac{B}{D}}$$

1 hectare = 104 sq metres

Cumec-day = 8.64 hectare-metres

Consider a field of 'D' hectare to be irrigated and Δ is the corresponding depth of water.

The total volume of water required to be supplied for the field of 'D' hectares, if cumec is to be supplied during the entire base period 'B'.

Problems 1:

Find the delta for a crop if the duty for a base period of 110 days is 1400 hectares/cumec.

Solution:

$\Delta = 8.64B/D$

$\Delta = 8.64x\ 110/1400 = 0.68\text{m} = 68\text{cm}$

Problems 2:

A crop requires a total depth of 9.2cm of water for a base period of 120 days. Find the duty of water.

Solution:

$D = 8.64B/\Delta$

$D = 8.64\ x\ 120/0.092 = 1300$ hectare/ cumec

4.10 Factors affecting duty of water crops

Methods and systems of irrigation: Perennial system of irrigation has more duty of water than inundation irrigation system the loss of water by deep percolation is minimum in the first case. In flow irrigation by channels the duty is less as conveyance losses are more. In lift irrigation the lands to be irrigated are very near to the source of water than any surface irrigation method.

Type of Crop: Different crops require varying quantities of water and therefore duty of water varies from crop to crop. Crops requiring large quantity of water have lower duty than crops requiring lesser quantity of water.

Climate conditions of the area: The climatic condition such as wind, temperature, humidity and rainfall affect the duty of water. At high temperature losses due to evaporation and transpiration are more and hence duty decreases. At higher wind velocity, rate of evaporation and transpiration are more thereby, duty decreased. But in humid conditions evaporations and transpiration losses are minimum, there by duty increases.

Canal conditions: In earthen canals, seepage losses are high resulting low duty. If canal is lined, losses are minimum and hence duty increases. If the length of the canal is very large before it reaches the irrigation fields (as in hilly areas) the duty of water decreases.

Quality of Water: If water contains harmful salts and alkali contents, then more water is to be applied liberally to leach out these salts and in turn duty of water decreases.

Characteristics of soil and subsoil in field and canals: If the soil and subsoil of the field and canals are made of coarse-grained soils the seepage and percolation losses are more and hence the duty of water decreases.

Topography of land: If the area to be irrigated is level, uniform water application is possible which will result in economic views and hence duty of water increases.

Method of Cultivation: If the land is properly tilled up to the required depth and soil is made loose before irrigation, water retaining capacity of soil increases. This reduces the number of watering or frequency of watering and hence duty increases.

4.11 Crop seasons of India:

Sowing of crops in irrigation of crops in India is usually done in 2 seasons, known as Crop seasons. They are

1. Kharif season
2. Rabi season

Kharif season begins with the onset of south west monsoons sowing of crops in Kharif season is done during June-July and these crops are harvested in October-November.

In Rabi season the crops are sown during September-October and harvested during march-April.

Sugarcane has a delta of about 90cms but has a period of 1 year or more. It is as such termed as a perennial crop.

Example:

1. The gross commanded area for a distributory is 20000 hectares, 75% of which can be irrigated. The intensity of irrigation for Rabi season is 40% that for Kharif season is 10%. If kor period is 4 weeks for rabi and 2.5 weeks for rice, determine the outlet discharge. Outlet factors for rabi and rice may be assumed as 1800 hectares/ cumec and 775 hectares/ cumec. Also calculate delta for each crop.

Solution:

Gross commanded area = 20000 hectares

Culturable commanded area = 0.75 x 20000 = 15000 hectares.

Area under irrigation in Rabi season at 40% intensity = 15000 x 0.4 = 6000 hectares

Area under irrigation in Kharif season at 10% intensity = 15000 x 0.1 = 1500 hectares.

Outlet Discharge for Rabi = 6000/1800 = 3.33 cumec

Outlet Discharge for Kharif = 1500/775 = 1.94 cumec

From the equation

Similarly, for rabi D = $8.64B/\Delta = 8.64(4x7)/1800 = 0.134m = 134mm$

Similarly for rice D = $8.64B/\Delta = 8.64(2.5x7)/775 = 0.195m = 195mm$

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

Solutions:

(A) For Crop A

Area under irrigation =1200 x 0.4 = 480 hectares

Kor period = B = 20days

Kor depth = D = 10cm = 0.1m

Duty = $\Delta = 8.64B/D$ = 8.64 x 20/0.1 = 1728 hectares/cumec

Hence discharge required = Area under irrigation/ outlet factor = 480/1728 = 0.278 cumec

(B) For Crop B
Area under irrigation =1200 x 0.35 = 420 hectares
Kor period = B = 15 days
Kor Depth = D = 16cm = 0.16m
Duty = Δ = $8.64B/D$ = 8.64 x15/0.16 = 810 hectares/cumec
Hence discharge required = Area under irrigation/ outlet factor = 420/810 = 0.519 cumec
Thus, the design discharge of water course = 0.278 +0.519 = 0.8 cumec

3. A water course commands an irrigated area of 1000 hectares. The intensity of irrigation for rice in this area is 70 %. The transplantation of rice crop takes 15days and during the transplantation period the total depth of water required by the crop on the field is 500mm. During the transplantation period, the useful rain falling on the field is 120mm. Find the duty of irrigation water for the crop on the field during transplantation, at the head of the field is and also at the head of the water course assuming losses of water to be 20% in the water course. Also calculate the discharge required in the water course.
Solutions:
Area under irrigation =1000 x 0.7 = 700 hectares
Depth of water required on the field during transplantation = 500mm
Useful rainfall during this period= 120mm
Depth of water required to be supplied by the water course= 500 - 120 = 380mm = 0.38m
Duty = Δ = $8.64B/D$
Duty of water on the field is = Duty = Δ = $8.64x$ 15/0.38 = 341 hectares/ cumec
Since the losses of water in the water course are 20%, a discharge of 1 cumec at the head of the water course will be reduced to 0.8cumec at the head of the field and hence will irrigate. = 341x 0.8 = 272.8 hectares.
Duty of water at the head of the water course = 272.8 hectares.
Discharge at the head of water course = 700 / 272.8 = 2.57 cumecs

4.12 Irrigation efficiency

Efficient use of irrigation water is an obligation of each user as well as of the planners even under the best method of irrigation, not all the water applied during irrigation & is stored in the root zone. In general, efficiency is the ratio of water output to the water input and is expressed as percentage. The objective of efficiency concepts is to show when improvements can be made which will result in more efficient irrigation. The following are the various types of irrigation efficiencies: (i) Water conveyance efficiency, (ii) Water application efficiency, (iii) Water use efficiency, (iv) Water storage efficiency, (v) Water distribution efficiency and.

4.12.1 Water Conveyance Efficiency (E_c): Water conveyance efficiency may be defined as the percentage ratio of the amount of water delivered to fields or farms to the amount of water diverted from sources. It is expressed as:

$$\mathbf{E_c = \frac{W_f}{W_r} * 100}$$

E_c = Water conveyance efficiency in percent

W_f = Amount of water delivered to the farm or irrigation plot
W_r = Amount of water diverted from sources

4.12.2 Water Application Efficiency (E_a): The water application efficiency is the percentage ratio of the amount of water stored in the crop root zone to the amount of water delivered to the field. It is expressed as:

$$\mathbf{E_a = \frac{W_s}{W_f} * 100}$$

E_a = Water application efficiency in percent
W_s= Amount of water stored in the crop root zone soil
W_f= Amount of water delivered to the farm.
The common sources of loss of irrigation water during water application are

(i) Surface run off R_f from the farm and
(ii) Deep percolation D_f below the farm root-zone soil. Hence

$$\mathbf{W_f = W_s + R_f + D_f}$$

In a well-designed surface irrigation system, the water application efficiency should be at least 60%. In the sprinkler irrigation system this efficiency is about 76%.

4.12.3 Water Use Efficiency (E_u): Water use efficiency is determined to evaluate the benefit of applied water through crop production. It is very important in crop production and irrigation water management. It is described in the following two ways.
1.12.3.1 Field water use efficiency: This may be defined as the ratio of amount of economic crop yield to the amount of water required for crop growing. It is expressed as:

$$\mathbf{E_u = \frac{Y}{WR}}$$

E_u = Field water use efficiency expressed in kilogram of economic yield per hectare-cm or hectare-mm of water
Y = Economic crop yield in kilogram per hectare
WR = Water requirement of the crop in hectare-cm or hectare-mm

4.12.3.2 Crop water use efficiency: This may be defined as the ratio of amount of economic yield of a crop to the amount of water consumptively used by the crop. It is expressed as:

$$\mathbf{E_{CU}\ (or\ WUE) = \frac{Y}{CU\ or\ ET}}$$

E_{CU} = Crop water use efficiency expressed in kilogram of economic yield per hectare-cm or hectare-mm of water
WUE = Water use efficiency of crop in kilogram of economic yield per hectare-cm or hectare-mm of water
Y = Economic crop yield in kilogram per hectare
CU = Consumptive use of water in hectare-cm or hectare-mm
ET = Evapotranspiration in hectare-cm or hectare-mm

4.12.4 Water Storage Efficiency (E_s): Water storage efficiency refers to the percentage ration of the amount of water stored in effective root zone soil to the amount of water needed to make up the soil water depleted in crop root zone prior to irrigation. It is expressed as:

$$\mathbf{E_s = \frac{W_s}{W_e} * 100}$$

E_s = Water storage efficiency in percent
W_s = Amount of water actually stored in root zone soil from the water applied
W_e = Amount of water needed to meet the soil water depleted in the crop root zone soil prior to irrigation = (field capacity - Available moisture).

4.12.5 Water Distribution Efficiency (E_d): Water distribution efficiency measure the extent to which water is uniformly distributed and stored in the effective root zone soil along the irrigation run. It is expressed as:

$$\mathbf{E_d = 100\left(1 - \frac{y}{d}\right)}$$

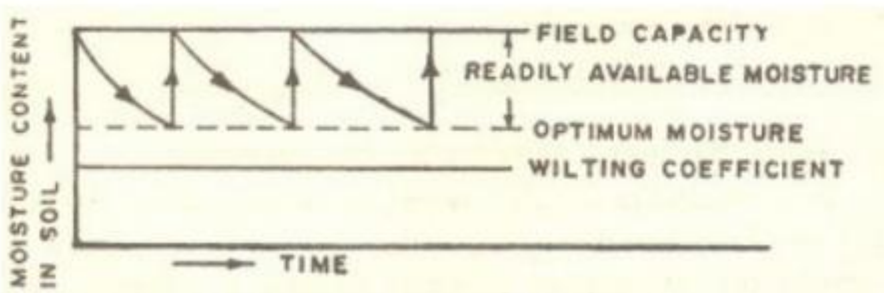
E_d = Water distribution efficiency in percent
y = Average numerical deviation in depth of water stored in root zone soil along the irrigation run from the average depth of water stored during irrigation.
d = Average depth of water stored during irrigation.

4.13 Frequency of irrigation:

The interval that can be safely allowed between two successive irrigations is known as frequency of irrigation.

$$\text{Irrigation interval} = \frac{\text{Allowable soil moisture depletion}}{\text{Daily water use}}$$

The amount of irrigation water applied should be such that the moisture content is raised to the field capacity. The moisture content in soil reduces due to consumptive use by plants. However, the moisture content should not be allowed to fall below lower limit of readily available moisture. When the moisture content reaches the lower limit of readily available moisture, water should be supplied by irrigation method to rise it to the field capacity or optimum moisture content.



The minimum depth of water to be applied during irrigation to maintain field capacity is given by,

$$\mathbf{D_w = \frac{D_s}{w} * d}$$

The frequency of irrigation is given by,

$$\mathbf{F_w = D_w/C_u \text{ (days)}}$$

Where Cu represents the consumptive use of water by crops expressed as depth of water in cm/day

4.14 Recommended Questions

1. Define irrigation? What is the necessity of irrigation?
2. Discuss in brief the benefit and ill effects of irrigation.
3. With a neat sketch explain Bhandhara irrigation scheme.
4. Explain irrigation efficiencies.
5. Define duty? What are the factors affecting duty of water? Explain.
6. Explain consumptive use of water. List the factors affecting consumptive use of water.
7. Explain irrigation requirements of crops.
8. Explain the following:

(a) Base period
(b) Crop period
(c) Time factor
(d) Gross command area
(e) Culturable command area

4.15 Outcomes

1. Find the benefits and ill-effects of irrigation.
2. Find the quantity of irrigation water and frequency of irrigation for various crops

4.16 Further Reading

1. https://pubs.usgs.gov/circ/circ1139/pdf/circ1139.pdf
2. http://www.fao.org/3/r4082e/r4082e06.htm
3. http://ecoursesonline.iasri.res.in/mod/page/view.php?id=124852
4. http://www.fao.org/3/a-s8376e.pdf

Module -02
Canals

Structure

5.0 Introduction
5.1 Objectives
5.2 Types of canals
5.3 Alignment of canals
5.4 Definitions
5.5 Canal lining
5.6 Design of Canal by Lacey's Theory
5.7 Design of canals by Kennedy's method
5.8 Investigations for Reservoir site
5.9 Storage Zones of a Reservoir
5.10 Economical height of dam:
5.11 Recommended questions
5.12 Outcomes
5.13 Further Reading

5.0 Introduction:

A canal is a manmade waterway that allows boats and ships to pass from one body of water to another. Canals are also used to transport water for irrigation and other human uses. While the advent of more efficient forms of transportation has reduced the need for canals, they still play a vital role as conduits for transportation and fostering global commerce.

5.1 Objectives

1. Design canals and canal network based on the water requirement of various crops.
2. Determine the reservoir capacity.

5.2 Classification of Canals:

The irrigation canals can be classified in different ways based on the following considerations.

5.2.1 Classification based on the nature of source of supply: In this method canals may be classified as

a) Permanent canals: A permanent canal is one which draws water from a permanent source of supply. The canal in such cases is made as a regular graded canal (fixed slope). It is provided with permanent regulation and distribution works. A permanent canal may also be perennial canal or non-perennial canal depending on whether the source supplying water is a perennial one or a non-perennial.

b) Inundation canals: An inundation canal is one which draws water from a river when the water level in the river is high or the river is in floods. These canals are not provided with any regulatory works, but an open cut is made in the banks of the canal to divert water.

5.2.2 Classification based on the function of the canal: Here the canals may be classified as

a) Feeder canals: A feeder canal is constructed for the purpose of supplying water to two or more canals only but not directly irrigating the fields.

b) Carrier canals: A carrier canal carries water for irrigating the fields and also feeds other canals for their needs.

c) Navigation canals: A canal serving the purpose of in-land navigation is called a navigation canal.

d) Power canals: A power canal supplies water to a hydroelectric power generation plant for generation of electrical power.

5.2.3 Classification based on the discharge and its relative importance in a given network of canals: On this basis an irrigation canal system consists of

a) Main canal: A main canal is the principal canal in a network of irrigation canals. It directly takes off from a river, reservoir or a feeder canal. It has large capacity and supplies water to branch canals and even to major distributaries.

b) Branch canal: Branch canals take off from a main canal on either side at regular intervals. They carry a discharge of about 5 cumec and are not usually used to directly irrigate the fields.

c) Major distributory: A major distributory takes off a branch canal or a main canal. It has a discharge capacity of 0.25 to 5 cumec. They are used for direct irrigation and also to feed minor distributaries.

d) Minor distributory: Minor distributaries are canals taking off from the branch canals and major distributaries. They carry a discharge less than 0.25 cumec. These canals supply water to field channels.

e) Water course or Field channel: A water course or field channel takes off from either a major or minor distributory or a branch canal also. These are constructed and maintained by the cultivators/farmers. The other canals are constructed and maintained by the government or the Command Area Development Authority.

5.2.4 Classification based on Canal alignment: On the basis of canal alignment, the canals are classified as

a) Ridge canal or watershed canal: A Ridge canal or watershed canal is one which runs along the ridge or watershed line. It can irrigate the fields on both sides. In case of ridge canals the necessity of cross drainage works does not arise as the canal is not intercepted by natural streams or drains.

b) Contour canal: A contour canal is one which is aligned nearly parallel to the contours of the country/area. These canals can irrigate the lands on only one side. The ground level on one side is higher and hence bank on the higher side may not be necessary. A contour canal may be intercepted by natural streams/drains and hence cross drainage works may be essential.

c) Side slope canal: A Side slope canal is one which is aligned at right angles to the contour of the country/area. It is a canal running between a ridge and a valley. This canal is not intercepted by streams and hence no cross drainage works may be essential. This canal has steep bed slope since the ground has steep slope in a direction perpendicular to the contours of the country/area.

5.2.5 Classification based on the financial output: On the basis of the financial output /revenue from the canals, the canals are called as

a) Productive canals: A productive canal is one which is fully developed and earns enough revenue for its running and maintenance and also recovers the cost of its initial investment. It is essential the cost of its initial investment is recovered within 16 years of construction.

b) Protective canals: Protective canals are those constructed at times of famine to provide relief and employment to the people of the area. The revenue from such a canal may not be sufficient for its maintenance. The investment may also not be recovered within the stipulated time.

5.2.6 Classification based on the soil through which they are constructed: On the above basis the canals are classified as

a) Alluvial canals: Canals constructed in alluvial soils are known as alluvial canals. Alluvial soils are found in the Indo-Gangetic plains of North India. The alluvial soils can be easily scoured and deposited by water.

b) Non-alluvial canals: Canals constructed through hard soils or disintegrated rocks are called non-alluvial canals. Such soils are usually found in Central and South India.

5.2.7 Classification based on lining being provided or not: On the above basis the canals are classified as

a) Unlined canals: An unlined canal is one which the bed and banks of the canal are made up of natural soil through which it is constructed. A protective lining of impervious material is not provided. The velocity of flow is kept low such that bed and banks are not scoured.

b) Lined canals: A lined canal is one which is provide with a lining of impervious material on its banks and beds, to prevent the seepage of water and also scouring of banks and bed. Higher velocity for water can be permitted in lined canals and hence cross sectional area can be reduced.

5.3 Canal Alignment

A canal has to be aligned in such a way that it covers the entire area proposed to be irrigated, with shortest possible length and at the same time its cost including cost of drainage works is minimum. A shorter length of canal ensures less loss of head due to friction and smaller loss of discharge due to seepage and evaporation, so that additional area may be brought under cultivation. A canal may be aligned as a contour canal, a side slope canal or a ridge canal according to the type of terrain and culturable area.

Irrigation canals can be aligned in any of the three ways:

(i) Watershed Canal

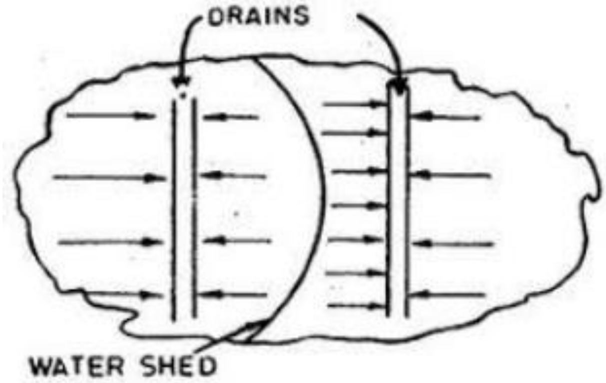
(ii) Contour Canal

(iii) Side slope Canal

Watershed Canal: The dividing line between the catchment area of two drains or streams is called watershed. Thus between two major streams, there is the main watershed which divides the drainage areas of the two. Similarly, between any tributary and the main stream and also between any two tributaries there are subsidiary watersheds, dividing the drainage between the two streams on either side.

For canal system in plain areas, it is often necessary as well as advantageous to align all channels on the watersheds of the areas, they are designed to irrigate. The canal which is aligned along any natural watershed is called a watershed canal. In such a canal, water flows by gravity, either side of the canal, directly or through small irrigation channels.

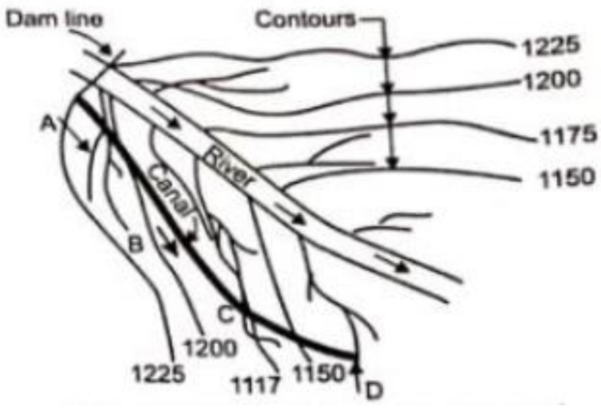
Moreover, cross drainage works avoided as the natural drainage will never cross a watershed, because all the drainage flows away from the watershed. Sometimes watershed may have to abandon in order to bypass localities settled on the watershed.



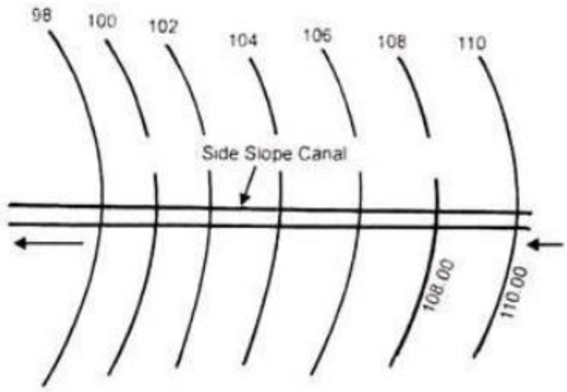
Contour Canal: The above arrangement of providing the watershed is not possible in hilly areas. In the hills, the river flows in the valley, while the watershed or the ridge line may be hundreds of meters above it. It becomes uneconomical to take the canal on top of such ridge. The canal in such cases is generally aligned parallel to the contours of the area except that the longitudinal slopes required to generate sufficient flow velocity, are given to it.

The maximum designed slope that can be provided in the canal without generating excessive velocity, is generally less the available country slope. The difference is accommodated by providing canal falls at suitable places. A contour channel irrigates only on one side because the areas on the other side is higher.

As the drainage flow is at right angle to the ground contours, such a channel would definitely have to cross drainage lines. Suitable cross drainage works are then provided.



Side slope Canal: A side slope channel is that which is aligned at right angles to the contours. i.e. along the side slopes. Such a channel is parallel to the natural drainage flow and hence, doesn't intercept cross drainage and no cross drainage works are required.



In aligning an irrigation canal, the following points must be considered.

1. An irrigation canal should be aligned in such a way that maximum area is irrigated with least length of canal.
2. Cross drainage works should be avoided as far as possible, such that the cost is reduced.
3. The off taking point of the canal from the source should be on a ridge, such that the canal must run as a ridge canal and irrigate lands on both sides.
4. Sharp curves in canals must be avoided.
5. In hilly areas, when it is not possible to construct ridge canals, the canal must be made to run as a contour canal.
6. The canal should be aligned such that the idle length of the canal is minimum.
7. The alignment should be such that heavy cutting or heavy filling are avoided. If possible balanced depth of cutting and filling is achieved.
8. It should not be aligned in rocky and cracked strata.
9. The alignment should avoid villages, roads, places of worship and other obligatory points.

5.4 Definitions

5.4.1 Canals: A canal is an artificial channel, generally trapezoidal in shape, constructed on the ground to carry water to the fields either from a river or tank or reservoir.

If the full supply level (FSL) of a canal is below the natural ground surface, an open cut or excavation is necessary to construct a canal. If the FSL of the canal is above the existing ground level, the canal is constructed by providing earthen banks on both sides. In the first case the channel is called a canal in cutting and in the second case it is called a canal in filling. Sometimes a canal can be of the intermediate type and the channel is called a canal in partial cutting and Partial filling.

5.4.2 Gross command area: It is the total area laying between the drainage boundaries which can be commanded or irrigated by a canal system.

5.4.3 Culturable command area: Gross command area may also contain villages, ponds, barrel lands, alkaline lands etc., and such areas are turned as unculturable area. The remaining area on which crops can be grown satisfactory is known as culturable command area.

5.4.4 Intensity of irrigation: Intensity of irrigation is decided according to water supply and area to be irrigated keeping in view the agriculture of the area. Intensity of irrigation means the area proposed to be irrigated per 100 hectares of culturable command area and is usually expressed in percent. When the supply of irrigation water is limited, intensity of irrigation is fixed before hand and the cropping pattern is evolved accordingly. Intensity is maximum in the perennial supply zone and minimum in the non-perennial zone.

5.4.5 Time factor: It is the ratio of number of days the canal has actually run to the number of days of irrigation period.
Time Factor = Number of days canal has actually run / Number of days of irrigation period

5.4.6 Crop factor: As plants grow larger, produce more leaf area, start producing fruit or approach maturity, the proportion of evaporation (Epan) or evapotranspiration (ET) that needs to be replaced by irrigation changes. Differences in water requirements and the proportion of Epan to be replaced are called crop factors (CF).

5.5 Canal lining
Canal Linings are provided in canals to resist the flow of water through its bed and sides. These can be constructed using impervious or fairly impervious lining material of sufficient strength such as compacted earth, cement, concrete, plastics, boulders, bricks etc.

Types of Lining:
The canals can be made fairly watertight by lining the canal section with various materials. The materials which are commonly used for lining are cement, bricks, puddle clay, stone blocks, and sodium carbonate, asphalt and road oils. The various types of linings vary in first cost, construction procedure.

5.5.1 Cement Concrete Lining:
There are various types of lining in which cement is used for lining the canal. This term is reserved for either plain or reinforced plastic concrete made with cement, coarse and fine aggregate or water. Experience has shown that this type of lining is very durable. It is capable of reducing the losses due to seepage by about 90 to 95 per cent. No general rule can be stated for determining the thickness of the concrete linings.
To relieve the canal of various unaccounted forces it is essential to limit the side slopes. The slopes adopted should be such that for the backfill the angle of internal fiction is not exceeded. Then the backfill will be stable and it will not exert any pressure on the lining. Thus the side slopes are limited to a range of 1.25: 1 to 1.5: 1. Sometimes side slopes as steep as 1: 1 may be adopted.

Other Types of Lining Using Cement:

Shotcrete lining: It is a type of lining in which slurry of Portland cement, processed sand and pure water is applied pneumatically through the nozzles on the surface of the subgrade. Ordinarily a mixture of cement and sand (1: 4) is used. The jet of cement mortar slurry is shot at the subgrade and hence this type of lining is called Shotcrete.

The thickness of this type of lining generally varies from 2.5 cm to 6 5 cm. This type of lining gives good working rate irrespective of the nature or condition of the subgrade. It works equally successfully on smooth, uneven or cracked sub-grade surfaces.

It is of course true that for irregular and cracked subgrade surfaces the amount required is more. This type is rich in cement and hence its cost is high. Satisfactory curing of shotcrete is very important requirement for successful lining. This type of lining has been found to be most successful.

Precast concrete lining: This type of lining is constructed with precast concrete slabs. The slabs are manufactured at a suitably located central place. The slabs may then be taken to the site at the time of construction. The size of concrete slab should be such that it can be handled by one or two men. The size may be 50 cm x 30 cm. The thickness of the slab may range from 5 to 6.5 cm. The blocks are manufactured with some interlocking arrangement at ends.

The provision of a suitable joint ensures continuity of the lining. The slabs are then laid on a well compacted sub-grade. The joints are sealed afterwards with asphalt or cement grout to prevent leakage.

Cement mortar lining: In this type of lining it is very essential to have well graded sand. The sand should range from fine to coarse to meet the requirements of durability and appearance. The amount of cement required is more and hence the cost is also more. The thickness of this type of lining may vary from 9 mm to 38 mm. The method of construction is similar to that of concrete lining.

This type is not durable hence it can be used only in conjunction with some other protective material. To give an example it may be sandwiched between two layers of bricks when later is used as a lining material. It has been proved that 25 mm thick cement mortar layer reduces seepage by about 75 per cent.

Soil cement lining: Sometimes cement may be mixed with the water and locally available soil. The soil before using should be thoroughly analyzed in laboratories by conducting various tests. The water soil and cement is mixed to get a workable mixture. After spreading this mixture on the sub-grade it is compacted to attain maximum density. This type of lining may be constructed with a travelling mixer with a slip form. It has been experienced that this type of lining can be constructed rapidly and has a low cost.

5.5.2 Brick Lining:

The canal is said to be lined with bricks when the sides and bed are protected with brick surfacing laid in cement mortar.

To make the lining successful the lining may be constructed of two layers of bricks laid flat in mortar. The first layer is laid on a 12 mm layer of 1: 6 cement mortar. This 12 mm thick layer of cement mortar is spread on the properly compacted and wetted subgrade.

Then on the top of first brick layer, a 12 mm layer of 1: 3 cement mortar is given. It is allowed to cure for two days. Finally second layer of bricks in mortar is laid on the top. Thus a 12 mm layer of rich cement mortar (1: 3) is sandwiched between two brick layers and this layer IS actually responsible to make the lining practically watertight.

The size of the bricks generally used is 30 cm x 15 cm x 6.5 cm. However, this size of bricks IS not standard. To give additional stability to the lining, masonry may be reinforced by using mild steel bars. This reinforcing steel is generally laid in the cement mortar layer sandwiched between the two brick layers.

The bricks used for lining should be manufactured with good earth. Good earth is one which contains about 10 to 20 per cent clay and in which calcium carbonate is less than 2 percent and salt content is not more than 0.3 per cent. The bricks should be thoroughly wetted with water before use.

The brick lining has been successfully done on various canals in Punjab, for example, Haveli canal and Bhakra canal. Sarda canal in Uttar Pradesh is also lined with bricks. The lining of Haveli canal damaged at many places within a year after its construction. The main reason for failure was that the lining was not designed and constructed to fulfill the safety requirements strictly. (Improper compaction, defective material and insufficient freeboard were the main reasons.)

5.5.3 Miscellaneous Types of Canal Lining:

(i) Clay puddle lining: The canal may also be lined with a clay puddle. The puddle clay is fairly impervious when properly pugged and saturated with water. It can be laid on the subgrade to form a fairly watertight coating. The thick layer of puddle clay is then protected with 30 cm thick layer of silty soil. It has been seen that it prevents about 80% seepage loss.

(ii) Lining with stone masonry: This type of lining is constructed with dressed stone blocks laid in mortar. Properly dressed stones are not available in nature. Irregular stone blocks are dressed and chipped off as per requirement. It makes the type costly. When roughly dressed stones are used for lining, the surface is rendered rough which may put lot of resistance to low. Technically the coefficient of rugosity will be higher. Thus the stone lining is limited to the situation where loss of head is not an important consideration and where stones are available at moderate cost.

(iii) Plastic lining: As a modern technique use of plastics in canal lining holds good promise. There are three types of plastic membranes which are used in canal lining, namely:

i. Low density polyethylene (LDPE)
ii. High molecular high density polythene (HM); and
iii. Polyvinyl chloride (PVC)

The plastic as a material for canal lining offers certain characteristic advantages like negligible weight, easy for handling, spreading and transport, immune to chemical action and speedy construction. The plastic film is spread on the prepared subgrade of the canal. To anchor the membrane on the banks 'V trenches are provided. The film is then covered with protective soil cover.

The plastic films are available like cloth in 3 m width. The thickness of plastic membrane varies according to its quality, for example, LDPE, HM and PVC qualities are used with 250, 100 and 15 micron thickness respectively. The plastic sheets can be welded together at site to increase the width. Considering factors like initial capital cost, installation cost and effectiveness plastic lining will go a long way in reducing loss of water in the water conveyance system.

5.6 Design of Canal by Lacey's Theory

Gerald Lacey, former Chief Engineer, U P Irrigation Department made a lot of investigations in this area and put forward this theory.

Lacey carried out a detailed study in designing suitable channels in alluvial soils. He developed the regime theory and formulated a number of expressions based on his observations. The salient features of Lacey's theory are stated as follows.

1. In a channel constructed in alluvial soil to carry a certain discharge, the bottom width, depth and bed slope of the channel will undergo modifications by silting and scouring till equilibrium is attained. The channel is now said to be a regime channel. (A regime channel is defined as a stable channel whose bed width, depth and side slope has undergone modifications by silting and scouring and are so adjusted that equilibrium is attained.)

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

   a) The Channel is flowing in unlimited incoherent alluvium of the same character as that of the transported sediment. (Incoherent alluvium is a soil composed of loose granular material which can be scoured and deposited with the same ease.)
   b) Silt grade (silt size) and silt charge (silt concentration) is the same throughout the channel.
   c) Discharge in the canal is constant.

2. The silt carried by the flowing water in the canal is kept in suspension by vertical eddies generated from the bed as well as from the sides of the canal.
3. The silt grade also plays an important role in controlling the regime conditions of the channel. The silt factor is given by the relationship $f=1.76\sqrt{d}$ where d represents diameter of silt particle in mm.

Lacey's procedures for designing unlined canals:

In Lacey's method for designing unlined canals in alluvial soils for a known discharge 'Q' and a mean diameter of silt particle 'd', the required quantities are calculated as follows.

1. The mean velocity of flow is computed from the relation

$$V = \left[\frac{Q * f^2}{140}\right]^{\frac{1}{6}}$$

Where Q is discharge and f is silt factor given by, f = 1.76 $\sqrt{d}$, d is the diameter of the silt particle in mm.

2. Calculate the cross sectional area of flow. A = (Q / V)

3. By knowing the side slopes express 'A' in terms of B and D

$$A = B*D + z*D^2 \text{…………… (A)}$$

4. Determine the required wetted perimeter from the relationship, P = 4.75 $\sqrt{Q}$

5. Express the wetted perimeter in terms of B and D,

$$P = B + \sqrt{5}\, D \text{……….. (B)}$$

6. From equation A and B solve for breadth (B) and depth (D).

7. Calculate hydraulic mean radius from the relationship

$$R = \frac{5}{2} * \frac{v^2}{f}$$

1. Also calculate hydraulic mean radius 'R' from the relationship

$$R = A / P$$

2. Calculate the bed slope from the equation,

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

5.6.1 Draw backs in Lacey's theory:

- The theory does not give a clear description of physical aspects of the problem.
- It does not define what actually governs the characteristics of an alluvial channel.
- The derivation of various formulae depends upon a single factor f and dependence on single factor f is not adequate.
- There are different phases of flow on bed and sides and hence different values of silt factor for bed and side should have been used.
- Lacey's equations do not include a concentration of silt as variable.
- Lacey did not take into account the silt left in channel by water that is lost in absorption which is as much as 12 to 15% of the total discharge of channel.
- The effect of silt accumulation was also ignored. The silt size does actually go on decreasing by the process attrition among the rolling silt particles dragged along the bed.
- Lacey did not properly define the silt grade and silt charge.
- Lacey introduced semi ellipse as ideal shape of a regime channel which is not correct.

5.7 Design of Canal by Kennedy's method

R G Kennedy, former Executive Engineer, Punjab Irrigation Department made a lot of investigations in this area and put forward this theory.

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

The bed of the canal offers frictional resistance to the flow of water, as a result critical eddies (Turbulences) arise from the bottom of the bed. These eddies keep the sediments carried by water in suspension. Some eddies also arise from the sides of the canal, but do not support the sediments. Hence, the sediment supporting capacity is proportional to the bed width of the canal.

The critical velocity or non-silting and non scouring velocity (V_k) is a function of the depth of the flowing water (D). It is given by the relationship

$$V_k = c * m * D^n$$

Where, ‘c’ is and ‘n’ are coefficients suggested by Kennedy for canals of Bari-Doad region.
The values of ‘c’ differs for different materials are
Light Sandy silt - c = 0.53
Coarse light sandy silt - c = 0.59
Sandy loam - c = 0.65
Coarse silt - c = 0.70

Note: Unless otherwise specified, values of c and n can be taken as c = 0.55 and n = 0.64
Thus, the equation for critical velocity becomes $V_k = 0.55\ m\ D^{0.64}$ in this above, ‘m’ represents critical velocity ratio which is given as $m = V_c / V_k$

Where, V_c represents mean velocity of flow.
The value of ‘m’ also varies with the silt material

Type of silt	Value of m
Silt of Indus rivers	0.7
Light sandy silt of North India	1.0
Coarse sandy silt	1.1
Sandy, Loamy silt	1.2
Coarse silt of hard rock	1.3

Note: Unless, otherwise specified m = 1.0

Case 1: When bed slope S is given

1. For the given discharge (Q) assume a trial value of the depth of flow (D)
   For different values of discharge (Q) the trail values of depth of flow (D) are given as follows

Q (m3/s)	0.283	0.708	1.416	2.832	7.079	14.158	28.317	56.634
D (m)	0.49	0.66	0.84	1.04	1.43	1.73	1.98	2.26

2. Calculate $V_k = 0.55 * m * D^{0.64}$
3. Determine ‘A’ from the equation $Q = A * V_k$
4. Knowing D and A calculate the Bed width ‘B’ using the equation
   $$A = B*D + z*D^2$$
   Assume side slope 1(V): 1/2(H), if not given.
5. Knowing ‘B’ and ‘D’ calculate wetted perimeter using the equation
   $$P = B + \sqrt{5D}$$
6. Knowing ‘A’ and ‘P’ calculate hydraulic mean radius

$$R = A/P$$

7. Calculate the mean velocity of flow from the equation

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

Where, C represents Chezy's constant, $C = \frac{23+\frac{1}{N}+\frac{0.0015}{S}}{1+(23+\frac{0.0015}{S})\frac{N}{\sqrt{R}}}$

Where, N represents Kutter's Rugosity coefficient,

S represents Bed slope of the canal

8. If critical velocity ratio ($m = V_c / V_k$) is equal to 1 (m=1), then the assumed value of 'D' is correct.
9. If not revise the depth 'D'.

Case 2: When B/D ratio is given

1. Let B/D = x, B = Dx
2. Calculate the cross sectional area in terms of 'D'

$$A = B*D + z*D^2$$

3. Calculate the critical velocity V_k in terms of 'D' by substituting in $V_k = 0.55 * m * D^{0.64}$
4. Substituting for 'A' and 'V_k' in $Q = A* V_k$, 'D' can be determined.
5. Knowing D, A and B calculate 'P' and 'R'
6. Calculate $V_k = 0.55 * m * D^{0.64}$
7. Assuming a trial value for 'S', calculate the Chezy's constant from equation

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

8. Calculate the mean velocity of flow from the equation $V_c = C\sqrt{RS}$
9. If critical velocity ratio ($m = V_c / V_k$) is equal to 1 (m=1), then the assumed value of 'S' is adequate.
10. If not revise the bed slope, 'S'.

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

Q (m3/s)	0.283	0.708	1.416	2.832	7.079	14.158	28.317	56.634
S (1 in ---)	3333	3636	4000	4444	4444	5000	5000	5714

N depends upon the boundary material

Channel condition	Very good	Good	Indifferent	Poor
N	0.0225	0.025	0.0275	0.03

Discharge (cumec)	14 – 140	140 – 280	> 280
N (in ordinary soil)	0.025	0.0225	0.02

5.7.1 Draw backs in Kennedy's theory:

- Kutter's equation is used for determining the mean velocity of flow and hence the limitations of Kutter's equation are incorporated in Kennedy's theory.
- The significance of B/D ratio is not considered in the theory

- No equation for the bed slope has been given which may lead to varied designs of the channel with slight variation in the bed slope.
- Silt charge and silt grade are not considered. The complex phenomenon of silt transportation is incorporated in a single factor are called critical velocity ratio.
- The value of m is decided arbitrarily since there is no method given for determining its value.
- This theory is aimed to design only an average regime channel.
- The design of channel by the method based on this theory involves trial and error which is quite cumbersome.

Problems

1. Design an irrigation channel to carry 5 cumec. The channel is to be laid on a slope of 0.2 m per kilometer. Assume N = 0.0225 and m = 1

Solution:

1. Assume a trial depth D equal to 1.0 m
2. $V_k = 0.55 * m * D^{0.64} = 0.55 \times 1.0 \times 1.0^{0.64} = 0.55$
3. Area = Q = A* V_k

   $A = Q / V_k = 5 / 0.55 = 9.09\ m^2$
4. $A = B*D + z*D^2$

   $9.09 = B * 1.0 + (1/2) * 1.0^2$

   B = 8.59 m
5. Perimeter = $P = B + \sqrt{5}\,D = 8.59 + \sqrt{5 * 1} = 10.83$ m

   $R = A / P = 9.09 / 10.83 = 0.84$ m
6. Mean velocity flow

   $V_c = C\sqrt{RS}$

   R= 0.84 m, S= 0.2/1000, N = 0.0225

$$C = \frac{23+\frac{1}{N}+\frac{0.0015}{S}}{1+(23+\frac{0.0015}{S})\frac{N}{\sqrt{R}}} = \frac{23+\frac{1}{0.0225}+\frac{0.0015}{(\frac{0.2}{1000})}}{1+\left(23+\frac{0.0015}{\left(\frac{0.2}{1000}\right)}\right)*\frac{0.0225}{\sqrt{0.84}}} = 42.85$$

   $V_c = 42.85\sqrt{0.84 * (\frac{0.2}{1000})} = 0.555$ m/s
7. Ratio of velocities found in step 6 and step2

   $V_c / V_k = 0.555 / 0.55 = 1.009 = 1.0$

   Hence assumed d is satisfactory.

2. Determine the dimensions of the irrigation canal for the following data B/D ratio = 3.7, N = 0.0225, m = 1.0 and S = 1/ 4000 side slopes of the channel is ½ H: 1V. Also determine the discharge which will be flowing in the channel.

Solution:

B / D = 3.7

B= 3.7D

For the channel with side slopes of 1/2H: 1V

$$R = \frac{A}{P} = \frac{B*D + z*D^2}{B + \sqrt{5}\,D} = \frac{3.7\,D*D + 0.5*D^2}{3.7\,D + \sqrt{5}\,D} = 0.71\,D$$

From Kennedy's equation

$V_k = 0.55 * m * D^{0.64} = 0.55 \times 1.0 \times D^{0.64} = 0.55\,D^{0.64}$ ---------- 1

$V_c = C\sqrt{RS}$

R= 0.71 D m, S= 1/4000, N = 0.0225

$$V_c = C\sqrt{RS} = \frac{23+\frac{1}{N}+\frac{0.0015}{S}}{1+(23+\frac{0.0015}{S})\frac{N}{\sqrt{R}}}\sqrt{RS} = \frac{23+\frac{1}{0.0225}+\frac{0.0015}{(\frac{1}{4000})}}{1+\left(23+\frac{0.0015}{\left(\frac{1}{4000}\right)}\right)*\frac{0.0225}{\sqrt{0.71\,D}}}\sqrt{0.71\,D*\frac{1}{1000}} =$$

$$= \frac{0.975\,D^{1/2}}{1+0.781\,D^{-1/2}}$$ ------------- 2

Equating the equations 1 and 2, we get

$0.55\,D^{0.64} = 0.975D^{1/2} / (1+0.781D^{-1/2})$

$0.55\,D^{0.64} + 0.4296D^{0.14} = 0.9795D^{1/2}$

Solving the above equation by trial and error , we get

D = 1.0 m

B = 3.7 m

V = 0.55 m/s

$A = B*D + z*D^2 = 3.7 * 1 + 0.5 * 1^2 = 4.2$

$Q = A* V_k = 4.2 * 0.55 = 2.31$ cumec

3. Design a irrigation channel in alluvial soil according to Lace's silt theory for the following data. Full supply discharge= 10 cumec, Lacey's silt factor = 0.9, Side slopes of channel= 1/2H: 1V

Solution:

The mean velocity of flow is computed from the relation

$$V = \left[\frac{Q*f^2}{140}\right]^{\frac{1}{6}} = \left[\frac{10*0.9^2}{140}\right]^{\frac{1}{6}} = 0.62 \text{ m/s}$$

Cross sectional Area of flow. A = (Q / V) = 10 / 0.62 = 16.12 m^3

Hydraulic mean radius from the relationship

$$R = \frac{5}{2}*\frac{V^2}{f} = \frac{5}{2}*\frac{0.62^2}{0.9} = 1.07 \text{ m}$$

Wetted perimeter, $P = 4.75\sqrt{Q} = 4.75\sqrt{10} = 15.02$ m

Bed slope from the equation,

$$S = \frac{f^{\frac{5}{3}}}{3340 * Q^{\frac{1}{6}}} = \frac{0.9^{\frac{5}{3}}}{3340 * 10^{\frac{1}{6}}} = 1 / 5836$$

4. Design an irrigation channel in alluvial soil from following data using Lacey's theory: Discharge = 15.0 cumec; Lacey's silt factor = 1.0; Side slope = ½: 1

Solution:

The mean velocity of flow is computed from the relation

$$V = \left[\frac{Q * f^2}{140}\right]^{\frac{1}{6}} = \left[\frac{15 * 1.0^2}{140}\right]^{\frac{1}{6}} = 0.689 \text{ m/s}$$

Cross sectional Area of flow. A = (Q / V) = 15 / 0.689 = 21.77 m^3

Hydraulic mean radius from the relationship

$$R = \frac{5}{2} * \frac{V^2}{f} = \frac{5}{2} * \frac{0.689^2}{1.0} = 1.186 \text{ m}$$

Wetted perimeter, P = 4.75 $\sqrt{Q}$ = 4.75 $\sqrt{15}$ = 18.4 m

Bed slope from the equation,

$$S = \frac{f^{\frac{5}{3}}}{3340 * Q^{\frac{1}{6}}} = \frac{1.0^{\frac{5}{3}}}{3340 * 15^{\frac{1}{6}}} = 1 / 5245$$

Difference between Kennedy's and Lacey's method

	Kennedy's Theory	Lacey's Theory
1	There can be many sections for a given discharge	Only one regime section is possible for a given discharge of silt factor
2	It introduces critical velocity ration (m)	It introduces silt factor (f)
3	Kennedy did not specify regime slope	Lacey specified regime slope for a given discharge and silt factor
4	Kennedy considered channel section as trapezoidal	Lacey considered channel section as semi elliptical
5	Kennedy channel section is wider and shallower	Lacey's channel section is tighter and deeper
6	Kennedy's theory applicable to irrigation channels only	Lacey's theory applicable to irrigation channels as well as rivers
7	Kennedy's theory depends on Kutter's equation	Lacey's theory doesn't depend on Kutter's equation

Reservoirs:

Definition: a natural or artificial place where water is collected and stored for use, especially water for supplying a community, irrigating land, furnishing power, etc.

5.8 Investigations for Reservoir site:

Following are the investigations that are usually conducted for reservoir planning.

1. Engineering Investigations / Surveys
2. Geological Investigations
3. Hydrologic Investigations

5.8.1 Engineering investigations / surveys:

- Generally Engineering Surveys are conducted for the dam, the reservoir and their associated works. During this investigation topographic survey of the area is carried out and the contour plan is prepared. The horizontal control is usually provided by triangulation survey and vertical control by precise leveling.
- At the dam site, very accurate triangulation survey is conducted and a contour plan to a scale of 1:250 or 1:500 is generally prepared with contour intervals in the range of 1 to 2 m. Such a survey should cover an area at hart upto 200 m upstream 400 m downstream and for adequate width beyond the two abutments.
- For the reservoir, the contour plan is generally prepared to a scale of 1:15,000 with contour intervals between 2 to 3 m. The area elevation and storage elevation curves are prepared for different elevations upto an elevation of 3 to 5 m higher than the anticipated maximum water level.

5.8.2 Geological investigations:

Following are the reasons for carrying out the Geological investigations at a reservoir site:

- Suitability of foundation for the dam.
- Water tightness of the reservoir basis.
- Location of quarry sites for the construction.

5.8.3 Hydrological investigations:

Following purposes demand the hydrological investigations:

- To study the runoff pattern and to estimate yield.
- To determine the maximum discharge at the site.

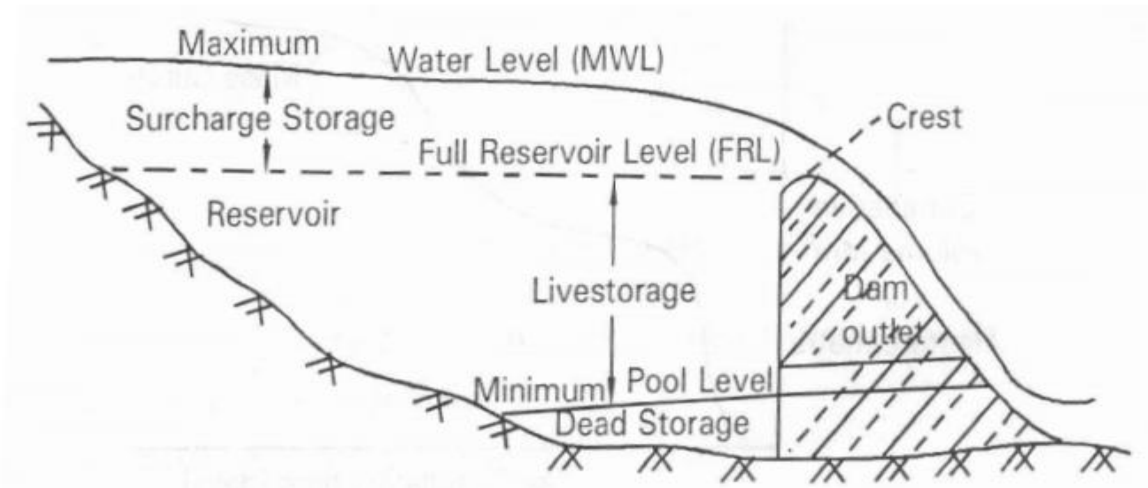
Selection of site for a Reservoir

A good site for a reservoir should have the following characteristics:

- Large storage capacity: The topography of the proposed site should be such that the reservoir has a large capacity for storing the water.
- Suitable site for the dam: A suitable site for the proposed dam should be available on the downstream side of the reservoir, with very good foundation; narrow opening in the valley to provide minimum length of the dam and also the cost of construction should be minimum.

- Water tightness of the reservoir: Geology at the proposed reservoir site should be such that the entire reservoir basin is water tight. They should have Granite, Gneiss, Schists, Slates, or Shales etc.
- Good hydrological conditions: The hydrological conditions of the river at the reservoir should give high yield. Evaporation, transpiration, and percolation losses should be minimum.
- Deep reservoir: The proposed site should be such that a deep reservoir is formed after the dam construction. The reason being evaporation losses would be minimum; in addition to low cost of land acquisition and less weed growth.
- Small Submerged area: At the proposed site, the submerged area should be minimum and should not affect the ecology of the area. Important places, monuments, roads, railway lines should not sub merge.
- Minimum silt inflow: The life of reservoir is defined by the quantity of silt inflow, which means that, if the silt inflow is large, the life would be less. Hence, it is necessary to select the reservoir site at such a place, where the silt inflow is minimum.
- No objectionable minerals: The proposed site should be free from soluble and objectionable salts, which may pollute the reservoir.
- Minimum acquisition and construction cost: The overall cost of the project should be minimum in terms of dam construction, land acquisition for reservoir, buildings, roads, railways etc.

5.9 Storage Zones of a Reservoir:



1. Live Storage or useful storage: Is that amount of water available or stored between the minimum pool level (LWL) and the full reservoir level (FRL). Minimum pool level or low water level is fixed after considering the minimum working head required for the efficient working of turbines.
2. Surcharge Storage: Is the volume of water stored above the full reservoir level (FRL) up to the maximum water level (MWL) In case of a multipurpose reservoir, useful storage or live storage is divided into A. Conservation storage B. Flood control storage
3. Dead storage: Is the volume of water held below the minimum pool level. This storage is not useful and hence cannot be used for any purpose under ordinary operating conditions.

4. Bank storage: Water stored in the banks of a river is known as bank storage. In most of the reservoirs the bank storage is small since the banks are generally impervious.
5. Valley storage: Is the volume of water held by the natural river channel in its valley upto the top of its banks before the construction of the reservoir. The valley storage depends upon the cross section of the river, the length of the river and its water level.

5.9.1 Determination of Storage Capacity using Mass Curves

Mass Curve is a graphical representation of cumulative volume of water in the reservoir Vs cumulative time. It will be a continuously raising curve.

Fixing Capacity of a reservoir Capacity of a reservoir depends on the inflow and demand. It is a fact that if the available inflow is more than the demand, there is no necessity of any storage. On the other hand, if the inflow is less and demand is high a large reservoir capacity is required. Capacity for a reservoir can be determined by the following methods

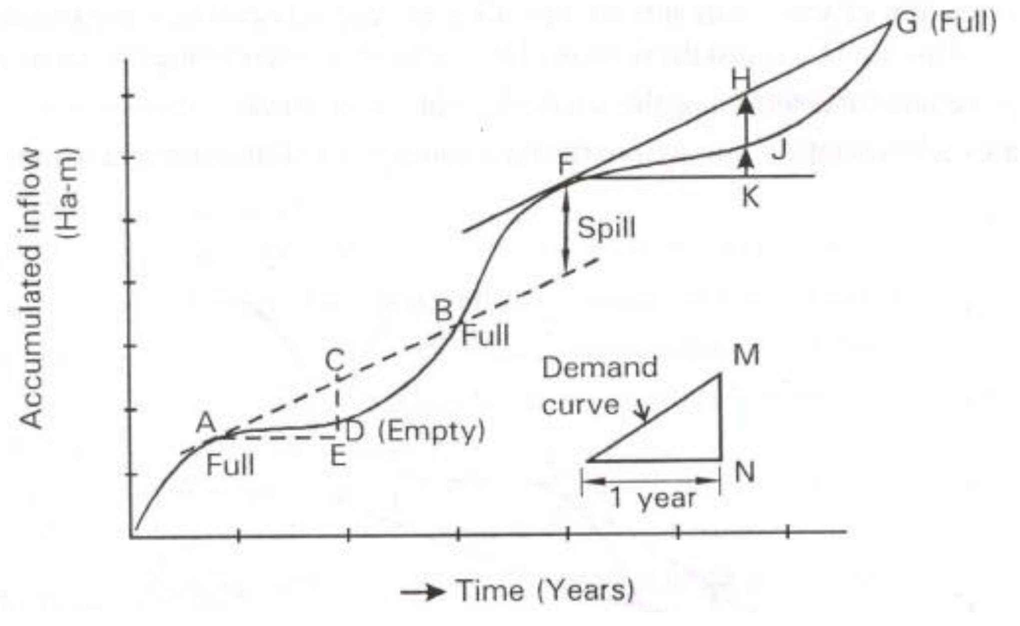
(i) Mass curve or graphical method
(ii) Analytical method
(iii) Flow duration curve method

5.9.2 Mass curve method or Graphical method

Storage required for uniform demand: In the case of uniform demand, the mass curve will be a straight line.

The procedure adopted will be as follows:

1. Prepare the mass inflow curve for the flow hydrograph of the site for a number of consecutive years including the most critical years (or the driest years) when the discharge is low, Fig shows the mass inflow curve.
2. Prepare the mass demand curve corresponding to the given rate of demand. If the rate of demand is constant, the mass demand curve is a straight line as shown in fig. The scale selected for plotting of the mass inflow and mass demand curve should be the same.



3. Draw the lines AB, FG etc. such that they are parallel to the mass demand curve, and they are tangential to the peak points or crests at A, F etc. of the mass inflow curve points A, F, etc. indicate the beginning of dry periods marked by the depressions.
4. Determine the vertical intercepts CD, HJ etc. between the tangential lines and the mass inflow curve. These intercepts indicate the volumes by which the inflow volumes fall short of demand, which can be explained as follows:

❖ Assuming that the reservoir is full at point A, the inflow volume during the period AE is equal to ordinate DE and the demand is equal to ordinate CE. Thus the storage required is equal to the volume intercepted by the intercept CD.

5. Determine the largest of the vertical intercept determined in step (4). The largest vertical intercept represents the storage capacity required. Following import points have to be noted:

❖ The capacity obtained in the net storage capacity which must be available to meet the demand. The gross capacity of the reservoir will be more than the net storage capacity. It is obtained by adding the evaporation and seepage losses to the net storage capacity.

❖ The tangential lines AB, FG etc. when extended forward must interest the inflow curve. This is necessary for the reservoir to get filled again. If these lines do not intersect the mass curve, the reservoir would not fill again. Many times very large reservoirs may not get refilled every year.

❖ The vertical distance such as FL between the successive tangents represents the volume of water flowing over the spillway.

5.10 Economical height of dam:

Economic height of a dam can be theoretically defined as that height for which the cost of the dam per million cubic meter of storage is minimum.

The height of a dam is determined by preparing approximate estimates of the cost of several heights of dams at a given site.

It is done somewhat above and below the level, where the elevation storage curve shows a fairly high rate of increase of storage per meter of elevation. While at the corresponding elevation, the cross section of the dam site shows the length of the dam to be moderate.

5.11 Recommended questions

1. What are the considerations made during alignment of canals?
2. Write a note on canal classification?
3. Write a short note on:
   (a) Critical velocity ratio (b) Regime Channel
4. Describe the procedure involved in design of canals using Kennedy’s method
5. Describe the procedure involved in design of canals using Lacey’s method
6. Explain the criteria for selection of reservoir site
7. Explain with neat sketch storage zones of reservoir.
8. Explain the different investigations conducted before selecting a reservoir site.
9. Explain the determination of storage capacity of reservoir by mass curves

5.12 Outcomes

1. Find the canal capacity, design the canal and compute the reservoir capacity.

5.13 Further Reading

a) https://dreamcivil.com/types-of-canal/
b) https://www.biologydiscussion.com/irrigation/reservoirs/reservoirs-types-planning-and-selection-of-site/73412

IRRIGATIONA ENGINEERING AND HYDRAULIC STRUCTURE (BCV602)

MODULE 03
GRAVITY DAMS

A **gravity dam** is a dam constructed from concrete or stone masonry and designed to hold back water by primarily utilizing the weight of the material alone to resist the horizontal pressure of water pushing against it. Gravity dams are designed so that each section of the dam is stable, independent of any other dam section

FORCES ACTING ON GRAVITY DAM:

In the design of a dam, the first step is the determination of various forces which acts on the structure and study their nature. Depending upon the situation, the dam is subjected to the following forces:

1. Water pressure
2. Uplift Pressure
3. Earthquake forces
4. Silt pressure
5. Wave pressure
6. Ice pressure
6. Self-weight of the dam.

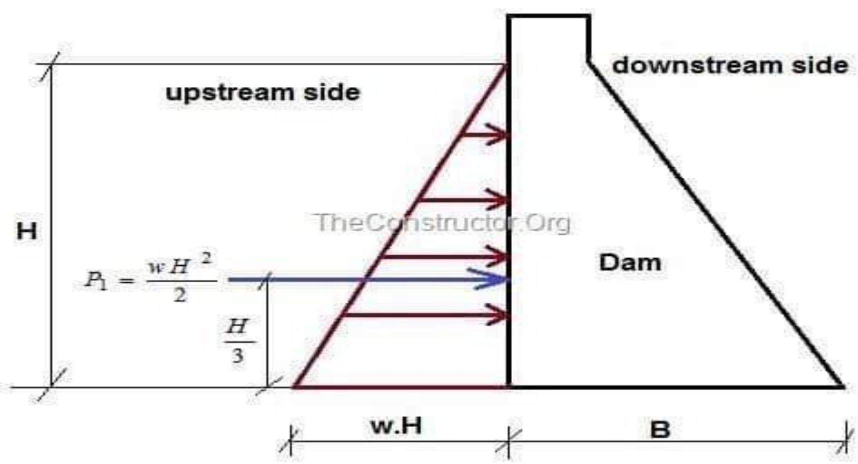
The forces are considered to act per unit length of the dam.

For perfect and most accurate design, the effect of all the forces should be investigated. Out of these forces, most common and important forces are water pressure and self-weight of the dam.

1. Water Pressure

It is the pressure of water on the upstream face of the dam. In this, there are two cases:

(I) Upstream face of the dam is vertical and there is no water on the downstream side of the dam (figure 1).



The total pressure is in horizontal direction and acts on the upstream face at a height from the bottom.

The pressure diagram is triangular and the total pressure is given by $\frac{H}{3}$

Where w is the specific weight of water. Usually, it is taken as unity. $P_1 = \frac{wH^2}{2}$

H is the height upto which water is stored in m.

(ii) Upstream face with batter and there is no water on the downstream side (figure 2).

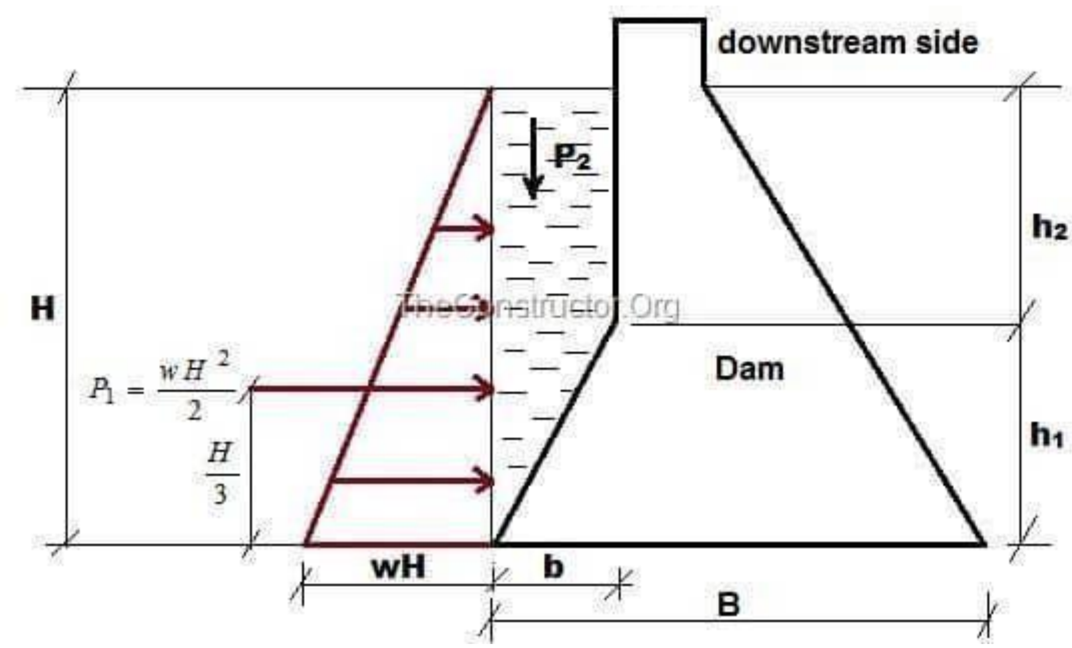


Figure 2

Here in addition to the horizontal water pressure P_1 as in the previous case, there is vertical pressure of the water. It is due to the water column resting on the upstream sloping side.

The vertical pressure P_2 acts on the length 'b' portion of the base. This vertical pressure is given by

$$P_2 = (b \times h_2 \times w) + \left(\frac{1}{2} b \times h_1 \times w\right)$$

Pressure P_2 acts through the center of gravity of the water column resting on the sloping upstream face.

If there is water standing on the downstream side of the dam, pressure may be calculated similarly. The water pressure on the downstream face actually stabilizes the dam. Hence as an additional factor of safety, it may be neglected.

2. Uplift pressure

When the water is stored on the upstream side of a dam there exists a head of water equal to the height upto which the water is stored. This water enters the pores and fissures of the foundation material under pressure. It also enters the joint between the dam and the foundation at the base and the pores of the dam itself. This water then seeps through and tries to emerge out on the downstream end. The seeping water creates hydraulic gradient between the upstream and downstream side of the dam. This hydraulic gradient causes vertical upward pressure. The upward pressure is known as uplift. Uplift reduces the effective weight of the structure and consequently the restoring force is reduced. It is essential to study the nature of uplift and also some methods will have to be devised to reduce the uplift pressure value.

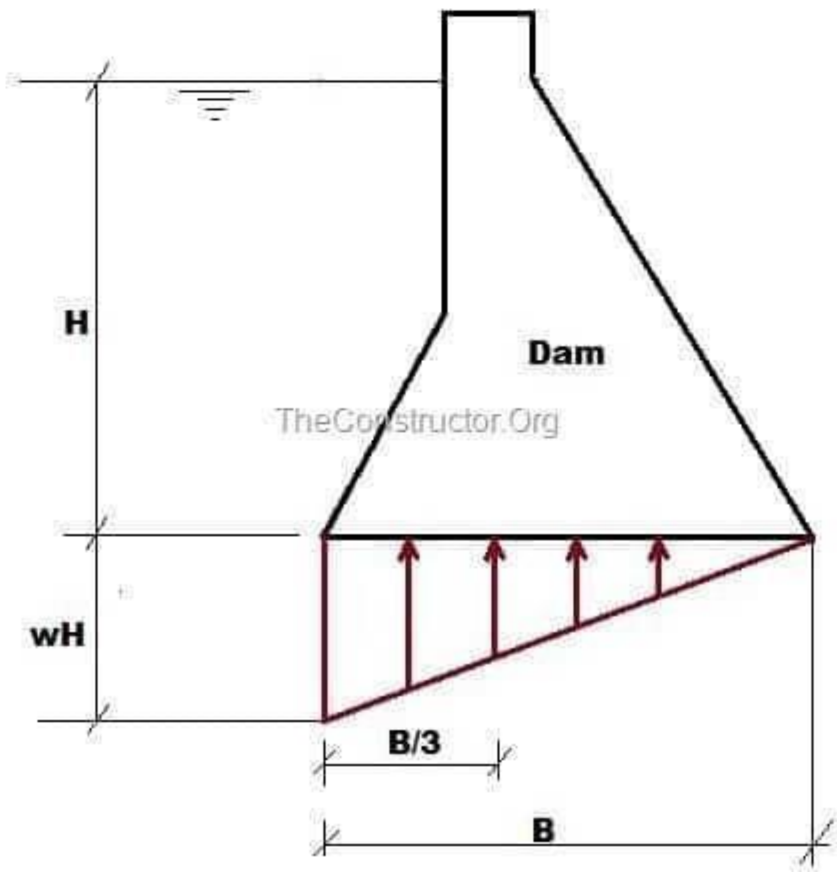


Figure 3

With reference to figure 3, uplift pressure is given by $P_u = \frac{wH \times B}{2}$

Where P_u is the uplift pressure, B is the base width of the dam and H is the height upto which water is stored.

This total uplift acts at $\frac{B}{3}$ from the heel or upstream end of the dam.

Uplift is generally reduced by providing drainage pipes or holes in the dam section.

Self-weight of the dam is the only largest force which stabilizes the structure. The total weight of the dam is supposed to act through the centre of gravity of the dam section in vertically downward direction. Naturally when specific weight of the material of construction is high, restoring force will be more. Construction material is so chosen that the density of the material is about 2.045 gram per cubic meter.

3. Earthquake Forces

The effect of earthquake is equivalent to acceleration to the foundation of the dam in the direction in which the wave is travelling at the moment. Earthquake wave may move in any direction and for design purposes, it is resolved into the vertical and horizontal directions. On an average, a value of 0.1 to 0.15g (where g = acceleration due to gravity) is generally sufficient for high dams in seismic zones. In extremely seismic regions and in conservative designs, even a value of 0.3g may sometimes by adopted.

Vertical acceleration reduces the unit weight of the dam material and that of water is to $(1 - k_v)$ times the original unit weight, where k_v the value of g accounted against earthquake forces, i.e. 0.1 is when 0.1g is accounted for earthquake forces. The horizontal acceleration acting towards the reservoir causes a momentary increase in water pressure and the foundation and dam accelerate towards the reservoir and the water resists the movement owing to its inertia. The extra pressure exerted by this process is known as hydrodynamic pressure.

4. Silt Pressure

If h is the height of silt deposited, then the forces exerted by this silt in addition to the external water pressure, can be represented by Rankine formula

$P_{\text{silt}} = \frac{1}{2}\gamma_s h^2 k_a$ acting at $\frac{h}{3}$ from the base. Where,

k_a = coefficient of active earth pressure of silt = $\frac{1 - \sin\phi}{1 + \sin\phi}$

ϕ = angle of internal friction of soil, cohesion neglected.

γ_s = submerged unit weight of silt material.

h = height of silt deposited.

5. Wave Pressure

Waves are generated on the surface of the reservoir by the blowing winds, which exert a pressure on the downstream side. Wave pressure depends upon wave height which is given by the equation

$h_w = 0.032\sqrt{PV} + 0.763 - 0.271 \times (F)^{1/4}$ for F < 32 km, and

$h_w = 0.032\sqrt{VF}$ for F > 32 km

Where h_w is the height of water from the top of crest to bottom of trough in meters.

V – wind velocity in km/hour

F – fetch or straight length of water expanse in km.

The maximum pressure intensity due to wave action may be given by $P_w = 2.4\gamma_w h_w$ and acts at $\frac{h_w}{2}$ meters above the still water surface.

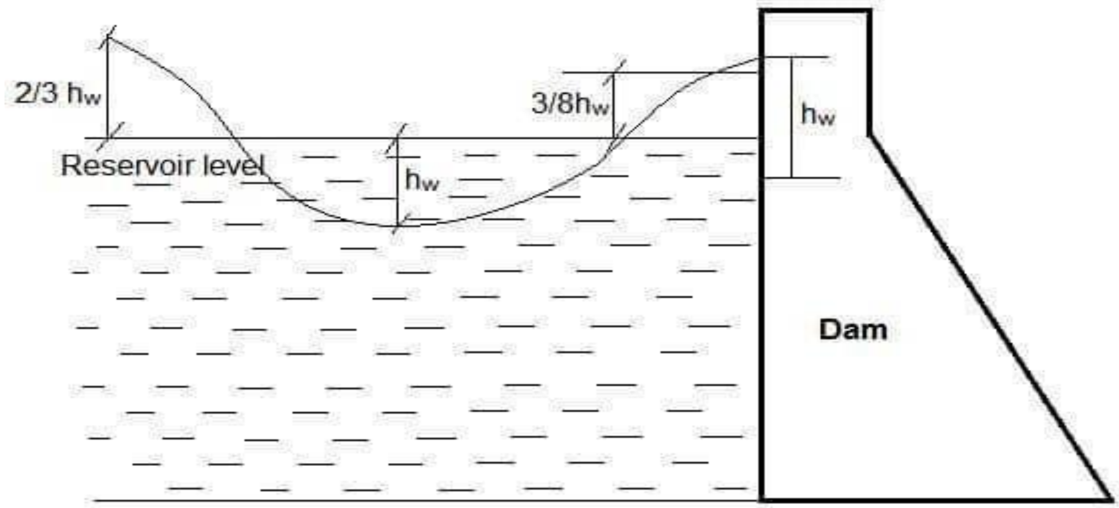


Figure 4

The pressure distribution may be assumed to be triangular of height $\frac{5h_w}{3}$ as shown in figure 4. Hence total force due to wave action P_w

$= \frac{1}{2} \times (2.4 \gamma_w h_w) \times \frac{5}{3} h_w$ acting at $\frac{3}{8} h_w$ above the reservoir surface.

6. Ice Pressure

The ice which may be formed on the water surface of the reservoir in cold countries may sometimes melt and expand. The dam face is subjected to the thrust and exerted by the expanding ice. This force acts linearly along the length of the dam and at the reservoir level. The magnitude of this force varies from 250 to 1500 kN/sq.m depending upon the temperature variations. On an average, a value of 500 kN/sq.m may be taken under ordinary circumstances.

7. Self-Weight of dam

The weight of dam and its foundation is a major resisting force. In two-dimensional analysis of dam, unit length of the dam is considered. The cross section of the dam may be divided into several triangles and rectangles and weights W1, W2, W3 etc are calculated along with their determination of lines of action. The total weight W of the dam acts at the C.G of its section.

FAILURES OF GRAVITY DAM

Failure of gravity dam occurs due to overturning, sliding, tension and compression. A gravity dam is designed in such a way that it resists all external forces acting on the dam like water pressure, wind pressure, wave pressure, ice pressure, uplift pressure by its own self-weight. Gravity dams are constructed from masonry or concrete. However, concrete gravity dams are preferred these days and mostly constructed.

The advantage of gravity dam is that its structure is most durable and solid and requires very less maintenance.

CAUSES OF FAILURE OF A GRAVITY DAM:

A gravity dam may fail in following modes:

1. Overturning of dam about the toe
2. Sliding – shear failure of gravity dam
3. Compression – by crushing of the gravity dam
4. Tension – by development of tensile forces which results in the crack in gravity dam.

Overturning Failure of Gravity Dam:

The horizontal forces such as water pressure, wave pressure, silt pressure which act against the gravity dam causes overturning moments. To resist this, resisting moments are generated by the self- weight of the dam.

If the resultant of all the forces acting on a dam at any of its sections, passes through toe, the dam will rotate and overturn about the toe. This is called overturning failure of gravity dam. But, practically, such a condition does not arise and dam will fail much earlier by compression.

The ratio of the resisting moments about toe to the overturning moments about toe is called the factor of safety against overturning. Its value generally varies between 2 and 3.

Factor of safety against overturning is given by

FOS = sum of resisting moments/ sum of overturning moments

$$F.S = \frac{\sum MR}{\sum MO}$$

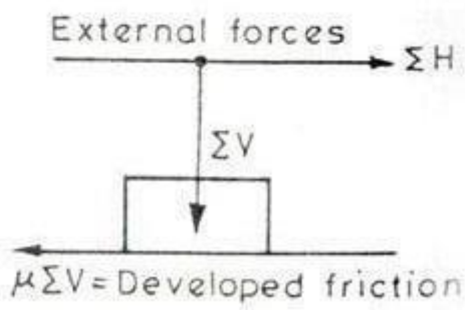


Fig: Sum of external horizontal forces greater than vertical self-weight of dam (overacting, sliding occurs)

Sliding Failure of Gravity Dam: When the net horizontal forces acting on gravity dam at the base exceeds the frictional resistance (produced between body of the dam and foundation), The failure occurs is known as sliding failure of gravity dam.

In low dams, the safety against sliding should be checked only for friction, but in high dams, for economical precise design, the shear strength of the joint is also considered

Factor of safety against sliding can be given based on Frictional resistance and shear strength of the dam

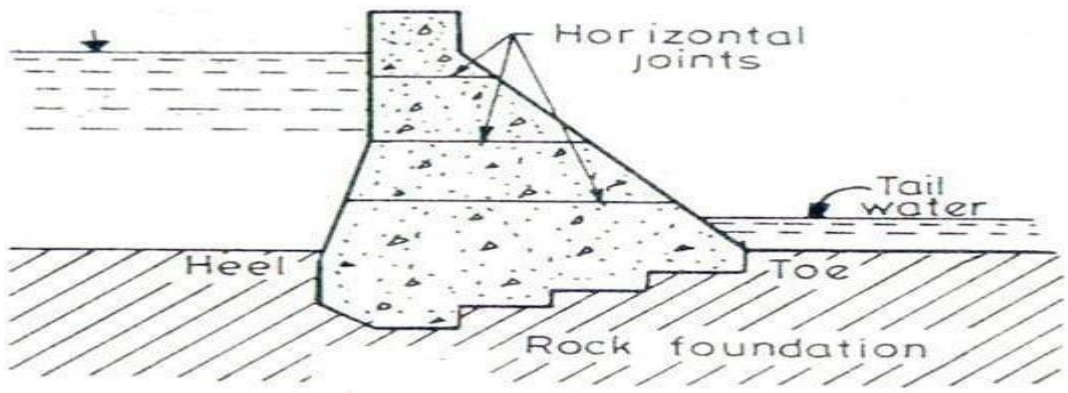
Factor of safety based on frictional resistance:

$$\text{FOS against sliding} = \text{FOS} = \frac{\mu \sum V}{\sum H}$$

μ=co-efficient of friction between two surfaces

$\sum V$ = sum of vertical forces acting on dam

$\sum H$ = sum of vertical forces acting on dam



Gravity Dam Failure due to Tension Cracks: Masonry and concrete are weak in tension. Thus masonry and concrete gravity dams are usually designed in such a way that no tension is developed anywhere. If these dams are subjected to tensile stresses, materials may develop tension cracks. Thus the dam loses contact with the bottom foundation due to this crack and becomes ineffective and fails. Hence, the effective width B of the dam base will be reduced. This will increase P_{max} at the toe. Hence, a tension crack by itself does not fail the structure, but it leads to the failure of the structure by producing excessive compressive stresses.

For high gravity dams, certain amount of tension is permitted under severest loading conditions in order to achieve economy in design. This is permitted because the worst condition of loads may occur only momentarily and may not occur frequently.

Gravity Dam Failure due to Compression: A gravity dam may fail by the failure of its material, i.e., the compressive stresses produced may exceed the allowable stresses, and the dam material may get crushed.

ELEMENTARY PROFILE OF A GRAVITY DAM (Refer class notes)

PROFILE OF A DAM FROM PRACTICAL CONSIDERATIONS

The elementary profile of a gravity dam, (i.e. triangle with maximum water surface at apex) is only a theoretical profile. Certain changes will have to be made in this profile in order to cater to the practical needs.

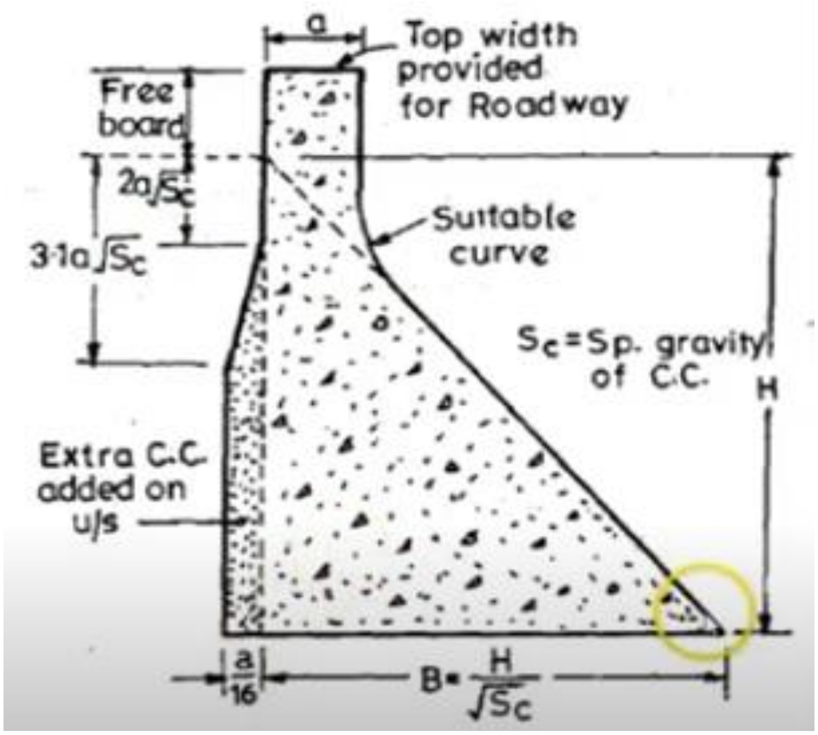
These needs are,

(i) Providing a straight top width for road construction over the top of the dam

(ii) Providing a free-board above the top water surface, so that water may spill over the top of the dam due to wave action, etc.

The addition of these two provisions, will cause the resultant force to shift towards the heel. The resultant force, when the reservoir is empty, was earlier passing through the inner middle third point. This will, therefore, shift more towards the heel, crossing the inner middle third point and consequently, tension will be developed at the toe. In order to avoid the development of this tension, some masonry will have to be

added to the upstream side., which shows the typical section along with the possible dimensions that can be adopted for a low gravity dam section. It should however, be checked for stability analysis.



LIMITING HEIGHT OF A HIGH AND LOW GRAVITY DAM (Refer Class Notes)

STABILITY ANALYSIS OF GRAVITY DAM

General Selection of the method of analysis should be governed by the type and configuration of the structure being considered. The gravity method will generally be sufficient for the analysis of most structures; however, more sophisticated methods may be required for structures that are curved in plan, or structures with unusual configurations. 3-4.2 Gravity Method The gravity method assumes that the dam is a 2-dimensional rigid block. The foundation pressure distribution is assumed to be linear. It is usually prudent to perform gravity analysis before doing more rigorous studies. In most cases, if gravity analysis indicates that the dam is stable, no further analyses need be done.

Stability Analysis Assumptions:

1. The dam is considered to be composed of a number of Cantilevers, each of which is 1 m thick and each of which acts independently of the other.
2. No load is transferred to the abutments by beam action
3. The foundation and the dam behave as a single unit, the joints being perfect.
4. The material in the foundation and the body of the dam are isotropic and homogeneous.
5. The stresses developed in the foundation and the body of the dam is isotropic and homogeneous.
6. No movements of dams are caused by the Transfers of loads.
7. Small openings made in the body of the dam do not affect the general distribution of stresses

Stability Analysis Procedure

Two-dimensional analysis can be carried out analytically or graphically

Analytical Method

1. Consider unit length of the dam
2. Work out the magnitude and direction of all the vertical forces acting on the dam and their algebraic sum i.e., $\sum V$
3. Similarly, work out all the horizontal forces and their algebraic sum, i.e. $\sum V$
4. Determine the level arm of all these forces about the toe
5. Determine the moments of all these forces about the toe and find out the algebraic sum of all those moments i.e... $\sum M$
6. Find out the location of the resultant force by determining its distance from the toe

   $\bar{x}=\sum M/\sum V$
7. Find out the eccentricity (e) of the resultant (R) using $e=B/2-\bar{x}$. It must be less than B/6 in order to ensure that no tension is developed anywhere in the dam.
8. Determine the vertical stresses at the toe and heel using the equation

   $P_v = \sum V/B[1\pm 6e/B]$

   Some stresses are found by ignoring uplift
9. Determine the maximum normal stresses i.e., principal stresses and shear stresses at the toe and heel using equations. The crushing strength of concrete varies between 1500 to 3000 KN/m^2.
10. Determine the factor of safety against overturning using the formula

    $FOS= \sum MR/\sum Mo$
11. Determine the factor of safety against Sliding using the formula

    $FS=\mu\sum V/\sum H$
12. Determine the factor of safety against Shear using the formula

    $FSS=\mu\sum V+ bq /\sum H$

    Sliding factor must be greater than unity and S.F.F. must be greater than 3 to 5.

    The analysis should be carried out for reservoir full case as well as for reservoir empty case.

[Refer Class notes for problems on Stability Analysis of Gravity Dam]

Graphical method

1. In the graphical method, the entire dam section is divided into number of horizontal sections at some suitable interval, particularly at the place where the slope changes.
2. For each section, the sum of the vertical forces $\sum V$ and the sum of all the horizontal forces $\sum H$ acting above that particular section, are worked out and line of action are graphically located.
3. The Resultant R is then found and its line of action is also located graphically.
4. This is done for each section and a line joining all the points where the individual resultants

cut the individual sections, is drawn.

5. This line represents the resultant force and should lie within the middle third, for no tension to develop.
6. The procedure should be repeated for reservoir full as well reservoir empty case.
7. Both the lines of resultant pressure so obtained should lie in the middle third portion as shown in figure below.

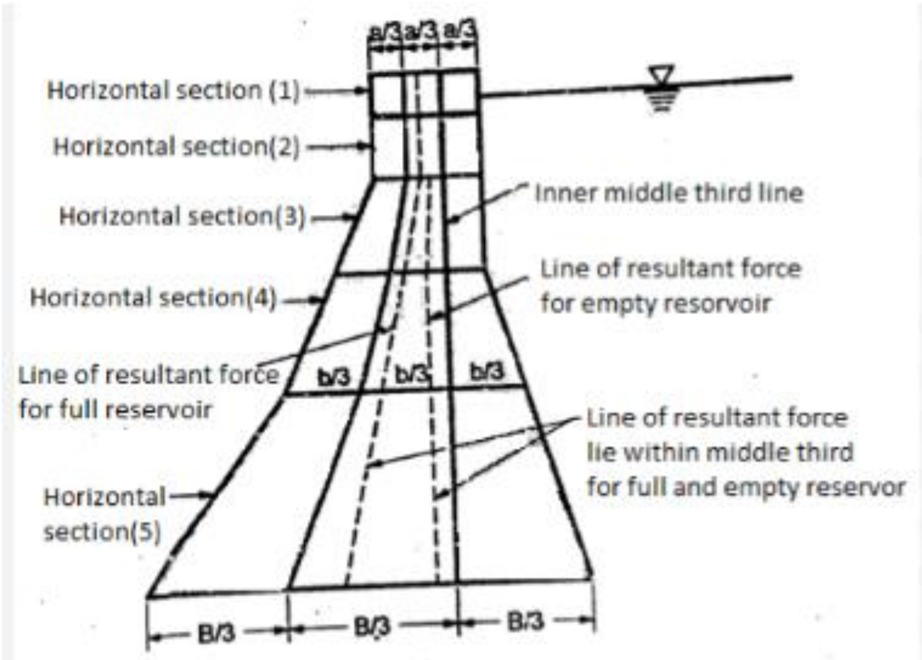


Figure: Graphical representation of a Gravity Dam

FOUNDATION FOR A GRAVITY DAM

The foundation of a gravity dam plays a critical role in ensuring the dam's stability and safety. A gravity dam relies on its weight to resist the horizontal forces exerted by the water it holds back. The foundation is crucial because it must support the entire weight of the dam and the water, while also preventing settlement or tilting.

Here are key components and considerations for the foundation of a gravity dam:

1. Location and Geology

The foundation should be placed on solid, stable rock or compacted soil capable of withstanding high compressive and shear forces.

In cases where the foundation material is weak or loose, remedial actions like grouting or deep foundations (e.g., piles) might be necessary.

2. Foundation Types

Rock Foundation: This is the most preferred foundation type, as it provides a strong, stable base. The rock should be sound and free from faults, fractures, and other weaknesses.

Soil Foundation: If the dam is built on soil, the soil must have high bearing capacity, good consolidation characteristics, and low permeability to avoid settlement and seepage. Often, soil foundations are improved through techniques like compaction, grouting, or providing a concrete slab.

Mixed Foundation: In some cases, parts of the dam may rest on rock, while other sections may be founded on soil or loose material, requiring special considerations for reinforcement.

3. Preparation and Excavation

The site should be carefully excavated to a sufficient depth to remove any loose material, and the foundation should be leveled and cleared of debris.

If necessary, an impermeable layer (like a concrete slab or cut-off trench) is constructed to minimize seepage through the foundation.

4. Cut-off Wall or Trench

A cut-off wall or trench may be built to prevent seepage and uplift pressure under the dam. It acts as a barrier that extends deep into the foundation rock or soil.

This trench can be filled with concrete or grout to ensure a watertight seal.

5. Uplift Forces

Water pressure beneath the dam (uplift pressure) is an important factor to consider in dam foundation design. Uplift forces can cause instability if not properly addressed. The foundation should be designed to resist these forces, often by providing a deep and solid foundation.

6. Seepage Control

Seepage through the foundation is a potential problem, especially if the foundation is in permeable material. Measures like grout curtains, cutoff walls, and drainage systems are used to control and manage seepage.

7. Settlement and Differential Movement

The foundation must be designed to ensure that it doesn’t settle unevenly, which could lead to tilting or cracking of the dam structure. Settlement analysis is done to determine if the soil or rock can accommodate the load without significant deformation.

8. Reinforcement

The foundation may need additional reinforcement, particularly in areas with weak or fractured rock. This could involve grouting, soil stabilization, or even the use of foundation piles.

9. Drainage

Drainage systems are essential to relieve water pressure that might build up beneath the dam and to control the uplift forces. These systems could include drain holes, galleries, and other features that allow water to flow away safely.

10. Analysis and Design

The dam’s foundation design is based on detailed geological and geotechnical studies, including site investigations to determine the properties of the soil or rock and the potential for seismic activity, erosion, and groundwater movement.

Galleries in Gravity Dams

Galleries are the horizontal or sloping openings or passages left in the body of the dam. • They may run longitudinally (i.e., parallel to dam axis) or traversely (i.e., normal to the dam axis) and are provided at various elevations. All the galleries are interconnected by steeply sloping passages or by vertical shafts fitted with stairs or mechanical lifts.
Function and types of galleries in Dams

(i) Foundation Gallery

A gallery provided in a dam may serve one particular purpose or more than one purpose. For example, a gallery provided near the rock foundation, serves to drain off the water which percolates through the foundations. This gallery is called a **foundation gallery or a drainage gallery**.

1. It runs longitudinally and is quite near to the upstream face of the dam. Drain holes are drilled from the floors of this gallery after the foundation grouting has been completed. Seepages is collected through these drain holes.
2. Besides draining off seepage water, it may be helpful for drilling and grouting of the foundations, when this cannot be done from the surface of the dam.
3. The primary function of a drainage gallery is to manage and control seepage (water flow through or under the dam) to reduce the risks of internal erosion, instability, and damage to the dam. It is designed to collect, control, and safely discharge water that may infiltrate the dam structure or its foundation.

(ii) Inspection Galleries

The water which seeps through the body of the dam is collected by means of a system of galleries provided at various elevations and interconnected by vertical shafts, etc. All these galleries, besides draining off seepage water, serves inspection purpose. They provide access to the interior of the dam and are, therefore, called inspection purposes. They generally serve other purposes along with this purpose.

1. They intercept and drain off the water seeping through the dam body
2. They provide access to dam interior for observing and controlling the behavior of the dam.
3. They provide enough space for carrying pipes, etc. during artificial cooling of concrete
4. They provide access to all the outlets and spillway gates, valves, etc. by housing their electrical and mechanical controls. All these gates, valves, etc., can hence be easily controlled by men, from inside the dam itself.
5. They provide space for drilling and grouting of the foundations, then it cannot be done from the surface of the dam.

An inspection gallery allows for easy access to inspect the interior of the dam structure. It provides a safe and accessible space for engineers and maintenance teams to assess the condition of the dam, its drainage system, and other critical components. The inspection gallery also facilitates monitoring for any signs of deterioration, seepage, cracks, or other potential issues

Recommended questions

1. Explain briefly with neat sketches the different forces that may act on gravity dam. Indicate their magnitudes, direction and location
2. Discuss in brief various modes of failure of a Gravity dam
3. What do you understand by elementary profile of a Gravity dam?
4. Derive expressions to determine base width of an elementary profile of a Gravity dam on
   i) Stress criterion ii) Sliding criterion
5. Give a brief note on practical profile of a gravity dam
6. Distinguish between Low and High gravity dam. Derive the expression used for such a distinction
7. Explain step by step procedure for stability analysis of a Gravity dam by analytical method. Mention its assumptions
8. With a neat sketch explain procedure for stability analysis of a Gravity dam by Graphical method.
9. Write a note on i) Foundation of a dam ii) Types of Galleries in Gravity dam

Module – 4: Earth dams

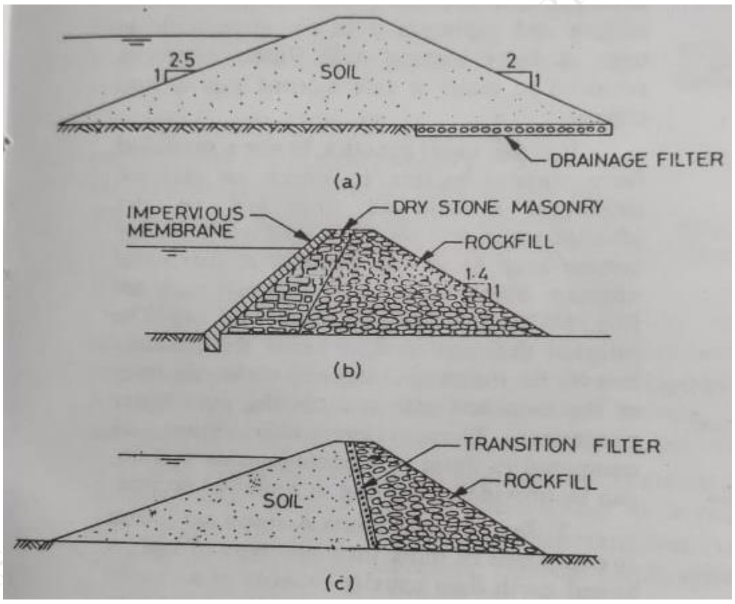
Spillways

Energy Dissipaters and Stilling Basins

EARTH DAMS

Embankment dams are built of soil or rockfill or both. As the soil and rockfill are no-rigid materials, the embankment dams are also called non-rigid dams. The embankments are broadly classified as follows:

- **Earth dams**: These are constructed mainly from earth or soil [fig (a)].
- **Rockfill dams**: These dams are constructed mainly from rockfill or pieces of rocks [fig (b)].
- **Composite earth and rockfill dam**: These dams are constructed from both earth and rockfill [fig (c)].



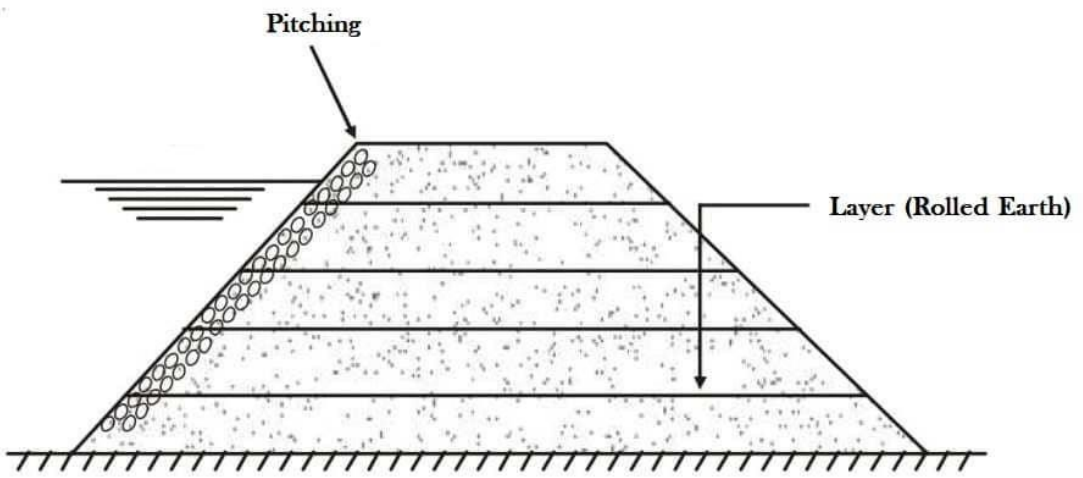
Earthen dams and earthen levees are the most ancient type of embankments, as they can be built with the natural materials with a minimum of processing and primitive equipment. They can be constructed on almost all type of foundations provided suitable measures are taken. But in ancient days, the cost of-carriage and dumping of the dam materials was quite high. How- ever, the modern developments in earth moving equipment have considerably reduced the cost of carriage and laying of the dam materials. The cost of gravity dams on the other hand, has gone up because of an increase in the cost of concrete, masonry, etc. Earthen dams are still cheaper as they can utilize the locally available materials, and less skilled labour is required for them.

4.1 TYPES OF EARTHEN DAMS

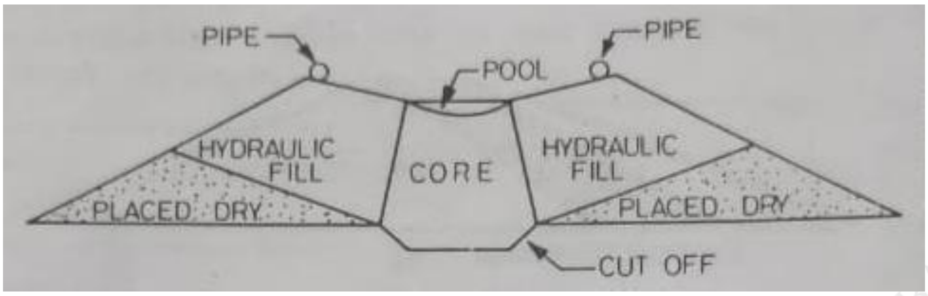
The earthen dam can· be of the following types:

- Based on method of construction
    - Rolled-fill earth dam
    - Hydraulic-fill earth dam
    - Semi-hydraulic earth dam
- Based on mechanical characteristics of earth materials used in making the section of da
    - Homogeneous Earthen Dam
    - Zoned Earthen Dam
    - Diaphragm Earthen Dam

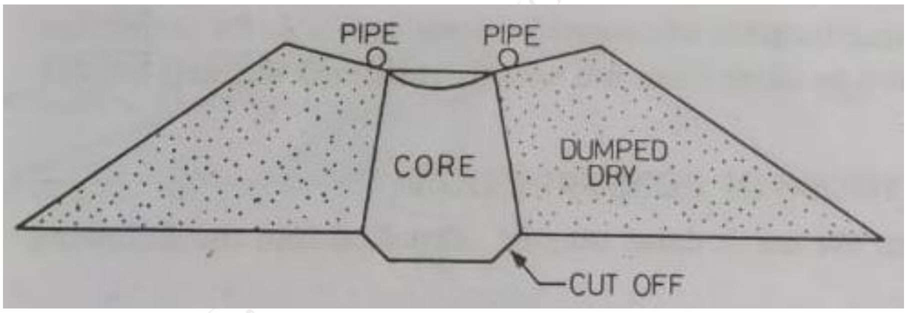
Rolled-fill earth dam: rolled-fill dams are most commonly used in practice. These dams are constructed by placing material in thin layers, about 15 to 45 cm thick, and compacting each layer to the required dry density with heavy rollers. Each layer should be properly bonded to the preceding layer.



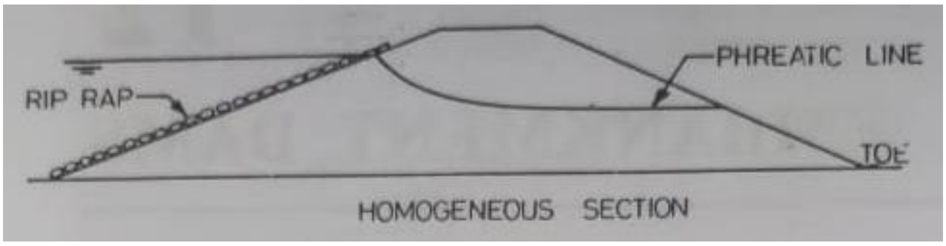
Hydraulic-fill earth dam: in a hydraulic –fill dam, water is used for transporting and placing the materials. No roller is required for compaction. The material at the borrow pits is mixed with a large quantity of water to form slush or mud. This slush is transported through flumes or pipes and discharged along the outside edges of the fill of the earth dam. As soon as the slush comes out of the pipe, the coarser materials are deposited near the exit. However, the finer materials are carried into the central pool at the location of the core. Thus a zoned section is formed with a relatively impervious core at the centre and a pervious shell at the shoulders. With proper control, a satisfactory zoned sections can be achieved.



Semi-hydraulic earth dam: in semi-hydraulic-fill dams, the coarse material is dumped from trucks into required position to form shells. The core is, however, constructed by hydraulic fill method. The fines are sluiced into a core. This method of constructions requires careful control to achieve a satisfactory embankment section. If the soil is not properly sluiced but it is dumped into a pool of water, some fines in the soil will be washed out and the core obtained will be pervious. As in the case of a hydraulic-fill dam, no compaction is required.

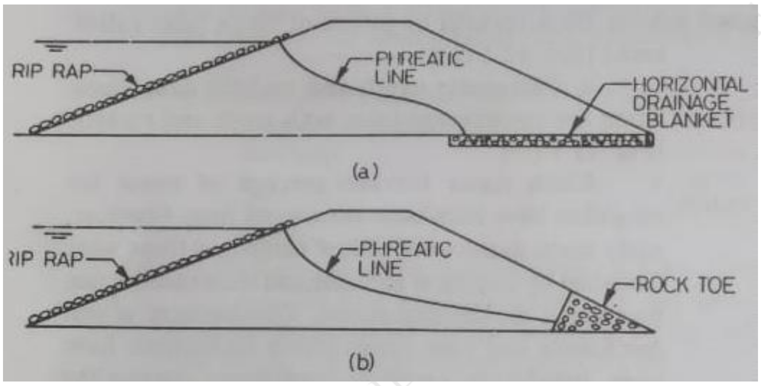


Homogeneous Earthen Dam: A homogeneous earth dam is composed of only one material. Generally, the material used is either semi-impervious or impervious soil TOE to limit the seepage through the dam. However, sometimes earth dams are built of relatively pervious soil such as sand and sand gravel mixture. Of course, the seepage through such dams is quite high. A homogeneous earth dam is usually constructed where only one type of material is economically available near the dam site and the height of the dam is low.



Modified homogeneous section: Modified homogeneous section A purely homogeneous section poses the problem of sliding and piping if it is not quite wide at base. A large section with flatter slopes is required to make it safe against piping and sliding.

It is the usual practice to use a modified homogeneous section in which an internal drainage system is provided in the homogeneous section. The internal drainage system may be in the form of a horizontal drainage blanket [Fig. (a)] or a rock toe [Fig. (b)] or a combination of the two. The internal drainage system keeps the phreatic line (or the saturation line) well within the body of the dam and also reduces the pore water pressures. Thus steeper side slopes, as compared to those in a homogeneous section, can be provided, resulting in a smaller section.

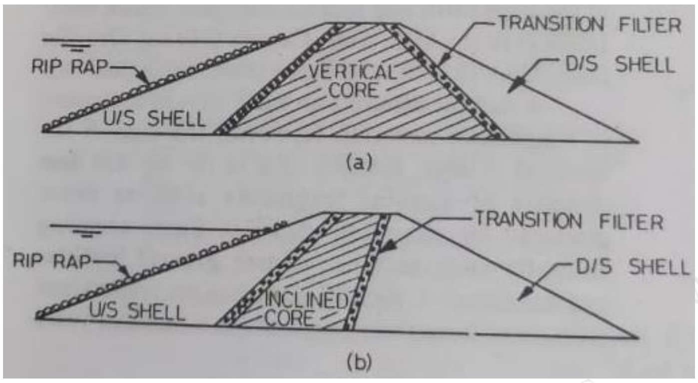


Zoned Earthen Dam: A zoned earth dam is composed of more than one type of soil. A zoned earth dam usually consists of a central impervious core flanked by shells of pervious materials on the upstream and downstream sides [Fig. (a)]. A transition filter is usually required between the core and the shell to prevent piping.

The central core checks seepage through the dam It is constructed of clay, silt, silty clay or clayey silt. The pervious shell gives stability to the dam and it consists of sand, gravel or a mixture of these materials. The upstream pervious zone provides free drainage during sudden drawdown. The downstream pervious zone acts as a drain to control the phreatic line. The pervious zones give stability to the core and also distribute the load over a large area of foundation. The transition filters prevent the migration of the core material into the pores of the shell material. The downstream transition filter is useful during the steady seepage conditions and the upstream filter is useful during the sudden drawdown conditions.

However, the transition filters are omitted if the difference in particle sizes of the core material and the shell material is not much or when the seepage gradient line through the dam is flat.

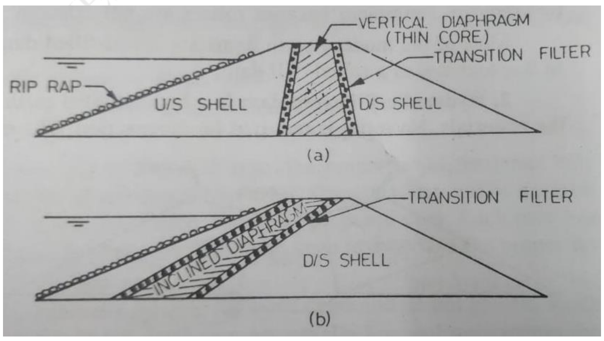
Sometimes, a sloping core is provided instead of the vertical core [Fig. (b)]. The main advantage of the sloping core is that the downstream portion of the dam can be constructed first and the core can be constructed later. However, a central core has a number of advantages and is commonly used in practice, as explained later. Zoned earth dams are commonly constructed in practice.

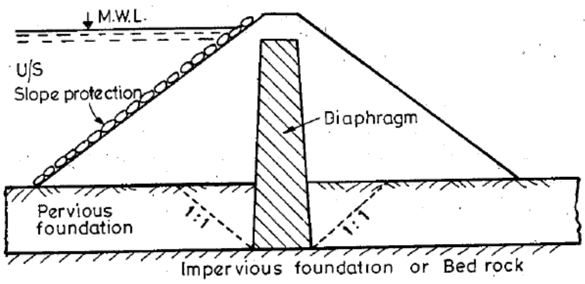


Diaphragm Earthen Dam: a diaphragm-type earth dam consists of a *thin* impervious core, called diaphragm, surrounded by pervious shells. The diaphragm-type earth dam is also called the *thin core earth dam* [Fig. (a)]

It may be noted that the difference between a zoned earth dam and a diaphragm-type earth dam is only in the thickness of the core. If the thickness of the core at any elevation is less than the height of the embankment above that elevation or 10 m, the dam is generally considered to be of the diaphragm type. On the other hand, if the thickness of the core equals or exceeds these limits, the dam is considered to be of the zoned type.

The position of the diaphragm may be anywhere from a central vertical position to an inclined position directly on the upstream face. The inclined diaphragm on the upstream face is also called a *buried blanket*, because a thin layer of pervious material is usually provided over it. The slope protection is provided over this pervious material [Fig. (b)].





4.2 CAUSES OF FAILURE OF EARTH DAM

Earth dams are less rigid and hence more susceptible to failure. Every past failure of such a dam has contributed to an increase in the knowledge of the earth dam designers. Earthen dams may fail, like other engineering structures, due to improper designs, faulty constructions, lack of maintenance, etc. The various causes leading to the failure of earth dams can be grouped into the following three classes:

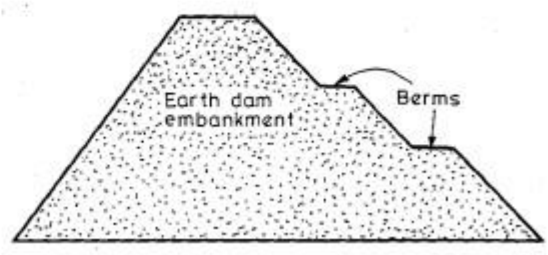
- Hydraulic failure.
- Seepage failure.
- Structural failure.

4.2.1 HYDRAULIC FAILURE

About 40% of earth dam failures have been attributed to these causes. The failure under this category, may occur due to the following reasons:

- **By over topping**: The water may overtop the dam, if the design flood is underestimated or if the spillway is of insufficient capacity or if the spillway gates are not properly operated. Sufficient freeboard should, therefore, be provided as an additional safety measure.
- **Erosion of upstream face**: The waves developed near the top water surface due to the winds, try to notch-out the soil from the upstream face and may even, sometimes, cause the slip of the upstream slope. Upstream stone pitching or riprap should, therefore, be provided to avoid such failures.
- **Cracking due to frost action**: Frost in the upper portion of the dam may cause heaving and cracking of the soil with dangerous seepage and· consequent' failure. An additional freeboard allowance upto a maximum of say 1.5 m should, therefore, be provided for dams in areas of low temperatures.
- **Erosion of downstream face by gully formation**: Heavy rains falling directly over the downstream face and the erosive action of the moving water, may lead to the formation of gullies on the downstream face, ultimately leading to the dam failure. This can be avoided by proper maintenance, filling the cuts from time to time especially during rainy season, by grassing the slopes and by providing proper berms at suitable heights (Fig.), so that the water has not to flow for considerable distances. The proper drainage

arrangements are made for the removal of the rain water collected on the horizontal berms. Since the provision of berms ensures the collection and removal of water before it acquires high downward velocities, the consequent erosion caused by the moving water (run off) is considerably reduced.

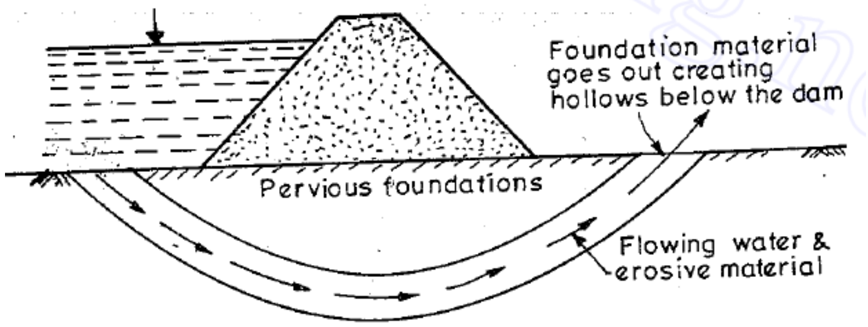


- **Erosion of the dis toe**: The dis toe of the earth dam may get eroded due to two reasons, i.e. (i) the erosion .due to cross currents that may come from the spillway buckets; and (ii) the erosion due-to tail water. This erosion of the toe can be avoided by providing a downstream slope pitching or a riprap up to a height slightly above the normal tail water depth. Side walls of the spillway (called diaphragm walls) must be of sufficient height and length, as so to prevent the possibility of the cross flow towards the earthen embankment.

4.2.2 SEEPAGE FAILURE

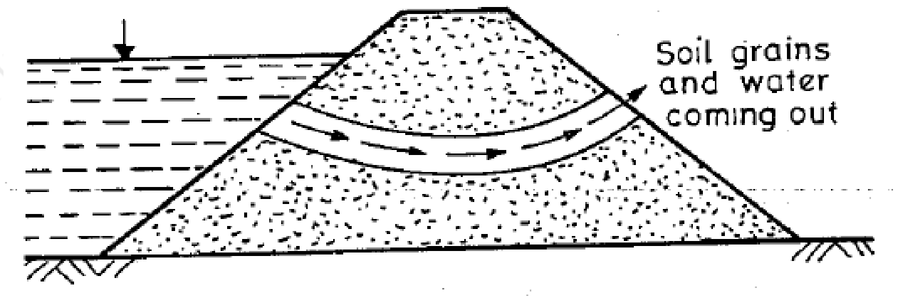
Controlled seepage or limited uniform seepage is inevitable in all earth dams, and ordinarily it does not produce any harm. However, uncontrolled or concentrated seepage through the dam body or through its foundation may lead to piping or sloughing and the subsequent failure of the dam. Piping is the progressive erosion and subsequent removal of the soil grains from within the body of the dam or the foundation of the dam. Sloughing is the progressive removal of soil from the wet downstream face. More than 1/3rd of the earth dams have failed because of these reasons.

- **Piping through foundations:** Sometimes, when highly permeable cavities or fissures or strata of coarse sand or gravel are present in the foundation of the dam, water may start seeping at a huge rate through them (Fig.). This concentrated flow at a high gradient, may erode the soil. This leads to increased flow of water and soil, ultimately resulting in a rush of water and soil, thereby creating hollows below the foundation. The dam may sink down into the hollow so formed, causing its failure.



- **Piping through the dam body**: When the concentrated flow channels get developed in the body of the dam, (Fig.) soil may be removed in the same manner as was explained in foundation piping, leading to the formation of hollows in the dam body, and subsequent subsidence of the dam. These flow channels may develop due to faulty construction, insufficient compaction, cracks developed in embankment due to foundation settlement, shrinkage cracks, animal burrows, etc. All these causes can be removed by better construction and better· maintenance of the dam embankments.

  Piping through the dam body, generally get developed near the pipe conduits passing through the dam body. Contact seepage along the outer side of conduits may either develop into piping, or seepage through leaks in the conduits may develop into piping. This can be avoided by thoroughly and properly compacting the soils near the outlet conduits and by preventing the possibilities of leakage through conduits, but preventing the formation of cracks in the conduits. These cracks in the conduits are caused by differential settlement and by overloading from the embankment. When these factors are controlled, automatically, the possibility of piping due to leakage through the conduits is reduced.



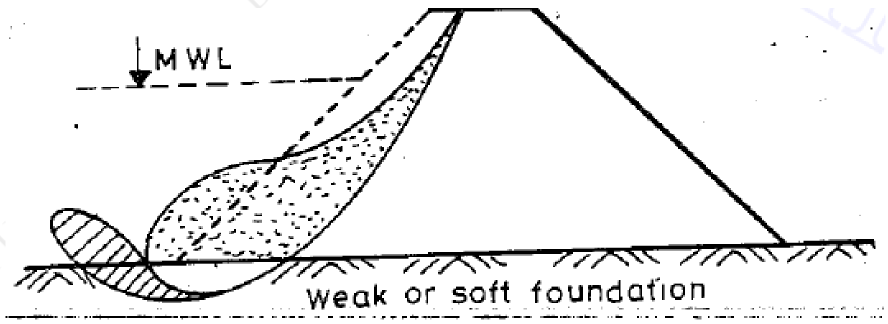
- **Sloughing of D/S Toe**: The process behind the sloughing of the toe is somewhat similar to that of piping. The process of failure due to sloughing starts when the downstream toe becomes saturated and get eroded, producing a small slump or a miniature slide.

The miniature slide leaves a relatively steep face which becomes saturated by the seep-age from the reservoir and slumps again, forming a more unstable surface. The process continues till the remaining portion of the dam is too thin to withstand the horizontal. water pressure, leading to the sudden failure of the dam.

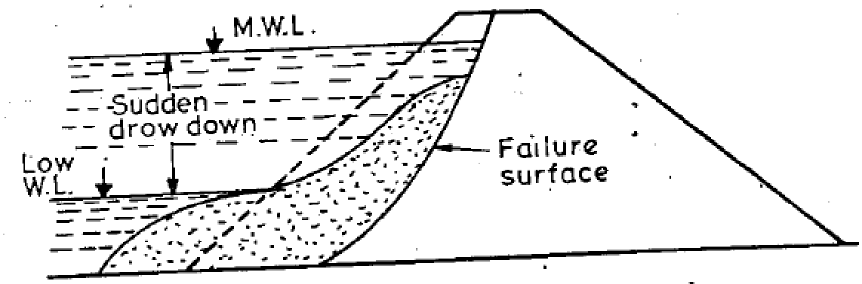
4.2.3 STRUCTURAL FAILURE

About 25% of the dam failures have been attributed to structural failures. Structural failures are generally caused by shear failures, causing slides.

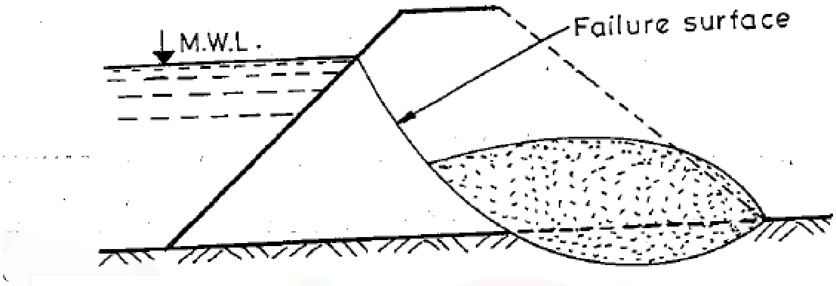
- **Foundation slide**: (i.e. overall stability of the dam). When the foundation of the earth dams are made of soft soils, such as fine silt, soft clay, etc., the entire dam may slide over the foundation. Sometimes, seams of fissured rocks, shales or soft clay, etc. may exits under the foundation, and the dam may slide over some of them, causing its fail-ure. In this type-of-failure;-the top of embankment gets cracked and subsides, the lower slope moves outward forming large mud waves near the heel, as shown in Fig. Excessive pore water pressure in confined seams of sand and silt, artesian pressure in abutments, or hydrostatic excess developed due to consideration of clay seams embed-ded between sands or silts, etc. may reduce the shear strength of the soil, until it be-comes incapable of resisting 'the induced shear stresses, leading to the failure of the dam foundation whithout warning. Loose sand foundations may fail by the liquefaction or flow slides.



- **Slide in. Embankments**: When the embankment slopes are too steep for the strength of the soil, they may slide causing dam failure. The most critical condition of the slide of the u/s slope is the sudden drawdown of the reservoir (Fig.); and the d/s slope is most likely to slide, when the reservoir is full (Fig.). The u/s slope failures seldom lead to catastrophic failures, but the d/s slope failures are very serious. These failures, generally occur due to development of excessive unaccounted pore pressures which may reduce the shearing strength of the soils as explained in the previous article. Many embankments may fail during the process of consolidation, at' the time of construction or after the construction



U/S slope slide due to sudden draw-down.



D/S. slope slide during full reservoir condition.

4.3 CRITERIA FOR SAFE DESIGN OF EARTHEN DAMS

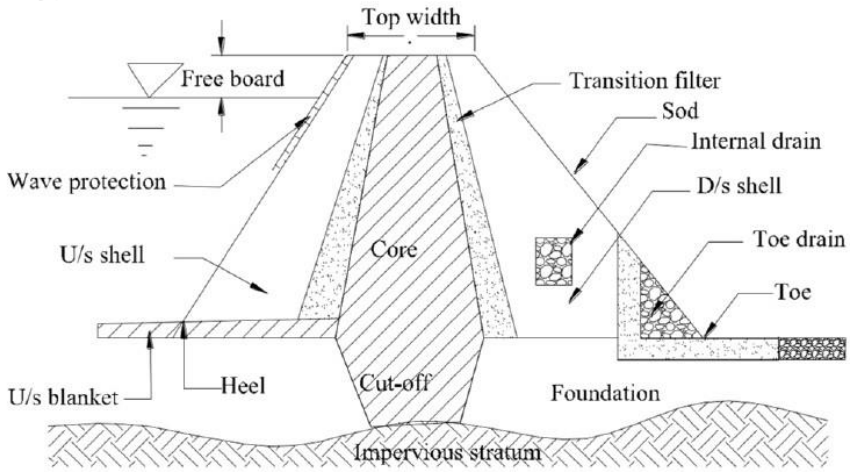
Various causes of failure of an earth dam have been discussed in the preceding section. The earth dam should be designed such that the failure of the dam or its foundation does not occur. The dam should be safe and stable during construction and throughout it life. For the safe design of an earth dam, the following basic criteria should be satisfied:

- **No overtopping**
  - The dam should be safe against overtopping during occurrence of the worst
  - An adequate free board should be provided so that the dam is not overtopped due to the wave action.
  - A suitable allowance in the height of the dam should be made to account for settlements.
- **No seepage failure**
  - The phreatic line (or the seepage line) should remain well within the downstream face of the dam so that no sloughing of the downstream face occurs.

- Seepage through the body of the dam, foundations and abutments should be controlled by adopting suitable measures.
- The dam and foundation should be safe against piping failure.
- There should be no opportunity for the free passage of water from the upstream to the downstream either through the dam or through the foundation.

- **No structural failure**
  - The upstream and downstream slopes should be safe during and immediately after construction.
  - The upstream slope should be safe during sudden-drawdown conditions.
  - The downstream slope should be safe during steady-seepage conditions.
  - The foundation shear stresses should be within the safe limits.
  - The dam as a whole should be earthquake-resistant
- **Proper slope protection**
  - The upstream slope should be protected against erosion by waves.
  - The downstream slope and the crest(i.e. top) should be protected against erosion due to rain and wind.
- **Proper drainage**
  - The portion of the dam downstream of the impervious core should be properly drained.
- **Economic section**
  - The dam should have an economic section. As far as possible, the materials available near the dam site should be used to reduce the cost.

4.4 PRELIMINARY SECTION OF EARTHEN DAM



4.5 SEEPAGE ANALYSIS

Seepage analysis is required to determine the quantity of water passing through the body of the earth dam and the foundation. It can be determined by theory of flow of fluids through porous media. Distribution of pore water pressure can also be obtained by the seepage analysis.

Assumptions made in seepage analysis:

- The soil in the embankment and foundation is incompressible and isotropic
- Water is incompressible
- Darcy's law is valid
- The flow is steady
- The soil is fully saturated
- The hydraulic boundary conditions at the entry and at the exit of the seepage flow are known.

Laplace equation --- $\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0$

Solution of the Laplace equation can be obtained by

- Graphical Method
- Experimental Method
- Analytical Method

4.5.1 GRAPHICAL METHOD

The graphical method makes use of the properties of the flow net. The net is drawn by trial and error. First a rough flow net is drawn and then it is modifies successively till a good flow net is obtained.

Flow net: Network of equipotential lines and flow lines

Properties of flow net are listed below:

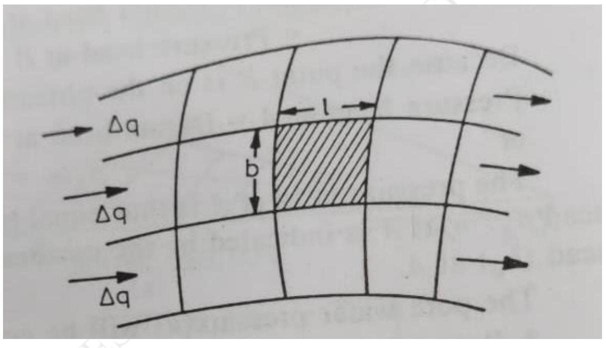
- The flow lines and equipotential lines meet at right angles to each other.
- Because the phreatic line is also a flow line, the equipotential lines intersect it at right angles.

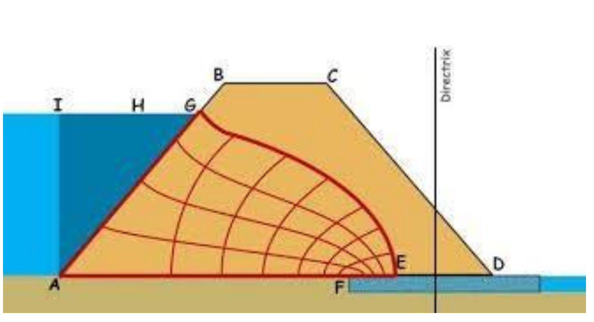
- Since the pressure at the phreatic line is zero, the successive equipotential lines make equal vertical intercepts on the phreatic line.
- The flow fields obtained by the intersections of the equipotential lines and the flow lines are approximately squares in shape. A circle can be approximately drawn in each square field touching all the sides of the square.
- The discharge between any two adjacent flow lines is constant.
- The potential drop between any two adjacent equipotential lines is also constant.
- The smaller the dimensions of the flow field, the greater is the hydraulic gradient and the velocity of flow through it.
- In a homogeneous soil, every transition in the flow lines and equipotential lines is smooth and gradual.

Flow net is used to determine the discharge through dam:

Figure shows a portion of the flow net. As already mentioned, the channel between the two successive flow lines is called the flow channel. The portion between two successive flow lines and two successive equipotential lines is called the flow field. It is approximately a square.

Let us consider flow through one field (shown hatched). Let b and I be, respectively, the width and length of the flow field. For the unit length of the earth dam, the distance perpendicular to the plane of the field is unity. From Darcy's law, the discharge through the field is given by





$\Delta q = k\ i\ A$

$\Delta q = k\ (\Delta h/l)\ (b \times 1) = k\ (\Delta h)\ (b/l) = k\ (h/N_d)\ (b/l)$

Δh – Head drop through the field

h – Total head causing flow

N_d – Number of equipotential drops

$q = \sum \Delta q = k\,(h/N_d)\,(b/l)\,N_f$

N_f – Total number of flow channels.

4.6 SEEPAGE CONTROL MEASURES

The water seeping through the body of the dam and foundation has the following adverse effects:

- Loss of water
- Pore water pressure
- Sloughing
- Piping

4.6.1 MEASURES TO CONTROL SEEPAGE THROUGH THE DAM

- Vertical core
- Inclined core

The minimum thickness of the core depends upon the following factors

- Permissible seepage through the dam
- Type of the material available for the core
- Design of the proposed filter
- Minimum practical thickness required for construction (b = 0.3 h to 0.5 h)

4.6.2 MEASURES TO CONTROL SEEPAGE THROUGH FOUNDATIONS

- Positive cutoff trench
- Concrete diaphragm
- Grout curtains
- Slurry trench cutoff
- Steel sheet piles
- U/S impervious blanket and relief walls
- Downstream loading berm

SPILLWAYS

4.7 SPILLWAYS

A spillway is a structure constructed at or near the dam site to dispose of surplus water from the reservoir to the channel downstream. Spillways are provided for all dams as a safety measures against overtopping and the consequent damages and failure. Spillway is thus safety valve for a dam.

Requirements of a Spillway are as listed below:

- The spillway must have sufficient capacity.
- It must be hydraulically and structurally adequate.
- It must be so located that it provides safe disposal of water.
- The bounding surfaces of spillway must be erosion resistant.
- Energy dissipater should be located in downstream side of the spillway for dissipation of energy.

Location of Spillway:

- Within the body of the dam.
- At one end of dam.
- Entirely away from it, independently in a saddle.

Required spillway capacity:

- Required spillway capacity is determined by flood routing.
- Spillway capacity should be equal to the maximum outflow rate determined by flood routing.

It requires the following data:

- Inflow hydrograph(plot of rate of inflow Vs time)
- Reservoir capacity curve(reservoir storage Vs reservoir water surface elevation)
- Discharge curve(rate of outflow through spillway Vs reservoir water surface elevation)

Factors affecting the required Spillway Capacity:

- Inflow flood hydrograph
- Available storage capacity
- Capacity of outlets
- Gates of spillways
- Possible damage, if the capacity is exceeded

4.8 COMPONENTS / PARTS OF SPILLWAY

- Approach channel
- Control structure
- Discharge carrier and discharge channel
- Terminal structure or Energy dissipator
- Exit channel

4.8.1 APPROACH CHANNEL

- Entrance structure or the path to draw water from reservoir and convey it to the control structure.
- It may be straight or curved in plan.
- Its banks may be parallel, convergent, divergent or combination of these and may be vertical or sloping.
- It may insure minimum head loss through the channel and to obtain uniformity of flow over the control structure.

4.8.2 CONTROL STRUCTURE

- Major component of spillway provided with bridge and gates.
- Regulates and controls the surplus water from the reservoir.
- It does not allow discharge of water below the fixed reservoir level.

4.8.3 DISCHARGE CARRIER AND DISCHARGE CHANNEL

- Discharge carrier is the waterway provided to convey the flows released from the control structure to the downstream side of spillway.
- The cross section may be rectangular, trapezoidal or of other shape.
- Waterway may be wide or narrow, long or short.
- Discharge channels are provided to convey the water from bottom of the discharge carrier to the downstream flowing river.
- It may be the downstream face of spillway itself.
- The width of discharge channel depends on amount of water to be conveyed.

4.8.4 TERMINAL STRUCTURE OR ENERGY DISSIPATOR

- Provided on downstream for dissipating the high energy of the flow, before the flow is returned to the river.

4.9 CLASSIFICATION OF SPILLWAYS

- Classification based on purpose
  - Main or service spillway
  - Auxillary spillway
  - Emergency spillway
- Classification based on control
  - Controlled or gated spillway
  - Uncontrolled or ungated spillway
- Classification based on prominent feature
  - Free overfall (or straight drop)
  - Overfall (or ogee spillway)
  - Chute (or open channel or trough)
  - Side channel spillway
  - Shaftwhich (or morning glory)
  - Siphon
  - Conduit (or tunnel)
  - Cascade spillway

4.9.1 CLASSIFICATION BASED ON PURPOSE

Main spillway

A main (or service) spillway is the spillway designed to pass a prefixed or the design flood. This spillway is necessary for all dams and in most of the dams, it is the only spillway.

Auxiliary spillway

It is provided as a supplement to the main spillway and its crest is so located that it comes into operation only after the floods for which the main spillway is designed are exceeded.

Total spillway capacity (Q) = Qm + Qa ,

Where, Q is the designed flood,

Qm is the capacity of main spillway,

Qa is the capacity of the auxiliary spillway

Emergency spillway

It is provided in addition to the main spillway but it comes into operation only during emergency which may arise at any time.

4.9.2 CLASSIFICATION BASED ON CONTROL

Controlled spillway

A controlled spillway is one which is provided with the gates over the crest to control the outflow from the reservoir. In the controlled spillway, the full reservoir level (F.R.L) of the reservoir is usually kept at the top level of the gates. Thus the water can be stored up to the top level of the gates.

Uncontrolled spillway

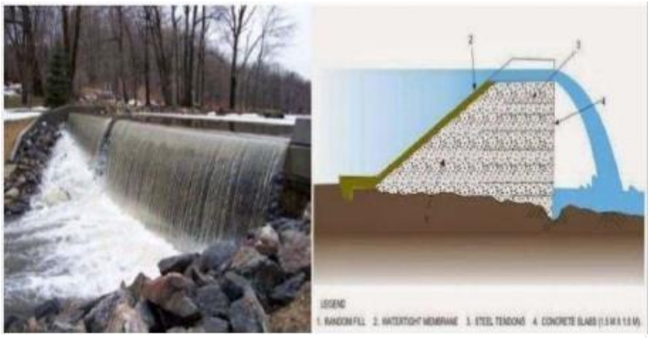
In an uncontrolled spillway the gates are not provided over the crest to control the outflow from the reservoir. The full reservoir level(F.R.L) is at the crest level of the spillway. The water escapes automatically when the water level rises above the crest level.

4.9.3 CLASSIFICATION BASED ON PROMINENT FEATURE

Free overfall (or straight drop)

- In this type of spillway, the water freely drops down from the crest. It is a low weir and simple vertical fall type structures.
- The water falls freely from the crest under the action of gravity.
- To prevent scouring at the downstream, an auxiliary dam of artificial pool is to be constructed at the place of fall of water.

- This type of spillway is not recommended for high head since the vibrations of falling-mjet might damage the structure.
- To direct the small discharge away frommthe face of the overfall section the crest is extended to form a overhanging clip.



Overfall (or ogee spillway)

- It represents the S-shape curve so, it is called ogee spillway.
- It is an improved form of straight drop spillway.
- It is mainly used in gravity dams.
- It has got the advantage over other spillways for its high discharging efficiency.
- The overflow water is guided smoothly over the crest so that water do not break the contact with the spillway surface.

Chute (or open channel or trough)

- It is often called as trough or open channel spillway.
- For earthen and rockfill dams, spillway is to be constructed separately away from the main valley.

- Chute Spillway is the simplest type of a spillway which can be easily provided independently and at low costs.
- It is lighter and adaptable to any type of foundations.

Side channel spillway

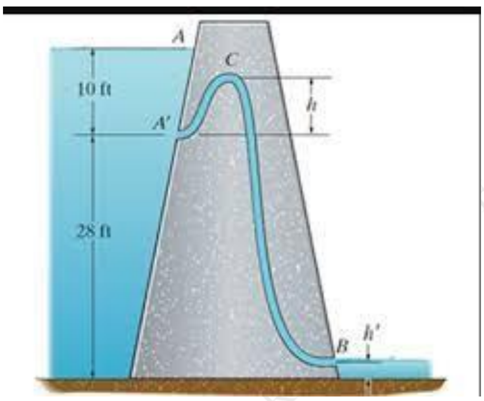
- The flow in this spillway is turned 90˚after passing the crest such that the flow is parallel to the weir crest.
- Best suitable for non rigid dams like earthen dams.
- It is preferred where space is not available for providing sufficient crest width for chute spillway.
- The discharge carrier may be an open channel type or a conduit type.

Shaftwhich (or morning glory)

- The water from the reservoir enters into a vertical shaft which conveys this water into a horizontal tunnel which finally discharges the water into the river downstream.
- This type of spillway is preferred where the space is not available for providing the above type of spillways
- If the inlet leg is provided in shape of a funnel, it is called Morning Glory Spillway.
- It has maximum discharge even at low heads.

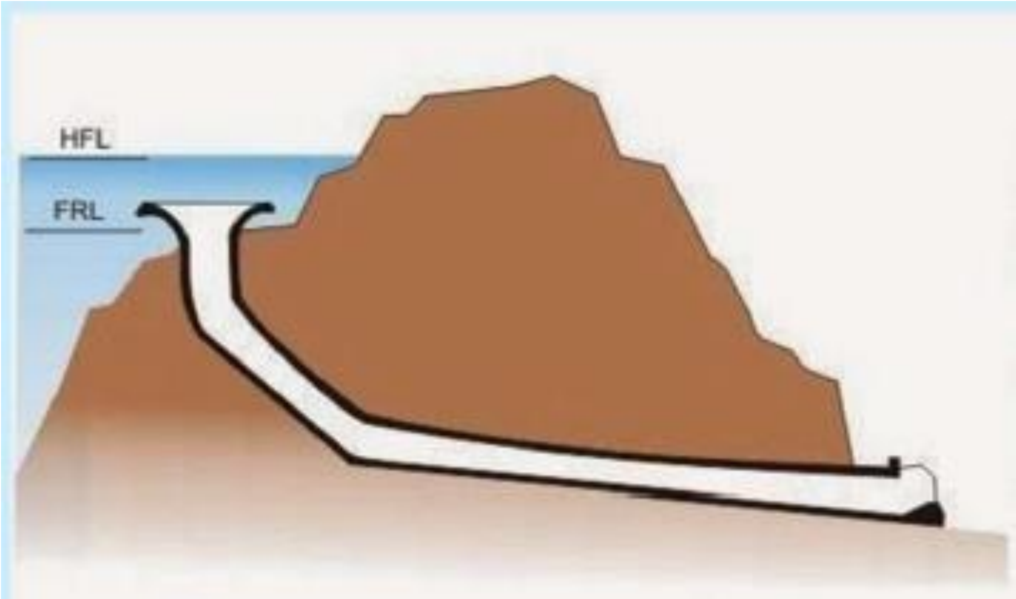

Siphon

- It works on the principle of syphonic action.
- It consists of a syphon pipe whose inlet leg is kept just below the normal pool level and an air vent kept at normal pool level is connected to the crown of syphon.
- When the water raises the pool level, syphonic action starts automatically and the water discharges to downstream side.
- When the water level falls below the pool level, air is entered through air vent and the discharging of water stops.



Conduit (or tunnel)

- A conduit Spill-way consist of a closed conduit to carry the flood discharge to the downstream channel . It is constructed in the abutment or under the dam.
- The closed conduit may take the form of a vertical or inclined shaft, a horizontal tunnel, or a conduit constructed in an open cut and then covered.
- To ensure the free flow in the tunnel, the ratio of flow area to total tunnel area is often limited to 75%of air vents are provided.
- Such a spill-way is suitable for dam sites in narrow canyons with steep abutments.



Cascade spillway

- A spillway consists of a cascade of falls, with a stilling basin at each fall.
- It is ideally suited for very high dams in which the energy cannot be dissipated by a hydraulic jump or a bucket. In the case of a high rockfill dams, already excavated quarry benches on d/s may be utilised for the formation of cascades.

4.10 DESIGN PRINCIPLES OF OGEE SPILLWAY

Ogee profile is acceptable as it provide

- Maximum possible hydraulic efficiency.
- Structural stability and economy.
- Avoid the formation of objectionable sub atmospheric pressures at surface.

For this shape, no negative pressure will develop on the spillway surface at the design head. Shape of the crest of the Ogee (overflow) spillway depends on:

- Head over the crest.
- Height of the spillway above the stream bed or the bed of the entrance channel.
- The inclination of the upstream face of the spillway.

Overflow or ogee spillways are classified as high & low depending on ratio value of 1.33

$$\frac{Height\ of\ the\ spillway\ crest\ from\ river\ bed\ (H)}{Design\ head\ (H_d)}$$

where,

Head (H): The distance between water surface to the crest axis.

Design head (H_d): It is the value of head for which ogee profile is designed.

Head due to velocity approach (H_a): It is the velocity head given by $Ha=Va^2/2g$

Total energy head (He): It is equal to the actual head plus the head due to velocity approach.

$$H_e = H + H_a, \quad \text{if } H=H_d,$$

$$H_e = H_d + H_a$$

Design criteria of downstream profile

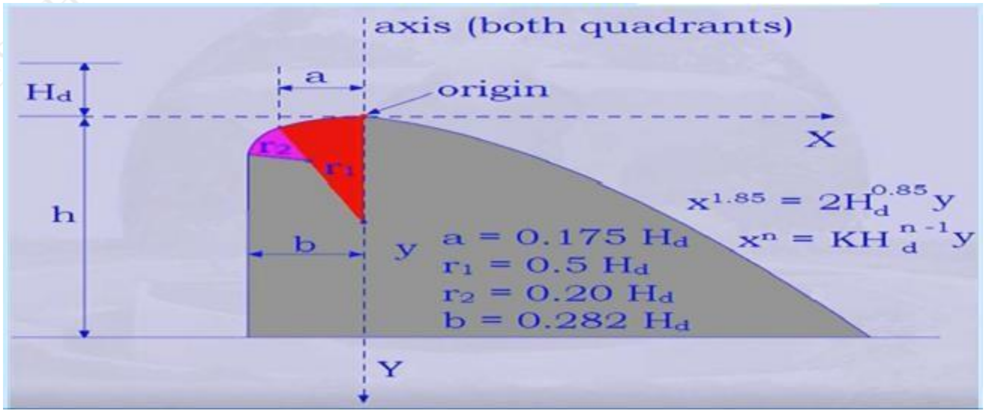
$$X^n = K\ H_d^{n-1}\ Y$$

Where,

X and Y are the coordinates of the point on the spillway surface, with the origin at the highest point of the crest,

H_d is the design head, excluding the head due to the velocity of approach,

K and n are constants, depends upon the inclination of the upstream face of the spillway.



Design criteria of upstream profile

$$Y = \frac{0.724\,(x + 0.27\,H)^{1.85}}{H^{0.85}} + 0.126\,H \; - 0.4315\,H^{0.375}\,(x + 0.27\,H)^{0.625}$$

It should be noted that u/s curve at crest should neither be made too sharp nor too broad.

- Broad crest supports the lower nappe, produces positive hydrostatic pressure & reduces discharge.
- If the curve is sharp the nappe leave the ogee profile, causing negative pressure & cavitation and increases discharge.

Discharge computation for an ogee spillway

$$Q = C_d\,L_e\,H_e^{3/2}$$

where,

Q is discharge(cumecs),

C_d is the coefficient of discharge,

L_e is the effective length

H_e is the actual effective head including the head due to the velocity of approach, i.e. $H_e = H_d + H_a$.

Effective length of crest

$$L_e = L' - 2(NK_p + K_a)\,H_e$$

where,

L_e is the effective length of crest,

L' is the net length of crest,

H_e is the actual total head of flow on crest,

N is the number of piers,

K_p is the pier contraction coefficient,

K_a is the abutment contraction coefficient

4.11 SPILLWAY GATES

Gates are placed to control the flow of water from the crest. This gives the spillway a controlled spillway. Types of spillway gates:

- Flash boards, stop logs and needles
- Radial gates
- Drum gates
- Vertical lift gates
- Bear trap gates
- Rolling gates

4.11.1 FLASH BOARDS, STOP LOGS AND NEEDLES

Flash Boards

- Flash boards are wooden boards or panels, placed side by side, on the crest of the spillways to form a continuous shutter.
- These are simplest and oldest types of gates.
- These are quite efficient and economical for small heights where they can be readily handled by the lifting arrangements.
- The flash boards can be temporary or permanent.

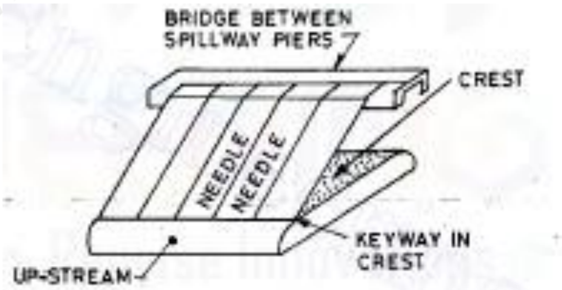
Stop Logs

- Stop logs are horizontal wooden timber beams which span the space between grooved piers constructed on the crest of the spillway.
- Stoplogs are pushed down into the grooves from top, one over the other.

- The logs may be raised by hands or with a hoist.
- Stoplogs are generally used for small spillways.

Needles

- Needles are wooden logs placed in an inclined position, with their lower ends resting in a keyway on the spillway crest and their upper ends supported on a bridge.
- These are placed and removed by hands.
- Needles are somewhat easier to remove than stop logs, but are more difficult to place in flowing water.
- Needles are also used for small spillways and weirs.

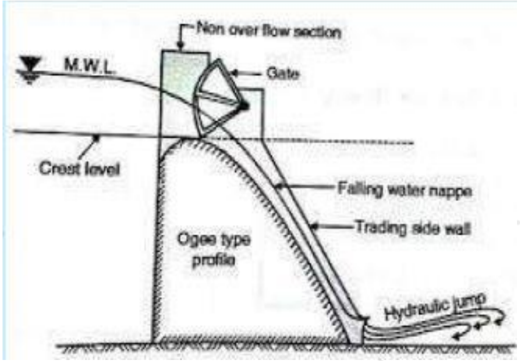


4.11.2 RADIAL GATES

- A tainter gate(or radial gate) has the upstream face cylindrical. The axis of the segment of the cylinder forming the u/s face is horizontal.
- The face is formed of a steel skin plate shaped to a segment of a hollow cylinder supported on a steel framework.
- Radial gates have a number of advantages over the vertical gates and are quite popular. The friction in the radial gates is concentrated at the pins and is usually much less than that in the vertical gates.
- Because of the face of radial gates is cylinder, the water pressure acts normal to the face and the resultant water pressure passes through its centre.

- These gates have been used up to 15m height and 20m span.
- These are usually more economical than the vertical gates of the same size.

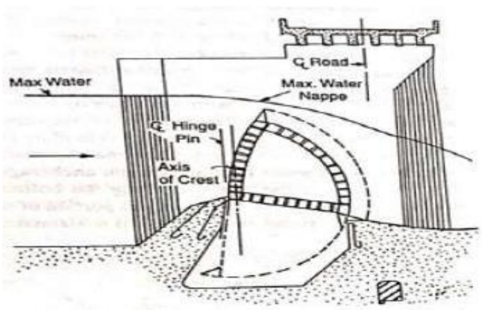
Radial gates



4.11.3 DRUM GATES

- A drum gate consists of a segment of cylinder of such a shape that when the gate is in the lowered position, it fits in a recess made in the top portion of the spillway, and the flood water passes over it.
- The drum gates is formed by fixing skin plates to an internal bracing systems. It is hinged on its upstream edge at the centre of curvature to the spillway crest.
- This type of gate can also be designed for automatic operation. Drum gates are suitable for long spans with moderate heights.

Drum gates



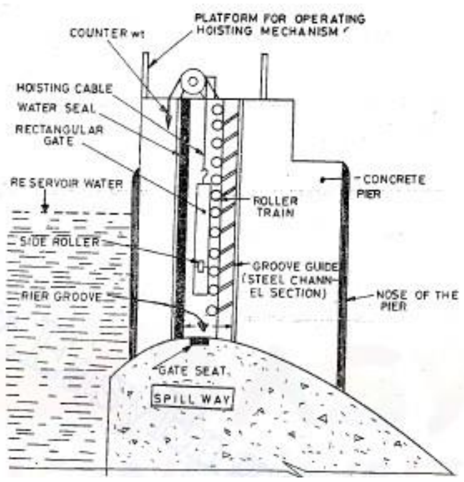
4.11.4 VERTICAL LIFT GATES

- Vertical rectangular gates are commonly used for spillways.
- A vertical lift gate consists of a vertical framework fabricated of steel members.
- A steel skin plate is fixed on the upstream side of the steel framework. The vertical gate can move vertically on its own plane in the grooves provided in piers.

- The vertical gates are raised or lowered by a hoisting arrangement through cables attached to them. These gates are usually provided with counterweights to reduce the lifting force.

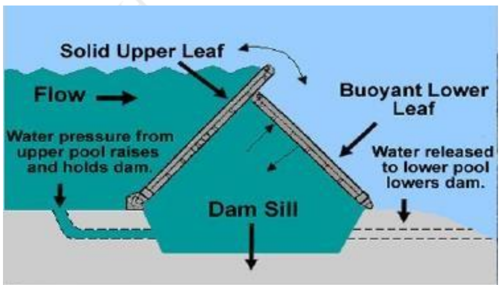
The vertical gates are mainly of the following types:

- Sliding gates
- Stoney gates
- Fixed- wheel gates

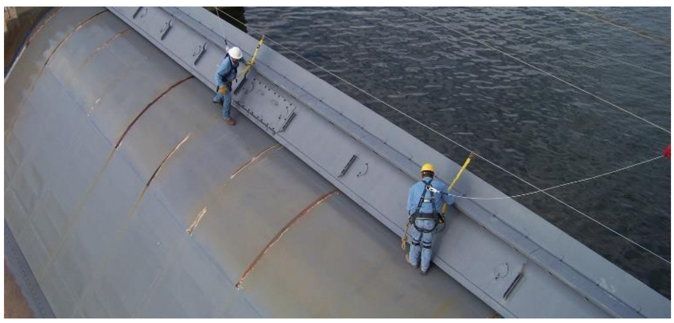


4.11.5 BEAR TRAP GATES

- Bear trap gates are also known as movable drum gates.
- A bear trap gate consists of two leaves of steel, with one leaf hinged on the upstream side and the other on the downstream side over the crest.
- The bear trap gates are suitable for low navigation dams.
- These gates have a wide base but a deep recess is not required.
- These gates may be designed to operate automatically.

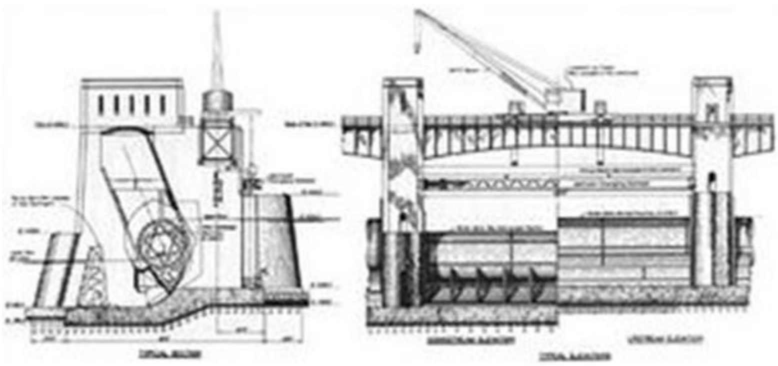


Bear trap gate



4.11.6 ROLLING GATES

- A rolling gate consists of a hollow steel cylinder or drum which spans between the piers constructed on the crest of the spillway.
- At each end of the hallow cylinder, there is a heavy annular rim having gear teeth on its periphery.
- Each pier has an inclined rack with teeth, which engages gear teeth encircling the ends of the cylinder. Hoisting cables are attached to the ends of the gate; the other ends of these cables are connected to a winch installed in the hoist room above the gates. When a pull is exerted on the hoisting cable, the gate rolls up the rack.
- The lower portion of the gate is attached with a cylinder segment projecting out from the cylindrical surface of the gate. When the gate is in the raised position, the cylindrical segment makes contact with the spillway crest and thus the overall height of the gate is increased.
- Rolling gates quite suitable for long spans of moderate heights. These gates have been used upto spans of 45 m and the height of 7 m.



ENERGY DISSIPATERS AND STILLING BASINS

4.12 ENERGY DISSIPATION BELOW SPILLWAYS

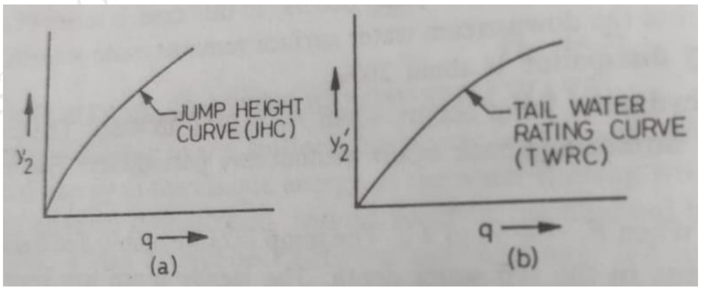
Water flowing over a spillway has a very high kinetic energy because of the conversion of the entire potential energy to the kinetic energy. If the water flowing with such a high velocity is discharged directly into the channel downstream, serious scour of the channel bed may occur. If the scour is not properly controlled, it may extend backward and may endanger the spillway and the dam. In order to protect the channel bed, it is discharged into the d/s channel. The energy-dissipating devices can be broadly classified into two types.

- Devices using a hydraulic jump for the dissipation of energy.
- Devices using a bucket for the dissipation of energy.

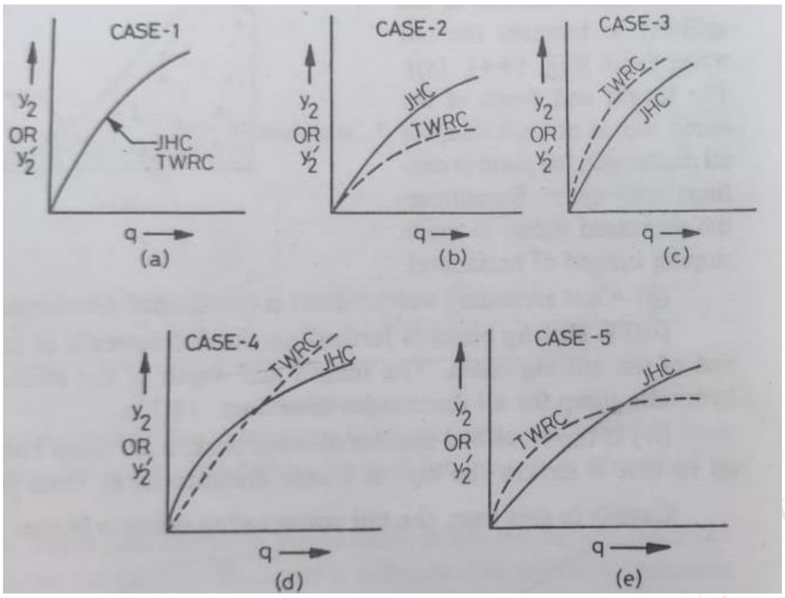
The choice of the energy-dissipating device at a particular spillway is governed by the tail water depth and the characteristics of the hydraulic jump, if formed, at the toe. If the tail water depth at the site is not approximately equal to that required for a perfect hydraulic jump, a bucket-type energy dissipating device is usually provided.

Jump Height Curve (JHC): A plot between discharge against depth of water after hydraulic jump.

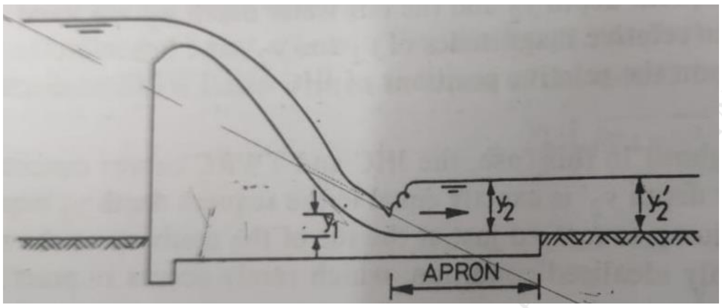
Tail Water Rating Curve (TWRC): A plot between discharge against actual depth of tail water (depth of water in the river in downstream).



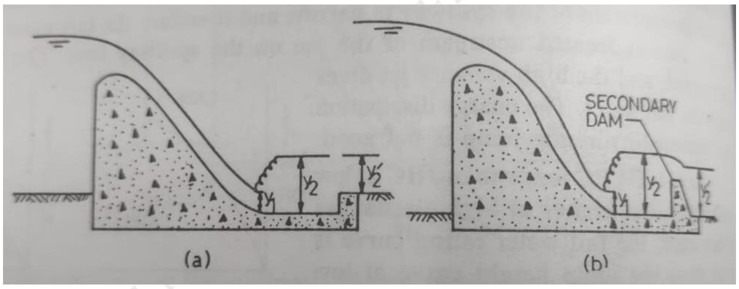
The location of hydraulic jump will depend upon JHC and TWRC. There are five cases, depending upon the relative positions of JHC and TWRC as shown below.



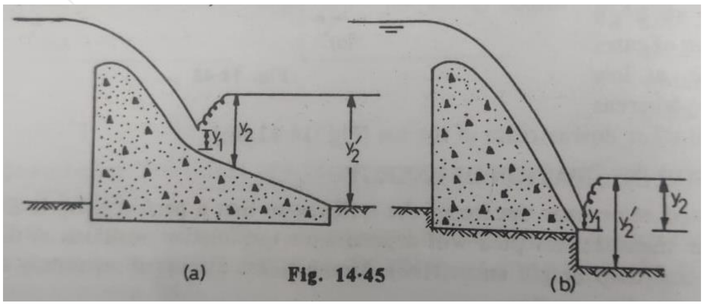
Case 1



Case 2



Case 3



Case 4

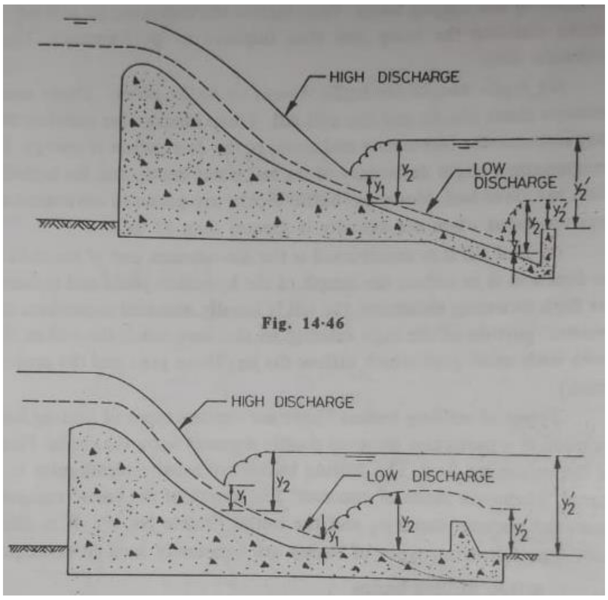
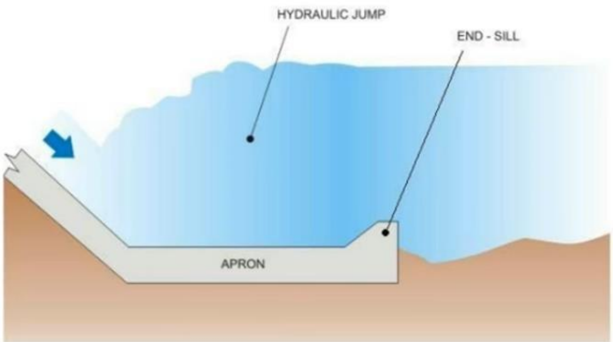
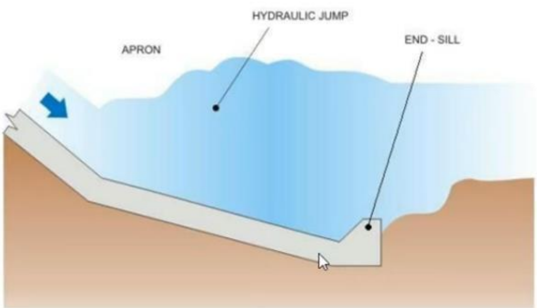


Fig. 14·46

4.13 STILLING BASIN

A stilling basin is a basin-like structure in which all or a part of the energy is dissipated. In a stilling basin, the kinetic energy causes turbulence and it is ultimately lost as heat energy. The stilling basing commonly used for spillways are of the hydraulic jump type, in which dissipation of energy is accomplished by a hydraulic jump.

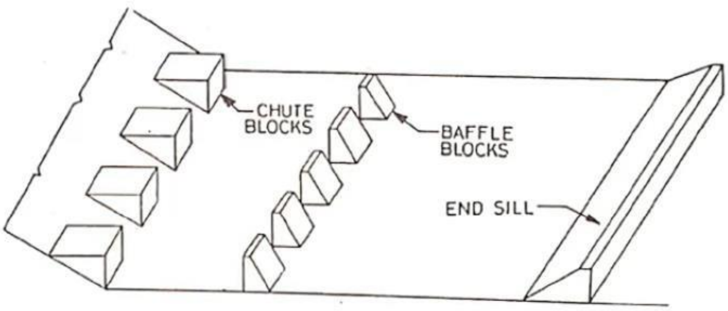




A hydraulic jump can be stabilised in the stilling basin by using appurtenances (or accessories) such as chute blocks, basin blocks and end sill.

- **Chute blocks**: These are triangular blocks with their top surfaces horizontal. These are installed at the toe of the spillway just at upstream end of the stilling basin. They act as a serrated device at the entrance to the stilling basin. They furrow the incoming jet and lift a portion of it above the floor. These blocks stabilise the jump and thus improve its performance. These also decrease the length of the hydraulic jump.

- **Basin blocks (or baffle blocks or baffle piers)**: These are installed on the stilling basin floor between chute blocks and the end sill. These blocks also stabilise the formation of the jump. Moreover, they increase the turbulence and assist in the dissipation of energy. For low flows, baffle blocks also help compensate a slight deficiency of the tail water depth, and for high flows, they help deflect the flow away from the river bed. However, baffle blocks are prone to cavitation on the downstream face, and are not recommended when the velocity is greater than 15 m/s.
- End sill It is constructed at the downstream end of the stilling basin. It may be solid of dentated. Its function is to reduce the length date and to control scour. For large bar diffusing for high incoming velocities, the sill is usually dentated to perform an additional function of dil thing the residual portion of the high velocity jet that may reach the end of the basin. In a dentated sill, there are teeth with small gaps which diffuse the jet. (These gaps and the projections between them look like human teeth).



4.14 TYPES OF STILLING BASINS

There are various types of stilling basins. The type of stilling basin most suitable at a particular location mainly depends upon the initial Froude number (F_1) and the velocity V of the incoming flow. The stilling basins are usually rectangular in plan. However, sometimes these are flared. These are made of concrete. The length of the basin, measured in the direction of flow, depends upon the sequent depth y_2 and the initial Froude No. F_1. It is different for different type of basins.

The following types of basins are commonly used in practice.

- U.S.B.R. Stilling basins
    - Type I basin
    - Type II basin
    - Type III basin

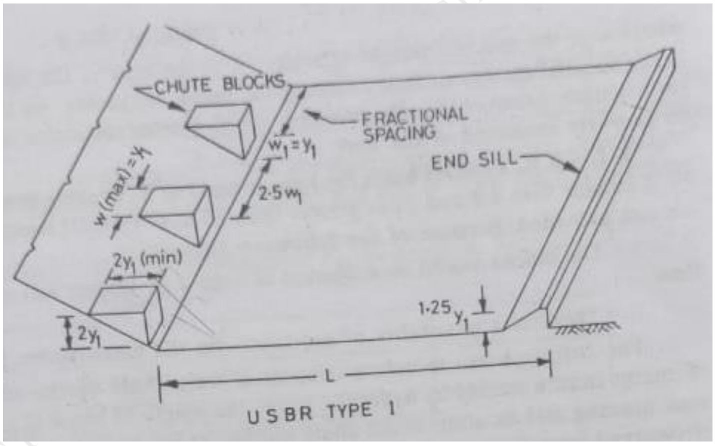
- Indian Standards basins
    - Horizontal floor - Type I
    - Horizontal floor - Type II
    - Sloping apron - Type III
    - Sloping apron - Type IV

4.14.1 U.S.B.R. STILLING BASINS

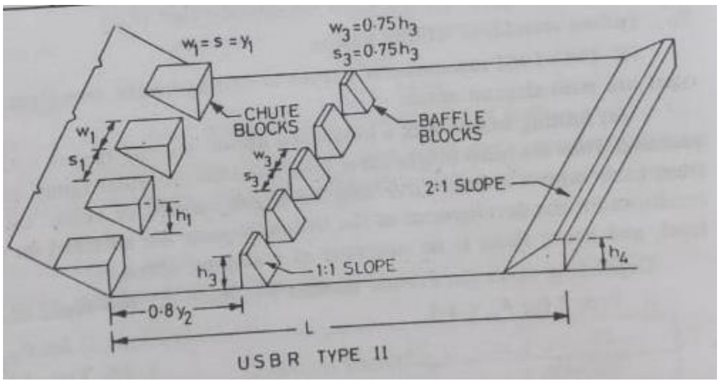
No special stilling basin is required to still flow if F_1 is less than 1.70. However, the channel length beyond the point from where the water depth starts increasing, should not be less than 4.00 y_2, where y_2 is the sequent depth.

For F_1 between 1.7 to 2.5, there is not much turbulence, only a horizontal apron is provided. However, the apron should be sufficiently long to contain the jump. A length of 5.0 y_2 is usually provided. No accessories such as baffles or sills are provided.

Type I: F_e b/w 2.5 to 4.5



Type II: F_e > 4.5 and Velocity of flow < 15 m/s



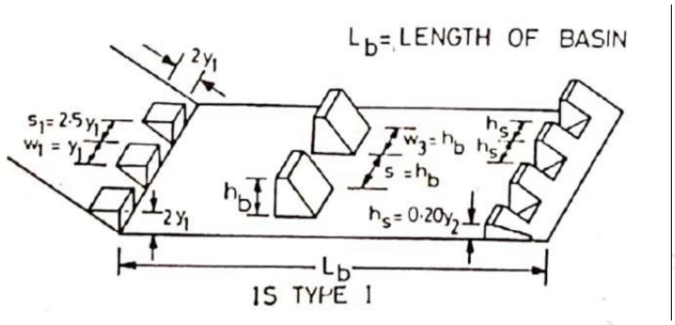
Type III: F_e > 4.5 and Velocity of flow > 15 m/s

4.14.2 INDIAN STANDARDS BASINS

IS: 4997 – 1968 recommends 4 types of stilling basins, Type I & II are with horizontal apron, and Type III & IV are with sloping apron.

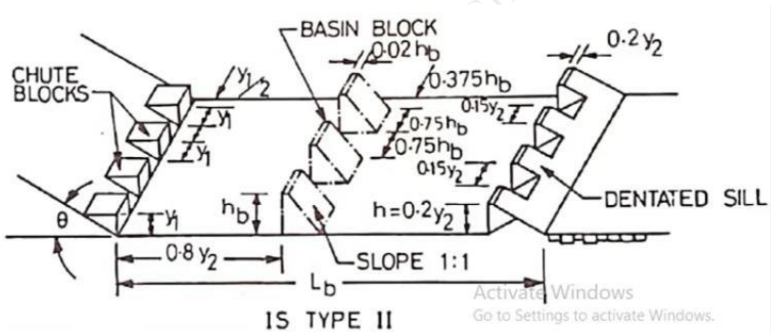
Type I: $F_e < 4.5$ and Velocity of flow < 15 m/s

Weirs, barrages and low dam

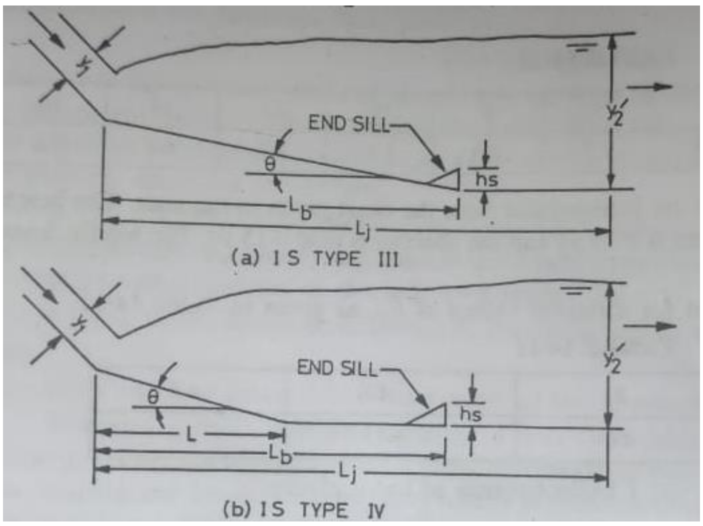


Type II: $F_e >= 4.5$ and Velocity of flow > 15 m/s

Medium and high dam



Type III and Type IV: Provided when actual tail water depth is greater than depth of water after hydraulic jump.



Recommended Questions

1. With a neat sketch explain the different types of earthen dams
2. Discuss in brief the causes of failure of earthen dam
3. What are the criteria for safe design of earth dam?
4. Explain the method of plotting flownet for seepage by graphical analysis
5. Write a note on seepage control measures
6. What is a spillway? What are its functions? Enumerate the various types of spillways
7. Write a note on ogee shaped spillway with its design principle
8. Explain components / parts of spillway
9. Enumerate the various classification of spillway gates
10. Write a note on energy dissipaters
11. Explain the different types of USBR stilling basins
12. Explain the different types of Indian standard stilling basins

Module – 5: Diversion Head works

A diversion headwork is a structure constructed across a river for the purpose of raisin water level in the river so that it can be diverted into the off taking canals (or) Any hydraulic structure which supplies water to the off-taking canal is called a headwork (or) The works which are constructed at the head of the canal, in order to divert the river water towards the canal, so as to ensure a regulated continuous supply of silt-free water with a certain minimum head into the canal, are known as diversion head works.

The canal head works may be classified into the following two types:

- Storage Head works: A storage headwork comprises the construction of a dam across the river. It stores water during the period of excess supplies in the river and releases it when demand overtakes available supplies.
- Diversion Head works: To divert required supply to canal from the river. They are two types.
    - Temporary spurs or bunds: which are temporary and constructed every year after floods.
    - Permanent weirs and barrages

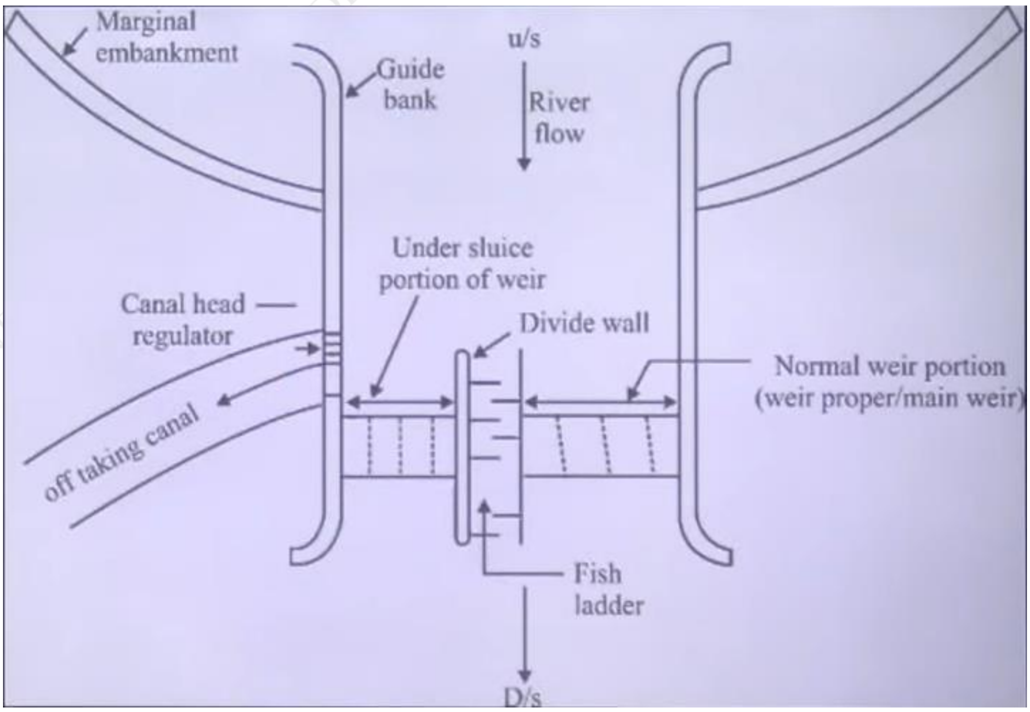
5.1 PURPOSE SERVED BY DIVERSION HEAD WORKS

- It rises the water level in the river.
- Divert the required quantity into the canal.
- Controls the entry of silt into canals.
- Creates a small pond (not reservoir) on its upstream and provides some pondage.
- It reduces the fluctuations in the level of supply in the river.

5.2 IDEAL SITE FOR DIVERSION HEAD WORKS

- The river section at the site should be narrow and well defined.
- The river should have high, well defined, inerodible and non-submersible banks so the cost of the river training works is minimum.
- The canals taking off from the diversion head works should be quite economical and should have a large commanded area.
- There should be suitable arrangement for the diversion of river during construction.
- The site should be such that the weir or barrage can be aligned at right angles to the direction of flow in the river.
- There should be suitable locations for the under sluice, head regulator and other com- ponents of the diversion head works.
- The diversion head work should not submerge costly land and property on its up- stream.
- Good foundation should be available at the site.
- The required materials of construction should be available near the site.
- The site should be easily accessible by road or rail.
- The overall cost of the project should be a minimum.

5.3 LAYOUT OF A DIVERSION HEAD WORKS



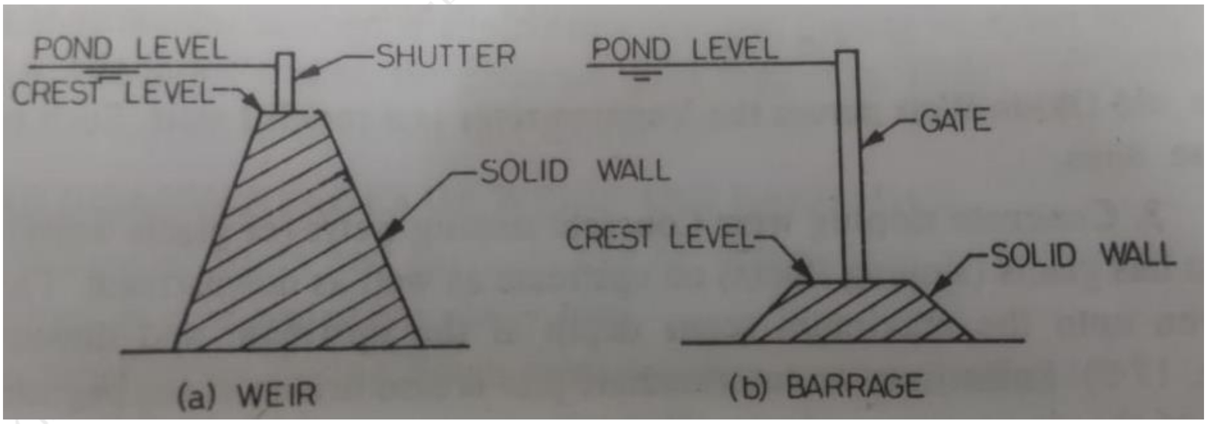
The components of diversion head works are as follows:

- Weir or barrage
- Undersluice
- Divide wall
- Fish ladder
- Canal head regulator
- Silt excluder
- Guide bunds
- Marginal bunds

5.3.1 WEIR OR BARRAGE

Weir: The weir is a solid obstruction put across the river to raise its water level and divert the water into the canal. If a weir also stores water for tiding over small periods of short supplies, it is called a storage weir. The main difference between a storage weir and a dam is only in height and the duration for which the supply is stored.

Barrage: The function of a barrage is similar to that of weir, but the heading up of water is affected by the gates alone. No solid obstruction is put across the river. The crest level in the barrage is kept at a low level.



Types of weirs

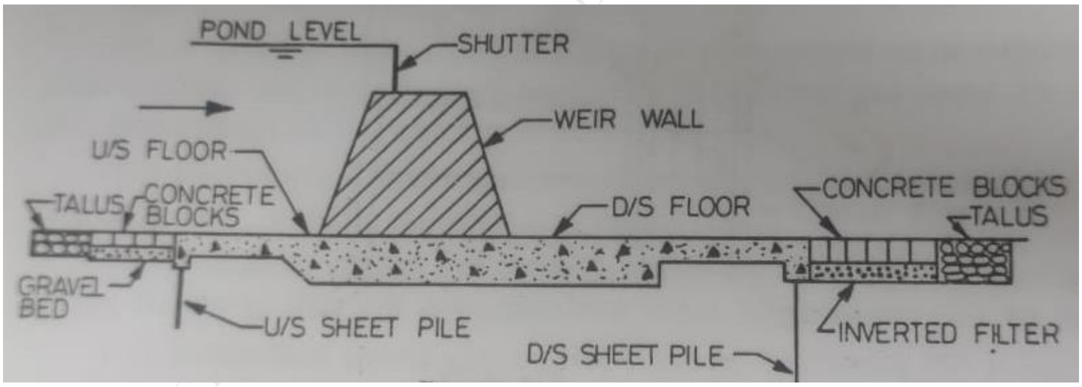
- **Gravity Weir:** When the weight of the weir balances uplift pressure caused by the head of water seeping below weir.
- **Non-gravity Weir:** The uplift pressure is largely resisted by weight of concrete slab with the weight of divided piers.

Depending on the materials and design features, gravity weirs are subdivided into following types:

- Masonry weirs with vertical drop or vertical drop weirs.
- Rockfill weirs with sloping aprons.
- Concrete weirs with downstream glacis.

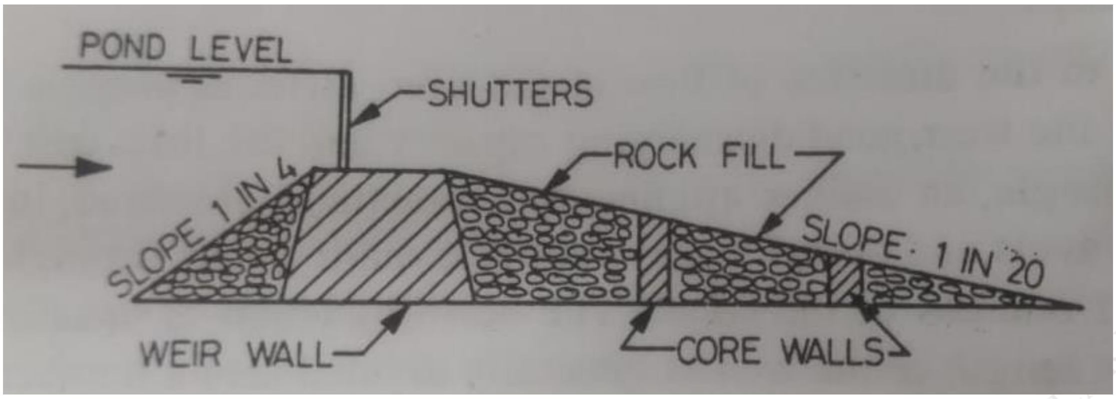
Masonry weirs with vertical drop or vertical drop weirs: This type of weir is suitable for any type of foundation.

- An impervious horizontal floor or apron
- A masonry weir wall (with both upstream and downstream faces vertical; or both faces inclined; or upstream face vertical and downstream face inclined)
- Block protection at upstream end of floor, and a graded inverted filter at the downstream end of floor
- Launching aprons or pervious aprons (or floors) after block protection and inverted filters.

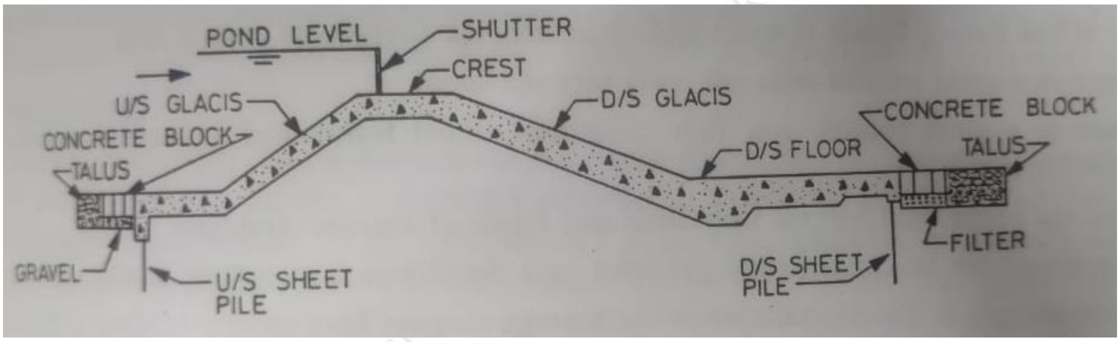


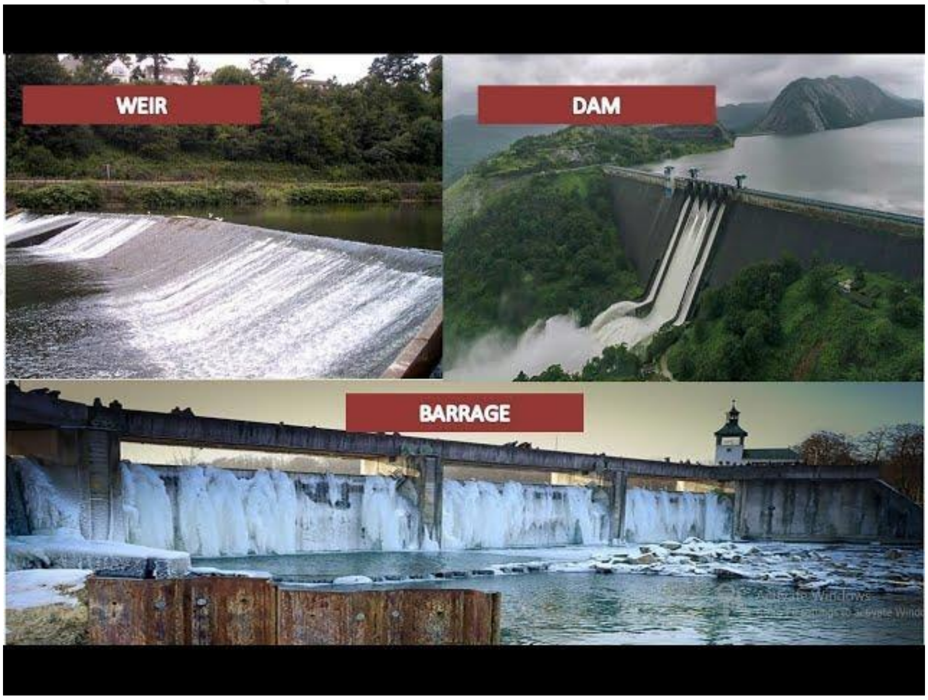
Rockfill weirs with sloping aprons: It is the simplest type of construction.

- Masonry weir wall
- Dry packed boulders laid in the form of glacis or sloping aprons in the upstream and downstream sides of the weir wall
- The downstream slope is generally made very flat. It requires a very large quantity of stone. It also has few intervening core walls.



Concrete weirs with downstream glacis: It is of recent origin and its design is based on sub-surface flow concept. Hydraulic jump is developed on the glacis due to which considerable energy is dissipated. Protection works such as inverted filter; block protection and launching apron are provided. May be constructed on pervious foundation. Sheet piles of sufficient depths are provided both at upstream and downstream ends of the floor.





5.3.2 UNDERSLUICE OR SCOURING SLUICES

They are the openings provided in the weir wall with their crest at low level. The openings are fully controlled by gates. They are located on the same side of the off-taking canal.

Functions of Undersluices:

- They preserve a clear and well defined river channel towards the canal head regulator;
- They scour the silt deposited on the river bed in the pocket upstream of the canal head regulator;
- They pass low floods without the necessity of dropping the weir crest shutters;
- They help to lower the high flood level by supplementing the discharge over the weir during high floods.

Capacity of Undersluices:

The discharging capacity is fixed from the following considerations:

- To ensure proper scouring, its capacity should be at least two times the maximum discharge of the off-taking canal;
- It should have sufficient capacity to discharge maximum winter flood – without the necessity of dropping the weir shutter;
- 10 to 20% of the maximum flood discharge – to supplement the discharge over the weir during high floods.

5.3.3 DIVIDE WALL

It is masonry or concrete wall with top width of 1.5 to 3m constructed at right angles to the axis of the weir and separates the 'weir proper' from under sluices. The divide wall extends on the upstream side beyond the beginning of the canal head regulator and on the down-stream side, it extends up to the end of downstream protection of the under sluices.


The main functions of a divide wall are:

- To separate the floor of undersluice which is at lower level from the weir proper.
- To help in proving the comparatively less turbulent pocket near the canal head regulator resulting in deposition of silt in this pocket and, thus, to help entry of siltfree water into the canal;
- To isolate the pocket upstream of the canal head regulator and facilitate scouring operation;
- To prevent formations of cross-currents to avoid their damaging effects on the weir.

5.3.4 FISH LADDER

This structure enables the fish to pass upstream. It is device by which the flow energy can be dissipated in such a manner as to provide smooth flow at sufficiently low velocity, not exceeding 3 to 3.5m/s. This object is generally accomplished by providing a narrow opening adjacent to the divide wall and provide suitable baffles or staggering devices in it, so as to control the flow velocity.


The various types of fish ladder are: (i) pool type, (ii) steep channel type, (iii) fish lock type and (iv) fish lift or elevator type.

Types (iii) and (iv) are suitable for high dams only. Types (i) and (ii) are generally provided for barrages.

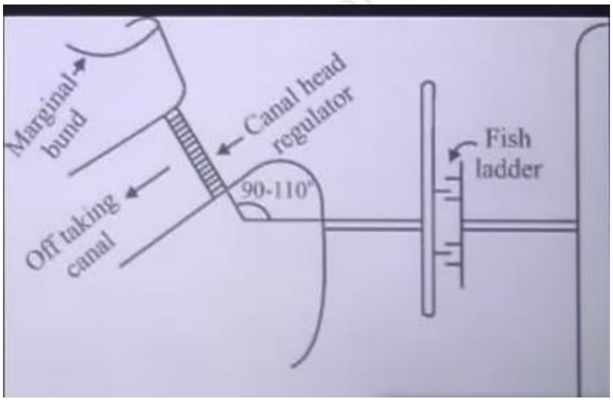
5.3.5 CANAL HEAD REGULATOR

The structure controlling diversion into a supply canal is called regulator. It is provided at the head of the offtaking canal and serves the following functions:

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

The head regulator is generally aligned at right angle to the weir, but slightly larger angles (between 90^0 and 110^0) are now considered preferable for providing smooth entry of water into the regulator. The regulation is done by means of gates.

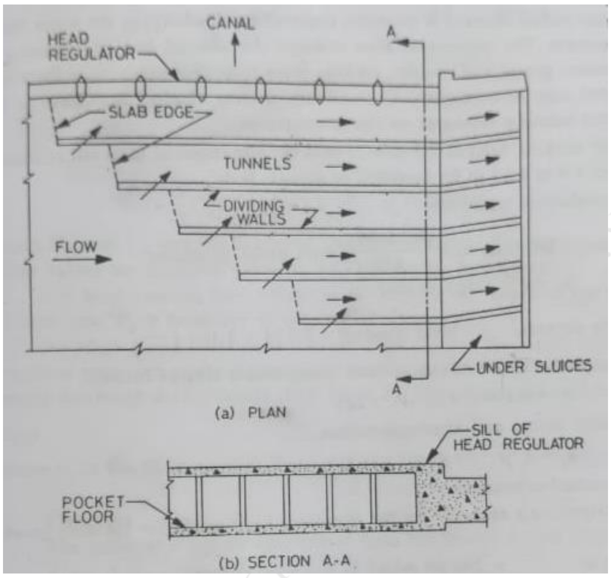
The design principles are the same as those used in the design of barrages, except that the regulators are a smaller version of barrages. An important consideration in designing the regulator is silt exclusion from canals. Silt-excluder tunnels are often provided in the barrage bays adjacent to the regulator, so that the heavier silt-laden bottom layers of water bypass through the tunnels.



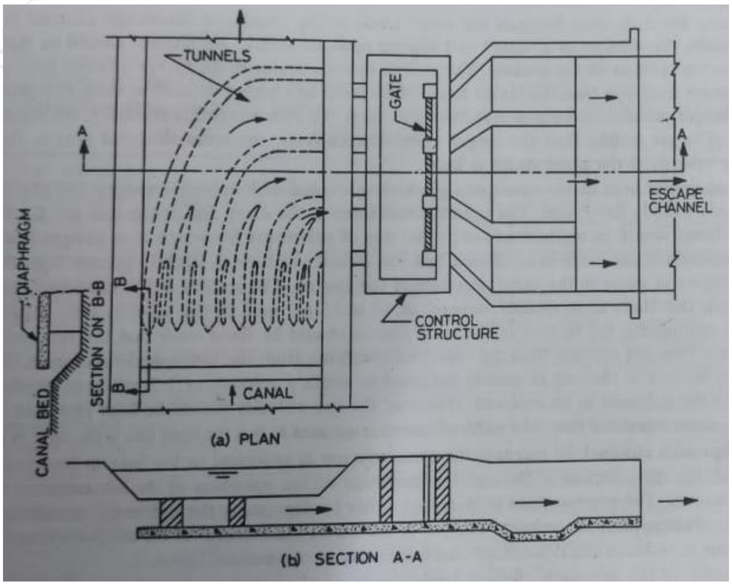
5.3.6 SILT EXCLUDER AND SILT EJECTORS

Silt excluders and ejectors are structures used in canals and rivers to control the amount of silt (sediment) entering the water system. Silt excluders are placed upstream of the head regulator to remove silt from the water before it enters the canal, while silt ejectors are located downstream in the canal to extract silt that has already entered.

Silt Excluder: These structures are built on the river bed, upstream of the head regulator (the point where water is diverted into the canal). They are designed to remove silt from the water before it flows into the canal. This helps prevent silt from accumulating in the canal and potentially clogging it or reducing its efficiency.



Silt Ejector: Silt ejectors are placed in the canal, a short distance downstream from the head regulator. They are used to remove silt that has already entered the canal and is being carried downstream. By removing silt from the canal, ejectors help maintain water flow and prevent silt buildup, which can lead to decreased canal capacity and potential damage to infrastructure.



5.3.7 GUIDE BUNDS

Guide banks direct the main river flow as centrally as possible to the diversion structure. They also safeguard the barrage from erosion and may be designed so that a desirable curvature is induced to the flow for silt exclusion from the canals. The side slopes of the guide banks must be protected by stone pitching, with a sufficient 'self-launching' stone apron at the lowest feasible level. The top levels of the guide banks will depend on the increase in the maximum flood level upstream of the barrage.

5.4 CAUSES AND FAILURE OF WEIRS AND BARRAGES ON PERMEABLE FOUNDATIONS

- Due to seepage or subsurface flow
    - By piping or undermining
    - By uplift pressure
- Failures due to surface flow
    - By suction due to standing wave or hydraulic jump
    - By scour on the upstream and downstream of the weir

5.4.1 BY PIPING OR UNDERMINING

If the water percolating through the foundation has sufficient force when it emerges at the downstream end of the impervious floor it may lift up the soil particles at the end of the floor. With the removal of the surface soil there is further concentration of flow into the resulting depression and more soil is removed which progressively result in subsidence of the floor in the hollows so formed.

To prevent these kinds of failures:

- Provide sufficient length of the impervious floor (so that the path of percolation is increased) and reduce exit gradient.
- Provide piles at upstream and downstream ends of the impervious floor

5.4.2 BY UPLIFT PRESSURE

If the uplift pressure is not counterbalanced by the weight of the floor, it may fail by rupture.

To prevent failure by uplift:

- Provide sufficient thickness of the impervious floor
- Provide pile at the upstream end of the impervious floor so that uplift pressure is reduced on the downstream side.

5.4.3 BY SUCTION DUE TO STANDING WAVE OR HYDRAULIC JUMP

The standing wave or hydraulic jump developed on the downstream side of the weir causes suction or negative pressure which also acts in the direction of uplift pressure. If the floor thickness is insufficient it may fail by rupture in suction.

The following measures may be taken to prevent such kind of failure:

- Providing additional thickness of the impervious floor to counterbalance the suction pressure due to standing wave.
- Constructing floor as monolithic concrete mass instead of in different layers of masonry.

5.4.4 BY SCOUR ON THE UPSTREAM AND DOWNSTREAM OF THE WEIR

Upstream and downstream ends of the impervious floor and bed of the river may be scoured during floods. If not prevented, lead to damage to the floor and an ultimately failure.

Preventive measures which should be taken against failure due to scour are:

- Providing deep piles both at upstream and downstream ends of the impervious floor. The piles should be driven much below the calculated scour depth.
- Providing launching aprons of suitable length and thickness at upstream and downstream ends of the impervious floor.

5.5 DESIGN OF IMPERVIOUS FLOOR FOR SUBSURFACE FLOW – THEORIES OF SUBSURFACE FLOW

- Hydraulic Gradient Theory
- Bligh's Creep Theory
- Lane's Weighted Creep Theory
- Khosla's Theory

5.5.1 HYDRAULIC GRADIENT THEORY

Upto the end of 19th century, the various irrigation structures were designed by empirical methods based on experience and intuition. Some of these structure failed because of subsurface flow. Following the damages of Khanki weir in 1895, Lt. Col. Clibborn, who was the Princiral of Thomson College, Roorkee, India, conducted experiments on flow of water through soils to study the subsurface flow and to evolve criteria for the safe design of weirs on permeable foundation. His experiments confirmed that the Darcy law is applicable except under very high heads. As a result of these experiments, in 1902, the hydraulic gradient theory of the subsurface flow was accepted for the design of irrigation structures.. According to this theory, the hydraulic gradient in the structure should be less than the allowable value. On the basis of these experiments and certain field observations at Narora weir, it was established that the subsurface flow (or foundation seepage) may cause the failure of the impervious floor either by piping or by uplift pressure.

5.5.2 BLIGH'S CREEP THEORY

In 1910, W.G. Bligh went a step forward and gave creep theory. According to this theory, the percolating water creeps along the cont et surface of the base profile of the structure with the subsoil. The length of the path thus traversed by the percolating water is called the length of creep or the creep length. As the water creeps from the upstream end to the downstream end, the head loss occurs. The head loss is proportional to the creep distance travelled.

5.5.3 LANE'S WEIGHTED CREEP THEORY

Lane analysed a large number of dams and weirs founded on pervious foundations which failed or did not fail. He brought out deficiencies in Bligh's creep theory and gave a new theory on statistical basis. The theory is known as Lane's weighted creep theory. This theory gives the vertical creep three times more weightage as compared to the horizontal creep. In other words, it is assumed that the vertical creep is three times more effective than the horizontal creep.

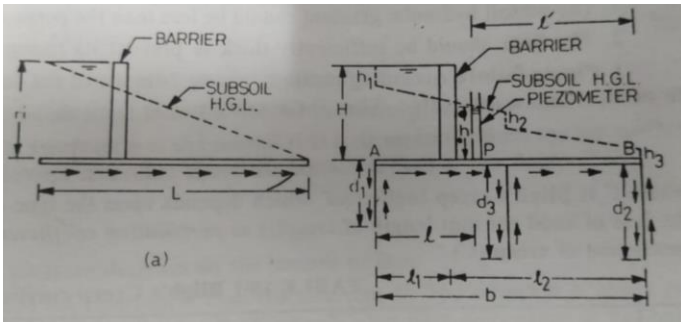
5.5.4 KHOSLA'S THEORY

In 1926-27, some siphons constructed on the Upper Chenab Canal, which were designed on the basis of Bligh's had undermining problems. During investigation t Structures, the actual uplift pressure measurements were made with the help of pipes inserted in the floor. These investigations indicated that the actual uplift pressures were quite different from those computed on the basis of Bligh's theory.

Khosla and his associates carried out further research to find an ultimate solution to the problem of subsurface flow. They conducted experiments at the Punjab Irrigation Institute. They provided a complete rational solution of the problem, which is now known as Khosla's uplift theory. The result of their research have been published by the Central Board of Irrigation and Power (CBIP) in the Publication Number 12, entitled "Design of weirs on permeable foundations". Although the theory was evolved with special reference to weirs, it is applicable to all hydraulic and irrigations structures constructed on permeable foundation. The Khosla theory gives uplift pressure at various points of the structure, depending upon its profile. It also gives the exit gradient. To ensure that the piping failure does not occur, there must be a downstream pile and the exit gradient should be safe. Moreover, the thickness of the floor should be adequate to resist uplift pressure.

5.6 BLIGH'S CREEP THEORY

Bligh assumed that the percolating water creeps along the base profile of the structure, which is in contact with the subsoil. The length of the path thus traversed by the percolating water is called the creep length. Bligh also assumed that the head loss per unit length of creep (called hydraulic gradient) is proportional to the distance of the point from the upstream of the foundation.



Shortcoming of this theory is that it does not discriminate between the horizontal and vertical creeps in estimating the exit hydraulic gradient.

The total creep length, L, is given by

$L = d_1 + d_1 + L_1 + d_3 + d_3 + L_2 + d_2 + d_2 = (L_1 + L_2) + 2d_1 + 2d_2 + 2d_3$

$L = b + 2(d_1 + d_2 + d_3)$

The hydraulic gradient or the loss of head per unit length of creep is,

$$\frac{H}{L} = \frac{H}{b + 2(d_1 + d_2 + d_3)}$$

Therefore, for any point the head loss is proportional to the creep length.

As the hydraulic gradient is constant, if L1 is the creep length up to any point, then head loss up to this point will be (H/L) L1 and the residual head at this point will be (H - (H/L) L1).

The head losses at the three vertical cutoffs will be:

$[(H/L)\,2d_1]$, $[(H/L)\,2d_2]$ and $[(H/L)\,2d_3]$

The reciprocal of the hydraulic gradient, i.e., L/H is known as Bligh's coefficient of creep, C.

5.6.1 SAFETY AGAINST PIPING AND UNDERMINING

According to Bligh, the safety against piping can be ensured by providing sufficient creep length, given by L = C.H, where C is the Bligh's Coefficient for the soil. Bligh recommended certain values of C for different soils. According to Bligh if the hydraulic gradient $H/L \leq 1/C$ (for the soil) there is no danger of piping.

Recommended values of Bligh coefficient of creep C and safe hydraulic gradient

Type of soil	Value of C	Safe Hydraulic Gradient
Fine micaceous sand	15	1/15
Coarse grained sand	12	1/12
Sand mixed with boulder and gravel; and for loam soil	5 to 8	1/9 to1/5
Light sand & mud	9	1/8

5.6.2 SAFETY AGAINST UPLIFT PRESSURE

The ordinate of the subsoil hydraulic gradient line above the bottoms of the floor at any point represents the residual seepage head or the uplift pressure at that point.

If h´ is the uplift pressure head at a point under the floor (i.e., h + t), the uplift pressure intensity is, $U = \rho\, g\, h` = \rho\, g\, (h + t)$

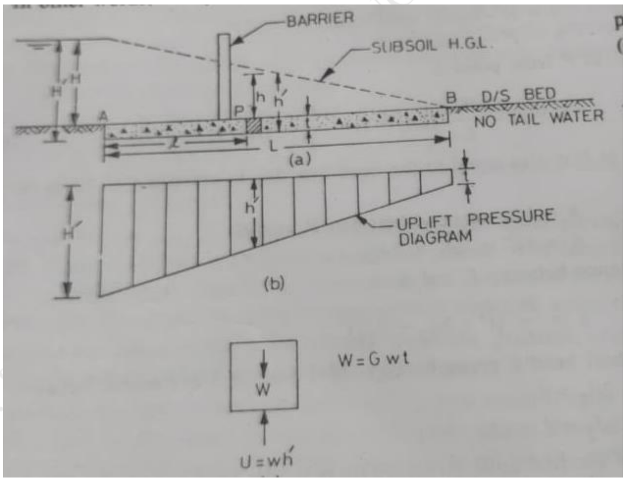
This is to be resisted by the weight of the floor, the thickness of which is t and density ρ_m (for concrete, ρ_m = 2400 kg/m^3). Downward force per unit area due to the weight of the floor is $W = \rho_m\, g\, t = G\, \rho\, g\, t$ (where G is the relative density / specific gravity of the floor material).

Therefore, equating

$G\, \rho\, g\, t = \rho\, g\, (h + t)$ $\quad$ $G\, t = h + t$

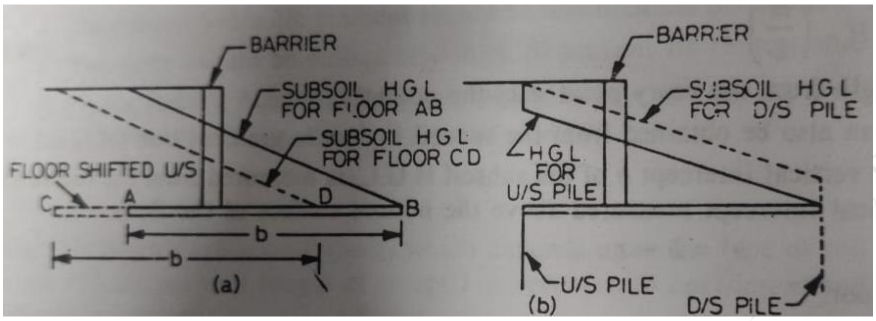
$t = h / (G – 1)$

Considering a safety factor of 4/3 $\quad$ **$t = 4/3 * h/(G – 1)$**



The design will be economical if the greater part of the creep length (i.e. of the impervious floor) is provided upstream of the weir where nominal floor thickness would be sufficient. The downstream floor has to be thicker to resist the uplift pressure. However, a minimum floor length is always required to be provided on the downstream side from the consideration of surface flow to resist the action of fast flowing water whenever it is passed to the downstream side of t

Moreover, the provision of maximum creep length on the upstream side of the weir (barrier) also reduces uplift pressures on the portion of the floor provided on the downstream side of the barrier. This is because a large portion of the total creep having taken place up to the barrier; the residual heads on the downstream floor are reduced. Further, (see Fig 4.10b) a vertical cutoff at the upstream end of the floor reduces uplift all over the floor. Thus, according to Bligh's theory a vertical cutoff at the upstream end of the floor is more useful than the one at the downstream end of the floor.



5.6.3 LIMITATIONS OF BLIGH'S THEORY

Bligh's theory is quite simple and convenient. A large number of early irrigation structures were designed using this theory. Some of these structures are existing even today, but unfortunately a few of them failed. The theory has a number of limitations, as discussed below. The theory is now rarely used for the design of large, important irrigation structures. However, sometimes it is used for the design of small structures or for the preliminary design of large structures. Even in these designs, care shall be taken to provide a pile (or cutoff) at the downstream end of the floor, otherwise piping failure would occur irrespective of the hydraulic gradient, as discussed later in Khosla's theory. Limitations of Bligh's theory as summarised below:

- The Bligh theory does not differentiate between the vertical creep and the horizontal creep and gives the same weightage to both, which is not correct. Actually, the vertical creep is more effective than the horizontal creep.
- The theory assumes that the head loss variation is linear, which is not correct. The actual head loss variation is non-linear.
- No distinction is made between the head loss on the outer faces and that on the inner faces of the sheet piles. Actually, the outer faces are more effective than the inner faces.
- The theory does not emphasise the importance of the downstream pile without which piping failure occurs. It considers the downstream pile only as a component of the total creep length and not as a controlling factor for the exit gradient and the piping.

- The theory does not give any theoretical or practical method for the determination of the creep coefficient C (or the safe gradient I/C). It has to be determined from experi-ence or from actual observations in the existing irrigation structures.
- Bligh did not consider the effect of the length of the intermediate pile. Later investiga-tions by Khosla indicated that the intermediate pile is ineffective if its length is shorter than that of the outer piles However, there is some local redistribution of uplift pressure.
- The theory does not give even the approximate results if the horizontal distance between the piles is less than twice their depths.

5.7 LANE'S WEIGHTER CREEP THEORY

Lane conducted a statistical analysis of 290 existing hydraulic structures founded on pervious foundations and gave a theory in 1932. The theory is now known as the Lane weighted creep theory. The theory gives different weightage to the vertical and horizontal creeps. Lane found that the vertical creep is 3 times more effective than the horizontal creep in reducing the uplift pressure. A weightage of unity was given to the vertical creep and of 1/3 to the horizontal creep. Thus the weighted creep length (L_W) is given by

$L_W = 1/3\ N + V$

where N is the sum of all the horizontal contacts and the flat sloping contacts making an angle less than 45° with the horizontal, V is the sum of all the vertical contacts and the steep sloping contacts making an angle greater than 45° with the horizontal.

According to Lane's weighted creep theory, an irrigation structure will be safe if the average weighted hydraulic gradient (H/L) is less than the safe hydraulic gradient ($1/C_1$) for that soil, where C_1 is Lane's creep coefficient.

Lane's Creep Coefficient

Type of Soil (Material)	Value of C_1	Safe Hydraulic Gradient $\left(\frac{1}{C^1}\right)$
Very fine sand or silt	8.5	1/8.5
Fine sand	7.0	1/7
Coarse sand	5.0	1/5
Gravel & Sand	3.5 to 3.0	1/3.5 to 1/3
Boulders, with some cobble & gravel	2.5	1/2.5
Boulders, gravel and sand	2.5 to 3.0	1/2.5 to 1/3
Clayey Soils	3.0 to 1.6	1/3 to 1/1.6

Thus $H/L_W = 1 / C_1$ or $L_W \geq C_1H$

where H is the seepage head.

In the limiting case, $L_W = C_1H$

In other words, the weighted creep length should be equal to (or greater than) the product of the Lane's creep coefficient C_1 and the seepage head H.

The thickness of the floor at any point can be determined by computing the uplift pressure head (h) and using above equation. Thus $t = 4/3 * (h/(G - 1))$

While computing the residual head (h), proper weightage should be given to creep length. For example, the residual head (h) at point P at a distance of / from the upstream in Fig. 18-1 b is given by $h = H – (H/L)*(l/3 + 2d_1)$

Lane's weighted creep theory is an improvement over Bligh's theory because it gives more weightage to the vertical creep. However, it also has the same limitations as Bligh's theory, except the limitation No. 1 listed in the preceding section. The theory is not accurate and is rarely used in India. However, the theory is quite popular in some other countries