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Hydraulic Structures

Hydraulic structures are engineering structures constructed for the purposes of harnessing and using water resources (groundwater, surface water, lakes, sea, etc.) or for the prevention of the negative and destructive actions (floods, shore erosion, etc.) of water on the surrounding environment.

There are a large variety of hydraulic structures to serve the many purposes for which water resources are put to use. There are several classifications of hydraulic structures, however, the most important is the classification by function as given below.
Table 1. Classification of Hydraulic Structures by function

<table>
<thead>
<tr>
<th>Type</th>
<th>Purpose</th>
<th>Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Storage structures</td>
<td>To store water</td>
<td>Dams, tanks</td>
</tr>
<tr>
<td>2. Flow control structures</td>
<td>To regulate the quantity and pass excess flow</td>
<td>Spillways, outlets, gates, valves</td>
</tr>
<tr>
<td>3. Flow measurement structures</td>
<td>To determine discharge</td>
<td>Weirs, orifices, flumes</td>
</tr>
<tr>
<td>4. Division structures</td>
<td>To divert the main course of water</td>
<td>Coffer dams, weirs, canal headworks, intake works</td>
</tr>
<tr>
<td>5. Conveyance structures</td>
<td>To guide flow from one location to another</td>
<td>Open channel, pressure conduits, pipes, canals and sewers</td>
</tr>
<tr>
<td>6. Collection structures</td>
<td>To collect water for disposal</td>
<td>Drain inlets, infiltration galleries, wells</td>
</tr>
<tr>
<td>7. Energy dissipation structures</td>
<td>To prevent erosion and structural damage</td>
<td>Stilling basins, surge tanks, check dams</td>
</tr>
<tr>
<td>8. Shore protection structures</td>
<td>To protect banks</td>
<td>Dikes, groins, jetties, revetments</td>
</tr>
<tr>
<td>9. River training and waterway stabilization structures</td>
<td>To maintain river channel and water transportation</td>
<td>Levees, cutoffs, locks, piers, culverts</td>
</tr>
<tr>
<td>10. Sediment and quality control structures</td>
<td>To control or remove sediments and other pollutants</td>
<td>Racks, screens, traps, sedimentation tanks, filters, sluiceways</td>
</tr>
<tr>
<td>11. Hydraulic machines</td>
<td>To convert energy from one from to another</td>
<td>Pumps, turbines, rams.</td>
</tr>
</tbody>
</table>

1. Storage structures - Dams

**Dam:** Dam is any artificial barrier and its appurtenant works constructed for the purpose of holding water or any other fluid.

There are three common classification schemes for dams. According to the function performed, dams are classified into:

i) storage dams for impounding water for developmental uses.

ii) Diversion dams for diverting streamflow into canals or other conveyance system and

iii) Detention dams to hold the water temporary to retard flood flows

From hydraulic design considerations, dams are classified as:

i) overflow dams to carry discharge over their crests, and
ii) non-overflow dams, which are not designed to be overtopped. The most common classification is one based on the materials of which dams are made. This classification makes further sub-classification by recognizing the basic type of design, such as concrete gravity or concrete arch dams. Types of dams include:

i) Earthfill dams
ii) Rockfill dams
iii) Concrete dams  
   a) concrete gravity dams  
   b) concrete arch dams  
   c) concrete buttress dams
iv) Stone masonry  
   a) stone-masonry gravity dams  
   b) stone-masonry arch dams
v) Timber dams
vi) Steel coffer dams

**SELECTION OF TYPE OF DAM**

There are many factors involved in the selection but at the beginning, physical factors play an important role. Topographically, a narrow stream section with high rocky walls suggest a suitable site for a concrete dam. Where the wall are strong enough to resist arch thrust, a concrete arch dam is adaptable. Low-rolling plains suggest an earthfill or rockfill dam.

When the geologic characteristics of foundation are comprised of solid rock, any type of dam can be constructed, although concrete gravity or arch dams are favourable. Gravel foundation are suitable for earthfill, rockfill, and low concrete gravity dams. Silt and fine sand foundations are used to support earthfill and low concrete gravity dams but not suitable for rockfill dams.

Availability of certain materials close to the site will effect considerable reduction in cost if the type of dam selected utilizes these materials in sufficient quantity. Size, type and natural restrictions in location of a spillway influence the choice of dam. A large spillway requirement indicates the adoption of a concrete gravity dam. A small spillway requirement favours the selection of rockfill dam. When the excavated material from a site channel spillway can be used in a dam embankment, an earthfill dam is advantageous.

Apart from the above factors, others, such as the cost of diverting the stream, availability of labour, and traffic requirements on top of the dam will favour one type or the other.
Chapter 2: EARTHFILL DAMS

Earthfill dams
Earthfill dams are the most common type of dam built to any height. They are designed as a non-overflow section with separate spillway. The reason for such wide spread use of earthfill dams are:

- The foundation requirements are not as rigorous as other dams
- Local available soil is the main construction material
- High skill not required
- No special plants are required. Most earth-moving machines can be used

Classification of earthfill dams
Earthfill dams are classified by many factors.

1. Based on the method of construction
   - Rolled fill earth dams
   - Hydraulic fill dam

2. Based on mechanical characteristics of earth materials making the section of the Dam
   - Homogeneous earth dams
   - Non-Homogeneous (zoned) earth dams
     i) non-homogeneous with inclined impervious zone (ekran) of artificial material
     ii) with impervious zone of soil with low permeability
     iii) with central core soil material of low permeability
     iv) with a central thin diaphragm of impervious material

Rolled Fill Earth dams
In this type of dams, successive layers of moistened or damp soils are laid one over the other. Each layer not exceeding 20 cm in thickness is properly consolidated at optimum moisture content, only then is the next layer laid.
Hydraulic Fill dams
In this type of dams, the construction, excavation, transportation of the earth is done by hydraulic methods. Outer edges of the embankments are kept slightly higher than the middle portion of each layer. During construction, a mixture of excavated materials in slurry condition is pumped and discharged at the edges. This slurry of excavated materials and water consists of coarse and fine materials. When it is discharged near the outer edges, the coarser materials settle first at the edges, while the finer materials move to the middle and settle there. Fine particles are deposited in the central portion to form a water tight central core. In this method, compaction is not required.

Homogeneous Earthen Dam
These dams are constructed with uniform and homogeneous materials. It is suitable for low height dams (up to 10m). These dams are usually constructed with soil and grit mixed in proper ratios. The seepage action of such dams are not favourable, therefore, for safety in case of rapid drawdown, the upstream slope is kept relatively flat (3:1).

Homogeneous section is modified by constructing rock toe at the downstream lower end and providing horizontal filter drain.
Zoned Earth Dams
These are dams with the central portions called core or hearting made from materials which are relatively impervious. The thickness of the core wall is made sufficiently thick to prevent leakage of water through the body of the dam.

Dam with a Diaphragm
This type of dam is constructed with pervious materials, with a thin impervious diaphragm in the central part to prevent seepage of water. The thin impervious diaphragm may be made of impervious clayey soil, cement concrete or masonry or any impervious material. The diaphragm can be constructed in the central portion or on the upstream face of the dam. The main difference in zoned and diaphragm type of dams depend on the thickness of the impervious core or diaphragm. The thickness of the diaphragm is not more than 10 m.

The criteria for the design of earth dams are:
1. Sufficient spillway capacity and freeboard are provided so that there is no danger of overtopping of the dam.
2. Seepage flow through the embankment is controlled so that the amount lost does not interfere with the objective of the dam and there is no erosion or sloughing of soil. In this respect, seepage line should remain well within the downstream face of the dam and the portion of the dam on downstream side of the impervious core should be well drained.
3. Uplift pressure due to the seepage underneath is not enough to cause piping.
4. The slopes of the embankment are stable under all conditions of reservoir operation, including rapid drawdown and during steady seepage under full reservoir.

5. The stresses imposed by the embankment upon the foundation are less than the strength of material in the foundation with a suitable factor of safety.

6. The upstream face is properly protected ((stone pitching, riprap, revetment) against erosion caused by wave action, and the downstream face is protected (counter-booms, turfs) against the action of rain

**Embarkment Materials**

*a. Earth-fill materials.*

(1) While most soils can be used for earth-fill construction as long as they are insoluble and substantially inorganic, typical rock flours and clays with liquid limits above 80 should generally be avoided. The term "soil" as used herein includes such materials as soft sandstone or other rocks that break down into soil during handling and compaction.

(2) If a fine-grained soil can be brought readily within the range of water contents suitable for compaction and for operation of construction equipment, it can be used for embankment construction. Some slow-drying impervious soils may be unusable as embankment fill because of excessive moisture, and the reduction of moisture content would be impracticable in some climatic areas because of anticipated rainfall during construction. In other cases, soils may require additional water to approach optimum water content for compaction. Even ponding or sprinkling in borrow areas may be necessary. The use of fine-grained soils having high water contents may cause high porewater pressures to develop in the embankment under its own weight. Moisture penetration into dry hard borrow material can be aided by ripping or plowing prior to sprinkling or ponding operations.

(3) As it is generally difficult to reduce substantially the water content of impervious soils, borrow areas containing impervious soils more than about 2 to 5 percent wet of optimum water content (depending upon their plasticity characteristics) may be difficult to use in an embankment. However, this depends upon local climatic conditions and the size and layout of the work, and must be assessed for each project on an individual basis. The cost of using drier material requiring a longer haul should be compared with the cost of using wetter materials and flatter slopes. Other factors being equal, and if a choice is possible, soils having a wide range of grain sizes (well-graded) are preferable to soils having relatively uniform particle sizes, since the former usually are stronger, less susceptible to piping, erosion, and liquefaction, and less compressible. Cobbles and boulders in soils may add to the cost of construction since stone with maximum dimensions greater than the thickness of the compacted layer must be removed to permit proper compaction. Embankment soils that undergo
considerable shrinkage upon drying should be protected by adequate thicknesses of non-shrinking fine-grained soils to reduce evaporation. Clay soils should not be used as backfill in contact with concrete or masonry structures, except in the impervious zone of an embankment.

(4) Most earth materials suitable for the impervious zone of an earth dam are also suitable for the impervious zone of a rock-fill dam. When water loss must be kept to a minimum (i.e., when the reservoir is used for long-term storage), and fine-grained material is in short supply, resulting in a thin zone, the material used in the core should have a low permeability. Where seepage loss is less important, as in some flood control dams not used for storage, less impervious material may be used in the impervious zone.

Some causes of failure of earthfill dams
Like most of engineering structures, earth dams may fail due to faulty design, improper construction and poor maintenance practices, etc

The various causes of failure may be classified as:

a) Hydraulic failure
b) Seepage failure
c) Structural failure

a) Hydraulic failure:
Hydraulic accounts for over 40% of earth dam failure and may be due to one or more of the following:

i) By overtopping: When free board of dam or capacity of spillway is insufficient, the flood water will pass over the dam and wash it downstream.

Fig. Dam failure by overtopping
ii). Erosion of downstream toe: The toe of the dam at the downstream side may be eroded due to i) heavy cross-current from spillway buckets, or ii) tail water. When the toe of downstream is eroded, it will lead to failure of dam. This can be prevented by providing a downstream slope pitching or a riprap up to a height above the tail water depth. Also, the side wall of the spillway should have sufficient height and length to prevent possibility of cross flow towards the earth embankment.

iii) Erosion of upstream surface: During winds, the waves developed near the top water surface may cut into the soil of upstream dam face which may cause slip of the upstream surface leading to failure. For preventing against such failure, the upstream face should be protected with stone pitching or riprap.

iv). Erosion of downstream face by gully formation: During heavy rains, the flowing rain water over the downstream face can erode the surface, creating gullies, which could lead to failure. To prevent such failures, the dam surface should be properly maintained; all cuts filled on time and surface well grassed. Berms could be provided at suitable heights and surface well drained.

Figure 3. Beginning of downstream failure

b). Seepage failure:
Seepage always occurs in the dams. If the magnitude is within design limits, it may not harm the stability of the dam. However, if seepage is concentrated or uncontrolled beyond limits, it will lead to failure of the dam. Following are some of the various types of seepage failure.

i) Piping through dam body. When seepage starts through poor soils in the body of the dam, small channels are formed which transport material downstream. As
more materials are transported downstream, the channels grow bigger and bigger which could lead to wash out of dam

ii) Piping through foundation: When highly permeable cavities or fissures or strata of gravel or coarse sand are present in the dam foundation, it may lead to heavy seepage. The concentrated seepage at high rate will erode soil which will cause increase flow of water and soil. As a result, the dam will settle or sink leading to failure.

iii) Sloughing of downstream side of dam:
The process of failure due to sloughing starts when the downstream toe of the dam becomes saturated and starts getting eroded, causing small slump or slide of the dam. The small slide leaves a relative steep face, which also becomes
saturated due to seepage and also slumps again and forms more unstable surface. The process of saturation and slumping continues, leading to failure of dam.

c) Structural Failure:

About 25% of failure is attributed to structural failure, which is mainly due to shear failure causing slide along the slopes. The failure may be due to:

i) Slide in embankment: When the slopes of the embankments are too steep, the embankment may slide causing failure. This might happen when there is a sudden drawdown, which is critical for the upstream side because of the development of extremely high pore pressures, which decreases the shearing strength of the soil. The downstream side can also slide especially when dam is full. Upstream embankment failure is not as serious as downstream failure.

ii) Foundation slide: When the foundation of an earthfill dam is composed of fine silt, clay, or similar soft soil, the whole dam may slide due to water thrust. If seams of fissured rocks, such as soft clay, or shale exist below the foundation, the side thrust of the water pressure may shear the whole dam and cause its failure. In such failure the top of the dam gets cracked and subsides, the lower slopes moves outward and forms large mud waves near the dam heel.

iii) Faulty construction and poor maintenance: When during construction, the compaction of the embankment is not properly done, it may lead to failure.

iv) Earthquake may cause the following types of failure to earthfill dams;

1. cracks may develop in the core wall, causing leakages and piping failure.
2. slow waves may set up due to shaking of reservoir bottom, and dam may fail due to overtopping
3. settlement of dam which may reduce freeboard causing failure by overtopping
4. slidding of natural hills causing damage to dam and its appurtenant structures
5. Fault movement in the dam site reducing reservoir capacity and causing overtopping.
6. Shear slide of dam
7. The sand below foundation may liquefy
8. Failure of slope pitching.
Some elements of earthfill dams

1. Crest of dam:
The crest width of dams should be sufficient to keep the seepage line within the dam, when the reservoir is full. The crest width of the dam if road is not envisaged should not be less than 3m for low and medium head dams and 6m for high head dams. If road is envisaged, then the width of the dam is determined according to the class of road and determined by the road code.

Top width could be determined by the following recommended formulae:

   a) For very low dams top width is given by

   \[ B = \frac{H}{5} + 3 \]

   b) For dams lower than 30m

   \[ B = 0.55(H)^{1/2} + \frac{H}{5} \]

   For dams higher than 30m,

   \[ B = 1.65(H + 1.5)^{1/3} \]

   \[ B = 1.67(H)^{1/2} \]

Balustrades are provided at the end of the roads to prevent car falling off the slopes.

2. Side slopes of dam: side slope of dams must satisfy the static stability. However, since the stability computations can be done only after defining the profile of the dam and determining the seepage line, it becomes necessary to give an initial side slopes. Initial slope could be taken from the tables below.

*(Taken from “hydraulic structures” – N. P. Rosanova)*

<table>
<thead>
<tr>
<th>Slope</th>
<th>Material of dam</th>
<th>Side slopes depending on height of dam</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Less than 5m</td>
<td>From 5m - 10m</td>
</tr>
<tr>
<td>Upstream</td>
<td>Clayey</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Sandy</td>
<td>2.5 - 2</td>
</tr>
<tr>
<td>Downstream</td>
<td>Clayey</td>
<td>1.5</td>
</tr>
<tr>
<td>With filter</td>
<td>Sandy</td>
<td>2</td>
</tr>
<tr>
<td>Downstream</td>
<td>Clayey</td>
<td>1.75</td>
</tr>
<tr>
<td>without filter</td>
<td>Sandy</td>
<td>2</td>
</tr>
</tbody>
</table>

In low head dams, usually one and constant side slope is used, however in medium and high head dams, different side slopes are usually adopted to reduce the volume of the dam.
Side Slope According to the recommendations of Terzaghi

<table>
<thead>
<tr>
<th>No.</th>
<th>Type of material</th>
<th>Upstream slope (H :V)</th>
<th>Downstream slope (H:V)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Well graded homogeneous soil</td>
<td>2.5:1</td>
<td>2:1</td>
</tr>
<tr>
<td>2</td>
<td>Homogeneous coarse silt</td>
<td>3:1</td>
<td>2.5:1</td>
</tr>
</tbody>
</table>
| 3   | Homogeneous silt clay:  
  i) for dam height less than 15m  
  ii) for dam height more than 15m | 2.5:1  
  3:1                  | 2.5:1  
  2:1                  |
| 4   | Sand or sand and gravel with clay core                | 3:1                   | 2.5:1                  |
| 5   | Sand or sand and gravel with reinforced concrete core wall | 2.5:1                 | 2:1                    |

3. **Berms**: Berms are constructed at both the upstream and downstream side of the dam for the purposes of observing the conditions of protections at the slopes and their repairs and also for increasing the width of dam at the base with aim of increasing seepage length. It is also done when constructions coffer dams are made part of the body of the dam. At the downstream side, berms are done at an interval of 10 – 15m high. Width of berm is taken between 1 – 2 m.

4. **Free Board**

Normal freeboard is the vertical distance between the normal pool level and the crest of the dam. Minimum freeboard is the vertical distance between the high flood level and the crest of dam.

The minimum height of freeboard is taken as 1.5 \( h_w \) where \( h_w \) is given by:

\[
h_w = 0.032 \left( V \cdot F \right)^{1/2} + 0.763 - 0.271 \left( F^{1/4} \right) \quad \text{for} \quad F, \ 32 \ \text{km} \quad (X)
\]

and

\[
h_w = 0.032 \left( V \cdot F \right)^{1/2} \quad \text{for} \quad F, \ 32 \ \text{km} \quad (Y)
\]
where \( h_w \) = wave height (height of water from top to trough of waves in meters)
\( V \) = velocity of wind in km/hr
\( F \) = fetch or straight length of water expanse in km.

Free board values as recommended by U.S.B.R are given in table below.

### Free Board by USBR

<table>
<thead>
<tr>
<th>Spillway Type</th>
<th>Dam Height in m</th>
<th>Minimum freeboard over M.W.L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free spillway</td>
<td>Any height</td>
<td>2 m to 3 m</td>
</tr>
<tr>
<td>Controlled spillway</td>
<td>Up to 60 m</td>
<td>2.5 m above top of gates</td>
</tr>
<tr>
<td>Controlled spillway</td>
<td>More than 60 m</td>
<td>3.0 m above top of gates</td>
</tr>
</tbody>
</table>

5. Slope Protection (Revetment)

**Upstream side protection:** For protecting the upstream slope from deterioration and damage from wave action, the slope is covered with different protective material.

Rock riprap, either dry dumped stone boulders or hand-packed stone boulders can be made. Stone pitching provided at slope of 1.5 : 1 to 2 : 1 for ordinary soil material of dam and 3 : 1 for poor soil material. The revetment stones are fixed at the toe of the dam to prevent slipping of the embankment. The thickness of the stone pitching is usually more than 60 cm. In most cases the stone pitching is placed over gravel then sand cushion. Big sizes stones with their broader face downwards are packed with each other by means of hammer.

Concrete, reinforced concrete slabs, steel plates, bituminous material pavement, brick tile pavement could also be used. However, extensive survey done by US
Corps of Engineers in the 1940s over 100 dams show that dry dumped riprap stone pitching has the best efficiency in terms of failure rate.
Downstream protection
One of the cheapest, simple and effective methods of protecting the downstream from rainfall and wind action is by planting green grass (turfs) on the slope. Counter-boom could also be done.

6. Drainage: Drainage in earth dams is meant for lowering the seepage curve; prevent seepage water from flowing onto the downstream slope, and conveying seepage water through the body of the dam to the downstream part of the dam. By its function, dam drainage must have two parts; an intake
structure (drainage trench) that allows seepage water from the body and foundation of the dam, while at the same time preventing deformation due to seepage and conveyance structure that transports the seepage water from the dam. However, in most drainage, it is difficult to see clearly these two parts.

Hydrotechnical construction practice has worked out many drainage systems depending on the type of dam, materials of the foundation and body of dam. Among some of the most commonly used drainage systems are:

i) Drainage prism: with many positive sides (advantages) but requires the use of large quantity of stones

ii) A type of drainage prism in which the filter material of the drainage system is laid to extend to a certain height on the downstream side. Such drainage system is used when there could be rise of the tail water above the crest of the prism.

iii) Flat horizontal drainage: It requires much smaller quantity of stones and simplifies construction. It has the advantage of draining both the foundation and body of dam and it is used mainly when the foundation is made up of saturated material

iv) Combination of horizontal drain with the prism.

v) Horizontal piped drainage: consists of a pipe (tube), laid parallel to base of the slope of dam.

vi) Horizontal stone drainage: a type of horizontal piped drain in which instead of the pipe, a stone prism is used.
Seepage through dam

Filtration (seepage) computation through dams are carried out with the aim of:

- determining the position of the seepage depression curve
- finding the value of the gradient and velocity of filtration
- determine the filtration flow (discharge)

The seepage pattern through a dam is shown above. The pattern is the same irrespective of the material (sand, clay, loam) of the dam, though the rate of seepage will depend on soil type. The emergence of seepage lines on the downstream slope tends to make the downstream slope unstable. Either the downstream slope has to be made very flat or the seepage must be diverted away from the downstream slope. The second alternative is favoured because it is economical.

The diagram below shows the seepage lines for a homogeneous earthfill dam with a horizontal gravel filter on the base of the dam at the downstream side. The flow lines
enter the blanket vertically. Cassagrande (1937) has shown that the phreatic line, which is the topmost seepage line, quite closely approximates a parabola.

The parabola

Seepage through a homogeneous dam with horizontal drainage blanket (filter)

Note: For D greater than 2m, see design dimensions in Nielson, 1985.

Insert drawing
intersects the water surface at A such that $AB = 0.3 \, CB$. Near the upstream face, the phreatic line diverges from the parabola and join B perpendicularly.

The focus of the parabola is at F and the directrix passes through D. Every point on the parabola with origin at the focus F can be given by

$$x = \frac{y^2 - y_0^2}{2y_0} \quad \text{----------------1}$$

At the point A, $x = d$ and $y = H$. Substituting in equation 1 yields

$$y_0 = \sqrt{H^2 + d^2} - d \quad \text{-----------------(2)}$$

$y_0$, which is also the distance FD, is determined from eq. (2)

How to get the point D graphically. With A as the centre and AF as radius, draw an arc to cut the water surface at J. From J drop a perpendicular to the base at D. D is the directrix. Locate the mid-point of FD at E which is the vertex of the parabola.

The phreatic line can be drawn by equation (1).

For an approximate estimate of seepage, the Darcy’s law at point G, per unit length of dam

$$q = K_i A = K \left( \frac{dy}{dx} \right) y_0 \quad \text{-----------------(3)}$$

From eq. (1) $\frac{dy}{dx} = \frac{y_0}{\sqrt{2xy_0 + y_0^2}} \quad \text{------------------(4)}$

Since at G, $x = 0$, and $y = y_0$ then

$$\frac{dy}{dx} = 1 \quad \text{------------------(5)}$$

Hence $q = K (1) (y_0) \quad \text{------------------(6)}$

The flow through the section at G is the same as at any other section. Consequently, the total seepage through the length L of the dam is given by:

$$Q = K(y_0)(L) \quad \text{------------------(7)}$$

**Worked example**

A homogeneous earthfill dam has a top width of 30ft and a height of 100ft with a freeboard of 10ft. The side slopes are 1V: 2.5H. It has a horizontal drainage blanket at the base that extends from the downstream toe to a distance of 100ft. The embankment has a permeability of $1.5 \times 10^{-5}$ ft/sec. Determine the seepage through the dam.

**Solution**

$H = 90 \, \text{ft} \quad CB = 90 \times 2.5 = 225\, \text{ft} \quad AB = 0.3(225) = 67.5\, \text{ft}$ and $CA = 157.5\, \text{ft}$

d = base length – blanket length – CA
\[ 22 = 530 - 100 - 157.5 = 272.5\text{ ft.} \]

From eq. (2) \[ y_0 = \sqrt{H^2 + d^2 - d} = \left[ (90)^2 + (272.5)^2 \right]^{1/2} - 272.5 = 14.5\text{ ft} \]

From eq. 6 \[ q = K \left( y_0 \right) = (1.5 \times 10^{-5})(14.5)(1) = 2.18 \times 10^{-4}\text{ cfs or 18.8 ft}^3/\text{day per foot of dam}. \]

**Seepage line of a homogeneous dam without filter**

For a homogeneous dam without filter, the focus point F of the parabola is at the downstream toe of the dam. The base parabola cuts the downstream slope of the dam and extends beyond the dam.

The seepage line emerges out at point C meeting the downstream face tangentially. The portion CF of the dam is known as the discharge face and always remains saturated.

The portion DC (\( \Delta a \)) and CF (a) are inter-related as follows:

\[ \Delta a = \left( a + \Delta a \right) \left( \frac{180^\circ - \alpha}{400^\circ} \right) \]

The table below gives the values of \( \Delta a/(a + \Delta a) \) for various values of \( \alpha \) (angles which the discharge face makes with the horizontal) as per Cassagrande

<table>
<thead>
<tr>
<th>( \alpha ) in degree</th>
<th>( \Delta a/(a + \Delta a) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>0.36</td>
</tr>
<tr>
<td>60</td>
<td>0.32</td>
</tr>
<tr>
<td>90</td>
<td>0.26</td>
</tr>
<tr>
<td>120</td>
<td>0.18</td>
</tr>
<tr>
<td>135</td>
<td>0.14</td>
</tr>
<tr>
<td>150</td>
<td>0.10</td>
</tr>
<tr>
<td>180</td>
<td>0.0</td>
</tr>
</tbody>
</table>

**Seepage rate calculation from flow net**

Flow net is a network form by streamlines (flow lines and equi potential (head lines) which are mutually perpendicular. In the network, the portion between any two successive flow lines is called flow channel and the portion between any two successive equi potential lines called flow field.

Then from Darcy’s law of flow through porous medium, considering unit thickness,

\[ \Delta q = k \frac{\Delta h}{l} (b.1) \]
If $\Delta h = \text{drop of head through a field},$
$h = \text{total head causing flow}$
$\Delta q = \text{discharge passing through the field}$

If the total number of potential drops in the flow net $= N_d$

Then $\Delta h = \frac{h}{N_d}$ ---------------b)

Putting b) into a), we obtain $\Delta q = k \cdot \frac{\frac{h}{N_d}}{\frac{b}{l}}$ ---------c)

Or Total flow $q = \sum \Delta q = k \cdot \frac{h}{N_d} \left( \frac{b}{l} \right) N_t = k h N_t \left( \frac{b}{l} \right)$ --------- d)

Where $N_t = \text{total number of flow channels in the net}.$

When the flow field is square, then $b = l$ and the discharge becomes

$q = k h \frac{N_t}{N_d}$ ---------------f)

**Seepage under the Dam**

If the foundation material is alluvial sand or gravel, seepage could occur underneath the dam. If the upward seepage pressure of water near the toe is greater than the effective weight of the soil, the surface of the soil will rise at a point of least resistance, and water and soil will start flowing away from dam.
This phenomena is known as piping and can result in the sliding of the toe or the settling of the whole dam. The submerged unit weight soil is given by
\[
\gamma_{sub} = \frac{G_s - 1}{1 + e}\gamma_w \quad ---------(Y)
\]
where \(G_s\) = specific gravity of soil; \(e\) = void ratio.

For a seepage line at a gradient, \(I\), the upward seepage force per unit volume is \(i\gamma_w\). When the two forces are in balance,
\[
i = \frac{G_s - 1}{i + e} \quad ------------------(Z)
\]
Eq. (Z) is known as the critical gradient and equal to unity. A gradient of slightly higher than unit value will cause piping or sand particles to be in an unstable condition known as quicksand. The actual gradient at the downstream end of the dam is evaluated from the flow net by dividing the head difference between the 1st two potential lines by the distance between these potential lines. This should be less than unity.

An empirical approach, the creep ratio, \(L/H\) is computed; here \(L\) is the length along the surface of contact between the soil and the base of the structure. This ratio is kept at 4 for gravels and 18 for sand and silt.

**Measures against seepage. Seepage Control.**

![Diagram of seepage control measures](image.png)

The following are some common measures for controlling seepage through the dam and embankments.

1. Prevention of Seepage through Foundation
   i) By providing drainage trenches
   ii). By providing downstream seepage berms
   iii). By providing impervious blanket layer on upstream slope
   iv). By providing impervious cutoff.
2. Prevention of Seepage through Embankment (Dam)
   i). By providing horizontal drainage filter
   ii). By providing toe filter
   iii). By providing filter downstream of toe
   iv). By providing downstream coarse section
   v). By providing chimney drains extending upwards into the embankment

![Chimney drain]

**Design of Filter**

The design of filters should be done in such a way that all the seepage water through the dam is effectively drained off. The filter consists of several layers. The first layer of the filter which comes in contact with the seeping water consists of fine sand material. Subsequent layers of filter are made of sand of increased fineness. The last layer of the filter is made of gravels. The soil of the earth dam and the foundation material surrounding the filter are known as the base material. The filters of filter drains are known as reverse or inverted filter.

The U.S. Bureau of reclamation, Washington 1960 recommended the following for materials to be used for filters;

i) Filter material should be fine and poorly graded so that the voids in the filter are small and thus prevent base material from entering the filter.

ii) The filter material should be coarse and pervious in relation to the base material. This aspect facilitates rapid removal of seeping water without building up any seepage forces within the filter.

iii) The filter material should be coarser than the perforations of openings in the drain pipes, so that filter material is not lost in the drains. The perforations (openings in the pipes drains should be adequate to admit all seeping water safely.

iv) The thickness of filter material should be sufficient to provide a good distribution of all particle sizes, also throughout the filter. The thickness should be adequate to provide safety against piping.

Terzaghi has recommended the following two requirements which should be fulfilled by the filter:
a). The $D_{15}$ size of the filter material must not be more than 4 to 5 times the $D_{85}$ of the base material. This prevents the foundation material from carrying through the pores of the filter material.
b). The $D_{15}$ size of the filter material should be at least 4 to 5 times the $D_{16}$ of the base material. This keeps seepage forces within the filter to permissible levels.
The above criteria can be expressed as follows:

\[
\frac{D_{15} \text{ of filter}}{D_{85} \text{ of base material}} < 4 \text{ to } 5 < \frac{D_{15} \text{ of filter}}{D_{16} \text{ of base material}}.
\]

The above criteria has been modified by USBR in “Design of small Dams” as:

a) \[
\frac{D_{15} \text{ of filter material}}{D_{15} \text{ of base material}} = 5 \text{ to } 40
\]
b). \[
\frac{D_{15} \text{ of filter material}}{D_{85} \text{ of base material}} = 5 \text{ or less}
\]
c). \[
\frac{D_{85} \text{ of filter material}}{\text{Max opening of perforations of pipes}} = 2 \text{ or more}
\]
d). The grain size curve of the filter material should be about parallel to the curve of the base material.

6. Stability of Earthfill dams

Stability computation for earthfill dams are done to check on the following:

2. stability of downstream slope during steady seepage
3. stability of upstream slope during drawdown
4. stability of downstream and upstream slopes during and immediately after construction
5. Stability of foundation against shear.

Earthfill dams usually fail due to the sliding of large soil mass along a curved surface.

The most common method used for examining the stability of earthfill dam embankment slopes is the Swedish slip circle or (the slice) method.

**Swedish slip Circle Method (Slide Method)**

In this method, the failure of the embankment surface is assumed to be cylindrical. The factor of safety against sliding, which is the ratio of average shearing stress as determined by the Coulomb equation $s = (c + \sigma \tan \phi)$ to the average shearing stress determined by statics on a potential sliding surface.

For testing the stability of a slope, the centre of the possible arc is assumed. It is necessary first to locate the centre of the Critical Circle to locate the Centre of Critical Slope.
Fellenious has given the following method to locate the locus on which the probable centre line may lie.

For a homogeneous soil, the centre of critical slip centre lies on the line PQ. The coordinates of point Q is H downwards from toe and 4.5 H horizontal away from toe as shown above. The location of point P is done with the help of directional angles $i$, $\alpha$ and $\beta$ given in the table below.

<table>
<thead>
<tr>
<th>Slope angle ($i^{\circ}$)</th>
<th>Slope</th>
<th>Directional angles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\alpha$</td>
</tr>
<tr>
<td>11.3</td>
<td>5 : 1</td>
<td>25</td>
</tr>
<tr>
<td>18.4</td>
<td>3 : 1</td>
<td>25</td>
</tr>
<tr>
<td>26.6</td>
<td>2 : 1</td>
<td>25</td>
</tr>
<tr>
<td>33.8</td>
<td>1.5 : 1</td>
<td>26</td>
</tr>
<tr>
<td>45</td>
<td>1 : 1</td>
<td>28</td>
</tr>
<tr>
<td>60</td>
<td>0.58 : 1</td>
<td>29</td>
</tr>
</tbody>
</table>

First extend the surface of the fluid WL. Draw angle $i$ from the toe of dam to cut the surface of fluid at S. From S draw the angle $\beta$ and from the toe draw angle $\alpha$ to intercept angle $\beta$ at the point P. Now by joining QP, we obtain the line on which the centre of critical slip centre lies.

Now on line QP we assume point O and an arc AB is drawn with radius OA. The earth mass above the arc is divided into a number of vertical segments known as slices. The number of slices may be from 6 to 12. Neglecting the forces between slices, each slice is assumed to act independently as a column of soil of unit
thickness and width. The weight of each slice is assumed to be acting at its centre. The weight is resolved into two components \( N \) and \( T \) (ie. normal component passing through the centre of the arc \( O \), and will not cause any driving moment on the slice. But the tangential component \( T \) causes a driving moment of \( T \times R \)). Near the toe of the dam, some of the slices may cause resisting moment and in such cases, \( T \) is negative.

Let the slices be numbered 1, 2, …6 and their weights be numbered \( W_1, W_2…W_6 \)
Now \( N = W \cos \alpha \) and \( T = W \sin \alpha \)
From Coulomb’s equation, the resisting force

\[
F_R = c \times \Delta L + N \tan \phi \quad \ldots \ldots 1)
\]
where \( c = \) unit cohesion ; \( \Delta L = \) curved length of slice ; \( \phi = \) angle of internal friction of soil.

The driving moment
\[
M_D = R \times \sum T \quad \ldots \ldots \ldots \ldots 2)
\]
The resisting moment
\[
M_R = R[c \sum \Delta L + \tan \phi \times \sum N]
\]
\( \sum \Delta L = \) sum of curved length of all components = 2\( \pi \theta /360 \) - length of arc \( AB \)

The factor of safety against sliding
\[ F.S = \frac{M_r}{M_p} = \frac{R[c \sum \Delta L + \tan \phi \sum N]}{R \sum T} \]

\[ F.S = \frac{[c \sum \Delta L + \tan \phi \sum N]}{\sum T} \]  \hspace{1cm} \text{(3)}

The values of \( \sum N \) and \( \sum T \) are generally calculated in a tabular form. Now if \( w_1, w_2, \ldots, w_n \) are the weight of slices, then
\[ \sum N = \cos \alpha (\sum w) \quad \text{and} \quad \sum T = \sin \alpha \sum (w) \]

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>Weight of slice</th>
<th>( N = w \times \cos \alpha )</th>
<th>( T = w \times \sin \alpha )</th>
<th>( c \times \Delta L )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>( w_1 )</td>
<td>( N_1 )</td>
<td>( T_1 )</td>
<td>( c \times \Delta L_1 )</td>
</tr>
<tr>
<td>2</td>
<td>( w_2 )</td>
<td>( N_2 )</td>
<td>( T_2 )</td>
<td>( c \times \Delta L_2 )</td>
</tr>
<tr>
<td>3</td>
<td>( w_3 )</td>
<td>( N_3 )</td>
<td>( T_3 )</td>
<td>( c \times \Delta L_3 )</td>
</tr>
<tr>
<td>4</td>
<td>( w_4 )</td>
<td>( N_4 )</td>
<td>( T_4 )</td>
<td>( c \times \Delta L_4 )</td>
</tr>
<tr>
<td>..</td>
<td>..</td>
<td>..</td>
<td>..</td>
<td>..</td>
</tr>
<tr>
<td>n</td>
<td>( w_n )</td>
<td>( N_n )</td>
<td>( T_n )</td>
<td>( c \times \Delta L_n )</td>
</tr>
<tr>
<td>( \sum N )</td>
<td>( \sum T )</td>
<td>[ c \times \sum \Delta L = ]</td>
<td>[ ] [ c \times 2\pi\theta/360 ]</td>
<td></td>
</tr>
</tbody>
</table>

The factor of safety is computed for the point O with equation (3). Different points on the line QP are chosen and the procedure gone through to determine the factor of safety. Various factor of safety so obtained are plotted as ordinates on the corresponding centres and a smooth curve of F.S drawn. The centre corresponding to the lowest factor of safety is the required centre of critical slip circle.

**Downstream Slope Stability during Steady Seepage**

The critical condition for downstream slope occurs when the reservoir is full and seepage is at its maximum rate. The seeping water below the phreatic surface exerts a pore pressure on the soil mass. Therefore, the shearing strength of the slices of the critical arc within the range of pore-pressure is reduced. The net shear strength of the affected slice is
\[ c \times \Delta L = (N - U) \tan \phi \]  \hspace{1cm} \text{(5)}

where \( U = \) the pore pressure

The factor of safety (F.S) for the entire slip circle is
\[ F.S = \frac{c \sum \Delta L + \tan \phi (\sum N - \sum U)}{\sum T} = \frac{c \sum \Delta L + \tan \phi \sum N'}{\sum T} \]  \hspace{1cm} \text{(6)}

where \( N' = \) normal components, calculated on the buoyant unit weight
\[ \gamma' = (\rho_s - \rho_w)g \] of the dam.
Values of \( \sum T \) is calculated on the basis of its saturated weight.

**Slope Stability during Sudden Drawdown**

If dam is suddenly emptied, it may cause critical conditions for the stability of the upstream slope. In this condition, the soil pores remain filled with water causing the water level to remain the same as before the sudden drawdown. The water weight within the soil tends to slide the upstream slope along a circular arc. The hydrostatic force acting along the upstream slope when reservoir is full is also removed with the sudden drawdown. Therefore, the tangential component of the saturated weight is the main force causing disturbance to the upstream slope. The shear resistance is considerably reduced due to development of pore pressure on the likely slip surface. The factor of safety can be calculated by the formula (6) above.

**Stability of Upstream and Downstream Slopes during and immediately after Construction**

During construction of the dam and embankment with relatively impervious soil, excess pore pressure is developed in the air and water entrapped in the pore space. This is due to the fact that the soil mass undergoes a change in volume during compaction during and after construction. With time, this initial excess pore pressure gets gradually dissipated.

**Stability of Foundation against Shear.**

At dam site, the silt and clay or fine, loose cohesionless material that form the foundation may have good imperviousness, but are weak in shear and always require checking. For increasing the shear area and keeping the shear stress within permissible limits, the embankment slopes are flattened or berms are added on both sides of the dam.
The method for determining the factor of safety, which is approximate is based on the assumption that earthfill materials have an equivalent liquid weight, which would produce the same shear stress as the material will develop itself.

The horizontal shear on left is given by:

\[ P = \frac{h_1^2 - h_2^2}{2} \cdot \gamma_m \cdot \tan^2 \left( \frac{45^\circ - \phi'}{2} \right) \]  

(7)

where \[ \gamma_m \cdot \tan^2 \left( \frac{45^\circ - \phi'}{2} \right) \]  

\( \phi' = \) equivalent angle of friction

\[ \tan \phi' = \frac{(\gamma_m \cdot h_1 \cdot \tan \phi + c)}{\gamma_m \cdot h_1} \]  

(8)

where \( \phi = \) angle of repose of foundation material
\( c = \) unit cohesion of foundation material
\( \gamma_m = \) mean unit weight of the dam and foundation weighted in proportion to the depth of each.

\[ \gamma_m = \frac{\gamma_d \cdot (h_1 - h_2) + \gamma_f \cdot h_2}{h_1} \]  

where \( \gamma_d = \) density of the dam; \( \gamma_f = \) density of foundation material

The average unit shear \( = S_{av} = P/b \)

Hence the maximum unit shear \( = S_{max} = 1.4 \times S_{av} \)
The maximum unit shear occurs at 0.4 B from point J.
The unit shear strength below toe K
\[ S_1 = c + \gamma_f \cdot h_2 \cdot \tan \phi \]
The unit shear strength at point J
\[ S_2 = c + \gamma_f \cdot h_1 \cdot \tan \phi \]
Therefore the average shear strength
\[ S = (S_1 + S_2)/2 \]
The factor of safety against shear
\[ F.S. = S/S_{av} \]
The factor of safety obtained must be more than 1.5 for stability of foundation against shear.
The F.S. at maximum shear, point L, let
\[ S = c + \gamma_{av} \cdot h \cdot \tan \phi = (\gamma_d \cdot h + \gamma_f \cdot h_2)/(h + h_2) \]
And factor of safety (F.S.) \( = S/S_{max} \)
For stability the value of F.S. so obtained must be more than 1.0.
Chapter 3

GRAVITY DAMS

Basically, gravity dams are solid concrete structures that maintain their stability against design loads from the geometric shape and the weight and strength of the structure. Generally, they are constructed on a straight axis, but may be slightly curved or angled to accommodate the specific site conditions. Gravity dams typically consist of a non-overflow section(s) and an overflow section or spillway. They are constructed with masonry or concrete but of late conventional concrete or roller-compacted concrete are popular.

The two general concrete construction methods for concrete gravity dams are conventional placed mass concrete and roller-compacted concrete (RCC).


(1) Conventionally placed mass concrete dams are characterized by construction using materials and techniques employed in the proportioning, mixing, placing, curing, and temperature control of mass concrete. Construction incorporates methods that have been developed and perfected over many years of designing and building mass concrete dams. The cement hydration process of conventional concrete limits the size and rate of concrete placement and necessitates building in monoliths to meet crack control requirements. Generally using large-size coarse aggregates, mix proportions are selected to produce a low-slump concrete that gives economy, maintains good workability during placement, develops minimum temperature rise during hydration, and produces important properties such as strength, impermeability, and durability. Dam construction with conventional concrete readily facilitates installation of conduits, penstocks, galleries, etc., within the structure.

(2) Construction procedures include batching and mixing, and transportation, placement, vibration, cooling, curing, and preparation of horizontal construction joints between lifts. The large volume of concrete in a gravity dam normally justifies an onsite batch plant, and requires an aggregate source of adequate quality and quantity, located at or within an economical distance of the project. Transportation from the batch plant to the dam is generally performed in buckets ranging in size from 4 to 12 cubic yards carried by truck, rail, cranes, cableways, or a combination of these methods. The maximum bucket size is usually restricted by the capability of effectively spreading and vibrating the concrete pile after it is dumped from the bucket. The concrete is placed in lifts of 5- to 10-foot depths. Each lift consists of successive layers not exceeding 18 to 20 inches. Vibration is generally performed by large one-man, air-driven, spud-type vibrators. Methods of cleaning horizontal construction joints to remove the weak laitance film on the surface during curing include green cutting, wet sand-blasting, and high-pressure air-water jet.
3) The heat generated as cement hydrates requires careful temperature control during placement of mass concrete and for several days after placement. Uncontrolled heat generation could result in excessive tensile stresses due to extreme gradients within the mass concrete or due to temperature reductions as the concrete approaches its annual temperature cycle. Control measures involve precooling and postcooling techniques to limit the peak temperatures and control the temperature drop. Reduction in the cement content and cement replacement with pozzolans have reduced the temperature-rise potential. Crack control is achieved by constructing the conventional concrete gravity dam in a series of individually stable monoliths separated by transverse contraction joints. Usually, monoliths are approximately 50 feet wide.

b. Roller-compacted concrete (RCC) gravity dams.

The design of RCC gravity dams is similar to conventional concrete structures. The differences lie in the construction methods, concrete mix design, and details of the appurtenant structures. Construction of an RCC dam is a relatively new and economical concept. Economic advantages are achieved with rapid placement using construction techniques that are similar to those employed for embankment dams. RCC is a relatively dry, lean, zero slump concrete material containing coarse and fine aggregate that is consolidated by external vibration using vibratory rollers, dozer, and other heavy equipment. In the hardened condition, RCC has similar properties to conventional concrete. For effective consolidation, RCC must be dry enough to support the weight of the construction equipment, but have a consistency wet enough to permit adequate distribution of the past binder throughout the mass during the mixing and vibration process and, thus, achieve the necessary compaction of the RCC and prevention of undesirable segregation and voids.

Site Selection

a. General. During the feasibility studies, the preliminary site selection will be dependent on the project purposes. Purposes applicable to dam construction include navigation, flood damage reduction, hydroelectric power generation, fish and wildlife enhancement, water quality, water supply, and recreation. The feasibility study will establish the most suitable and economical location and type of structure.

b. Selection factors.

(1) A concrete dam requires a sound bedrock foundation. It is important that the bedrock have adequate shear strength and bearing capacity to meet the necessary stability requirements. The foundation permeability and the extent and cost of foundation grouting, drainage, or other seepage and uplift control measures should be investigated.
(2) The topography is an important factor in the selection and location of a concrete dam and its appurtenant structures. Construction at a site with a narrow canyon profile on sound bedrock close to the surface is preferable, as this location would minimize the concrete material requirements and the associated costs.

(3) The criteria set forth for the spillway, powerhouse, and the other project appurtenances will play an important role in site selection. The relationship and adaptability of these features to the project alignment will need evaluation along with associated costs.

(4) Additional factors of lesser importance that need to be included for consideration are the relocation of existing facilities and utilities that lie within the reservoir and in the path of the dam. Included in these are railroads, powerlines, highways, towns, etc. Extensive and costly relocations should be avoided.

(5) The method or scheme of diverting flows around or through the damsite during construction is an important consideration to the economy of the dam. A concrete gravity dam offers major advantages and potential cost savings by providing the option of diversion through alternate construction blocks, and lowers risk and delay if overtopping should occur.

Forces on Gravity Dams

3-3. Loads

a. General. In the design of concrete gravity dams, it is essential to determine the loads required in the stability and stress analysis. The following forces may affect the design:

1. Dead load. 2. Water Pressure (Headwater and tailwater pressures).
3. Uplift. 4. Temperature. 5. Earth and silt pressures. 6. Ice pressure.
10. Wave pressure. 11. Reaction of foundation.

b. Dead load.

Dead load comprises the major resisting force.
The dead loads considered should include the weight of concrete, superimposed backfill, and appurtenances such as gates and bridges.
In the computation of the dead load, relatively small voids such as galleries are normally not deducted except in low dams, where such voids could create an appreciable effect upon the stability of the structure. The cross section of the dam is divided into several triangles and rectangles. Weight of each triangle and rectangle and their points of application at respective centre of gravity are computed. The resultant of all these downward forces is thus found by taking moments of the component forces which constitute the total weight of the dam.
acting at its centre of gravity. Unit weight of concrete and masonry is taken as 2400kg/m³ and 2300kg/m³.

c. Water Pressure; (Headwater and tailwater).

Water pressure is the major external force acting on the dam. As the water is stored in the reservoir, and stands against the body of the dam, it exerts horizontal pressure on the dam.

**Fig. G.D. 1 Headwater pressure with vertical upstream face**

When the upstream face of the dam is vertical, the water pressure is resolved in two components:
- Horizontal pressure: \( P = \frac{1}{2} \gamma h \) acting at \( h/3 \) from base.

**Fig. G.D. 2 Water pressure for slanted upstream face and water at tailrace**

When the upstream face of the dam is vertical, the water pressure is \( P = \frac{1}{2} \gamma h^2 \) and acting at \( h/3 \) from base.

When the upstream face is slanted, the water pressure is resolved in two components:
- Horizontal pressure: \( P = \frac{1}{2} \gamma h^2 \) and acting at \( h/3 \) from base.
Vertical pressure \( W_w = \) weight of water on slanted side and acting at centre of gravity of volume of water.

**Uplift Pressure.**

It is the second major external force acting upwards on the dam. Uplift pressure resulting from headwater and tailwater exists through cross sections within the dam, at the interface between the dam and the foundation, and within the foundation below the base. This pressure is present within the cracks, pores, joints, and seams in the concrete and foundation material. Uplift pressure is an active force that must be included in the stability and stress analysis to ensure structural adequacy. These pressures vary with time and are related to boundary conditions and the permeability of the material.

(1) Along the base.

(a) General. The uplift pressure will be considered as acting over 100 percent of the base. A hydraulic gradient between the upper and lower pool is developed between the heel and toe of the dam. The pressure distribution along the base and in the foundation is dependent on the effectiveness of drains and grout curtain, where applicable, and geologic features such as rock permeability, seams, jointing, and faulting. The uplift pressure at any point under the structure will be tailwater pressure plus the pressure measured as an ordinate from tailwater to the hydraulic gradient between upper and lower pool.

![Diagram of Uplift Distribution Without Foundation Drainage](image)

**Fig. G.D.3 Uplift distribution without foundation drainage**

(b) Without drains. Where there have not been any provisions provided for uplift reduction, the hydraulic gradient will be assumed to vary, as a straight line, from headwater at the heel to zero or tailwater at the toe. Determination of uplift, at any point on or below the foundation, is demonstrated in Figure G.D.3 above.
(c) With drains. Uplift pressures at the base or below the foundation can be reduced by installing foundation drains. The effectiveness of the drainage system will depend on depth, size, and spacing of the drains; the character of the foundation; and the facility with which the drains can be maintained. This effectiveness will be assumed to vary from 25 to 50 percent, and the design memoranda should contain supporting data for the assumption used.

Along the base, the uplift pressure will vary linearly from the undrained pressure head at the heel, to the reduced pressure head at the line of drains, to the undrained pressure head at the toe, as shown in Figure G.D.4

![Diagram of uplift distribution with drainage gallery](image)

**Figure G.D. 4   Uplift distribution with drainage gallery**

Where the line of drains intersects the foundation within a distance of 5 percent of the reservoir depth from the upstream face, the uplift may be assumed to vary as a single straight line, which would be the case if the drains were exactly at the heel. This condition is illustrated in Figure G.D.5

If the drainage gallery is above tailwater elevation, the pressure of the line of drains should be determined as though the tailwater level is equal to the gallery elevation.
(d) Grout curtain. For drainage to be controlled economically, retarding of flow to the drains from the upstream head is mandatory. This may be accomplished by a zone of grouting (curtain) or by the natural imperviousness of the foundation. A grouted zone (curtain) should be used wherever the foundation is amenable to grouting. Grout holes shall be oriented to intercept the maximum number of rock fractures to maximize its effectiveness. Under average conditions, the depth of the grout zone should be two-thirds to three-fourths of the headwater-tailwater differential and should be supplemented by foundation drain holes with a depth of at least two-thirds that of the grout zone (curtain).

Figure G.D.5. Uplift distribution with foundation drains near upstream face

Figure G.D.6. Uplift distribution cracked base with drainage, zero compression zone not extending beyond drains (3-4)
Where the foundation is sufficiently impervious to retard the flow and where grouting would be impractical, an artificial cutoff is usually unnecessary. Drains, however, should be provided to relieve the uplift pressures that would build up over a period of time in a relatively impervious medium. In a relatively impervious foundation, drain spacing will be closer than in a relatively permeable foundation.

(e) Zero compression zones. Uplift on any portion of any foundation plane not in compression shall be 100 percent of the hydrostatic head of the adjacent face, except where tension is the result of instantaneous loading resulting from earthquake forces. When the zero compression zone does not extend beyond the location of the drains, the uplift will be as shown in Figure G.D.6. For the condition where the zero compression zone extends beyond the drains, drain effectiveness shall not be considered. This uplift condition is shown in Figure G.D.7

![Diagram of uplift distribution]

Figure G.D.7. Uplift distribution cracked base with drainage, zero compression zone extending beyond drains (3-5).

When an existing dam is being investigated, the design office should submit a request to CECW-ED for a deviation if expensive remedial measures are required to satisfy this loading assumption.

(2) Within dam.
(a) Conventional concrete. Uplift within the body of a conventional concrete-gravity dam shall be assumed to vary linearly from 50 percent of maximum headwater at the upstream face to 50 percent of tailwater, or zero, as the case may be, at the downstream face. This simplification is based on the relative
impermeability of intact concrete which precludes the buildup of internal pore pressures. Cracking at the upstream face of an existing dam or weak horizontal construction joints in the body of the dam may affect this assumption.

(b) RCC concrete. The determination of the percent uplift will depend on the mix permeability, lift joint treatment, the placements, techniques specified for minimizing segregation within the mixture, compaction methods, and the treatment for watertightness at the upstream and downstream faces. A porous upstream face and lift joints in conjunction with an impermeable downstream face may result in a pressure gradient through a cross section of the dam considerably greater than that outlined above for conventional concrete.

**Wave Pressure**
The portions of the dam is subjected to the impact of waves, which is produced in water surface during wind

![Wave Pressure Diagram](image)

**Figure G.D. 8**
The wave pressure is determined by the following formula developed by D. A. Molitor

Wave height, \( h_w = 0.032(V\cdot F)^{1/2} + 0.763 - 0.271(F)^{1/4} \) for \( F<32 \text{ km} \) ------ (A)

and \( h_w = 0.032(V\cdot F)^{1/2} \) for \( F>32 \text{ km} \) ------ (B)

where \( h_w \) = height of wave in metres from top of crest to bottom of trough.
\( F \) = fetch or straight length of water expanse in km
\( V \) = velocity of wind in km.hr

The maximum pressure intensity due to wave pressure is given by

\[ P_w = 2.4 \times \gamma x h_w \] and act at \( h_w/2 \)m above still water.

Total force due to wave pressure

\[ P_w = \frac{1}{2}(2.4 \cdot w \cdot h_w) \times 5/3 \cdot h_w = 2 \cdot w \cdot h^2 = 2 \cdot 1000 \cdot h_w^2 \text{ kg/m} \]

This force act at 3/8\( h_w \) above the still water level of the reservoir.
Wind Load
Wind load in stability analysis is usually ignored

Earth and Silt

Earth pressures against the dam may occur where backfill is deposited in the foundation excavation and where embankment fills abut and wrap around concrete monoliths. The fill material may or may not be submerged. Silt pressures are considered in the design if suspended sediment measurements indicate that such pressures are expected. Whether the lateral earth pressures will be in an active or an at-rest state is determined by the resulting structure lateral deformation.

Earthquake Forces
(1) General.

(a) The earthquake loadings used in the design of concrete gravity dams are based on design earthquakes and site-specific motions determined from seismological evaluation. As a minimum, a seismological evaluation should be performed on all projects located in seismic zones.

(b) The seismic coefficient method of analysis should be used in determining the resultant location and sliding stability of dams. In strong seismicity areas, a dynamic seismic analysis is required for the internal stress analysis.

(c) Earthquake loadings should be checked for horizontal earthquake acceleration and, if included in the stress analysis, vertical acceleration. While an earthquake acceleration might take place in any direction, the analysis should be performed for the most unfavorable direction.

(2) Seismic coefficient. The seismic coefficient method of analysis is commonly known as the pseudostatic analysis. Earthquake loading is treated as an inertial force applied statically to the structure. The loadings are of two types: inertia force due to the horizontal acceleration of the dam and hydrodynamic forces resulting from the reaction of the reservoir water against the dam (see Figure G.D.9). The magnitude of the inertia forces is computed by the principle of mass times the earthquake acceleration. Inertia forces are assumed to act through the center of gravity of the section or element. The seismic coefficient is a ratio of the earthquake acceleration to gravity; it is a dimensionless unit, and in no case can it be related directly to acceleration from a strong motion instrument. The coefficients used are considered to be the same for the foundation and are uniform for the total height of the dam.
Figure G.D. 9. Seismically loaded gravity dam, nonoverflow monolith

(a) Inertia of concrete for horizontal earthquake acceleration. The force required to accelerate the concrete mass of the dam is determined from the equation:

\[ P_e = M a_x = (W/g) a g = W a \]

Where

- \( P_e \) = horizontal earthquake force
- \( M \) = mass of dam
- \( a_x \) = horizontal earthquake acceleration = \( g \)
- \( W \) = weight of dam
- \( g \) = acceleration of gravity
- \( a \) = seismic coefficient

(b) Inertia of reservoir for horizontal earthquake acceleration. The inertia of the reservoir water induces an increased or decreased pressure on the dam concurrently with concrete inertia forces. Figure G.D.9 shows the pressures and forces due to earthquake by the seismic coefficient method. This force may be computed by means of the Westergaard formula using the parabolic approximation:

\[ P_{ew} = 2/3 \ C_e (a) y (hy)^{1/2} \]

where

- \( P_{ew} \) = additional total water load down to depth \( y \) (kips)
- \( C_e \) = factor depending principally on depth of water and the earthquake vibration period, \( t \), in seconds \( e \)
- \( h \) = total height of reservoir (feet)
Westergaard's approximate equation for \( Ce \), which is sufficiently accurate for all usual conditions, in pound-second feet units is:

\[
Ce = \frac{51}{\sqrt{1 - 0.72\left(\frac{h}{1,000t_e}\right)^2}}
\]

where \( t \) is the period of vibration.

COMBINATION OF FORCES FOR DESIGN

The design of a gravity dam is performed through an interactive process involving a preliminary layout of the structure followed by a stability and stress analysis. If the structure fails to meet criteria then the layout is modified and reanalyzed. This process is repeated until an acceptable cross section is attained.

Analysis of the stability and calculation of the stresses are generally conducted at the dam base and at selected planes within the structure. If weak seams or planes exist in the foundation, they should also be analyzed.

Basic Loading Conditions

Dams are designed for the most adverse combination of load conditions as have reasonable probability of simultaneous occurrence. The following basic loading conditions are generally used in concrete gravity dam designs (see Figure G.D 10).
(1) Load Condition No. 1 - unusual loading condition - construction.
(a) Dam structure completed. (b) No headwater or tailwater.

(2) Load Condition No. 2 - usual loading condition - normal operating.
(a) Pool elevation at top of closed spillway gates where spillway is gated, and at spillway crest where spillway is ungated. (b) Minimum tailwater. (c) Uplift. (d) Ice and silt pressure, if applicable.

(3) Load Condition No. 3 - unusual loading condition - flood discharge.
(a) Pool at standard project flood (SPF). (b) Gates at appropriate flood-control openings and tailwater at flood elevation. (c) Tailwater pressure. (d) Uplift. (e) Silt, if applicable. (f) No ice pressure.

(4) Load Condition No. 4 - extreme loading condition - construction with operating basis earthquake (OBE).
(a) Operating basis earthquake (OBE). (b) Horizontal earthquake acceleration in upstream direction. (c) No water in reservoir. (d) No headwater or tailwater.

(5) Load Condition No. 5 - unusual loading condition - normal operating with operating basis earthquake.
(a) Operating basis earthquake (OBE). (b) Horizontal earthquake acceleration in downstream direction. (c) Usual pool elevation. (d) Minimum tailwater. (e) Uplift at pre-earthquake level. (f) Silt pressure, if applicable. (g) No ice pressure.

(6) Load Condition No. 6 - extreme loading condition - normal operating with maximum credible earthquake.
(a) Maximum credible earthquake (MCE). (b) Horizontal earthquake acceleration in downstream direction. (c) Usual pool elevation. (d) Minimum tailwater. (e) Uplift at pre-earthquake level. (f) Silt pressure, if applicable. (g) No ice pressure.

(7) Load Condition No. 7 - extreme loading condition - probable maximum flood.
(a) Pool at probable maximum flood (PMF). (b) All gates open and tailwater at flood elevation. (c) Uplift. (d) Tailwater pressure. (e) Silt, if applicable. (f) No ice pressure.

b. In Load Condition Nos. 5 and 6, the selected pool elevation should be the one judged likely to exist coincident with the selected design earthquake event. This means that the pool level occurs, on the average, relatively frequently during the course of the year.
Stability Considerations

a. General requirements. The basic stability requirements for a gravity dam for all conditions of loading are:

(1) That it be safe against overturning at any horizontal plane within the structure, at the base, or at a plane below the base.

(2) That it be safe against sliding on any horizontal or near-horizontal plane within the structure at the base or on any rock seam in the foundation.

(3) That the allowable unit stresses in the concrete or in the foundation material shall not be exceeded.

Characteristic locations within the dam in which a stability criteria check should be considered include planes where there are dam section changes and high concentrated loads. Large galleries and openings within the structure and upstream and downstream slope transitions are specific areas for consideration.

b. Stability criteria. The stability criteria for concrete gravity dams for each load condition are listed in Table G.D -1.

<table>
<thead>
<tr>
<th>Load Condition</th>
<th>Resultant Location at Base</th>
<th>Minimum Sliding FS</th>
<th>Foundation Bearing Pressure</th>
<th>Concrete Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Compressive</td>
</tr>
<tr>
<td>Usual</td>
<td>Middle 1/3</td>
<td>2.0</td>
<td>≤ allowable</td>
<td>0.3 ( f'_c )</td>
</tr>
<tr>
<td>Unusual</td>
<td>Middle 1/2</td>
<td>1.7</td>
<td>≤ allowable</td>
<td>0.6 ( f'_c )</td>
</tr>
<tr>
<td>Extreme</td>
<td>Within base</td>
<td>1.3</td>
<td>≤ 1.33 x allowable</td>
<td>0.9 ( f'_c )</td>
</tr>
</tbody>
</table>

Overturning Stability

a. Resultant location.

The overturning stability is calculated by applying all the vertical forces (SV) and lateral forces for each loading condition to the dam and, then, summing moments (SM) caused by the consequent forces about the downstream toe. The resultant location along the base is:

\[
\text{Resultant location} = \sum_{M} \frac{M}{V}
\]

b. Criteria. When the resultant of all forces acting above any horizontal plane through a dam intersects that plane outside the middle third, a noncompression zone will result.

For usual loading conditions, it is generally required that the resultant along the plane of study remain within the middle third to maintain compressive stresses in the concrete. For unusual loading conditions, the resultant must remain within the middle half of the base. For the extreme load conditions, the resultant must remain sufficiently within the base to assure that base pressures are within prescribed limits.
Sliding Stability

a. General. The sliding stability is based on a factor of safety (FS) as a measure of determining the resistance of the structure against sliding. The horizontal components of the loads acting on a dam are resisted by frictional or shearing forces along horizontal or nearly horizontal planes in the body of the dam, on the foundation or on horizontal or nearly horizontal seams in the foundation. It follows that the total magnitude of the forces tending to induce sliding shall be less than the minimum total available resistance along the critical path of sliding. The sliding resistance is a function of the cohesion inherent in the materials and at their contact and the angle of internal friction of the material at the surface of sliding.

Definition of sliding factor of safety.
(1) The sliding FS is conceptually related to failure, the ratio of the shear strength (tF), and the applied shear stress (t) along the failure planes of a test specimen according to Equation 4-2:

\[
F.S. = \frac{\tau_F}{\tau} = \frac{(\sigma \tan \phi + c)}{\tau} = \frac{1}{P} \left[ \frac{(w-u)\tan \phi + CA}{F_{\phi}} \right]
\]

where \( t_F = s \tan f + c \), according to the Mohr-Coulomb Failure Criterion; \( w \) = total weight of dam; \( u \) = total upthrust force; \( \tan \phi \) = coefficient of internal friction of material; \( c \) = cohesion of the material at the plane considered; \( A \) = area under consideration for cohesion; \( F_{\phi} \) = partial factor of safety in respect of friction; \( F_c \) = partial factor of safety in respect of cohesion and \( P \) = total horizontal force.


<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Loading conditions</th>
<th>( F_{\phi} )</th>
<th>( F_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>For dams and the contact plane with foundation</td>
<td>For foundation Thoroughly investigated Others</td>
<td></td>
</tr>
<tr>
<td>(i)</td>
<td>1, 2, 3</td>
<td>1.5</td>
<td>3.6</td>
</tr>
<tr>
<td>(ii)</td>
<td>4, 5</td>
<td>1.2</td>
<td>2.4</td>
</tr>
<tr>
<td>(iii)</td>
<td>6, 7</td>
<td>1.0</td>
<td>1.2</td>
</tr>
</tbody>
</table>

Safety against Crushing
Safety against crushing is ensured if the compressive stresses produced are within the allowable stresses.
Maximum compressive stress = Direct stress + Bending moment

Bending moment \( M_B \)

\[
M_B = \frac{W}{A} + \frac{6W.e}{A.B} = \frac{W}{A} \left(1 + \frac{6.e}{B}\right)
\]

where \( W \) = weight of dam, \( A \) = area of dam section, \( e \) = eccentricity

**ELEMENTARY PROFILE OF A GRAVITY DAM**

The elementary profile of the gravity dam to bear only the water pressure, will be triangular in section as shown below. The width of the profile will be zero at the water surface, where the pressure is zero and maximum at the base, where the pressure is maximum. Thus the shape of the elementary profile is the same as that of the hydrostatic pressure distribution. When the reservoir is empty, the only force acting is self weight \((W)\) of the dam acting at a distance \(B/3\) from the heel. It is the maximum possible inner-most position of the resultant so that no tension develops and provides the maximum possible stabilizing force against overturning without causing tension at toe under empty dam condition. If any triangular profile other than the right-angled is provided, its weight will act closer to the upstream face to provide a higher stabilizing force but will cause tension to develop at the toe.

Vertical stresses developed when dam is empty will be:

\[
P_{\text{max}} = \frac{W}{A} \left(1 + \frac{6.e}{B}\right) \text{ ...at...toe}
\]

\[
P_{\text{min}} = \frac{W}{A} \left(1 - \frac{6.e}{B}\right) \text{ ...at...heel}
\]

Now when reservoir is full and downstream empty, forces acting on elementary profile will be:
a) Dam weight $W = \frac{1}{2} B \cdot h \cdot s_g \cdot \gamma_w$
   where $s_g =$ specific gravity of dam material (for concrete, $= 2.4$)
   $\gamma_w =$ unit weight of water ($9.81 \times 1000$ kg/m$^3$)

b). Water Pressure, $P = \frac{1}{2} \gamma_w h^2$ acting at 1/3 from the base.

c). Uplift pressure $u = c \cdot \gamma_w \cdot B \cdot h$
   where $c =$ uplift pressure intensity coefficient.

The base width, ‘B’ of the elementary profile is determined by the following two criteria:
   a) Stress criteria
   b) Stability or sliding criteria.

a) Stress criteria. When the reservoir is empty, there is no tension in the dam, the resultant is acting at the inner 1/3$^{rd}$ point $J$. When the reservoir is full, for no tension, the resultant must pass at outer 1/3$^{rd}$ point $K$.

Now taking moment of all forces about point $K$,

\[
\frac{1}{2} \cdot \gamma_w h^2 \cdot h/3 + \frac{1}{2} c \cdot \gamma_w B \cdot h \cdot B/3 - \frac{1}{2} B \cdot h \cdot s_g \cdot \gamma_w \cdot B/3 = 0
\]

Multiplying both sides by $6/\gamma_w h$

\[
h^2 + cB^2 - B^2 s_g = 0 \quad \text{or}
\]

\[
B^2(s_g - c) = h^2 \quad \text{from whence}
\]

\[
B = h/(s_g-c)^{1/2} \quad \text{----------------------------- (B)}
\]
By considering the force triangle, using similar triangles, we have:

\[
(W - u)/P = (h/3)/(B/3)
\]

or

\[
\frac{1}{2} Bhsg\gamma_w - \frac{1}{2} \gamma c.B.h = \frac{h}{B}
\]

or \( B^2 (sg - c) = h^2 \)

\[
B = \frac{h}{\sqrt{(sg - c)}}
\]

When the uplift force is not considered, \( c = 0 \) or \( B = h/ (sg)^{1/2} \)

Stability or Sliding Criteria
For no sliding of the dam, the horizontal forces causing sliding should be equal to the frictional forces, i.e.

\[
P = \mu (W - u)
\]

or \( 1/2 \gamma_w h^2 = \mu (1/2.B.h.sg. \gamma_w - 1/2. c.B. \gamma_w.h) \)

\[
B = h/\mu(sg - c)
\]

And neglecting uplift pressure,

\[
B = h/\mu sg
\]

The base width “B” of the elementary profile should be greater of the widths obtained in equation (B) or (C)

Stresses in the elementary profile
The normal stressing the dam is given by:

\[
p = \left( \frac{W - u}{B} \right) \left[ 1 + \frac{6e}{B} \right] = \cdots - - - - - - - (D)
\]

when the reservoir is full, the normal stress at toe is

\[
p = \left( \frac{W - u}{B} \right) (1+1) = \frac{2(W - u)}{B}
\]

\[
p = \frac{2}{B} \left[ 1/2 B.h.sg\gamma_w - 1/2 c.B.\gamma_w.h \right] = \gamma_w h (sg - c)
\]

The corresponding stress at the heel is:
\[ p = \left( \frac{W - u}{B} \right)(1 - 1) = 0 \quad \text{---} \quad (F) \]

When the reservoir is empty, the only force acting on the elementary profile is its weight, acting through \( J \). In this case, the maximum compressive stress at the heel = \( \frac{W}{B}(1+1) = 2\frac{W}{B} \) and the corresponding normal stress at toe is \( \frac{W}{B}(1-1) = 0 \).

The Practical Dam Profile

The elementary profile of a gravity dam is a triangle with maximum water surface at its apex. This profile is only theoretical one. For meeting the practical requirements certain changes have to be made namely: i) for communication, road has to be provided and therefore a top width; ii) for wave action, free board above the high flood level must be provided. The addition of the above will cause the resultant force to shift towards the heel. Earlier, when the reservoir was empty, the resultant was passing through the inner middle third. The above changes will shift it towards the heel, crossing the inner middle third point; this will create tension in the toe. To prevent this tension, some concrete is added in the dam body towards upstream side.

![Diagram of Practical dam profile](image_url)

Figure G.D. : Practical dam profile
Check drawing from main notes

The figure above shows the dam with possible pressure distribution on it. The maximum normal stress in the dam is the major principal stress which will be generated on the major principal plane. When the dam is full, the vertical direct stress is the maximum at the toe as the resultant is near the toe. The principal stresses near the toe is shown on the second diagram with a small element ABC.

Let \( d_r, d_s \) and \( d_b \) be the lengths of AB, AC and BC respectively, and let

\[
\begin{align*}
\sigma_1 &= \text{principal stress on plane AB} \\
\tau &= \text{shear stress}
\end{align*}
\]

Now considering unit length of the element ABC of the dam, the normal forces on the planes AB, AC and BC are \( \sigma_1 dr, \sigma_1 dr, \) \( PV db \) and \( p ds \sin \alpha \) respectively. Resolving all the forces in the vertical direction,

\[
p \cdot v \cdot db = p \cdot ds \sin \alpha + \sigma_1 \cdot dr \cos \alpha \quad \text{--------- (G)}
\]

But \( dr = db \cos \alpha \) and \( ds = db \sin \alpha \)

Therefore \( p \cdot v \cdot db = p \cdot ds \sin^2 \alpha + \sigma_1 \cdot db \cos^2 \alpha \)

Or \( p \cdot v = p \cdot \sin^2 \alpha + \sigma_1 \cdot \cos^2 \alpha \)
Therefore \[ \sigma_1 = (p \cdot v - p \cdot \sin^2 \alpha) / \cos^2 \alpha \]

\[ \sigma_1 = p \cdot v \cdot \sec^2 \alpha - p \cdot \tan \alpha \]  \hspace{1cm} \text{Equation (H)}

Equation (H) is known as the principal stress relationship, which is applicable to both upstream and downstream faces. For the downstream side the worst condition will be when there is no tail water, and hence \( p \) will be zero. In this case the major principal stress \( \sigma_1 \) is given by

\[ \sigma_1 = p \cdot v \cdot \sec^2 \alpha \]  \hspace{1cm} \text{Equation (I)}

If \( p_{e} \) is the intensity of hydrodynamic pressure of tail water due to an earthquake, the principal stress at the downstream is given by:

\[ \sigma_1 = p \cdot v \cdot \sec^2 \alpha - (p - p_{e}) \cdot \tan^2 \alpha \]

In the same way, considering the hydrodynamic pressure for the upstream side, in the horizontal direction:

\[ T \cdot db = \sigma_1 \cdot dr \cdot \sin \alpha + p \cdot ds \cdot \cos \alpha \]

\[ \tau = \sigma_1 \cdot \sin \alpha (dr/db) + p \cdot (ds/db) \cdot \cos \alpha \]

\[ \tau = (\sigma_1 - p) \cdot \sin \alpha \cdot \cos \alpha. \]

Now substituting the value of \( \sigma_1 \) from eq. (H)

\[ \tau = (p \cdot v \cdot \sec^2 \alpha - p \cdot \sec^2 \alpha) \cdot \cos \alpha \cdot \sin \alpha \]

or

\[ \tau = (p \cdot v - p) \cdot \tan \alpha \]

The shear stress for the upstream side has the same value but with reversed direction.

For the upstream side, \( \tau = -(p \cdot v - p) \cdot \tan \alpha \)

**STABILITY ANALYSIS OF GRAVITY DAMS**

The stability analysis of a gravity dam section can be done by any one of the following methods:

- a) Gravity method of two-dimensional method
- b) Slab analogy method
- c) Trial load twist method
- d) Lattice analogy method

Gravity Method or Two-dimensional Method.

Being an approximate method, it is used for the preliminary calculations.

The gravity method can be carried out by:
i) Graphical method or 
ii) Analytical method. 

We shall consider only the analytical method.

For the analytical method, the following steps are carried out:

a) Considering unit length of the dam, all vertical loads are determined the algebraic sum of all vertical forces $\sum V$ is calculated.

b) Considering unit length of the dam, all horizontal forces are determined and their algebraic sum $\sum H$ calculated.

c) The sum of the righting moments ($\sum M_R$) and the sum of overturning moments ($\sum M_O$) at the toe of the dam are calculated.

The difference between the algebraic sum of the overturning and righting moments is determined i.e $\sum M = \sum M_R - \sum M_O$

d) The location of the resultant force $F_R$ from the toe of the dam is also calculated by the following:

$$x_e = \frac{\sum M}{\sum V}$$ \hspace{1cm} (J)

e) Now the eccentricity of the resultant force is determined by:

$$e = \left(\frac{B}{2}\right) - x_e$$ \hspace{1cm} (K)

f) The normal stress at the toe of the dam is determined by:

$$N_{\sigma T} = \frac{\sum V}{B} \left(1 + \frac{6e}{B}\right)$$ \hspace{1cm} (L)

g) The normal stress at the heel is determined by

$$N_{\sigma H} = \frac{\sum V}{B} \left(1 - \frac{6e}{B}\right)$$ \hspace{1cm} (M)

h) The principal and shear stresses at the toe and heel are determined from

$$\sigma_T = p_v \cdot \sec^2 \alpha$$ \hspace{1cm} (l)

$$\tau = -(p_v - p) \tan \alpha$$

i) The factor of safety against overturning is calculated by:
\[(F.S.) = \frac{\sum M_R}{\sum M_O}\]

j) The factor of safety against sliding is calculated by:

\[\text{Sliding factor} - (F.S) = \frac{\mu \sum V}{\sum H}\]

\[\text{Shear friction factor} - (F.S) = \frac{\mu \sum V + Bq}{\sum H}\]

LOW AND HIGH GRAVITY DAMS

Low Dam: is one of limiting height such that the resultant of all forces passes through the middle third and the maximum compressive stress at the toe does not exceed the permissible limit i.e

\[\sigma_1 = \gamma_w H(s-c+1) = f_c\]

\[H = \frac{f}{\gamma_w (s.g - c + 1)} \quad \text{or} \]

or limiting height

\[H = \frac{f}{\gamma_w (s.g + 1)}\]

The limiting height, for the usual stress of dam material i.e. \(\gamma_w = 9.81 \times 1000 \text{ kg/m}^3\), \(\rho = 2.4\) and \(f = 30 \text{ kg/cm}^2\) (or 300 tonnes /m²) gives:

\[H = \frac{300 \times 1000}{9.81 \times 1000 (2.4 + 1)} = 8.99 \text{m}\]

s = Specific gravity of material; c = coefficient of uplift pressure, \(\gamma_w =\) specific weight of water; f = permissible compressive stress of material.

High Dam: A dam the height of which exceeds the limiting height of low dams is termed as high. In a high dam, the allowable stress are often exceeded if the resultant of all the forces were to pass through the middle third; to avoid excessive stresses the resultant is maintained still near the centre of the base for which purpose the downstream slope is flattened and the upstream slope is also provided with a batter.
Design of Gravity Dams

Before starting, one must establish whether it is low or high dam using the above relationship.

\[ B = \frac{h}{\sqrt{S - c}} \]

Figure G.D.  13. Economic section of Low Gravity Dam

Top width is chosen according to Creager must be about 14% of the dam’s height.
**Free Board**: is provided on the basis of height of waves and other practical considerations. In practice a free board of between (3 – 5)% of dam height is used.

\[ B_1 = \frac{h}{\sqrt{S_s - c}} \quad \text{or} \quad B_1 = \frac{h}{\mu(S_s - c)} \]

**WORKED EXAMPLES**

The diagram below shows the cross section of a masonry dam. Determine the stability of the dam. Also determine the principal stress at the toe and heel of the dam. Take unit weight of dam material as 2250 kg/m³, density of water 1000 kg/m³ and the permissible shear stress of joint = 15 kg/cm². Assume value of coefficient of friction \( \mu = 0.75 \)

**SOLUTION**

Stability of dam is tested assuming no free board.

A. Vertical forces
   i) Self weight of dam  = \((12 + 2.5)/2 \times 15 \times 1 \times 2250 = 244,687.5 \text{ kg}\)

   \[ \text{ii) Weight of water in column DD'A} = (1 \times 15)/2 \times 1000 = 7500 \text{ kg} \]

   \[ \text{iii) Uplift force on dam} = (15 \times 12)/2 \times 1000 = 90,000 \text{kg} \]
iv) Therefore \( \sum V = 244687.5 + 7500 - 90,000 = 162.187.5 \text{ kg} \)

v) Horizontal water pressure \( = (\gamma \times h^2)/2 = (1000 \times 15^2)/2 = 112,500 \text{ g} \)

Calculation of moments due to various forces about toe of dam

vi) Moment of self weight
\[
(1 \times 15)/2(1 + 2.5 + 8.5) + (2.5 \times 15 \times 2250)/(2.5/2 + 8.5) + (8.5 \times 15)/2(2/3 \times 8.5)
\]
\[
= 194,062.5 + 822,656.25 + 406,406.25 = 1,423,124.9 \text{ kg-m (+ve)}
\]

vii) Moment due to weight of water in DD'A
\[
7500 \times (1 + 2.5 + 8.5) = 78,750 \text{ kg-m (+ve)}
\]

viii) Moment due to uplift force
\[
82,500 \times 2/3 \times 12 = 660,000 \text{ kg-m (-ve)}
\]

ix) Moment due to horizontal water pressure
\[
112,500 \times 1/3 \times 15 = 562,500 \text{ kg-m (-ve)}
\]

Therefore \( \sum M = 1,423,124.9 + 78,750 - 660,000 - 562,500 = + 279,374.9 \text{ kg-m} \)

Factor of safety calculation

x) Factor of safety against overturning \( = \frac{\text{Resisting Moments}}{\text{Overturning Moments}} \)
\[
\frac{(1423124.9 + 78750)}{(660,000 + 562,500)} = 1.228 < 2 \text{ unsafe.}
\]

xi) Factor of safety against sliding \( = \frac{\mu \sum V}{\sum H} = \frac{0.75 \times 162,187.5}{112,500} = 1.08 > 1.0 \text{ safe} \)

xii) Shear friction factor \( = \frac{\mu \sum V + b \cdot q}{\sum H} = \frac{0.75 \times 162,187.5 + 12 \times 15 \times 10^4}{112,500} = 17.08 \)

Stress calculation

Let the resultant be acting at \( x_{av} \) from the toe
\[
X_{av} = \frac{\sum M}{\sum H} = \frac{279,374.9}{162,187.5} = 1.72 \text{ m}
\]

Distance of resultant force from centre of dam, (eccentricity, \( e \))
\[
e = B/2 - x_{av} = 12/2 - 1.72 = 4.28
\]

Compressive stress at toe
\[
f_t = \frac{\sum V (1 + 6.0 \times e/B)}{162,187.5 (1 + 6 \times 4.28/12)} = 509,268.7 > 50 \text{ kg/cm}^2 \text{ unsafe}
\]
Tensile stress at heel,
\[ f_h = \frac{\sum V(1 - 6.28/12)}{B} = \frac{162,187.5(1-6x4.28/12)}{B} = -15407.81 = -1.54 \text{ kg/cm}^2 \text{ unsafe} \]

In masonry dams, there should not be any tensile stress, therefore the section is not safe.

Calculation of principal stresses
From the diagram, \( \tan \beta = 8.5/15 = 0.567 \)
\[
\begin{align*}
\sec \alpha &= \frac{1 + 15^2}{15} = 1.002 \\
\sec \beta &= \frac{(8.5^2 + 15^2)}{15} = 1.149 \\
\tan \alpha &= \frac{1}{15} = 0.067 
\end{align*}
\]

Principal stress at toe
\[ \sigma = p_n \sec^2 \beta = 509,268.7 \times 1.149 = 672,336 \text{ kg/m}^2 \]
Shear Stress at toe
\[ T = p_n \tan \beta = 509,268.7 \times 0.567 = 288,755 \text{ kg/m}^2 \]
Principal stress at heel
\[ \sigma_h = p_n \sec^2 \alpha - p \tan^2 \alpha = -15407.81 \times (1.002)^2 - 1000 \times 15 \times (0.067)^2 
= (15469.5 - 67.33) = 15402.16 \text{ kg/m}^2 \]
Shear stress at heel
\[ = -(p_n - p) \tan \alpha = -(15407.81 - 10,000) \times 1/15 = -(25407.81)/15 = 1693.8 \text{ kg/m}^2 \]

Example 2: From the data given below, design a stone masonry gravity dam of practical profile.

Ground level, R.L = 1130.5m
R.L of HFL = 1155.5m
Wave height = 1.0m
Specific gravity of masonry = 2.5
Permissible compressive stress for stone masonry = 125t/m²

Solution
Free board height = 1.5 x height of wave = 1.5 x 1.0 = 1.5 m
Therefore required level of top of dam = 1155.5 + 1.5 = 1157m
Height of dam = 1157 – 1130.5 = 26.5m

Limiting height of dam = \( \frac{f_c}{\gamma_w(S + 1)} = \frac{125 \times 1000}{1000(2.5 + 1)} = 35.71 \text{m} \)

Therefore the dam is a low gravity dam.
The design of the dam can be done with respect to the details.

Depth of water  = 1155.5 – 1130.5 = 25m

Top width of dam = 14% height of dam = 14/100 x 26.5 = 3.71m

Assume a roadway width of 4.5 m

Therefore provide top width MN of dam = 4.5m

Base width of dam PS = $B_1 = \frac{h}{\sqrt{(S_s - c)}} = \frac{25}{\sqrt{2.5 - 0}} = 15.81m$;  assume 16m

Extra width JS = MN/16 = 4.5/16 = 0.281m  (assume 0.3m)

Vertical distance LK = $2a\sqrt{S_s} = 2 \times 4.5 \sqrt{2.5} = 14.23$ assume 14m

Vertical distance LR = $3.1a\sqrt{S_s} = 3.1 \times 4.5 \sqrt{2.5} = 22m$

Example 3

What should be the maximum height of elementary profile of a dam, if the safe limit os stress on the masonry should not exceed 350 tonnes/m². Assume weight of masonry 2.4 tonnes/m³. Determine the base width also. Determine $H$ and $B$ if uplift intensity factor is 0.67 and factor of safety is 2

Solution: The limiting height of elementary profile of a masonry dam
\[ H = \frac{f_c}{\gamma_w (s_g + 1)} = \frac{350 \times 1000 \times xg}{1000 \times xg (2.4 + 1)} = 103m \]

Base width \( B = \frac{H}{\sqrt{S_s - c}} = \frac{103}{\sqrt{2.4}} = 66m \)

\( \text{ii) F.S} = 2, c = 0.67: \quad H = \frac{f_c}{F_S \gamma_w (S_s - c + 1)} = \frac{350 \times 1000 \times xg}{2.0 \times 1000 \times xg (2.4 - 0.67 + 1)} = 64m \)

\[ B = \frac{H}{\sqrt{S_s - c}} = \frac{64}{\sqrt{2.4 - 0.67}} = 49m \]

Example 4.

A concrete gravity dam has maximum water level 305.0m, bed level 225.0m, top required level of dam 309.0m, downstream face slope starts at required level 300.0m, downstream slope 2:3, tail water is nil, upstream face of dam is vertical, centre line of drainage gallery is 8m downstream of upstream face, uplift pressure is 100% at heel, 50% at line of gallery and zero at toe, specific gravity of concrete is 2.4. Considering only weight, water pressure and uplift, determine

i) Maximum vertical stresses at toe and heel of dam

ii) major principal stresses at toe of dam and

iii) Intensity of shear stress on a horizontal plane near the toe

Solution:

Height of dam \( H = 309.0 - 225.0 = 84m \)

Depth of water \( h = 305.0 - 225.0 = 80m \)

1. Top width of dam = 14% of height = 0.14 \( \times 84 = 12m \)

2. Bottom width of dam = 12 + (300 – 225) x 2/3 = 62m

Calculating weight and moments by considering unit length of dam
Designation | Force | Moment arm | Moment about toe
---|---|---|---
Weight of dam, $w_1=12 \times 84 \times 1 \times 2.4 \times \gamma_w$ | $=2419 \gamma_w$ | 50 + 12/2 = 56 | 13546 $\gamma_w$
$w_2=50 \times 75/2 \times 1 \times 2.4 \gamma_w$ | $=4500 \gamma_w$ | 50 x 2/3 = 33.33 | 149985 $\gamma_w$
$w = w_1 + w_2$ | $= 6919$ | | $\sum M_1= (+) 285449 \gamma_w$
Uplift | | | |
$U_1=40 \times 8 \times 1/2 \gamma_w$ | $=160 \gamma_w$ | 54+2/3x8=59.33 | 9493
$U_2=40 \times 8 \gamma_w$ | $=320 \gamma_w$ | 54 + 4 =58 | 18560
$U_3=54 \times 40/2$ | $=1080 \gamma_w$ | 54x2/3=36 | 38880
$\sum u$ | $=1560 \gamma_w$ | | $\sum M_2= (-) 66933 \gamma_w$
Water pressure | | | |
$P = \gamma_w h^2/2=80 \times 80/2 \gamma_w$ | $=3200 \gamma_w$ | 80 x 1/3=26.67 | $\sum M_3= (-) 85344 \gamma_w$

$\sum V = W - U = 6919 - 1560 = 5359 \gamma_w \quad \sum M = M_1-M_2-M_3 = (+) 133172 \gamma_w$

Position of resultant from toe $x_{av} = M/V= 133172 \gamma_w /5359 \gamma_w=24.85$

Eccentricity, $e = B/2 - x_{av} = 62/2 - 24.85=6.15m$

Normal compressive stress at toe $p_n=V/B(1+6e/B)= 5359 \gamma_w /62(1+6x6.15/62)= 138 \gamma_w N/m^2$

Normal compressive stress at heel $p_n= V/B(1-6e/B)= 5359 \gamma_w /62(1-6x6.15/62)= 35 \gamma_w N/m^2$
Principal stress at toe \( \sigma = p_n \sec^2 \alpha - p' \tan^2 \alpha \)  
\( p' = \) zero because tail water is zero;  
\( \tan \alpha = 2/3 \)  
\( \sec^2 \alpha = 1 + \tan^2 \alpha = 1 + (2/3)^2 = 13/9 \)  
Therefore  
\( \sigma = p_n \sec^2 \alpha = 138 \, \gamma_w \times 13/9 = 199 \, \gamma_w \, N/m^2 \)

Intensity of shear stress on a horizontal plane near toe  
\[ T_{\tau_o} = (p_n - p') \tan \alpha = (138 \, \gamma_w - 0) \times 2/3 = 92 \, \gamma_w \, N/m^2 \]

Example 5. A concrete gravity dam has maximum reservoir level 150.0m, base level of dam = 100.0m, tail water elevation 110.0m, base width of dam 40m, location of drainage gallery 10m from upstream face which may be assumed as vertical. Compute the hydrostatic thrust and the uplift force per metre length of dam at its base level. Assume 50% reduction in net seepage head at the location of the drainage gallery.

Solution:

Free board = 5% of dam height = 0.05x50 = 2.5  adopt 3m

Dam height \( h = 50 + 3 = 53m \)

Top width \( a = 14\%xh = 0.14 \times 53 = 7.5m \) adopt 7.5m

<table>
<thead>
<tr>
<th>Designation</th>
<th>Force x ( \gamma_w )</th>
<th>Moment arm</th>
<th>Moment about toe x ( \gamma_w )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
**Weight of dam**

\[ W_1 = 7.5 \times 53 \times 1 \times 2.4 = 954 \]

\[ W_2 = 32.5 \times 43/2 \times 1 \times 2.4 = 1677 \]

\[ W_{\text{tail water}} = 8 \times 10/2 \times 1 \times 1 = 40 \]

\[ \sum V_1 = 2671 \]

\[ 34583 \]

**Uplift pressure**

\[ U_1 = 20 \times 10/2 = 100 \]

\[ U_2 = 10 \times 30 \times 1 = 300 \]

\[ U_3 = 10 \times 30 \times 1 = 300 \]

\[ U_4 = 20 \times 30/2 \times 1 = 300 \]

\[ \sum V_2 = 1000 \]

\[ 71031 \]

**Water pressure**

\[ U/S = \gamma_w h^2/2 = \gamma_w \times 1.10.10/2 = 1250 \]

\[ D/S = \gamma_w h^2/2 = \gamma_w \times 1.10.10/2 = 50 \]

\[ \sum H_3 = 1200 \]

\[ (-) 20,833 \]

\[ 167 \]

\[ \sum M_3 (-) 20,666 \]

**Position of resultant from toe**

\[ x_{av} = M/V = 25698/1671 = 15.38 \text{m} \]

**Eccentricity**

\[ e = B/2 - x_{ac} = 40/2 - 15.38 = 4.62 \text{m} \]

**Normal compressive stress at toe**

\[ p_n = V/B(1 + 6e/B) = 1671/40(1 + 6.4 \times 62/40) = 70.73 \gamma_w \]

**Normal compressive stress at heel**

\[ p' = V/B(1 - 6e/B) = 1671/40(1 - 6 \times 4.62/40) = 12.82 \gamma_w \]

**Maximum principal stress**

\[ \sigma = p_n \sec^2 \alpha - p' \tan^2 \alpha \quad p' = 10 \gamma_w \]

\[ \tan \alpha = 40/50, \quad \sec^2 \alpha = 1.64 \]

\[ \sigma = 70.73 \times 1.64 - 10(40/50)^2 = 110 \gamma_w \]

**Intensity of shear stress on a horizontal plane near toe**

\[ T_{\tau_0} = (p_n - p') \tan \alpha \]

\[ \tau_0 = 70.73 - 10 \times 40/50 = 49 \gamma_w \]
SPILLWAYS AND GATES

Spillway is a passageway to convey past the dam flood flows that cannot be contained in the allotted storage space or which are in excess of those turned into the diversion systems. Spillways function infrequently, at times of flood or sustained high runoff, when other facilities are inadequate. However its ample capacity is of prime importance for the safety of the dam and other hydraulic structures. Hydraulic aspects of spillway design relates to design of the three spillway components: control structure, discharge channel and terminal structure. The control structure regulates outflows from the reservoir and may consist of a sill, weir section, orifice, tube or pipe. Design problems here relate to determining the shape of the section and computing discharge through the section. The flow released from the control structure is conveyed to the streambed below the dam in a discharge channel. This can be the downstream face of the overflow section, a tunnel excavated through an abutment, or an open channel along the ground surface. The channel dimensions are fixed by the hydraulics of channel flow. Terminal structures are energy dissipating devices that are provided to return the flow into the river channel without serious scour or erosion at the toe of the dam.

Spillways are usually located at the normal flood level and the water just overflows the crest when it gets to that level. However, some spillways are equipped with gates, which are temporary barriers installed over the permanent crest of the spillway for storing additional water during period of low water seasons. All small flows exceeding the barrier top level are allowed to pass over the barrier, but during large flood flows, the barrier is removed and full spillway capacity is used to discharge the flood water.

Types of Spillways

Spillways may be classified, depending upon the type of structure as:

i) Side channel spillway
ii) Straight drop spillway
iii) Overflow or Ogee spillway
iv) Chute or Trough spillway
v) Shaft (Morning glory) spillway
vi) Siphon spillway

**Side channel spillway**

This type of spillway is most suited for earthfill and rockfill dams, in narrow canyons, where construction of other types of spillway is not possible. Also when a long overflow crest is required for limiting the surcharge head, this type is suited. It is also useful when its discharge is to be connected to a narrow discharge channel or tunnel. In this type the control weir is kept along the side and approximately parallel to the upper portion of the spillway. The discharge after passing over the crest turns at about 90° before flowing into a trough to be discharged.

Straight Drop (free overfall) Spillway. In this type, water is allowed to fall freely from a low weir and vertical fall structure. In some cases the crest of the spillway is extended in
the shape of overhanging lip which keeps the discharge away from the straight drop section. Ogee (Overflow) Spillway: is a special form of a weir whose shape is made to conform to the profile of lower nappe of a ventilated sheet of water falling from a sharp-crested weir. The profile is so shaped that the discharging water always remains in touch with the spillway surface.
Accordingly, the profile of the ogee spillway is made to the shape of the lower nappe of a free falling jet. The downstream curve of the ogee has the equation:

$$x^{1.85} = 2y(h)^{0.85}$$

(S-1)

where x and y are the co-ordinates of the crest profile measured from the apex of the crest, and h is the design water head.

Fig. S-1 Definition sketch of overflow spillway

The upstream curve may be approximate to the following:

- $a = 0.175h$
- $b = 0.292h$
- $r_1 = 0.50h$
- $r_2 = 0.20h$
\[
 y = \frac{0.724(x + 0.27h)^{0.85}}{(h)^{0.85}} \quad \text{(S-2)}
\]

For larger discharges, (e.g., flows beyond H.F.L.), the nappe may leave the ogee profile and will cause negative pressure resulting in cavitation and increase of discharge.

**Discharge over an Overflow Spillway**

The discharge equation of the ogee-shaped spillway is given by

\[
 Q = C.L_e(h_e)^{3/2} \quad \text{---------------------------(S-3)}
\]

Where,
- \(Q\) = discharge over the ogee spillway
- \(L_e\) = effective length of the crest
- \(h_e = h + \frac{v^2}{2g}\) = total water head at crest including the velocity approach
- \(C\) = a variable coefficient of discharge, whose value varies from 2.1 to 2.5 depending on various factors.

The effective length of the crest is given by the equation:

\[
 L_e = L - 2(NK_p - K_a) \quad \text{---------------------------(S-4)}
\]

Where
- \(L\) = Total clear length of crest
- \(N\) = No. of piers in the spillway
- \(K_p\) = pier contraction coefficient
- \(K_a\) = abutment contraction coefficient

**Table S-2 Pier Contraction Coefficient \(K_p\)**

<table>
<thead>
<tr>
<th>Condition of Pier</th>
<th>Value of (K_p)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Pointed nose piers</td>
<td>0.00</td>
</tr>
<tr>
<td>2 Round nose piers</td>
<td>0.01</td>
</tr>
<tr>
<td>3 Square-nose pier with corners rounded on a radius 0.1 times pier thickness</td>
<td>0.02</td>
</tr>
</tbody>
</table>

**Table S-3 Abutment Contraction Coefficient (\(K_p\))**

<table>
<thead>
<tr>
<th>Condition of Abutment</th>
<th>Value of (K_p)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Rounded abutment with head wall as 90° to the direction of flow, when (0.5h &gt; r &gt; 0.15h)</td>
<td></td>
</tr>
<tr>
<td>2 Rounded abutment where (r &gt; 0.5H_o) and the angle of head wall with direction of flow is (&lt;45°)</td>
<td>0.00</td>
</tr>
<tr>
<td>3 Square abutment with head wall at 90° to the direction of flow of water</td>
<td>0.20</td>
</tr>
</tbody>
</table>

**Example 9.10**
Design an overflow spillway section for a design discharge of 50,000cfs. The upstream water surface is at Elv. 800 and the channel floor is 680. The spillway, having a vertical face, is 180ft long.

**Solution**
1. Assume $C = 3.95$

2. from the discharge equation

$$h_c^{3/2} = \frac{Q}{CL} = \frac{50,000}{3.95(180)} = 70.32$$

$$h_c = 17.1$$

3. Depth of water upstream = $800 – 680 = 120$ ft.
   Velocity of approach $v_o = \frac{50,000}{120(180)} = 2.31$ ft/sec
   Velocity head $= v^2/2g = (2.31)^2/(32.2) = 0.08$ ft

4. Maximum water head = $17.1 – 0.08 = 17.0$ ft

5. Height of crest, $P = 120 – 17.0 = 103$ ft.

6. Since $h_c < 30$ ft, design head, $h_d = 17.1/1.42 = 12.0$ ft

7. $P/h_d = 103/12 = 8.58 > 1.33$, high overflow section

8. Downstream quadrant of the crest shape

$$y/12 = \frac{1}{2} (x/12)^{1.85} \text{ or } y = 0.06x^{1.85}$$

<table>
<thead>
<tr>
<th>X (select) ft</th>
<th>Y (computed) ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>1.18</td>
</tr>
<tr>
<td>10</td>
<td>4.25</td>
</tr>
<tr>
<td>15</td>
<td>9.00</td>
</tr>
<tr>
<td>20</td>
<td>15.30</td>
</tr>
<tr>
<td>30</td>
<td>32.40</td>
</tr>
</tbody>
</table>

9. Point of tangency assume a downstream slope of 2:1

$$(X_{DT})/h = 0.485(K\alpha)^{1.176}$$

where $X_{DT} =$ horizontal distance from the apex to the downstream tangent point

$\alpha =$ slope of the downstream face

$X_{DT} = 0.485[2(2)]^{1.176} (12) = 30$ ft

10. Upstream quadrant. $A/h = 0.28, B/h = 0.165$

$A = 0.28(12) = 3.36; B = 0.165(12) = 2.00$ ft

$$(x^2)/(3.36)^2 + (2.0 - y)^2/(2.0)^2 = 1$$

<table>
<thead>
<tr>
<th>X (selected) ft</th>
<th>Y (computed) ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0.09</td>
</tr>
<tr>
<td>2.0</td>
<td>0.39</td>
</tr>
<tr>
<td>3.0</td>
<td>1.10</td>
</tr>
<tr>
<td>3.36</td>
<td>2.00</td>
</tr>
</tbody>
</table>

CHUTE OR TROUGH SPILLWAY

Chute spillway is a type of spillway in which the discharge is conveyed from a reservoir to the downstream river level through a steep open channel placed either along the dam abutment or through a saddle. In a chute spillway, the velocity of flow is always greater than the critical. The name chute applies, regardless of the control device used at the
head of the chute, which can be an overflow crest, a gated orifice or a side channel crest. The chute spillway consists of four parts: an entrance channel, a control structure or crest, the sloping chute and a terminal structure. The entrance structure is usually an open channel of sub-critical flow. The critical velocity occurs when the water passes over the control.

Flow in the chute is maintained at supercritical stage until the terminal structure DE is reached. It is desirable that from B to C, where a heavy cut is involved, the chute is placed on a light slope. From C to D it follows the steep slope and ends with an energy dissipating device placed at the bottom of the valley D. The axis of the chute is kept as straight as possible; otherwise, the floor has to be super-elevated to avoid the piling up high-velocity flow around the curvature.

It is preferable that the width of the control section, the chute, and the stilling basin are the same.

To prevent hydrostatic uplift under the chute, a cutoff wall is provided under the control structure and a drainage system of filters and pipes is provided. When the stilling basin is operating, there is substantial uplift under the lower part of the chute and upstream part of the stilling basin floor. The floor must be made sufficiently heavy or anchored to the foundation.

**Slope of Chute Channel**

It is important that the slope of the chute in the upstream section BC chute, should be sufficiently steep to maintain a supercritical flow to avoid the formation of a hydraulic jump. Therefore slope should always be above the critical slope.

For the Manning’s formula;
\[ Q = \frac{1}{n} AR^{2/3} I^{1.2} \]  \hspace{1cm} (S-5) 

For a rectangular channel under critical flow conditions,

\[ h^3 = \frac{q^2}{g} \]  \hspace{1cm} (S-6)

By putting this into Manning’s formula, the critical slope could be determined. The slope to be given to the chute must be greater than this critical slope. A review of existing spillways indicates that the actual slopes of the upstream section of the chute are 1 or 2 % or more.

**Chute Sidewalls**

Except for the diverging or converging sections, chute channels are designed with parallel vertical sidewalls, commonly of reinforced concrete 30 to 45cm thick; designed as retaining wall. The height of the walls is designed to contain the depth of flow for the spillway design flood by gradually flows hydraulics equation. Free board or allowance is made for pier end waves, roll waves and air entrainment. In view of uncertainties involved in the evaluation of surface roughness, pier end waves, roll waves and air entrainment, the following empirical equation is added to the computed depth of water surface profile

\[ \text{Freeboard (ft)} = 2.0 + 0.002V h^{1/3} \]  \hspace{1cm} (S-7)

where \( V \) - mean velocity in chute section under consideration, \( h \) – mean depth.

**SIDE CHANNEL SPILLWAYS**

In side-channel spillway, the overflow weir is placed along the side of the discharging channel, so that the flow over the crest falls into a narrow channel section (trough) opposite the weir, turns through a right-angle, and continues in the direction approximately parallel to the weir crest.
This type of weir is adoptable to certain special conditions, such as when a long overflow crest is desired but the valley is narrow, or where the overflow are most economically passed through a deep narrow channel or a tunnel. The crest is similar to an overflow or ordinary weir section. A control section downstream from the trough is achieved by constricting the channel or elevating the channel bottom to induce critical flow. Downstream from the control section functions as a chute spillway. Thus the side-channel design is concerned only with the hydraulic action in the trough upstream of the control section, where varied flow takes place. The water surface profile in the trough is determined from the momentum equation applied to gradual varied flow. If flow rate and velocity in left is $Q_1$ and $V_1$ while those at the right is $Q_2$ and $V_2$, then by applying the momentum equation, the rate of change of momentum in the reach $dx$ is equal to the external forces acting in the reach.

Rate of momentum at left (upstream) = $\rho Q_1 V_1$

Rate of momentum at downstream = $\rho Q_2 V_2$

Change in momentum = $\rho (Q_2 V_2 - Q_1 V_1)$ \hspace{1cm} ------------------------------------------(S-8)

Due to smallness of friction forces and weight component in the direction of flow, the only force acting is the hydrostatic pressure force.

The resultant hydrostatic force $F_{p1} - F_{p2} = \gamma (A_1, h_1 - A_2 h_2) = \gamma (Q_1/V_1 - Q_2/V_2) dy$ \hspace{1cm} -(S-9)
Equating equations (S-8) and (S-9) and rearranging

\[
dy = \frac{Q_2}{g} \left( \frac{V_1}{Q_1} + \frac{V_2}{Q_2} \right) \left[ (V_1 - V_2) + \frac{V_1(Q_2 - Q_1)}{Q_2} \right]
\]

---(S-10)

Equation (S-10) is solved by a trial and error procedure.

**Worked example:**
Design a side-channel trough for a spillway of 100ft length for a maximum discharge of 2500 cfs. The side-channel trough has a length of 100ft and a bottom slope of 1ft in 100 ft. A control section of 10 ft width is placed downstream from the trough with the bottom of the control at the same elevation as the bottom of the trough floor at the downstream end.

**Solution**
1. Critical depth at the control section, \( y_c = \left( \frac{q_1^2}{g} \right) \)
2. \( q_1 = \frac{2500}{10} = 250 \text{ cfs/ft} \)
3. \( y_c = \left( \frac{250^2}{32.2} \right)^{1/3} = 12.44 \text{ ft} \)
4. \( v_c = \frac{q_1}{y_c} = \frac{250}{12.44} = 20.1 \text{ ft/sec} \)
5. Velocity head, \( h_c = \frac{v_c^2}{2g} = \frac{(20.1)^2}{2(32.2)} = 6.27 \text{ ft} \)
6. For a side-channel trough, assume a trapezoidal section with \( 2V:1H \) slope and a 10ft bottom width. Also, assume that the transition loss from the end of side-channel trough to the control section is equal to 0.2 of the difference in velocity heads between the end of the transition.
7. The following energy equation may be written between the trough end and the control section:

\[ y_{100} + h_{100} = y_c + h_c + 0.2(h_c - h_{100}) \quad \text{or} \quad y_{100} + 1.2h_{100} = 12.44 + 1.2(6.27) = 19.96 \quad \text{(S-11)} \]

where the subscript \( c \) refers to “critical” and “100” refers to the distance of the trough from the upstream end of the spillway. Equation (S-11) is solved by trial and error. Assume \( y_{100} = 19.2 \); then \( A = 376.3 \text{ ft}^2 \), \( v = Q/A = 2500/376.3 = 6.64 \text{ ft/sec} \), and \( h_{100} = (6.64)^2/(2(32.2))0.68 \). Thus Eq. (S-11) is satisfied.

8. The known values above relate to section 1 at the downstream end of the trough. Section 2 is taken at the upper end of a selected increment \( dx \), 25 ft in this case. A value of the change in water level, \( dy \), is assumed for each reach, all terms of equation (S-10) are evaluated, and \( dy \) is computed, as in table below. The assumed and computed values of \( dy \) should match; otherwise, a new value is assumed for \( dy \).

9. The process is repeated until the upstream end of the channel is reached.

<table>
<thead>
<tr>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>dx (selected)</td>
<td>Bottom Level</td>
<td>( dy ) (Assume)</td>
<td>Water Level</td>
<td>( y ) (col 4-col 2)</td>
<td>( A )</td>
<td>( Q = q(L-dx) )</td>
<td>( v = Q/A )</td>
</tr>
<tr>
<td>D/S end</td>
<td>100.0</td>
<td>119.2</td>
<td>19.20</td>
<td>376.3</td>
<td>2500</td>
<td>6.64</td>
<td></td>
</tr>
<tr>
<td>25</td>
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<td>1.0</td>
<td>120.2</td>
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<td>1875</td>
<td>4.71</td>
</tr>
<tr>
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<td>1250</td>
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<td></td>
<td>0.40</td>
<td>120.22</td>
<td>19.72</td>
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<td>0.25</td>
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<td>625</td>
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<tr>
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<td>100.90</td>
<td>0.10</td>
<td>120.57</td>
<td>19.67</td>
<td>390.2</td>
<td>250</td>
<td>0.64</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.07</td>
<td>120.54</td>
<td>19.64</td>
<td>389.3</td>
<td></td>
<td>0.64</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>( Q_1 + Q_2 )</th>
<th>( Q_2 - Q_1 )</th>
<th>( v_1 + v_2 )</th>
<th>( v_2 - v_1 )</th>
<th>dy computed</th>
<th>Remarks on assumed ( dy )</th>
</tr>
</thead>
<tbody>
<tr>
<td>4375</td>
<td>-625</td>
<td>11.35</td>
<td>-1.93</td>
<td>0.63</td>
<td>High</td>
</tr>
<tr>
<td>4325</td>
<td>-625</td>
<td>11.48</td>
<td>-1.80</td>
<td>0.61</td>
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<tr>
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<td>4.79</td>
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<tr>
<td>875</td>
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<td>2.24</td>
<td>-0.96</td>
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<td>875</td>
<td>-375</td>
<td>2.24</td>
<td>-0.96</td>
<td>0.07</td>
<td>ok</td>
</tr>
</tbody>
</table>

Note: 1. \( q = Q/L = 2500/100 = 25 \text{ cfs/ft} \); bottom slope = 1 in 100 ft given
2. Bottom level (col. 2) = slope \( x \) channel length + datum
3. Water level (col. 4) = final water level at the preceding station (section) + assumed \( dy \).
4. \( A \) (col. 6) = area of cross-section of trough computed for depth \( y \) in col. 5
5. \( Q \) (col. 7) = \( q(L - \sum dx) \), \( L \) = crest length
6. \( Q_1 + Q_2 \) (col. 9) = Col. 7 + value in col. 7 at the preceding station (section)
MORNING GLORY OR SHAFT SPILLWAYS

In this type of spillway, water enters over a horizontal circular crest and then drops through a circular shaft, and then through a horizontal tunnel or conduit. This type of spillway is suitable under the following situations:

1. For damsites with steeply rising abutments particularly where a diversion tunnel can be utilized as discharge carrier.
2. For sites in narrow canyons.
3. For sites where there is inadequate space for locating other type of spillway.

Advantages: The main advantage of such type of spillways are as follows:

1. The nearly maximum capacity may be attained at relatively low heads.
2. It is ideal for sites where maximum spillway overflow is to be limited.

This type of spillway consists of four parts: 1) a circular weir at the entry, 2) a flared transition conforming to the shaft of the lower nappe of a sharp-crested weir, 3) a vertical drop shaft, and 4) a horizontal or near-horizontal outlet conduit or tunnel. As the head increases, the control shifts from weir crest, to drop shaft, and to outlet conduit.

Table S-3. Discharge characteristics of shaft spillway

<table>
<thead>
<tr>
<th>Control Point</th>
<th>Condition</th>
<th>Characteristics</th>
<th>Discharge relations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weir crest</td>
<td>Unsubmerged flow</td>
<td>Weir flow</td>
<td>$Q = CLh^{3/2}$ ------(S-12)</td>
</tr>
<tr>
<td>Throat of drop shaft</td>
<td>Partially submerged</td>
<td>Orifice flow</td>
<td>$Q = c_d A_t \sqrt{2gH_a} ... C_d = 0.95$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>------------------------- (S-13)</td>
</tr>
<tr>
<td>Downstream of outlet</td>
<td>Submerged flow</td>
<td>Pipe flow</td>
<td>$Q = A_t \sqrt{2gH_T \Sigma K}$ ----(S-14)</td>
</tr>
<tr>
<td>conduit</td>
<td></td>
<td></td>
<td>$\Sigma K =$ loss coefficient through pipe</td>
</tr>
</tbody>
</table>

Condition 1: a free-discharging weir prevail as long as the nappe forms to converge into the shape of a solid jet.
Condition 2: weir crest is drowned out. The US Bureau of Reclamation (1977) indicated that this condition is approached when $H_d/R_s > 1$, where $H_d$ is the design head and $R_s$ is the radius of the crest.
Condition 3: Spillway is flooded out, showing only a slight depression and eddy at the surface. Under condition 3, the head rises rapidly for a small increase in discharge. Thus the design is not recommended under this condition (i.e., under the design head, the outlet conduit should not flow more than 75% full).

The U.S. Bureau of Reclamation suggested that the following weir formula may be used for flow through the shaft spillway entrance regardless of the submergence, by adjusting the coefficient to reflect the flow conditions.

$$Q = C(2\pi R_s)H^{3/2} \text{ -------------------- (S-15)}$$
Where \( C = \) discharge coefficient related to \( H_d/R_s \) and \( P/R_s \), where \( H_d = \) design head and \( P = \) crest height from the outlet pipe; \( R_s = \) radius to the circular crest; \( H = \) head over the weir (Gupta page 495).

Equation (S-15) can be used to determine the crest size (radius), \( R_s \) for a given discharge under the maximum head.

Equation (S-13) can be used to determine the shape of the transition (drop shaft) that is required to pass the design discharge with the maximum head over the crest.

**Worked Example:** A shaft spillway is to discharge 2000 cfs under a design head of 10 ft. Determine the minimum size of the overflow crest. Also determine the shape of the transition if the control section is 4 ft below the crest level.

**Solution**

1. Since the coefficient \( C \) is related to \( P \) and \( R_s \), assume that \( P/R_s > 2 \) and determine \( R_s \) by trial and error.
2. Try \( R_s = 7 \) ft.
   \[ H_d/R_s = 10/7 = 1.43 \]
   From the curve of \( C \) as a function of \( H_d/R_s \), we choose \( C = 1.44 \)
3. From equation (S-15)
   \[ Q = C(2\pi R_s) H^{3/2} = 1.44(2\pi) \left( \frac{10}{7} \right)^{3/2} = 2002 \text{ cfs} \]
   This is practically the same as the required discharge. Hence the crest radius = 7 ft.
4. Depth of the control section from the water surface;
   \[ H_a = 10 + 4 = 14 \text{ ft.} \]
5. From equation (S-13),
   \[ Q = c_d A_t \sqrt{2gH_a} \ldots \]
   \[ C_d = 0.95 \]
   \[ R^2 = 2000/(0.95\pi(2gH_a)^{1/2}) \]
   or \( R = 9.14/(H_a)^{1/4} \)

<table>
<thead>
<tr>
<th>( H_a ) (select), ft</th>
<th>( R ) (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>4.73</td>
</tr>
<tr>
<td>16</td>
<td>4.57</td>
</tr>
<tr>
<td>18</td>
<td>4.44</td>
</tr>
<tr>
<td>20</td>
<td>4.32</td>
</tr>
</tbody>
</table>

**SIPHON SPILLWAYS**

A siphon spillway is a short enclosed duct whose longitudinal section is curved. When flowing full, the highest point in the spillway lies above the liquid level in the upstream reservoir, and the pressure at that point must therefore be sub-atmospheric. This is the essential characteristics of a siphon. Siphon spillways can be saddle type or volute type but the volute is not very common. The siphon has usually three parts, 1) the inlet or mouth; 2) throat, and 3) lower limb.
The siphon spillway functions as follows:
When the water level exceeds the crest level, the water commences to spill and flows over the downstream slope in much the same way as a simple Ogee spillway. As the water rises further, the entrance is sealed off from the atmosphere. Air is initially trapped within the spillway, but the velocity of flow of water tends to entrain the air (giving rise to aeration of the water) and draws it out through the exit. When all the air has been expelled, the siphon is primed and is therefore acting as a simple pipe. There are thus three possible operating conditions depending on upstream depth.

1. Gravity spillway flow
2. Aerated flow (during self priming)
3. Pipe flow (after priming).

Operational problems with siphon spillways. The aerated condition is unstable and is maintained while the siphon begins to prime. In a simple siphon, a small change in \( H \) produces a sharp increase or decrease in the discharge through the spillway. Depending on the discharge entering the reservoir, the siphon could go through the following cycle:

a) if the spillway is initially operating with gravity flow, then the upstream (reservoir) level must rise,
b) when the upstream level has risen sufficiently, the siphon primes and the spillway discharge increases substantially,
c) the upstream level falls until the siphon de-primes and its discharge drops.

The cycle (a) to (c) is then repeated. This obviously can give rise to radical surges and stoppages in the downstream flow.

Other potential problems encountered with siphon spillway are:

i) blockage of spillway entrance by debris (this problem could be overcome by submerging the inlet of the hood into the water or installing a trash-intercepting grid in front of the intake).

ii) Substantial foundations required to resist vibrations during operation of siphon.

iii) Waves arriving in the reservoir during storms may alternately cover and uncover the entry, thus interrupting smooth siphon action.

The discharge of a saddle siphon can be calculated from the following formula;

\[
Q = CA\sqrt{2gH}
\]

where \( A = \text{area of cross section at crown}; \quad A = L \times b \)

\( L = \text{length of hood (going into the paper)} \)

\( b = \text{height of throat} \)

\( H = \text{operating head} \)

\( H = \text{Reservoir level – downstream tail water level if outlet is submerged} \)

\( H = \text{Reservoir level - downstream centre of outlet if the outlet is discharging freely.} \)

\( C = \text{Coefficient of discharge. Its average value may be taken as 0.65.} \)
SPILLWAY OR CREST GATES

Spillway gates are the temporary barrier installed over the permanent crest of the spillway, for storing additional water during dry weather season. The small flows in excess above the spillway gates is allowed to pass over the gates, but in case of large flood, the spillway gates are opened and the full capacity is used to remove excessive flood water.

Spillway gates can be provided on all types of spillways except siphon spillway. Following types of spillway gates are commonly used:

a) Flash board gates
b) Stop logs or needle gates
c) Radial gates
d) Drum gates
e) vertical lift gates
Flash Board
Rubber stopper
Flash Board Gate
Spillway crest
Stop logs or needle gates
Piers
Groove
Stop log
Chapter 5

RESERVOIRS

Reservoirs are the most important man-made storage elements in water systems because their capacity and operational schedules determine the rates and volumes of flow in streams. They have enabled humans to make the desert bloom and flourish and provide water supplies for large and concentrated populations. A reservoir is usually created by the construction of a dam across a flowing stream. Storing water is necessary because when water occurs naturally in streams and is not stored, it is sometimes not available when needed. Reservoirs solve this problem by capturing water when it occurs and making it available at latter times. However, with today’s environmental awareness, reservoirs are sometimes negative symbols of “man taking dominion over nature” and work against sustainable development as they interfere with natural ecosystems.

While the concept of a reservoir may bring to mind a large body of water, many small reservoirs are also in service. These include urban water tanks, farm ponds, regulating lakes, and small industrial or recreational facilities. These small reservoirs can have important cumulative effects especially in rural areas.

A reservoir is created with the impounding of runoff from the catchment upstream by the construction of a dam across a river or stream. Storage is done during the period the flow is in excess of the demand for release during the lean period. Storage reservoirs are constructed for the purposes of 1) flood control or 2) conservation of water. Reservoir construction may be contemplated under one of the following conditions:
i) Running water is available in sufficient quantity, but running to waste or causing damage. In this case, an attempt is made to utilize the available water for developmental purposes of the area.

ii) Water is needed all the time either for the generation of hydro-power to meet the requirement of power for development or for irrigation, however the flow in the stream is unable to meet the demand all the time.

Once it is decided to construct a reservoir, it is important to know that it is cheaper to construct a larger reservoir and combine several functions than to construct series of separate dams for each purpose. The following factors must be investigated before deciding to put up a reservoir:

   a) Availability of water

   b) Availability of suitable site for the construction of a dam to create the reservoir.

   c) Availability of construction material in sufficient quantity, equipment, labour, etc.

Investigations if carried out in details, will be time consuming and expensive. Sometimes, it may happen that the investigations made may reveal that the project is not economical or technically sound. In such a case, all the labour and resources spent is a waste. Therefore an investigation should be planned and executed such that the soundness of the project will be determined as early and as cheap as possible. To achieve this aim, investigation may be divided as follows:

1. Reconnaissance or preliminary investigations: which seeks to collect and analyze the following information: i) A not very precise topographic map of the site; ii) Some information of over burden; iii) Characteristics of the foundation (few samples are needed); iv) A preliminary geological survey of site; v) Hydrological studies; vi) Investigation of the available construction materials; vii) Checking of high flood marks and their use to determine spillway capacity.

The purpose of this preliminary investigation is to obtain sufficient data to carry out the office studies and estimate cost sufficiently accurate to select the most economical and suitable site amongst the several options that may be available.

2. Final investigations: is limited to the point that is necessary to confirm the relative merits of few selected options and working out details. The investigations should include: i) to select the final option; ii) to determine the nature of foundation; iii) to determine availability and quantity of construction materials; iv) to obtain all other necessary information useful for the design purpose; v) requirement for coffer dam, pumping and other provisions for dewatering the site; vi) transport facilities and accessibility to site; vii) suitable site for construction camp and repair of construction equipments.
Investigations for Reservoir Planning

1. Engineering Surveys; 2) Geological Surveys; 3) Hydrological surveys

1. Engineering surveys of the reservoir and work areas are required to determine the capacity and spread of the reservoir at various elevation; for laying out the lines of communications and other various works and for estimating quantities of materials and their cost.

The contour interval is 2.5 m but at the site of dam an interval of 1.5 m is used and a map on a scale of 1/250 to 1/500.

Area Capacity Curves.

From the contour map of the reservoir area, the water spread of the reservoir at any elevation may be directly determined by measuring the area at that contour with a planimeter. From these measurements, an area – elevation curve can be drawn. The capacity of the reservoir may be determined by taking contour areas at equal interval and summing up these areas by any of the following methods:

2. Geological Surveys is essential to determine 1) the suitability of foundation; 2) the tightness of the reservoir basin and 3) location of quarry sites.

Hydrological investigation is necessary to 1) have accurate estimate of the run-off pattern at the proposed dam site for the purposes of determining the height of
Site Selection for Reservoir

The storage capacity of site should be sufficient to meet the requirement for which it is designed.

i) Site where the width of river is narrow but rapidly widening upstream

ii) Site where sufficient quantity of water is available. This will depend on the intensity of rainfall, run-off and catchment area.

iii) As much as possible, site should be such that water can flow under gravity from reservoir to demand points

iv) Site must be close to demand points

v) The geological conditions at the site should permit minimum percolation losses, with maximum run-off.

vi) Site must be close to useful construction materials.

vii) The topography should be favourable for site for spillway.

viii) Site should be such that the run-off water has minimum percentage of sediment

ix) Site must be free from such minerals and salts, which may make the water unfit for the purpose, for which the dam built.

Classification of Reservoirs

a) Storage reservoir

b) Flood Control (flood mitigation) Reservoir

c) Distribution Reservoir

d) Single Purpose reservoir,

e) Multipurpose Reservoir.

Storage Reservoir: are small storage capacity reservoir, which are constructed to store water for meeting the demand for water supply, irrigation, power generation etc.

Flood Control (Flood Mitigation) Reservoir: The main function of such reservoirs is to temporary store flood waters and release it at such a safe rate that it may not flood downstream side.

Distribution Reservoir: These are small capacity reservoirs used for water supply of towns. These reservoirs may store raw water for treatment. As the water is required at varying rate during the day, they permit the water treatment plant to function at a constant rate.
**Single Purpose Reservoir:** It is a reservoir to serve only one purpose which may be: a) Municipal Water Supply, b) Power generation; c) Flood Control; d) irrigation; e) Navigation; f) Recreation.

**Multipurpose Reservoir:** In this reservoir, the storage and release cater for a combination of two or more purposes.

**STORAGE ZONES AND LEVELS OF A RESERVOIR**

1. **Dead Storage:** us about 10 – 25 % of the gross storage and it is provided to cater for sediment deposition by the impounded sediment –laden waters. Usually this volume is below sluiceway, and therefore below it, the reservoir is not susceptible to release water by the built in outlet means. The dead storage is equivalent to the volume of sediment expected to be deposited in the reservoir during the design life of the reservoir.

2. **Live (Active or Useful) Storage:** It is the storage capacity above the inactive storage, which constitutes useable portion of the total storage. It is thus the difference of gross storage capacity and the sum of dead storage capacity and inactive capacity Live storage has to be sufficient so that the project is successful for i) 75% of its life period in an irrigation project; ii) 90% for hydro- power and iii) 100% for water supply.

3. **Flood Storage:** is the storage contained between the normal reservoir level and the full reservoir level. It is the storage space provided in a reservoir for storing flood water temporary to moderate the releases downstream.

4. **Valley Storage:** is the storage in the river in floods after it gas overflowed its banks. It is important element in the design of large size flood control reservoirs where it could be a significant proportion in respect to the reservoir storage volume.

5. **Surcharge Storage:** It is the storage between the Normal full reservoir level and the maximum possible level in the reservoir. This storage is usually difficult to control since it depends on maximum floods, rains and the resulting run-offs.

6. **High Flood Level (Maximum Pool Level):** The level to which the water will rise during the design flood.

7. **Full Reservoir Level** The level to which water will rise during ordinary conditions of operation of reservoir.

Minimum Pool Level: The level to which water from the reservoir in ordinary conditions may be drawn.

**Storage Capacity and Yield**

**Yield:** It is the volume of water that can be supplied from the reservoir in a specific interval of time. The time interval may vary from a day to a year.

**Dependable or Firm Yield:** The maximum guaranteed supply of water during the worst dry period.
Secondary Yield: The quantity of water available in excess of dependable yield during the flood period.

Designed Yield: It is the quantity of water for which the project is designed after ascertaining the availability of water. It is usually kept lower than firm yield.

Average Yield: It is the arithmetic average of safe yield and secondary yield over a long period.

**Reservoir Storage Capacity Determination**

The storage capacity of a reservoir to meet the demand of continuous supply is determined with the help of observed discharge data of a stream on which the dam is to be constructed. The flow values for the driest years in as long a period as is available, eg, 25 to 30 years are used.

There are two approaches for the determination of the size of reservoir storage. The simplified method, which are commonly used in planning stage studies, comprise mass curve analysis.

The detail method, used at the time of developing reservoir operating plans, performs a sequential reservoir routing of the historical flows, which will not be treated here.

A third, by Bar Graph Method. In this method, the inflows of the driest year are plotted as ordinates against time as abscissa to get a stepped graph as bar graph. The area under the bar graph represents total volume of inflow into reservoir. The average demand is likewise plotted. The area between the two plots indicates either surplus or deficit volume. The area of maximum deficit between the demand and inflow represents the minimum storage required.
Simplified Procedure for Reservoir Storage Capacity.

There are two methods of analyses: i) the sequential mass-curve method and ii) the non-sequential mass-curve method. Mass curves are useful in reservoir studies since they provide a ready means of determining storage capacity necessary for a particular average drawoff.

Properties of the mass curve

1. Any point on the curve indicates the total inflow from the beginning of the period up to the given point.
2. The slope of the tangent to the mass curve at any time gives the inflow rate at that time.
3. The slope of a line joining any two points on the mass curve gives the average inflow rate within the period.
4. Dry spells (periods) are indicated as hollows on the curve.

A sequential mass-curve method (Rippl Method) considers the most critical period of recorded flow, which might be a severe drought period. The cumulative difference between the inflow and outflow to the reservoir during successive periods are evaluated, the maximum value of which is the required storage.

\[ S = \text{maximum} \sum (I_t - O_t) \quad \text{-------------------------R-1} \]

Where \( S \) – required storage capacity

\( I_t \) – inflow during period \( dt \), \( O_t \) – outflow (draft) during period \( dt \)

Equation (R–1) can be solved either analytically or graphically.

Analytical Method: In this method, the inflow and demand values in each month are determined. The demand includes prior rights, if any, evaporation, seepage,
etc. The deficit and surplus of water which is the departure of inflow volume from demand volume is determined. The maximum value of cumulative excess of demand over inflow represents the minimum storage necessary to meet the demand. The cumulative excess inflow volume starting from each demand withdrawal from the storage is also determined which indicates the filling up of the reservoir and the volume in excess of storage to be spilled over.

**Example:** In the table below, (col. 2) gives the yield of a river, (col. 3 & 4) gives total losses including evaporation, percolation, etc., and total estimated consumption of water. Determine the storage capacity of the proposed reservoir.

<table>
<thead>
<tr>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Yield of River</td>
<td>Total Losses</td>
<td>Total consumption</td>
<td>Total (3 + 4)</td>
<td>Surplus (2 – 5)</td>
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<tr>
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<td>0.2746</td>
<td>-</td>
<td>-</td>
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<td>0.1863</td>
<td>-</td>
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<td>0.4832</td>
<td>0.0615</td>
<td>0.1513</td>
<td>0.2128</td>
<td>0.2704</td>
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Graphical Method: there are two graphical procedures. In the first method, (Mass Curve method), $I_t$ is accumulated separately as a mass inflow curve and $O_t$ as a mass yield curve. For a constant draft rate, the yield curve is a straight line having a slope equal to the draft rate. At each high point on the mass inflow curve, a line is drawn parallel to the yield curve and extended until it meets the inflow curve. The maximum vertical distance between the parallel yield line and the mass inflow curve represents the required storage. Assuming the reservoir is full at A; and going from A to F along the inflow curve. From A to B, the draft is more than inflow resulting in a lowering of reservoir; from B to C, the inflow is
higher than the draft, but not enough to refill the reservoir; from C to D, the draft is more, once again causing a drop in reservoir level; from D to E, however, the inflow is very high, thus filling the reservoir at E.

The second graphical procedure plots the difference of successive accumulated values of inflow and yield $\sum(I_t - O_t)$ against time. The maximum vertical difference is the storage.
RESERVOIR SEDIMENTATION

Every stream (river) carries some sediment load along with the flow. The larger solid particles roll along the bed as **bed load**. Smaller particles are kept in suspension by the upwards component of the turbulent forces and will only settle by gravitational force when velocity reduces. Such particles are called **suspended load**.

When sediment laden water reaches a reservoir, in the vicinity of a dam, the velocity and the turbulence are greatly reduced due to enlarged area of the channel. The larger suspended particles and most of the bed load gets deposited in the head reaches of the reservoir. The smaller particles remain in suspension for longer periods and are deposited further down in the reservoir. Very fine particles may remain in suspension for still longer period and some may pass over the dam along with the water discharged through spillway or sluiceways. If the water stored in the reservoir is clear and the inflow is charged with sediment or is muddy, the heavier water with sediment will flow along the channel bottom towards the dam under the influence of gravity and the clear light water will flow on the upper surface of the turbid water. This condition is known as stratified flow and the under flow of sediment laden water is known as **density current**. The process whereby sediments are deposited at the bed of the reservoir due to reduced turbulence and velocity is known as reservoir sedimentation. The deposition of sediments will reduce the water storing capacity of the reservoir and if this process continues for a long time, a stage will be reached when the whole reservoir will be silted and would be rendered useless.

**Mechanism of Sedimentation in Reservoirs**

Silt deposition in reservoirs follow a typical pattern just like the formation of deltas at the confluence of a river at the sea or lake. There are four patterns:
1. Top set bed;
2. Fore set bed;
3. Bottom set bed; and density currents.
   1. Top set bed: The velocity of a river is reduced considerably as it approaches the reservoir even before the river enters the reservoir due to back water effect. Due to the reduction of velocity of flow, the silt carrying capacity of the river reduces and it deposits the heavy sediment particles in the channel above the highest level, just at the entry into the reservoir. This is called the top set bed.
   2. Fore set bed: As the river water enters the relatively calmer waters of the reservoir, the coarser particles and most medium size sediment particles settle down at the toe of the top set bed. This is called the fore set bed.
   3. Bottom set bed: The fine particles (silt) do not settle till they have moved through a sufficient distance into the reservoir where they may settle and get deposited in thin layers as bottom set bed.
   4. Density current: is the turbid water which flows under the clear water of the reservoir due to its relative dense nature

In reservoir engineering, a special storage is allocated to sediment called the dead storage under the lowest sluice-way. The dead is can be as much as a fourth of the total capacity of the reservoir.
Estimation of the Rate of Sedimentation
The amount of sediment load carried by a stream is determined by taking samples of water carrying silt at various depths. The samples are filtered, sediment removed and dried. The sediment load per day may be computed by:

\[ W_d = \frac{86400 \times C \times Q}{1000} \text{ tones per day} \]

where \( W_d \) = dry weight of sediment  
\( Q \) = mean daily discharge in cumecs  
\( C \) = dry sediment weight in grams per litre of water.

Khosla’s Studies.
In the absence of any information on sediments, one could use the studies of Dr. A. N. Khosla. On the basis of studies on some catchment areas, Khosla suggested the rate of silting per 100 square kilometer of catchment to be 0.036 million cubic metres.
Recent studies have shown that Khosla’s value is on the low side. The average sediment rate of reservoirs in USA is given below:

<table>
<thead>
<tr>
<th>No.</th>
<th>Area of watershed (Km(^2))</th>
<th>No. of measurement</th>
<th>Average annual sediment rate (in mil m(^3) per 100 sq. km area).</th>
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<tr>
<td>2</td>
<td>25-250</td>
<td>205</td>
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</tr>
<tr>
<td>3</td>
<td>250–2500</td>
<td>123</td>
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</tr>
<tr>
<td>4</td>
<td>Above 2500</td>
<td>118</td>
<td>0.029</td>
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</table>

TRAP EFFICIENCY AND LIFE OF A RESERVOIR
Trap efficiency \( \eta \) of a reservoir is defined as the ratio of sediments retained in reservoir to the total sediment brought in the reservoir by the stream.
Trap efficiency ($\eta$) = \frac{\text{Sediment deposited in reservoir}}{\text{Total sediment brought into reservoir by river}}

It has been observed that most reservoirs trap about 95 – 100% sediment brought to it. It is not possible to reduce it below 90 % irrespective of sediment control measures taken.

Capacity-Inflow Ratio
Capacity inflow ratio is the ratio of the capacity (volume) of a reservoir to the total inflow (volume) of water in a given time (usually one year). It is an important factor as detailed observations of reservoir sedimentation has shown that trap efficiency is a function of this factor.

Thus: $\eta = f \left( \frac{\text{Reservoir capacity}}{\text{Total inflow}} \right)$

The figure below shows a graph of trap efficiency and capacity inflow ratio based on observations of existing reservoirs. From the figure, it will be noted that for a given inflow rate, the trap efficiency decreases with age of reservoir as the capacity of the reservoir reduces due to sediment deposition. Hence, the rate of silting is higher in the initial stages and it decreases as silting takes place. Complete filling of the reservoir may take a long time, but actually the useful life of the reservoir is terminated when its capacity is reduced to 20 % of the designed capacity or sediment is so much as to prevent the from serving intended purpose.

For small reservoirs on large rivers having large inflow rates, the trap efficiency is low, because the capacity inflow ratio is very small. The silting in such reservoirs is low, because most of the sediments are passed to the downstream.
The procedure for determination of the life of a reservoir is as follows:
i) The capacity of the reservoir is determined. Next, the capacity inflow ratio and
the trap efficiency are determined for the full capacity of the reservoir with the
help of the curve.
ii) Divide the total capacity into 10 parts. Assuming that 10% capacity has been
reduced due to deposits of sediments, determine the trap efficiency for the
reduced capacity (90% of the total capacity) and the inflow ratio.
iii) For the above 10% interval of capacity, determine the average trap efficiency
by taking into account the trap efficiency (η) obtained in the above steps.
iv) By collecting water samples and drying the sediments, determine the
sediment inflow rate. The total annual sediment collected in the reservoir per
year is multiplied with the trap efficiency determined in step (iii) above. The
sediment quantity so obtained is converted into hectare-meter units deposit per
year.
v) Now the reservoir volume interval (i.e. 10% of the capacity is divided by the
sediment deposited per year, obtained in step (iv) above. It will give the number
of years required to fill this volume interval of 10% capacity.
vi) Now repeat the procedure (i) to (v) above for further capacity intervals i.e.
80%, 70%, 60% etc of the total reservoir capacity. The total life of the reservoir
will be the total number of years required to fill each of the volume intervals.

Examples
The following data is available from a reservoir regarding its trap efficiency and
capacity inflow ratio;

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<th>0.2</th>
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<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.8</th>
<th>0.9</th>
<th>1.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trap efficiency</td>
<td>87</td>
<td>92</td>
<td>93</td>
<td>95.5</td>
<td>96</td>
<td>96</td>
<td>96.5</td>
<td>97</td>
<td>97.5</td>
<td>98</td>
</tr>
</tbody>
</table>

Find the probable life of reservoir with an initial reservoir capacity of 40 million
cubic meters and with an annual average flood inflow of 80 million cubic meters.
The annual sediment inflow is found to be $22 \times 10^4$ tonnes. The specific weight of
sediment may be taken as 1.15 tones per m$^3$. The useful life of reservoir may be
assumed when its capacity is reduced to 20% of its designed capacity.

Solution.

Initial reservoir capacity = $40 \times 10^6$ m$^3$
Average annual inflow = $88 \times 10^6$ m$^3$
Therefore capacity inflow ratio = $40/80 = 0.5$
Average annual sediment inflow = $22 \times 10^4$ tonnes

Let’s assume that 20% of the initial capacity is filled in the first interval, i.e. 20/100
$\times 40 \times 10^6 = 8 \times 10^6$ m$^3$ capacity is filled at a time.
For 0.5 C-I-R the trap efficiency is 96%.

At the end of the first interval, the capacity of reservoir = \((40 - 8) \times 10^6 = 32 \times 10^6\) m³
At the end of the first interval the C-I-R = \(32/80 = 0.4\)

From the curve, this correspond to a trap efficiency of 95.5%.
Therefore the average trap efficiency for the entire period = \((96 + 95.5)/2 = 95.75\%\)

Weight of sediment deposited annually = \(22 \times 10^4 \times 0.975 = 0.2107 \times 10^6\) tonnes
Volume of sediment deposited annually = \((0.2107 \times 10^6)/1.15 = 0.1823 \times 10^6\) m³
Therefore No. of years to fill 20% \((8 \times 10^6\) m³) capacity = \((8 \times 10^6)/0.1823 \times 10^6 = 43.9\) years.

Similarly, the silting period for the other intervals can be calculated. The rest of the calculations are done in the table below

<table>
<thead>
<tr>
<th>Capacity x 10^6 m³</th>
<th>Capacity Inflow Ratio (C-I-R)</th>
<th>Trap Efficiency</th>
<th>Sediment Trapped</th>
<th>Incremental Volume (x 10^6 m³)</th>
<th>Years to fill (Col.7)/(Col. 6)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>At indicated volume</td>
<td>Average for increment</td>
<td>Tonnes x 10^6 tonnes</td>
<td>Volume x 10^6 m³</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>40</td>
<td>0.5</td>
<td>96.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>32</td>
<td>0.4</td>
<td>95.5</td>
<td>95.75</td>
<td>2.107</td>
<td>0.1831</td>
</tr>
<tr>
<td>24</td>
<td>0.3</td>
<td>95.0</td>
<td>95.25</td>
<td>2.095</td>
<td>0.1822</td>
</tr>
<tr>
<td>16</td>
<td>0.2</td>
<td>92.5</td>
<td>93.75</td>
<td>2.062</td>
<td>0.1793</td>
</tr>
<tr>
<td>8</td>
<td>0.1</td>
<td>87.5</td>
<td>90.0</td>
<td>1.98</td>
<td>0.1722</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Example 2.

Below are data regarding trap efficiency and capacity inflow ratio of a reservoir

<table>
<thead>
<tr>
<th>Capacity Inflow ratio</th>
<th>0.8</th>
<th>1.0</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.8</th>
<th>0.9</th>
<th>1.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trap efficiency</td>
<td>85</td>
<td>86</td>
<td>92</td>
<td>94</td>
<td>95</td>
<td>95.5</td>
<td>96</td>
<td>96.5</td>
<td>97</td>
<td>97.5</td>
<td>97.7</td>
</tr>
</tbody>
</table>

Determine the probable life of the reservoir with an initial reservoir capacity of 25 million m³, if the annual inflow is 50 million m³ and the average annual sediment inflow is 300,000 tonnes. Assume the density of the sediment as 1250 kg/m³. The useful life of the reservoir will terminate when 85% of the initial capacity is filled with sediment.

Solution:
Average annual sediment inflow = 300,000 x 1000 kg
Volume of sediment inflow = 300,000,000/1250 = 0.24 x 10^6
Initial reservoir capacity = 25 x 10^6 m^3
Annual inflow = 55 x 10^6 m^3
Therefore Initial capacity = \( \frac{25 \times 10^6}{50 \times 10^6} = 0.5 \)

The capacity inflow ratio goes on decreasing as the sediment takes place and trap efficiency also decreases. The volume of interval chosen is 5 x 10^6 i.e. 20% of initial capacity. To obtain the 85% filling of initial capacity the volume interval in last two readings is taken as 5%.

Similarly, for the calculation of the years to fill the last 5% capacity is determined by taking \( \frac{1}{S_i} = \frac{1}{0.252} = 4.87 \) years

<table>
<thead>
<tr>
<th>Capacity %</th>
<th>Volume x 10^6 m^3</th>
<th>Cap. Inflow Ratio</th>
<th>Trap Efficiency η</th>
<th>Sediment Trapped Tonnes x 10^6</th>
<th>Volume x 10^6 m^3</th>
<th>Incremental Vol. x 10^6 m^3</th>
<th>Years to fill col 8/col 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>25</td>
<td>0.5</td>
<td>0.955</td>
<td>2.8575</td>
<td>0.2286</td>
<td>5</td>
<td>21.7</td>
</tr>
<tr>
<td>80</td>
<td>20</td>
<td>0.4</td>
<td>0.95</td>
<td>2.835</td>
<td>0.2268</td>
<td>5</td>
<td>22.04</td>
</tr>
<tr>
<td>60</td>
<td>15</td>
<td>0.3</td>
<td>0.94</td>
<td>2.79</td>
<td>0.2232</td>
<td>5</td>
<td>22.40</td>
</tr>
<tr>
<td>40</td>
<td>10</td>
<td>0.2</td>
<td>0.92</td>
<td>2.67</td>
<td>0.2136</td>
<td>5</td>
<td>23.40</td>
</tr>
<tr>
<td>20</td>
<td>5</td>
<td>0.1</td>
<td>0.86</td>
<td>2.565</td>
<td>0.2052</td>
<td>121.7</td>
<td>4.87</td>
</tr>
<tr>
<td>15</td>
<td>4</td>
<td>0.08</td>
<td>0.85</td>
<td>2.565</td>
<td>0.2052</td>
<td>121.7</td>
<td>4.87</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>94.58</strong></td>
</tr>
</tbody>
</table>

Example 3. A reservoir has a drainage area of 49,000 acres. The reservoir has a capacity of 10,000 acre-ft. Streamflow runoff averages 15.5 in. per year. The annual sediment production is 9.5 x 10^6 ft^3. Determine the life of the reservoir, assuming that the life is over when 80% of the original capacity is lost

Solution
1. Annual inflow \( I = (49,000) \times (15.5/12) = 63,290 \) acre-ft
2. Annual sediment production = 9.5 x 10^6 ft^3 or 218.5 acre-ft
3. Unfilled capacity (20%) = \( \frac{20}{100} \) (10,000) = 2,000 acre-ft
4.
Control of Silting in Reservoirs
It is utmost necessary to reduce the depositions of sediments in the reservoir bed for increasing its life. Sediment control methods may be classified as:
   Pre-construction methods
   Post-construction methods.
1. Pre-construction Methods includes:
i) Site Selection: For silting control, the site of reservoir should be such that the catchment area should have firm soil which would not be easily erodable.

ii) The ratio of reservoir capacity and drainage size: If the storage capacity of the reservoir is much less than the annual inflow into reservoir, large quantity of water will flow out of reservoir and with it large volume of sediment.

iii) Design of Reservoir: For the release of density currents, adequate outlets such as sluices should be provided in the design of the dam. I this way, sufficient quantity of sediment may be flashed fro the reservoir.

iv) Vegetation Screen: If the ground around reservoir is covered by vegetation, grass, etc., it will prevent the erosion of soil particles as well as it will trap large amount of sediments. Therefore, if vegetations are grown in the catchment area, the silting will be reduced to considerable extent. The vegetations covering the ground are known as vegetation screens and are the cheap and effective means of silting control.

Post-Construction Methods:
i) Mechanical Stirring of Sediment: The sediments which are deposited in the reservoir bed are scrapped and stirred by mechanical equipments and kept in moving state. The sediments mixed water is pushed towards the sluices and is flashed out.

ii) Excavation: It is a very expensive method. Iy is expensive to excavate and also expensive to dispose of excavated material.

iii) Dredging: It is also expensive as it require the purchase and maintenance of costly machines as dredger.
Watershed Erosion Control

a) By afforestation: In this method, vegetation is planted to provide a cover to soil, which reduces the scoring effect of the soil in three different ways:
   i) It provides a cushion to the falling rain drops and reduces their impact and consequent damage due to erosion.
   ii) The roots of the plants penetrate deep into the soil and bind soil particles enabling it resist water and wind erosion.
   iii) Plants offer resistance to flow and thereby reduces the velocity of flow.

b) Controlling Overgrazing

OPERATION OF RESERVOIRS

After a dam is built, it must be operated correctly. The key person is the operator, who makes decisions about when to release or store water. The operator may be a part-time worker who lives near a lake and who occasionally operates a gate, or a highly trained engineer working at a remote location who makes system decisions based on computer forecast. In the past, reservoir operating decisions were made by RULES CURVES, which provided the operator with simple guidelines about how much water to release and what lake levels to maintain. As the science of forecasting and the use of computers has become more complex, however, reservoir operation has become more sophisticated. It is not uncommon to have a reservoir control center where operators use computers to monitor weather forecasts furnished from satellite data and simulate future demands for water to make decisions about water releases. They may also be bound by legal requirements to release water for downstream users, including fish and wildlife.

Basically what is required in the operations problem is an analytical tool to illustrate the time rate of inflows, outflows, and change in storage. In other words, it is necessary to consider the storage equation both numerically and graphically. Recall that the storage equation relates inflow, outflow and change in storage as a function of time, where for any time period such as an hour, day month or year.

\[ Q_i - Q_o = DS \]

Where \( Q_i \) – is the rate of inflow; \( Q_o \) – is the rate of outflow; and DS – change in storage.

A simple way to view the operation plan is to consider a rule curve such as one shown below, which is taken from the corps of engineers operating plan for lake Lanier in Georgia. A rule curve illustrates a typical operations time period, usually a year, and shows the boundaries within which operations should occur.
8.1.2 Reservoir Operations

Reservoir operation in the CRYSTAL model is based on the Snohomish County PUD Jackson Operations model (JKSOPR). Release policies depend on the "state" of both Spada Reservoir and Lake Chaplain. The state of the reservoir is determined by comparing the storage to maximum storage capacity, rule curves, and minimum storage capacity. When water is in abundance, releases are dependent on the capacity of Jackson Powerhouse transmission line. Otherwise, releases are functions of instream flows, demands, and relative storage.

Snohomish County PUD No. 1 (SnoPUD) operates Spada Reservoir using two rule curves based on storage (Figure 8.3). In addition,"poweroff" storage may be used during extreme conditions (Spada reservoir has yet to reach the poweroff storage). Comparing the reservoir storage to the Full Storage condition (FS), Upper normal rule curve, and Lower normal rule curve determine the state of the reservoir. If reservoir storage is between the rule curves (State 3), then a variable amount of flow (based on a ratio of storage to lower rule curve and time of year) is conveyed through the Powerhouse pipeline. If storage in Spada is below the Lower normal rule curve (State 4) then minimum flow to meet instream flow requirements and M&I demand is conveyed. The
maximum flow of 1300 cfs is conveyed if the reservoir is above the Upper normal rule curve (States 1&2). No flow is conveyed if storage is below the "Power off" level.

FLOOD ROUTING

Flood routing is the process whereby the shape of a flood hydrograph is modified as it passes through a hydraulic system (such as a river channel, reservoir, lake, etc). The hydrograph resulting after passing through the hydraulic system is one with considerably reduced peak and enlarged time base. The process can be represented as:

1. Studying the effect of a hydraulic system on the modification of a flood peak
2. Determining the design elevations of flood walls and levees
3. Determining the site for a spillway
4. Derivation of unit hydrograph and synthetic hydrographs
5. For flood forecasting
6. Any other flood flow-related objectives.

In flood routing, all the components of the routing process (inflow, storage and outflow) are connected by the storage (continuity) equation in the form:

\[
I\Delta t + \Delta S = O\Delta t
\]  

where \( I \) and \( O \) are the average inflow and outflow rates within the time \( \Delta t \). The hydraulic system can be a reservoir (lake) or a section of a river channel, and so the routing process is, accordingly classified into two namely: reservoir routing and channel (streamflow) routing.

In the above equation (1) \( I \) - the inflows are known quantities but the outflows, \( O \) and the storage, \( S \) are unknown parameters. To solve the equation, either both \( O \) and \( S \) have to be related to a common unknown parameter or \( S \) has to be defined in terms of \( O \) to a common unknown parameter or \( S \) has to be defined in terms of \( O \). The former approach is applicable to reservoir routing whiles the latter to channel routing.

Equation (1) when expressed in the differential form can be integrated, however, the terms in the equation have a form that is not amenable to direct solution. Therefore, the numerical solution expressed as numerical approximation can be written as:

\[
\frac{I_1 + I_2}{2} + \frac{S_1 - S_2}{\Delta t} = \frac{O_1 + O_2}{2}
\]  

where subscript 1 and 2 denote beginning and end of routing period \( \Delta t \)

**RESERVOIR FLOOD ROUTING**

Reservoir routing is the process of determining the reservoir stage, storage volume and outflow rate corresponding to a particular hydrograph of inflow. The flood routing procedure is used in the detention and storage reservoir for studies related to the variations in reservoir levels with inflow and outflow discharge with time with a view to deciding the location and capacity of proposed reservoirs, determining the spillway outlet capacities, height of dam, extent of land submergence in the reservoir area.

In reservoir routing, the volume of storage can be expressed as a function of water surface elevation in the reservoir, so also is the outflow, which can be expressed as a function of elevation.

In routing through a reservoir, equation (2) is re-organised to the form:
\[(I_1 + I_2) + \left( \frac{2S_1}{\Delta t} + O_1 \right) - 2O_1 = \left( \frac{2S_2}{\Delta t} + O_2 \right) \]  \hspace{1cm} (3)

Since the outflow and the storage are both functions of water surface elevation, the storage equation becomes a relation between the known inflow and the unknown elevation, from which elevation can be computed as a function of time.

**Procedure**

1. What is available at the beginning.
   i) An inflow hydrograph \( I = f(t) \)
   ii) An elevation – capacity curve \( H = f(S) \) or elevation – area curve of the reservoir
   \( H = f(A) \)
   iii) An elevation – outflow curve \( H = f(O) \)

2. From the above, the routing curve is prepared and plotted
   \[ H = f\left( \frac{2S}{\Delta t} + O \right) \]
   \[ H = f(O) \]

3. At the initial time, \( t = 0 \) (start of the routing, just before flood arrives, we assume a steady state condition, so that \( I_1 = I_2 = O_1 \) and \( S_1 \) corresponds to the storage corresponding to elevation of \( O_1 \).

4. Select the time interval \( \Delta t \) for the routing. This is usually taken as the interval of time given for the inflow hydrograph. Otherwise, the time interval is so chosen so that one does not miss the peak values.

5. At the beginning of the routing, \( I_1 = I_2 = O_1 \). With \( O_1 \) known, read off \( H \) from \( H = f(O) \) curve. With \( H \) now known, read off \( 2S/ \Delta t + O \) from the \( H = f(2S/ \Delta t + O) \) curve. So at time \( t = 0 \), the following quantities are known; \( H, O_2 \) and \( (2S_2 + O_2) \) and they will become the quantities at the end of a fictitious previous period.

<table>
<thead>
<tr>
<th>Time</th>
<th>Inflow</th>
<th>( (I_1 + I_2) )</th>
<th>( (2S/ \Delta t + O) )</th>
<th>-2( O_1 )</th>
<th>( (2S_2/ \Delta t + O_2) )</th>
<th>( H )</th>
<th>( O )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>( I_1 )</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>( X )</td>
<td>( H )</td>
<td>( O )</td>
</tr>
<tr>
<td>6</td>
<td>( I_2 )</td>
<td>( I_1 + I_2 )</td>
<td>( X )</td>
<td>-2( O )</td>
<td>( X )</td>
<td>( H )</td>
<td>( O )</td>
</tr>
<tr>
<td>16</td>
<td>( I_3 )</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>( I_4 )</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>( I_5 )</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>( I_6 )</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6. The values in row corresponding to time \( t = 0 \), being the values at the end of a routing period is transferred to become the initial values on the left hand side of the routing equation (3) for the next time step of the routing period.
7. The left hand side of the routing equation now has known values that yield a value for \((2S_2 + O_2)\). With this known, \(H\) is read from the plot and \(O\) is also determined.

8. The procedure is repeated until all the inflow values are used up.

Example: Given an inflow hydrograph and storage vs elevation data for a reservoir below. The spillway discharge \(Q = 3LH^{3/2}\). The crest height of the spillway is 50 ft and the length of the spillway is 35 ft.

<table>
<thead>
<tr>
<th>Time(days)</th>
<th>0.5</th>
<th>1.0</th>
<th>1.5</th>
<th>2.0</th>
<th>2.5</th>
<th>3.0</th>
<th>3.5</th>
<th>4.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flows(cfs)</td>
<td>70</td>
<td>185</td>
<td>360</td>
<td>480</td>
<td>300</td>
<td>165</td>
<td>80</td>
<td>0</td>
</tr>
</tbody>
</table>

**Storage data**

<table>
<thead>
<tr>
<th>Elevation(ft)</th>
<th>50</th>
<th>50.5</th>
<th>51.0</th>
<th>51.5</th>
<th>52.0</th>
<th>52.5</th>
<th>53.0</th>
<th>53.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storage(acre-ft)</td>
<td>231</td>
<td>247</td>
<td>277</td>
<td>313</td>
<td>353</td>
<td>400</td>
<td>452</td>
<td>509</td>
</tr>
</tbody>
</table>

Solution:

1. The discharge data, which is the outflow, is computed from the given equation \(Q = 3 \times (35) \times H^{3/2}\) and the storage data are listed in tabular form.

<table>
<thead>
<tr>
<th>Water surface Elevation (ft)</th>
<th>Head on crest ft</th>
<th>Storage (acre-ft)</th>
<th>Outflow (cfs) (Q=3 \times (35) \times H^{3/2})</th>
<th>((2S/ \Delta t + O)) (\Delta t = 0.5) day</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>0</td>
<td>231</td>
<td>0</td>
<td>465.6</td>
</tr>
<tr>
<td>50.5</td>
<td>0.5</td>
<td>247</td>
<td>37.1</td>
<td>536.4</td>
</tr>
<tr>
<td>51.0</td>
<td>1.0</td>
<td>277</td>
<td>105.0</td>
<td>664.4</td>
</tr>
<tr>
<td>51.5</td>
<td>1.5</td>
<td>313</td>
<td>192.9</td>
<td>825.3</td>
</tr>
<tr>
<td>52.0</td>
<td>2.0</td>
<td>353</td>
<td>297.0</td>
<td>1009.4</td>
</tr>
<tr>
<td>52.5</td>
<td>2.5</td>
<td>499</td>
<td>415.0</td>
<td>1224.3</td>
</tr>
<tr>
<td>53.0</td>
<td>3.0</td>
<td>452</td>
<td>545.6</td>
<td>1459.5</td>
</tr>
<tr>
<td>53.5</td>
<td>3.5</td>
<td>509</td>
<td>687.5</td>
<td>1715.7</td>
</tr>
</tbody>
</table>

The routing is done in the table below

<table>
<thead>
<tr>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time</td>
<td>Inflow</td>
<td>(I_1 + I_2)</td>
<td>(2S_1 + O_1)</td>
<td>(-2O_1)</td>
<td>(2S_2 + O_2)</td>
<td>(H)</td>
<td>(O)</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>465.6</td>
<td>50</td>
<td>0</td>
</tr>
<tr>
<td>0.5</td>
<td>70</td>
<td>70</td>
<td>465.6</td>
<td>0</td>
<td>535.6</td>
<td>50.45</td>
<td>35</td>
</tr>
<tr>
<td>1.0</td>
<td>185</td>
<td>255.0</td>
<td>535.8</td>
<td>-70</td>
<td>720.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>360</td>
<td>545.0</td>
<td>720.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>480</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>165</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.5</td>
<td>80</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
i) Column 1 & 2 are the given inflow hydrograph
ii) Column 3 is obtained by adding the two successive values in column (2)
iii)Column 4 is obtained by transferring col. 6 of the previous routing period
iv)Col. 5 is obtained by transferring 2 times col. 8
v)Col. 6 is obtained by adding columns (3)+(4)+(5) to obtain the right hand side of eq.(3)
vi. Col. 7 is obtained by reading off H from the plot of $H = f(2S/\Delta t + O)$
vii Col 8 is obtained by reading off O from the plot of $H = f(O)$

5. After the computation, a plot of the inflow and the outflow hydrographs are made from which the extent of peak reduction can be seen and when the peak occurs.

Example: An impounding reservoir enclosed by a dam has a surface area that varies with elevation as given below. The dam is equipped with two circular gated discharge ports each of 2.7 m diameter, whose centres are at elevation 54.0 with discharge coefficient $c_d=0.8$ and a free overflow spillway 72.5 m long with crest level at elevation 66.0. Live storage at elevation 54 is $5.5 \times 10^6$ m³ discharge gates are opened and the surface water level is at elevation 63.5 at time $t=0$. The flood hydrograph table below is forecast. What will the maximum reservoir level be and when will it occur.

<table>
<thead>
<tr>
<th>Elevation (m)</th>
<th>54</th>
<th>56</th>
<th>58</th>
<th>60</th>
<th>62</th>
<th>64</th>
<th>66</th>
<th>68</th>
<th>70</th>
<th>72</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface area</td>
<td>4.12</td>
<td>4.19</td>
<td>4.28</td>
<td>4.36</td>
<td>4.45</td>
<td>4.58</td>
<td>4.71</td>
<td>4.88</td>
<td>5.05</td>
<td>5.22</td>
</tr>
</tbody>
</table>
Chapter 6

DIVERSION HEAD WORKS

The construction work done at the river or canal for diversion of water to the off-taking canal is known as head-work. Depending on the purpose, head-works may be:

1. storage head-work or
2. diversion head-work

Storage head-work is constructed to store water for some purpose.

The main purpose of diversion head-work is to divert the required quantity of water into the off-taking canals for irrigation purpose. A diversion head-works may also serve the following purposes:

i) to raise the water level in the river for increasing its command
ii) to regulate the intake of water into the canal
iii) to control silt entry into the canal
iv) to store water for meeting emergency requirements
v) to prevent fluctuations in the level of supply of the river.
i). Diversion head-work provides an obstruction across a river, so that the water level is raised and water is diverted to the channel at required level. The increase water level helps the flow of water by gravity and results in increasing the commanded area and reducing the water fluctuations in the river.

ii). Diversion head-work may serve as silt regulator into the channel. Due to the obstruction, the velocity of the river decreases and silt settles at the bed. Clear water with permissible percentage of silt is allowed to flow through the regulator into the channel.

iii). To prevent the direct transfer of flood water into the channel.

### 6.1.1. Headwork for direct river offtake

In rivers with a stable base flow and a high enough water level throughout the year in relation to the bed level of the intake canal, one can resort to run-off-river water supply (Figure 38 and Example 1 in Figure 37).
A simple offtake structure to control the water diversion is sufficient. The offtake should preferably be built in a straight reach of the river (Figure 39). When the water is free from silt, the centre line of the offtake canal could be at an angle to the centre line of the parent canal. When there is a lot of silt in the system, the offtake should have a scour sluice to discharge sediments or should be put at a 90° angle from the parent canal. If it is not possible to build the offtake in a straight reach of the river, one should select a place on the outside of a bend, as silt tends to settle on the inside of bends. However, erosion usually takes place on the outside of the bend and therefore protection of the bank with, for example, concrete or gabions might be needed.

The offtake can be perpendicular, at an angle or parallel to the riverbank, depending on site conditions, as illustrated in Figure 40.
The functions of the offtake structures are:
- To pass the design discharge into the canal or pipeline
- To prevent excessive water from entering during flood
Considering these functions, the most important aspect of the structure is the control arrangement, which can be a gate, stop logs, or other structures. When the gate is fully opened, the intake behaves like a submerged weir (Figure 41) and its discharge is given by equation 23

**Equation 23**

\[
Q = C \times B (h + h_d)^{1/3}
\]

Where: 
- \(Q\) = Discharge in intake \((\text{m}^3/\text{sec})\)
- \(C\) = Weir coefficient
- \(B\) = Width of the intake \((\text{m})\)
- \(h\) = Difference between river water level and canal design water level \((\text{m})\)
- \(h_d\) = Difference between canal design water level and sill level of the intake \((\text{m})\)

### Example 11

A discharge of 1.25 \(\text{m}^3/\text{sec}\) has to be abstracted from a river, into an open conveyance canal. The base flow water level of the river is 125.35 m. The design water level in the canal is 124.90 m and the water depth is 0.60 m. The weir coefficient is 1.60. The width of the intake is 1.50 m and the length of the weir is 0.50 m (Figure 41). What will be the sill level?

- \(Q\) = 1.25 \(\text{m}^3/\text{sec}\)
- \(C\) = 1.60
- \(h\) = 125.35 - 124.90 = 0.45 m
- \(B\) = 1.50 m

The next step would be to substitute these values in Equation 23:

\[
1.25 = 1.60 \times 1.50 (0.45 + h_d)^{1/3} \Rightarrow h_d = 0.20 \text{ m}
\]

Thus the sill level should be at an elevation of 124.90 - 0.20 = 124.70 m

### River offtake using a weir

Figure 42 shows an example of a river diversion structure, in this case a weir (Example 2 in Figure 37). Structures constructed across rivers and streams with an objective of raising the water level are called cross regulators (see Section 6.4).
A weir should be located in a stable part of the river where the river is unlikely to change its course. The weir has to be built high enough to fulfill command requirements. During high floods, the river could overtop its embankments and change its course. Therefore, a location with firm, well defined banks should be selected for the construction of the weir. Where possible, the site should have good bed conditions, such as rock outcrops. Alternatively, the weir should be kept as low as possible. Since weirs are the most common diversion structures, their design aspects will be discussed below.

Diversion head-works may be a weir or a barrage. Weirs are solid walls constructed across a river for the purpose of raising the water level. Barrage is a structure constructed either to store water or raise the level of water. In case of barrages, no wall is constructed across the river, but there is an arrangement of gates which can be used to store water to the required level.

Diversion Head-Works consists of:

1. Weir (Barrage) may be masonry; rockfill or cement concrete. All weirs normally have the following components: i) body wall usually with shutters; ii) upstream rough stone or boulder pitching; iii) upstream curtain wall; iv) upstream impervious apron; v) crest shutters; vi) downstream impervious apron; vii) downstream curtain wall; viii) downstream apron for channel bed.
Rockfill weir

Figure 45
Types of weirs

(a)  
\[
\begin{align*}
\text{V} & \quad \text{hu} \\
\text{L > 2h} & \\
\text{hd} & 
\end{align*}
\]

a. Broad-(long)crested weir

(b)  
\[
\begin{align*}
\text{hu} & \\
\text{V} & \\
\text{hd (<0)} & 
\end{align*}
\]

b. Short-crested weir

(c)  
\[
\begin{align*}
\text{hu} & \\
\text{V} & \\
\text{hd (<0)} & 
\end{align*}
\]

c. Sharp-crested weir
General Equation of Weirs.
The flow over any type of weir is given by

**Equation 24**

\[ Q = C_1 \times C_2 \times B \times H^{3/2} \]

**Figure 44. C2 coefficient for different types of weirs in relation to crest shape**
Where: \( Q \) = Discharge (m\(^3\)/sec)  
\( C_1 \) = Coefficient related to condition of submergence and crest shape (Figure 43)  
\( C_2 \) = Coefficient related to crest shape (Figure 44)  
\( B \) = Weir length, i.e. the weir dimension across the river or stream (m)  
\( H \) = Head of water over the weir crest (m)
Example 12

In Example 11 the weir coefficient $C$, which is the product of $C_1$ and $C_2$, was assumed to be 1.60. Can this be confirmed by calculating $C_1$ and $C_2$ respectively?

From Example 11 the difference between the water level in the river and the sill elevation can be calculated as follows:

$$h_u = 125.35 - 124.70 = 0.65m$$

The weir length $L$ is 0.50 m, thus $\frac{L}{h_u}$ in Figure 44 is:

$$\frac{L}{h_u} = \frac{0.50}{0.65} = 0.77$$

This relates to a weir type between 1c and 1d in Figure 44. By interpolation, $C_2$ is approximately 1.8.

The difference between the canal design water level and the sill elevation $h_a = 0.20$ m.

Thus $\frac{h_u}{h_a}$, which is the y-axis in Figure 43, is:

$$\frac{h_u}{h_a} = \frac{0.20}{0.65} = 0.31$$

Using the curve for weir type 1b-d in Figure 43, gives a value for $C_1$ of approximately 0.9.

Thus $C = C_1 \times C_2 = 0.9 \times 1.8 = 1.62$, which is almost the same as the weir coefficient 1.60 used in Example 11.
Design of weir

a) Design of Weir Walls. Weir walls are designed as a retaining wall or masonry dam of solid gravity type. The two main loads to be considered are the horizontal water thrust, \( P \), and the self-weight of the weir body wall, \( W \). The resultant of forces \( P \) and \( W \) should pass through the middle third of the base of the weir, for the weir to be safe.

Bligh has recommended the following:

If \( b = \) bottom width of the body wall (to be designed);
\( a = \) top width (to be fixed)
\( H = \) height of weir above the floor or the height of the body wall, and
\( d = \) depth of water above the weir crest or height of weir crest wall,
\( S = \) specific gravity of the body wall, then
\[ b = \frac{H + d}{\sqrt{S}} \]  \hspace{1cm} (D.H -1)

\[ a = 0.552\left(\sqrt{H} + \sqrt{d}\right) \]

From the above recommendations, stability are checked under the following conditions:

I) The water in upstream is at crest level or the crest of shutter and there is no flow over the weir

II) The weir is submerged and water is passing over it.

III) Water is passing over weir crest and the weir is discharging with a clear overfall.

Design of Impervious Apron

a) **The upstream Apron**

The length of the upstream impervious apron according to Bligh is:

\[ L_{u/s} = 2.208\sqrt{CH} \quad \text{------------------ (1)} \]

where \( C \) = creep coefficient

\( H \) = head of water stored against the body wall

The thickness of this apron is about 30 cm placed over 30 to 50 cm thick concrete foundation.

b). **Downstream Apron.**

The length of this apron is:

\[ L_{d/s} = \frac{2.208C\sqrt{H}}{13} \quad \text{------------------ (D.H - 2)} \]
Thickness of apron: 
\[ t_{D/s} = \frac{4}{3} \left( \frac{H - h}{SG - 1} \right) \]  
------------------(D.H - 3)

Where \( t_{D/s} \) = in metres; \( H \) = total head causing seepage, 
\( h \) = head lost by the creeping water u to the point where thickness is to be calculated. 
\( SG \) = specific gravity of material of the apron.

c). **Upstream Curtain Wall**: This is usually 2.0 to 2.5 m in depth.

d) **Downstream Curtain Wall**: usually 2.5 to 3.0 m depth

**Two major causes of failure of weirs**

i) **Piping** (or Undermining): Since there is always a differential head between upstream and downstream, water is constantly moving from upstream to downstream from under the base of weir. However, if the hydraulic gradient becomes big, greater than the critical value, then at the point of exist of water at the downstream end, it begins to dislodge the soil particles and carry them away. In due course, when this erosion continues, a sort of pipe or channel is formed within the floor through which more particles are transported downstream which can bring about failure of weir.

The main preventive measure of piping is to reduce the hydraulic gradient by increasing the length of travel of the fluid by either increasing the length of the impervious floor or providing curtains or piles at both upstream and downstream.

2. **Uplift Pressure Rupturing the Floor of the Weir**

The floor of the weir may burst due to inadequate weight of the weir and the heavy uplift pressure. This bursting of the floor reduces the effective length of the impervious floor, which will resulting increasing exit gradient, and can cause failure of the weir.

Rapture of floor could be prevented by:

i) providing adequate length of impervious floor
ii) providing sufficiently thick impervious apron
iii) by providing a pipe at the upstream end to reduce the uplift pressure at the downstream end.

**BLIGH’S CREEP THEORY**

Water from upstream percolates and creeps (or travel) slowly through weir base and the subsoil below it. The head lost by the creeping water is proportional to the distance it travels (**creep length**) along the base of the weir profile. The creep length must be made as big as possible so as to prevent the piping action. This can be achieved by providing deep vertical cut-offs or sheet piles.

According to Bligh’s theory, the total creep length for first drawing: \( L = B \) and for second drawing: \( L = B + 2(d_1 + d_2 + d_3) \)
If $H$ is the total loss of head, then the loss of head per unit length of the creep shall be

$$C' = \frac{H}{L} = \frac{H}{B + 2(d_1 + d_2 + d_3)}$$

Bligh called the loss of head per unit length of creep as Percolation coefficient. The reciprocal, $(L/H)$ of the percolation coefficient is known as the coefficient of creep $C$.

Table D-1. Coefficient of creep $C$ for various soils.

<table>
<thead>
<tr>
<th>No.</th>
<th>Type of soil</th>
<th>Coefficient of creep</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Boulder or shingle, gravel and sand mixed</td>
<td>5 – 9</td>
</tr>
<tr>
<td>2</td>
<td>Coarse grained sand</td>
<td>12</td>
</tr>
<tr>
<td>3</td>
<td>Fine micaceous sand</td>
<td>15</td>
</tr>
<tr>
<td>4</td>
<td>Light sand and mud</td>
<td>18</td>
</tr>
</tbody>
</table>

Design Criteria.
For design, Bligh proposes the following two important criteria:

Method of increasing creep length
1. **Safety Against Piping**: For safety against, the creep length L should be sufficient to provide a safe hydraulic gradient, depending on the type of soil.

Safe creep length, \( L = CH \)

Where \( L = \) safe creep length; \( H = H_1 - H_2 \) Total loss of head for the length of creep;
\( H_1 = \) upstream head; \( H_2 = \) downstream head; \( C = \) creep coefficient

2. **Safety against Uplift**: The uplift pressure \( = \gamma_w H^l \); where \( H^l = \) the uplift pressure head at any point of the apron.

The downward resisting force per unit length of apron \( F_R = t \cdot \gamma_w \cdot S \)
where \( t = \) thickness of the apron floor; \( S = \) specific gravity of floor material.

Equate the two forces, we determine the limiting floor thickness

\[
\gamma_w H^l = t \cdot \gamma_w \cdot S \\
H^l = t \cdot S \\
\text{Or } H^l - t = t \cdot S - t = t(S - 1) \\
\text{Or } \frac{H^l - t}{S - 1} \approx \frac{h}{S - 1} \text{ - the limiting thickness}
\]

where \( h \) is the ordinate of the hydraulic gradient line measured above the top of the floor.

Now applying a safety factor of 4/3

\[ t = \frac{4}{3} \left( \frac{h}{S - 1} \right) \]

**Divide Wall**

This is a long solid wall of stone masonry built at right angles to the axis of the body of the weir. Its function is to divide the river channel into two portions and to create a still pond near the canal head regulator end. It is usually built in stone masonry but it could be done in earth with stone pitching or in concrete. Because of the divide wall a still pond is created. This helps the deposition of silt before the water is allowed to pass through the head regulator. It also protects the regulator from the direct turbulence of river flow.
6.1.4. Scour gates for sedimentation control

Many rivers carry substantial sediment loads, especially during the rainy season, in the form of sand, silt, weeds, moss and tree leaves. Approximately 70% of all suspended and bed load sediments travel in the lower 25% of the flow profile. While suspended silt can be beneficial to the scheme by adding nutrients to the farmland, coarse sediments usually cause problems once they are blocked by a weir or other diversion structure. Headworks have to be adapted to these sediment loads to avoid silting of canals and structures. A properly-designed intake should divert only the relatively clean upper part of the water flow into the canal and dispose of the lower part down the river. A sluice should therefore be incorporated into the diversion structure design. It should be placed in line with the weir near the canal intake (Figure 51). Its seal level is generally placed at the river bed level while the floor to the intake gate should be located higher (Figure 52).

The control arrangement in the scour sluice generally consists of a series of stop logs (timber, concrete) or a sluice gate. This arrangement allows the water to be raised when there are very few or no sediments in the water. During the flood season, the sluice is permanently open or opened at regular intervals so that depositions of sediments can be flushed away. The guide wall prevents lateral movement of sediments deposited in front of the weir and separates the flow through the sluice and the flow over the weir.

Under Sluices (Scour Sluices): These are openings provided in the body wall of the weir. Their main function is to prevent the obstructions to the flow of water through the main sluice. They also transport deposited silt in front of the head regulator at the upstream side, to the downstream side, thus preventing the entry of bed silt in the canal. They create a clear, unobstructed river channel at the approach portion of the head regulator. They also help to reduce maximum flood level. They are provided near the wing walls of the weir. They are operated by
means of gates provided for this purpose. The gates are operated by levers provided at the top of the weir. In designing the scour sluices, the following are to be noted:

i) the design flow should be at least twice the discharge in the off-take channel

ii) the crest of the under sluice should be one metre lower than the crest of the head regulator.

iii) The downstream portion of the under sluice must be well protected to prevent downstream erosion.

**Canal Head Regulator**

A canal head regulator (or head sluice) is a structure constructed at the head of an off-take channel of a weir. This may consist of a number of spans created by a number of piers and covered by a number of gates. The functions of canal head regulator are:

i) to regulate the quantity of water passing into the canal

ii) to control the amount of silt entering the channel

iii) for shutting out flood flows.

The head regulator consists of a raised crest with abutments on both sides. Piers may be constructed on the crest to create a number of spans and to support a roadway and a platform at its top to operate the gates.

Marginal Bunds: are earthen embankments constructed parallel to the river along the banks. They are located near the approaches of the diversion head works. Their main function is to control the floods and prevent submergence of the areas behind the hydraulic structures. Their design and construction are similar to earth dams.

Guide Banks (or Bell’s bunds): are constructed almost parallel to the flow of the river, near the hydraulic structure. They are bunds with a straight length and a curved end at their flanks. Their function is to guide the flow into and protect the hydraulic structure by controlling and regulating the flow of the river.
Chapter 7

DISCHARGE MEASUREMENT

Discharge measurement in irrigation schemes is important for the following reasons:

- To ensure the maintenance of proper delivery schedules
- To determine the amount of water delivered for water pricing, where it is applicable
- To detect the origin of water losses and to estimate the quantity
- To ensure efficient water distribution
- To conduct applied research

Almost any kind of obstacle that partially restricts the flow of water in an irrigation canal and provides a free fall, to ensure that upstream and downstream flow are independent, can be used as a measuring device, provided that it can be calibrated. Standard structures, which have already been accurately described and calibrated, exist. Weirs, flumes and orifices are the devices that are normally used for discharge measurement.

6.6.1. Discharge measurement equations
The three fundamental equations used to solve discharge problems in canals are based on the principles of conservation of mass, energy and momentum. For our purposes, only the conservation of mass and energy equations will be dealt with.

Conservation of mass

Conservation of mass leads to the Continuity Equation 12 to be constant:

\[ Q = A \times V = \text{Constant} \]

Conservation of energy

Conservation of energy applied along a streamline results in the Bernoulli Equation:

\[ \frac{P}{\gamma} + \frac{V^2}{2g} + z = \text{Constant} \]

Where: \( P \) = Pressure (kgf/m^2) \( \gamma \) = Density of water (kg/m^3) \( V \) = Water velocity (m/sec) \( g \) = Gravitational force (9.81 m/sec^2) \( z \) = Elevation above reference line (m)
Equation 38 sums up the pressure head, velocity head and gravitational head to give the total head. For an open canal, the pressure head equals the water depth \( h \). When there is frictional loss along the flow path, an expression for frictional head loss must be included. Thus applying the Bernoulli Equation to two successive cross sections along a flow path results in:

**Equation 39**

\[
h_1 = \frac{V_1^2}{2g} + z_1 = h^2 + \frac{V_2^2}{2g} + z_2 + HL
\]

The numbers 1 and 2 refer to the first and second cross section. HL is the frictional head loss.

---

**Specific energy**

The concept of specific energy is used in the analysis of critical flow. At any cross-section of a canal, the energy with respect to the canal bed is referred to as specific energy. It is derived from the Bernoulli Equation according to the following equation:

\[
E = \frac{h}{2g} + V^2
\]
Where:  $E = \text{Specific energy (m)}$,  $h = \text{Depth of flow (m)}$,  $g = \text{Gravitational force (9.81 m/sec}^2)$

$V = \text{Water velocity (m/sec)}$

Assuming a uniform velocity distribution, the specific energy is constant across the section. Combining the above equation and the Continuity Equation gives:

$$E = \frac{h}{2g} + \frac{Q^2}{A}$$

The cross-sectional area varies with the depth of flow only if the geometry of the canal is constant. Therefore, for a given discharge the specific energy is a function of depth alone.

**Specific energy can be determined for different structures:**

**Rectangular canal**
Plotting $E$ against $h$ for different values of $(Q/b)$ gives curves as shown in diagram above. The curves show that, for a given discharge and specific energy, there are two alternate depths of flow, which coincide at a point where the specific energy is a minimum for a given discharge. Below this point, flow is physically not possible. At this point flow is critical and it occurs at critical depth and velocity. At a greater depth, the velocity is low and flow is sub-critical. At the lesser depth, the velocity is high and flow is super-critical. For sub-critical flow, the mean velocity is less than the velocity of propagation of stream disturbances such as waves. Thus, stream effects can be propagated both upstream and downstream. This means that downstream conditions affect the behaviour of flow. When flow is supercritical, the velocity of flow exceeds the velocity of propagation. Consequently, stream effects (for example, waves) cannot be transmitted upstream, and downstream conditions do not affect the behaviour of the flow. For critical flow, the specific energy is a minimum for a given discharge. In this case, a relationship exists between the minimum specific energy and the critical depth. This relationship is found by differentiating Equation 41 with respect to $h$, while $Q$ remains constant. This gives:

$$v_c = \left[ \frac{g \times A_c}{b_c} \right]^{1/2}$$

Froude Number

The Froude Number is calculated according to:

$$Fr = \frac{v}{(g \times h)^{1/2}}$$

Where:  $Fr = 1$ for critical flow,  $Fr = > 1$ for super-critical flow,  $Fr = < 1$ for sub-critical flow

If a structure is built in a canal which has sub-critical flow, it may cause the flow to pass through the critical to the super-critical state. This means that the state
upstream of the structure becomes independent of the state downstream. This can either be achieved if the structure narrows the canal, which means increasing the \((Q/b)\)-ratio without altering the specific energy, or if it raises the canal bed, which means reducing the specific energy without altering the discharge per unit width. That is how critical flow is obtained with a measuring device. A control section in a canal is a section that produces a definitive relationship between water depth and discharge.

Hydraulic jump

If, through a structure, super-critical flow is introduced in a canal where the normal flow is sub-critical, flow adjusts back to the sub-critical state through a hydraulic jump in which the water level rises over a short distance with much visible turbulence. This situation occurs, for example, downstream of a sluice gate or a flume. It is undesirable to have a hydraulic jump in an unlined canal because of the risk of scour. In such cases, a jump is usually induced over a concrete apron by means of a sill or baffle blocks set in the floor, as shown in Figure 67.

The relationship between depths just upstream and downstream of a hydraulic jump is found by the application of the momentum theory to the simplified situation shown in Figure 68. It is assumed that boundary frictions are negligible over the length of the jump. For a rectangular canal it can be shown that:

**Equation 43**

\[
h_2 = -\frac{h_1}{2} + 0.5 \times \left[ h_1^2 + \left( 8 \times V_1^2 \times \frac{h_1}{9} \right) \right]^{1/2}
\]

6.6.2. Weirs

**FLOW OVER WEIRS AND NOTCHES**
A weir in general is any regular obstruction over which flow occurs. Weirs, especially the sharp-crested ones are commonly used for measuring large and small open flows in the field or laboratory.

The zone of fluid before the wall of the weir is called the head race while the zone of water after the weir is called the tail race.

The distance $L_B$ from the wall of the weir to the point where a fall in the level of stream is noticeable, known as the drawdown section is given by:

$$L_B = (3-5)H$$

$H$ - measured at the section $B-B$ is called the geometric head which is the rise of fluid above the crest of the weir.

$B$ – width of weir going into the paper.

$\delta$ – thickness of the wall of the weir.

$z$ – geometric drop in the level of fluid at the weir. It is the difference between the head race and the tail race.

$v_o$ – approach velocity.

$H_o$ – The total head which is given by the sum of the geometric head and the velocity head.

$z_o$ – total drop in head over the weir which is the sum of the geometric drop plus the velocity head.
Classification of weirs

1. By form of the opening
   - a) Rectangular
   - b) Triangular
   - c) Trapezoidal
   - d) Circular
   - e) Parabolic
   - f) With inclined crest

2. By the thickness of the cross-section of weir wall
   - a) Sharp-crested weir \( \delta \leq (0.1 - 0.5)H \): In sharp-crested weirs, the nappe springs free as it leaves the upstream face
   - b) Broad – crested weir \( 2H \leq \delta < 8H \): In broad-crested weir, the falling nappe is supported on the crest and does not allow the nappe to fall free.
   - c) Weir with practical profile (Orgee weir)

3. By width of obstacle in relation to width of stream
   - a) Suppressed weir: - When the length of the weir is the same as the width of the channel (stream) It is so called because in this case, the effect of sides or ends of the weir on contraction of the nappe is eliminated or suppressed. The width of the nappe is the same as the width of the stream.
   - b) Weirs with end contractions: - In this case, the length of the weir is less than the width of the channel. Therefore there is a lateral contraction of the nappe so that the length of the nappe is less than the length of the weir crest.

4. By the level of downstream water level relative to the crest of the weir.
   - a) Un-submerged weir: - When the water level on the downstream side of the weir is below the crest of the weir, it is said to be un-submerged.
   - b) Submerged weir: When the water level on the downstream side of the weir rises above the level of the crest, the weir is said to be submerged.

The flow of fluid over weirs is extremely complex phenomenon and virtually impossible of exact analytical solution. As result, all weir formula are derived by making lots of
simplification of the problem which leads to an approximate solution which is somehow rectified by the use of experimental coefficients derived from experimental set ups.

To derive a simplified relationship for a weir, the following assumptions are made:

i) velocity distribution upstream from the weir is uniform

ii) all fluid particles move horizontally as they traverse pass the weir crest

iii) the pressure in the nappe is zero; ie atmospheric

iv) the influence of viscosity, surface tension, turbulence are all neglected

After simplification, the sharp-crested weir is considered as a large orifice of rectangular shape placed in the channel such that the head on its upper edge is zero. As a result, the upper edge is eliminated leaving only the lower edge or the crest. By considering the weir as a large orifice and by considering an elementary strip of of area $Bdh$ below the free surface, the velocity of flow is given by:

$$u = c_s \sqrt{2gh}$$

The flow rate through the strip

$$dQ = Bdhc_s \sqrt{2gh}$$
The theoretical flow passing over the weir is obtained by integrating the above equation over the geometric head.

\[ Q = \frac{2}{3} Bc \sqrt{2gh} dh = \frac{2}{3} Bc \sqrt{2gH^{3/2}} \]

To obtained the actual flow rate over the weir due to the above assumptions, we introduce an experimental coefficient of discharge

\[ Q = \frac{2}{3} C_D B \sqrt{2gH^{3/2}} \]

If the approach velocity \( v_0 \) is appreciable (>1.0), then instead of geometric head one has to use the total head \( H_0 \).

As can be seen, the formula for flow over weir seems simple and easy to use however, the difficulty is with the correct determination of the discharge coefficient. In the laboratory, this could be determined quite easily and accurately, however, it becomes extremely difficult to determine this accurately in the field. A few empirical formulae exist for the determination of the discharge coefficient for a rectangular weir.

\[ C_D = 0.40 + 0.05 \frac{H}{P} \quad \text{---Chugaev R. R. (2/3 in front of flow rate absorbed)} \]

\[ C_D = 0.405 + 0.003 \frac{1}{H} \quad \text{---Basen (2/3 in front of formula absorbed)} \]

\[ C_D = 0.397 + 0.00015 \frac{1}{H^{3/2}} \quad \text{---Fteley and Stearn (2/3 in front of formula absorbed)} \]

\[ C_D = 605 + 0.08 \frac{H}{P} + 1/1000H \quad \text{---Rehbock} \]

\[ C_D = 0.611 + 0.075 \frac{H}{P} \quad \text{---taken from A. K. Jain.} \]

**Contracted Weirs**

For weir with end contractions, Francis observed that the effect of the contraction is to reduce the length of nappe by 0.1H. Thus if there are \( n \) end contractions, then the length of the nappe will be reduced by 0.1nH and will be equal to \( B - 0.1nH \) and the discharge equation will then be given as:

\[ Q = \frac{2}{3} C_D (B - 0.1nH) \sqrt{2gH^{3/2}} \]

**Flow over a Triangular Weir (Notch)**
A triangular weir (notch) is particularly useful where the discharge is to vary over a large range and the same accuracy is desired for both small and large discharges because of the following advantages:

i) The coefficient of discharge for a triangular weir is practically independent of the head. This is because, for all heads the ratio of the head to the wetted length of crest is constant. But in a rectangular weir, the ratio of head to the wetted length of crest is not constant.

ii) When the discharge is small, a triangular notch provides a greater head than the rectangular notch. Hence head measurement can be done more accurately over the triangular notch than over the rectangular notch.

iii) When the flow rate is small, there is the tendency of a clinging nappe in a rectangular notch. However, for the same flow rate over a triangular notch, the head will be greater and clinging nappe will be avoided.

The equation for the discharge over the vee-notch can be obtained just like we did in the rectangular weir by considering an elementary strip of width $b$ and height $dh$.

Considering the notch as a large orifice, the velocity of flow is given by:

$$u = c_r \sqrt{2gh}.$$  

The flow rate through the strip

$$dQ = bdhc_r \sqrt{2gh}$$

But $b = 2(H - h) \tan \theta/2$
Putting $b$ into the discharge equation, $dQ = 2(H - h) \tan \frac{\theta}{2} c_r \sqrt{2gh} dh$

\[ Q = \int_0^H 2 \tan \frac{\theta}{2} c_r \sqrt{2g} \left( Hh^{1/2} - h^{3/2} \right) dh = \frac{8}{15} c_r \sqrt{2g} \tan \frac{\theta}{2} H^{5/2} \]

The above equation represents the theoretical discharge without consideration of the contraction of the flow area. The actual discharge is obtained by applying the coefficient of discharge $C_D$, to obtain:

\[ Q = \frac{8}{15} C_D \sqrt{2g} \tan \frac{\theta}{2} H^{5/2} \]

By representing $k = (8/15)C_D(2g)^{1/2}$ and taking the average value of $C_D = 0.60$

\[ k = 8/15 \times 0.60 \times 4.43 = 1.42 \text{ and hence} \]

\[ Q = 1.42 \times \tan(\theta/2) H^{5/2} \]

For a right angled Vee-notch, $\theta = 90^\circ$ and hence

\[ Q = 1.42 H^{5/2} \]

Empirical formulae 1) By Thompson

\[ Q = 1.4 H^{5/2} \]

2. By King

\[ Q = 1.343 H^{2.47} \]

By Grave When $22^\circ \leq \theta \leq 118^\circ$ \[ Q = 1.331 (\tan \theta/2)^{0.996} H^{2.47} \]

The coefficient of discharge is made up mainly of the contraction coefficient and the velocity coefficient. The contraction coefficient of a notch depends on the wetted perimeter. For a rectangular notch, the wetted perimeter does not vary with head because the base, which forms part of the wetted perimeter remains constant for all heads. Consequently, the coefficient of contraction and hence the coefficient of discharge will not be constant for all heads. In a triangular notch, there is no base and therefore the contraction is due to the sides only. The wetted perimeter depends on the length of the sides which in turn depend on the head. Consequently, the coefficient of discharge is fairly constant in a triangular notch.

**Trapezoidal Weir**
A trapezoidal weir has an opening of a trapezoidal shape and may be considered as a combination of a rectangular and a triangular weirs.

The trapezoidal notch is equivalent to two notches; one rectangular with length \( b \) and the other a triangular with an apex angle of \( \theta \). The total discharge over the trapezoidal notch would be equal to the sum of discharges over the equivalent rectangular and triangular notches.; thus
\[
Q_{\text{trap}} = Q_1 + Q_2 = b\sqrt{2gH^{3/2}} + \frac{8}{15} C_D \sqrt{2g} \tan \frac{\theta}{2} H^2 = C_D' \sqrt{2g} H^{3/2} \left( \frac{2}{3} b + \frac{8}{15} H \tan \frac{\theta}{2} \right)
\]

Cippoletti Weir

The principle behind the Cippoletti weir is based on the fact that in a rectangular weir, due to end contractions, the effective length of the weir is reduced and consequently the discharge. On the other hand, by the addition of side slopes to a rectangular weir, gives an increase in discharge. Therefore if the side slopes of a trapezoidal weir are so adjusted such that the reduction in discharge due to end contractions is just equal to the increase in discharge due to the addition of side slopes, the net effect I discharge is zero and therefore the standard weir formula without end contraction can be used.

The Cippoletti Weir is a trapezoidal weir with side slopes 1 horizontal to 4 vertical such that the discharge in effect equal to that of a rectangular weir without end contraction.

For Cippoletti weir, 
\[
Q = \frac{2}{3} C_D b \sqrt{2gH^{3/2}}
\]

Cippoletti gave the equation for the discharge through such a weir as:
\[ Q = 1.86 \cdot b \cdot H^{3/2} \]

**Submerged Weir**

A submerged weir (notch) is one in which the level of fluid in the downstream side of the weir is above the crest of the weir.

The flow over such a weir can be obtained by dividing the flow into two portions:

i) Flow over the upper part of the section (above line AB) of depth \( H_1 - H_2 \) may be considered as a free from a weir into the atmosphere and

ii) Flow through the lower part of depth (below line AB) may be considered as a discharge through a submerged orifice.

The discharge over the upper portion behaving as a free weir is given by

\[ Q_1 = \frac{2}{3} C_D b \sqrt{2g} (H_1 - H_2)^{3/2} = \frac{2}{3} C_D b \sqrt{2g} H^{3/2} \]

and the lower portion acting as an orifice is given by:

\[ Q_2 = C_D H_2 \sqrt{2g} (H_1 - H_2) = C_D b H_2 \sqrt{2gH} \]

Therefore the total flow is given by:

\[ Q = \frac{2}{3} C_D b \sqrt{2g} H^{3/2} + C_D b H_2 \sqrt{2gH} \]
Experimental works by Herschel, Villemonte and Mavis on the discharge characteristics of a number of sharp-crested submerged weirs on rectangular, triangular, parabolic and proportional weirs showed the results for all types could be represented by the equation:

\[ Q = Q_1 \left[ 1 - \left( \frac{H_2}{H_1} \right)^n \right]^{0.385} \]

where \( Q \) = discharge for submerged condition
\( Q_1 \) = free discharge given by \( Q = K H_1^n \)
\( n \) = exponent in the free discharge equation
\( n = 1.44 \) for contracted weir
\( n = 1.5 \) for suppressed weir
\( n = 2.50 \) for 90° notch.

**Broad-Crested Weir**

Broad-crested weirs (long-based weirs) are weirs that have crest lengths that are sufficiently longer (\( \delta > 0.67H \)) to prevent the nappe from springing free. These weirs are usually constructed of concrete, have rounded edges, and are capable of handling much larger discharges than sharp-crested weirs.

Some advantages of broad-crested weirs.

i) It is cheaply and easily fabricated (can be done in-situ or off-site)
ii) It is easy to install
iii) It possesses a wide modular range;
iv) It produces minimum afflux (increases in upstream depth due to weir installation)
v) It requires a minimum of maintenance.

Flow over a broad-crested weir must pass through the critical depth somewhere near the downstream corner. Broad-crested weirs are characterized by the presence of two drops in the water level over the weir.
By taking Bernoulli’s equation in sections 1 and 2, the velocity of flow on the weir is obtained as

\[ v = c_s \sqrt{2gZ_{B(0)}} = c_s \sqrt{2g(H_o - h)} \]

For a rectangular weir, the discharge is given by:

\[ Q = Av = b \times h \times v = bh c \sqrt{2g(H_o - h)} \]  \[ \text{-------}(*) \]

\[ q = hc \sqrt{2g(H_o - h)} = f(h) \]  \[ \text{-------------}(**) \]

As can be seen from the above equation, in order to determine the discharge, there is the need to first find \( h \), the depth of flow on the weir. There are many methods of finding \( h \) and \( Q \) but the Belanch’s method based on the principle of maximum discharge is much more appealing.

As can be seen from the above equation, the discharge is only a function of \( h \) and the value of \( h \) varies between \( 0 \leq h \leq H_o \) however the discharge goes to zero as \( h = 0 \) and as \( h = H_o \).

Since the function \( (***) \) is a continuous function and the function goes to zero at the two limits, then it follows that there must be a turning point (maximum or minimum) within the limits. This is the basis of Belanch’s principle which states that: for a given head \( H_o \) on a broad-crested weir, the depth that will be established on the weir is such as to result in the maximum discharge on the weir.

With the above postulate, the required depth is obtained by:
\[
\frac{dq}{dh} = \frac{d}{dh} \left[ c_r h \sqrt{2g(H_o - h)} \right] = 0
\]

Neglecting the constants and differentiating

\[
\frac{d}{dh} \left( h \sqrt{H_o - h} \right) = \sqrt{H_o - h} - \frac{1}{2} \frac{h}{\sqrt{H_o - h}} = 0
\]

which gives \( h = \frac{2}{3} H_o \)

Substituting this value of \( h \) into (*) we

\[
Q = b \frac{2}{3} H_o c_v \sqrt{2g \left( H_o - \frac{2}{3} H_o \right)} = \frac{4}{27} b c_v \sqrt{2g H_o^{3/2}} = 0.385 c_v b \sqrt{2g H_o^{3/2}}
\]

\[
Q = C_D b \sqrt{2g H_o^{3/2}}
\]

\[
q = C_D \sqrt{2g H_o^{3/2}}
\]

where \( C_D = 0.385 \) \( C_v \), \( C_D \) ranges between 0.32 to 0.36.

**Ventilation of Weirs**

When flow takes place over a suppressed weir with wing walls on the downstream side, the free access of air below the nappe is prevented. The air entrapped between the nappe and the weir face is gradually evacuated by the dynamic action of the flowing fluid. This eventually leads to subatmospheric pressure in the underside of the nappe bringing about increases in the discharge.

Investigations by Bazin have shown that if the flow is sufficient to prevent the air having access to the underside of the nappe, it may assume one of the forms shown below.
**Fully aerated (free, ventilated) nappe:** In this type, the weir discharges free as the pressure below the nappe is maintained at atmospheric by proper ventilation.

**Drowned (depressed) nappe:** When the underside of the nappe is poorly aerated, the air enclosed between the nappe and the downstream face of the weir gradually mixes up with the flowing fluid and is carried away. This causes the pressure under the nappe to fall below atmospheric. The pressure difference on the two sides of the nappe—(the upper side exposed to atmospheric while the bottom side has sub-atmospheric pressure)—causes the nappe to deflect towards the weir.

**Clinging nappe:** This type occurs when there is no ventilation and all the air under the nappe has been carried away by the dynamic action of the flowing fluid. The nappe is forced to adhere to the downstream face of the weir by the atmospheric pressure on the upper side. In this type, the discharge is 20 to 30 per cent more than in free nappe.

Knowledge of these types of nappes is necessary especially when the weir is used for floe (discharge) measurement. All the discharge formulae have been derived for free nappe and therefore some adjustment in the measured flow must be made when other forms of nappe occurs during discharge measurement. For discharge measurement it is always advisable to ensure that the nappe is fully aerated.

**FLUMES**

Although weirs are the cheapest and simplest structures for flow measurement in open channels, however the relatively high losses caused by weirs and the tendency of sedimentation of suspended particles due to reduced velocities in the vicinity of the weir, could in certain cases pose important limitations Flumes provide a convenient alternative...
to weirs for flow measurement in open channels where high head losses and sedimentation are of concern. Such cases include flow measurement in wastewater treatment plants and irrigation channels with suspended particles.

Flumes are devices in which the flow is locally accelerated by a streamlined lateral contraction in the channel sides. A flume has: a convergent section, in which the flow accelerates, a throat section and a divergent section, in which the flow returns to normal. Flumes are of two types: non-modular or the venturi flume and the modular or the standing wave flume.

In the non-modular flume, the velocity at the throat is maintained below the critical value so that no standing wave is produced. However, in the modular type, the flume is designed such that the velocity of flow at the throat is greater than the critical velocity thereby resulting in a standing wave within the flume.

**Non-Modular Flume (Venturi Flume)**

When the width of a channel is reduced while the bed remains flat, the discharge per unit width increases. If the losses are neglected, the specific energy remains constant. If the conditions are made such that the free surface does not pass through the critical depth, the arrangement forms a Venturi flume.

Referring to the diagram above, for continuity of flow,

\[
h_1 \times b_1 \times v_1 = h_2 \times b_2 \times v_2 \quad --continuity \, equation \, for \, section \, 1-1 \, and \, 2-2.
\]

Applying Bernoulli’s equation to sections 1-1 and 2-2.

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\[ h_1 + \frac{v_1^2}{2g} = h_2 + \frac{v_2^2}{2g} \]

Substituting \( v_1 \) for continuity equation,
\[ h_1 - h_2 = \frac{v_2^2}{2g} \left[ 1 - \left( \frac{b_2 h_2}{h_1 b_1} \right)^2 \right] \]

\[ v_2 = \sqrt{\frac{2g(h_1 - h_2)}{1 - \left( \frac{b_2 h_2}{h_1 b_1} \right)^2}} \]

\[ Q = v_2 A_2 = b_2 h_2 \sqrt{\frac{2g(h_1 - h_2)}{1 - \left( \frac{b_2 h_2}{b_1 h_1} \right)^2}} \]

Introducing the discharge coefficient, \( C_D = \frac{\text{(Actual discharge)}}{\text{(Theoretical discharge)}} \)

\[ Q = C_D b_2 h_2 \sqrt{\frac{2g(h_1 - h_2)}{1 - \left( \frac{b_2 h_2}{b_1 h_1} \right)^2}} = C_D \frac{A_1 A_2}{\sqrt{A_1^2 - A_2^2}} \sqrt{2g(h_1 - h_2)} \]

\( C_D = 0.95 \text{ to } 0.99 \)

Therefore to measure the flowrate, water depth are measured at two locations, one at section 1-1 and the other at section 2-2.

**Modular Flume or Standing Wave Flume (eg. Parshall Flume)**

As already mentioned, modular flumes are designed to make the flow pass through the critical depth in some section of the throat.

By considering a section upstream with depth \( h_1 \) and another section at the throat with depth \( h_2 \), then applying Bernoulli’s equation,

\[ h_1 + \frac{v_1^2}{2g} = h_2 + \frac{v_2^2}{2g} = H \]

\[ v_2 = (2g(H-h_2)^{1/2} \]

Therefore the discharge \( Q = b_2 h_2 v_2 = b_2 h_2 (2g(H-h_2))^{1/2} \)

\[ Q = b_2 \sqrt{2g(Hh_2^2 - h^3)} = b_2 \sqrt{2g(Hh_2 - h^3)^{1/2}} \]

For the maximum discharge, the quantity \((Hh_2^2 - h^3)^{1/2}\) should be maximum
Therefore \[
\frac{d\left(Hh_2^2 - h^3\right)}{dh} = 0
\]

Therefore \[2Hh_2 - 3h_2^2 = 0\]

\[h_2 = \frac{2}{3}H\]

Thus for the flow to be maximum, the depth at the throat section should be two-thirds the total energy.

For this condition, the velocity \[v_2 = \sqrt{2g(H - h_2)} = \sqrt{2g\left(\frac{3h_2}{2} - h_2\right)} = \sqrt{gh_2}\]

This implies that for maximum discharge, the depth at the throat must be equal to the critical depth and the velocity critical velocity.

Substituting \[h_2 = 2/3H\] in the discharger expression, we shall obtain

\[Q = b_2\sqrt{2g \left(\frac{2}{3}H \left(H - \frac{2}{3}H\right)\right)^{1/2}} = \frac{2}{3\sqrt{3}}\sqrt{2gb_2H^{3/2}} = 1.705b_2H^{3/2} = 1.705b_2(h_i + v_i^2/2g)^{3/2}\]

Although flat bed modular flume is easier to construct, it is sometimes necessary to raise the invert in the throat to attain critical conditions. For the flat or raised floor, the throat length should ideally be sufficient to ensure that the curvature of water surface is small to make the water surface parallel to the invert. Such ideal flumes become relatively long and expensive. A more compact, short and less expensive groups of flumes have been constructed. However, the surface water profile varies rapidly and as such the theoretical analysis for ordinary flumes is not wholly applicable. Therefore empirical relationship are usually developed for such flumes (e.g; Parshall Flume), which make each such flume standard with its unique calibration curve.

The Parshall Flume
The Parshall flume is a widely-used discharge measurement structure. Figure 76 shows its general form. The characteristics of Parshall flumes are:

i) Small head losses, ii) Free passage of sediments, iii) Reliable measurements even when partially submerged and iv) Low sensitivity to velocity of approach

The Parshall flume consists of a converging section with a level floor, a throat section with a downward sloping floor and a diverging section with an upward sloping floor. Flume sizes are known by their throat width and each size has its own characteristics, which is unique (see Table 31).

The flow through the Parshall flume can occur either under free flow or under submerged flow conditions. Under free flow the rate of discharge is solely dependent on the throat width and the measured water depth, ha. The water depth is measured at a fixed point in the converging section.
The upstream water depth-discharge relationship, according to empirical calibrations, has the following general form:

\[ Q = K \times (h_a)^u \]

*Where:* \( Q \) - Discharge (m³/s); \( h_a \) - Water depth in converging section (m); \( K \) - A fraction, which is a function of the throat width; \( u \) - Variable, lying between 1.522 and 1.60

Table 32 gives the values for \( K \) and \( u \) for each flume size.

When the ratio of gauge reading \( h_b \) to \( h_a \) exceeds 60% for flumes up to 9 inches, 70% for flumes between 9 inches and 8 feet and 80% for larger flume sizes, the discharge is reduced due to submergence. The upper limit of submergence is 95%, after which the flume ceases to be an effective measuring device because the head difference between \( h_a \) and \( h_b \) becomes too small, such that a slight inaccuracy in either head reading results in a large discharge measurement error.
The discharge under submerged conditions is:

$$Q_s = Q - Q_c$$

Where: $Q_c$ – Reduction of the modular discharge due to submergence

Figure 77 gives the corrections $Q_c$ for submergence for flumes with 6 inch, 9 inch and 1 foot throat width. The correction for the 1 foot flume is made applicable to other sizes by multiplying the correction $Q_c$ for the 1 foot by the factors given in Figure 77 (1 foot flume).
<table>
<thead>
<tr>
<th>Throat width b feet + inches</th>
<th>Discharge range (m³/sec x 10⁻⁹)</th>
<th>Equation Q = K x hₚ^n</th>
<th>Head range (m)</th>
<th>Modular limit hₚ/hₐ (m)</th>
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<tbody>
<tr>
<td></td>
<td>Minimum</td>
<td>Maximum</td>
<td>Minimum</td>
<td>Maximum</td>
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<tr>
<td>1'</td>
<td>0.09</td>
<td>5.4</td>
<td>0.0004 hₚ1.66</td>
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<table>
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<td>59.33</td>
<td>0.09</td>
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</tr>
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<td>40'</td>
<td>0.60</td>
<td>74.70</td>
<td>0.09</td>
<td>1.63</td>
</tr>
<tr>
<td>50'</td>
<td>0.75</td>
<td>93.04</td>
<td>0.09</td>
<td>1.63</td>
</tr>
</tbody>
</table>
Figure 77
Discharge corrections due to submergence for Parshall flumes with different throat width

a. Parshall flume with a throat width \( b \) of 6 inch or 15.2 cm
Figure 77
Discharge corrections due to submergence for Parshall flumes with different throat width

b. Parshall flume with a throat width b of 9 inch or 22.9 cm
Figure 77
Discharge corrections due to submergence for Parshall flumes with different throat width

c. Parshall flume with a throat width $b$ of 1 foot or 30.5 cm
The weir is the most practical and economical device for water measurement. Weirs are simple to construct, easy to inspect, robust and reliable. Discharge measurement weirs can either be sharp-crested (Figure 69, 70, 71) or broadcrested (Figure 72).
Sharp-crested weirs

Sharp-crested weirs, also called thin plate weirs, consist of a smooth, vertical, flat plate installed across the channel and perpendicular to the flow (Figure 69). The plate obstructs flow, causing water to back up behind the weir plate and to flow over the weir crest. The distance from the bottom of the canal to the weir crest, \( p \), is the crest height. The depth of flow over the weir crest, measured at a specified distance upstream of the weir plate (about four times the maximum \( h_1 \)), is called the head \( h_1 \). The overflowing sheet of water is known as the nappe.

Thin plate weirs are most accurate when the nappe springs completely free of the upstream edge of the weir crest and air is able to pass freely around the nappe. The crest of a sharp-crested weir can extend across the full width of channel or it can be notched. The most commonly used notched ones are:

- Rectangular contracted weir
- Trapezoidal (Cipoletti) weir (Figure 70)
- Sharp sided 90° V-notch weir (Figure 71)

The type and dimensions of the weir chosen are based on the expected discharge and the limits of its fluctuation. For example, a V-notch weir gives the most accurate results when measuring small discharges and is particularly adapted to the measurement of fluctuating discharges.

Calibration curves and tables have been developed for standard weir types. The conditions and settings for standard weirs are as follows:

1. The height of the crest from the bottom of the approach canal (\( p \)) should preferably be at least twice the depth of water above the crest and should in no
case be less than 30 cm. This will allow the water to fall freely, leaving an airspace under and around the jets.

ii. At a distance upstream of about four times the maximum head a staff gauge is installed on the crest with the zero placed at the crest elevation, to measure the head $h_1$.

iii. For the expected discharge, the head ($h_1$) should not be less than 6 cm and should not exceed 60 cm.

iv. For rectangular and trapezoidal weirs, the head ($h_1$) should not exceed 1/3 of the weir length.

v. The weir length should be selected so that the head for the design discharge will be near the maximum, subject to the limitations given in (ii) and (iii).

vi. The thickness of the crest for sharp-crest weirs should be between 1-2 mm. In sediment-laden canals, a main disadvantage of using weirs is that silt is deposited against the upstream face of the weir, altering the discharge characteristics. Weirs also cannot be used in canals with almost no longitudinal slopes, since the required difference in elevation between the water levels upstream and downstream side of the weir is not available. Discharge equations for weirs are derived by the application of the Continuity and Bernoulli Equations (Equation 12 and 38 respectively). In each case, a discharge coefficient is used in order to adjust the theoretical discharge found by laboratory measurement.

**Rectangular contracted weir**

A rectangular contracted weir is a thin-plate weir of rectangular shape, located perpendicular to the flow. To allow full horizontal contraction of the nappe, the bed and sides of the canal must be sufficiently far from the weir crest and sides. Many practical formulae have been developed for computing the discharge, amongst which are the following.

**Equation 44**

Hamilton-Smith formula:  
$$ Q = \left[ 0.618 \times \left( 1 - \frac{0.1h}{b} \right) \right] \times \frac{2}{3} \times (2g)^{1/2} \times b \times h^{3/2} $$

**Equation 45**

Francis formula:  
$$ Q = 1.838 \times (b - 2h) \times h^{3/2} $$

Where:
Q = Design discharge over weir (m³/sec)  b = Length of weir crest (m)  h = Design water depth measured from the top of the weir crest (m)  Table 28 gives discharge data related to length of crest, b, and water head, h, over a weir.

**Trapezoidal (Cipoletti) weir**
The trapezoidal weir has a trapezoidal opening, the base being horizontal. The Cipoletti weir is a trapezoidal weir with the sides having an outward sloping inclination of 1 horizontal to 4 vertical (Figure 70). This side slope is such that the water depth-discharge relationship is the same as that of a full width rectangular weir.

The discharge equation for a Cipoletti weir is:

**Equation 46**

\[ Q = 1.859 \times b \times h^{0.2} \]

Where:
Q = Design discharge over weir (m³/sec)  b = Length of weir crest (m)  h = Design water depth measured from the top of the weir crest (m)

Table 29 shows discharge data, related to the design water depth, h, and weir length, b.
A V-notch weir has two edges that are symmetrically inclined to the vertical to form a notch in the plane perpendicular to the direction of flow. The most commonly used V-notch weir is the one with a 90° angle. Other common V-notches are the ones where the top width is equal to the vertical depth (1/2 x 90° V-notch) and the one where the top width is half of the vertical depth (1/4 x 90° V-notch) (Figure 71). The V-notch weir is an accurate discharge-measuring device, particularly for discharges less than 30 l/sec, and it is as accurate as other types of sharp-crested weirs for discharges from 30 to 300 l/sec (U.S. Department of Interior, 1975). To operate properly, the weir should be installed so that the minimum distance from the canal bank to the weir edge is at least twice the head on the weir. In addition, the distance from the bottom of the approach canal to the point of the weir notch should also be at least twice the head on the weir (U.S. Department of Interior, 1975).

The general and simple discharge equation for a V-notch weir is:

\[ Q = 0.0783 \times b \times (b - 0.2 \times 0.15) \times b^{1.5} = 0.1068 \times b - 0.0032 \]

where \( Q \) is the discharge in l/sec, \( b \) is the length of the crest in m, and the term \( b - 0.2 \times 0.15 \) adjusts for the depth of the notch.

### Table 28

Discharge Q (m³/sec) for contracted rectangular weir, depending on h and b

<table>
<thead>
<tr>
<th>Head h (m)</th>
<th>Length of crest b (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.30</td>
</tr>
<tr>
<td>0.0025</td>
<td>0.0001</td>
</tr>
<tr>
<td>0.015</td>
<td>0.0010</td>
</tr>
<tr>
<td>0.030</td>
<td>0.0028</td>
</tr>
<tr>
<td>0.045</td>
<td>0.0051</td>
</tr>
<tr>
<td>0.060</td>
<td>0.0078</td>
</tr>
<tr>
<td>0.075</td>
<td>0.0108</td>
</tr>
<tr>
<td>0.090</td>
<td>0.0140</td>
</tr>
<tr>
<td>0.105</td>
<td>0.0175</td>
</tr>
<tr>
<td>0.12</td>
<td>0.0211</td>
</tr>
<tr>
<td>0.15</td>
<td>0.0268</td>
</tr>
<tr>
<td>0.18</td>
<td>0.0511</td>
</tr>
<tr>
<td>0.21</td>
<td>0.0777</td>
</tr>
<tr>
<td>0.24</td>
<td>0.1795</td>
</tr>
<tr>
<td>0.27</td>
<td>0.2084</td>
</tr>
<tr>
<td>0.30</td>
<td>0.2692</td>
</tr>
<tr>
<td>0.36</td>
<td>0.4564</td>
</tr>
<tr>
<td>0.42</td>
<td>0.5527</td>
</tr>
<tr>
<td>0.54</td>
<td>0.9635</td>
</tr>
<tr>
<td>0.60</td>
<td></td>
</tr>
</tbody>
</table>

**Example 29**

A rectangular contracted weir has to be placed in a lined canal. The design discharge is 0.0783 m³/sec and the maximum allowable water depth, h, at the measuring gauge can be 0.15 m. What should be the minimum weir crest length, b, calculated using the Francis formula?

Using Equation 45:

\[ Q = 0.0783 = 1.838 \times (b - 0.2 \times 0.15) \times b^{1.5} = 0.1068 \times b - 0.0032 \]

\[ \Rightarrow b = 0.76 m. \]
Equation 47

\[ Q = 1.38 \times \tan\left(\frac{1}{2} \times \theta \right) \times h^{5/2} \]

Where:
- \( Q \) = Design discharge over the weir (m³/sec)
- \( \theta \) = Angle included between the sides of the notch (degrees)
- \( h \) = Design water depth (m)

Table 30 gives discharge data for the three common V-notches related to water depth (head) and angle°.

**Example 31**

A design discharge of 0.0783 m³/sec has to pass through a V-notch weir with an angle \( \theta \) of 90°. What will be the water depth over the weir?

Substituting the above data in Equation 47:

\[ 0.0783 = 1.38 \times \tan\left(\frac{1}{2} \times 90\right) \times h^{5/2} \Rightarrow h^{5/2} = 0.0783 \Rightarrow h = 0.317 \text{ m}. \]
Broad-crested weir

A broad-crested weir is a broad wall set across the canal bed. The way it functions is to lower the specific energy and thus induce a critical flow (Figure 72).

<table>
<thead>
<tr>
<th>Head (m)</th>
<th>Discharge (m³/sec x 10)</th>
<th>Head (m)</th>
<th>Discharge (m³/sec x 10)</th>
<th>Head (m)</th>
<th>Discharge (m³/sec x 10)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.050</td>
<td>0.003</td>
<td>0.160</td>
<td>0.142</td>
<td>0.270</td>
<td>0.523</td>
</tr>
<tr>
<td>0.055</td>
<td>0.010</td>
<td>0.165</td>
<td>0.153</td>
<td>0.275</td>
<td>0.548</td>
</tr>
<tr>
<td>0.060</td>
<td>0.012</td>
<td>0.170</td>
<td>0.165</td>
<td>0.280</td>
<td>0.573</td>
</tr>
<tr>
<td>0.065</td>
<td>0.015</td>
<td>0.175</td>
<td>0.177</td>
<td>0.285</td>
<td>0.599</td>
</tr>
<tr>
<td>0.070</td>
<td>0.018</td>
<td>0.180</td>
<td>0.190</td>
<td>0.290</td>
<td>0.626</td>
</tr>
<tr>
<td>0.075</td>
<td>0.022</td>
<td>0.185</td>
<td>0.203</td>
<td>0.295</td>
<td>0.653</td>
</tr>
<tr>
<td>0.080</td>
<td>0.025</td>
<td>0.190</td>
<td>0.217</td>
<td>0.300</td>
<td>0.681</td>
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<td>0.085</td>
<td>0.029</td>
<td>0.195</td>
<td>0.232</td>
<td>0.305</td>
<td>0.710</td>
</tr>
<tr>
<td>0.090</td>
<td>0.034</td>
<td>0.200</td>
<td>0.247</td>
<td>0.310</td>
<td>0.739</td>
</tr>
<tr>
<td>0.095</td>
<td>0.039</td>
<td>0.205</td>
<td>0.263</td>
<td>0.315</td>
<td>0.770</td>
</tr>
<tr>
<td>0.100</td>
<td>0.044</td>
<td>0.210</td>
<td>0.279</td>
<td>0.320</td>
<td>0.801</td>
</tr>
<tr>
<td>0.105</td>
<td>0.050</td>
<td>0.215</td>
<td>0.296</td>
<td>0.325</td>
<td>0.832</td>
</tr>
<tr>
<td>0.110</td>
<td>0.056</td>
<td>0.220</td>
<td>0.313</td>
<td>0.330</td>
<td>0.865</td>
</tr>
<tr>
<td>0.115</td>
<td>0.062</td>
<td>0.225</td>
<td>0.332</td>
<td>0.335</td>
<td>0.898</td>
</tr>
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<td>0.120</td>
<td>0.069</td>
<td>0.230</td>
<td>0.350</td>
<td>0.340</td>
<td>0.932</td>
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<tr>
<td>0.125</td>
<td>0.077</td>
<td>0.235</td>
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<td>0.345</td>
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<td>0.130</td>
<td>0.084</td>
<td>0.240</td>
<td>0.390</td>
<td>0.350</td>
<td>1.002</td>
</tr>
<tr>
<td>0.135</td>
<td>0.093</td>
<td>0.245</td>
<td>0.410</td>
<td>0.355</td>
<td>1.038</td>
</tr>
<tr>
<td>0.140</td>
<td>0.102</td>
<td>0.250</td>
<td>0.432</td>
<td>0.360</td>
<td>1.075</td>
</tr>
<tr>
<td>0.145</td>
<td>0.111</td>
<td>0.255</td>
<td>0.454</td>
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<td>1.113</td>
</tr>
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<td>0.150</td>
<td>0.121</td>
<td>0.260</td>
<td>0.476</td>
<td>0.370</td>
<td>1.152</td>
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<td>0.155</td>
<td>0.131</td>
<td>0.265</td>
<td>0.499</td>
<td>0.375</td>
<td>1.181</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>0.380</td>
<td>1.231</td>
</tr>
</tbody>
</table>

Flumes

Discharge measurement flumes are extensively used in irrigation schemes mainly because they:

- Can be used under almost any flow condition.
Have smaller head-losses than weirs, thus are more accurate over a large flow range
Are insensitive to the velocity of approach
Are relatively less susceptible to sediment and debris transport

However, major disadvantages of flumes include the relative large sizes and the accurate manufacturing/construction workmanship required for optimum performance (James, 1988).
A canal section that causes flow to pass from sub-critical through critical to the super-critical state forms a control and the discharge is a single valued function of the upstream water level. Critical flow can be achieved by raising the canal bed, thereby reducing the specific energy, or by decreasing the canal width, thereby increasing the discharge per unit width (see Section 6.6.1). This latter technique is the one used by flumes. A flume has:

- A convergent section, in which the flow accelerates
- A throat, in which critical flow occurs
- A divergent section, in which the flow returns to normal

Super-critical flow passing from the throat will return to sub-critical flow downstream of the flume. This occurs due to the development of a hydraulic jump, which is induced within the divergent section by a sill or other barrier. Where there is sufficient head available, the divergent section of the flume could be avoided as the flow could fall freely in a stilling basin. In this case, weirs could also be used. However, if canals are expected to carry a lot of sediment, the flume should be the better choice. Flumes are most commonly rectangular or trapezoidal in cross-section. The former type is the most simple to construct, but if the canal cross-section is not rectangular there is a risk that unpredictable flow patterns will result from an abrupt change of cross-section.

The most commonly used flumes are:
- Parshall flume
- Trapezoidal flume
- Cut-throat flume

Trapezoidal flume

Whenever the canal section is not rectangular, trapezoidal flumes such as those shown in Figure 79, are often preferred, especially for measuring smaller discharges. A typical trapezoidal flume has an approach, a converging section, a throat, a diverging and an exit section. A minimum transition will be required. An additional advantage is the flat bottom, which allows sediment to pass through fairly easily. Furthermore, the loss in head may be less for comparable discharges. Trapezoidal flumes are particularly suited for installation in concrete-lined canals. The flume should normally be put on top of the lining, thus constricting the flow section to the extent required for free flow conditions over a
whole range of discharges up to the canal design discharge. As a rule of thumb, one can say that the lower the canal gradient the higher the elevation of the flume above the canal bed level. The flow characteristics of the flume can be determined experimentally. This allows for the calibrations of the flume.

As an example, a flume with dimensions such as those given in Figure 79 can be located in a canal with a bed width of 0.30 m (1 foot), having side slopes of 1:1. The range of calibrated water depth is 6-37 cm and the range of calibrated discharge is 1.4-169 l/sec. This will suit most conditions in a typical small-scale irrigation canal.

Cut-throat flume
The cut-throat flume has a converging inlet section, throat and diverging outlet section. The flume has a flat bottom and vertical walls (Figure 80). It is preferable
to have the cut-throat flume operating under free flow conditions. This facilitates
measurements and ensures a high degree of accuracy.

Free flow conditions through the cutthroat flume are described by the following

**Equation 52**

\[ Q = C \times (h_a)^9 \]

**Equation 53**

\[ C = K \times W^{1.025} \]

Where:
- \( Q \) = Discharge (m\(^3\)/sec)
- \( C \) = Free flow coefficient
- \( h_a \) = Upstream water depth (m)
- \( K \) = Flume length coefficient
- \( W \) = Throat width (m)
Example 34

A cut-throat flume is to be installed with a length $L = 1.22\, m$ and throat width $W = 0.36\, m$. The maximum discharge through the structure is $0.20\, m^3/sec$. How should it be installed in order to operate under free flow conditions?

From Figure 81, it follows that for a flume length $L = 1.22\, m$:

$S_t = 68.2\%$

$K = 3.1$

$n = 1.75$

Using Equations 53 and 52 respectively:

$C = 3.1 \times 0.36^{1.025} = 1.068$

$Q = 1.088 \times h_a^{1.76} = 0.200 \Rightarrow h_b = 0.38\, m$

$S_t = \frac{h_b}{h_a} = 0.682 \Rightarrow h_b = 0.682 \times 0.38 = 0.26\, m$

Therefore the floor of the flume should be placed not lower than $0.26\, m$ below the normal water depth, in order to let pass the maximum discharge of $0.20\, m^3/sec$ under free flow conditions.

Figure 80

Cut-throat flume (Source: FAO, 1975b)

[Diagram of a cut-throat flume showing maximum water surface, actual water surface profile, and various sections including converging and diverging outlet sections.]
Orifices

Orifices, such as gates and short pipes, are also used as water measuring devices (Figure 82). However, they do not offer any advantage over the use of weirs or flumes. Furthermore, their calibrations are not as accurate nor as stable as other types of measuring devices. For weirs the discharge is proportional to the head above the crest raised to the power 3/2 (Equations 44, 45, 46, 48). Therefore, they are sensitive to the fluctuations in the upstream water level. For orifices, including gates and short pipes, the discharge is proportional to the head of water above the crest raised to the power 1/2, as shown by Equation 34 (see Section 6.1.3). Therefore, they are less sensitive to small fluctuations of the upstream water level. Under submerged conditions both the upstream and downstream sides of the structure need water level recordings. For free flow conditions, the discharge is a function of the upstream water depth alone.
The general discharge equation for a free flow orifice is (Equation 34):

\[ Q = C \times A \times (2gh_1)^{1/2} \]

Where:

\( Q \) = discharge

\( C \) = discharge coefficient

\( A \) = area

\( 2gh_1 \) = energy head
Q = Design discharge through orifice (m3/sec)  C = Design coefficient (approximately 0.60)
A = Cross-sectional area of the orifice (m2)  g = Gravitational force (9.81 m/sec2)
h1 = Water depth upstream of orifice over reference level (m) (Figure 83)

Partially-opened sluice gates could be used for discharge measurements, in which case they will be acting like submerged orifices (Figure 84). For partially-opened sluice gates and submerged orifices the discharge equation reads:

**Equation 54**

\[ Q = C \times A \left(2g(h_1 - h_2)\right)^{1/2} \]

Where:
Q = Design discharge through orifice (m3/sec),  C = Discharge coefficient, which is 0.63 for sluice gates and submerged orifices and 0.85 for short pipes,  A = Cross-sectional area of the orifice (m2)  g = Gravitational force (9.81 m/sec2),  h1 = Water depth upstream of orifice over reference level (m),  h2 = Water depth downstream of the structure (m)

**Figure 84**
Sluice gate under submerged conditions

Current meters

Current meters are used to measure the velocity in a canal, from where the discharge can be calculated using the Continuity Equation 12 (see Section 5.1). Most current meters have a propeller axis in the direction of the current. The flowing water sets the propeller turning. On a meter, forming part of the equipment, the number of revolutions per time unit can be read and, by means of
a calibrated graph or table, the velocity can be determined. A well known type of current meter is the Ott instrument C31 for velocities up to 10 m (Figure 85). Propeller meters are reliable and accurate, but rather expensive. In measuring the velocities, the number of points per vertical and the number of verticals per cross-section should be determined. For this purpose, the quantity of work and the time required should be weighed against the degree of accuracy (Euroconsult, 1989). For example, measurements can be taken at 10 cm horizontal distance over the cross-section and at 0.2h and 0.8h depth at each 10 cm (h is the water depth). The velocity is the average of the velocity at 0.2h and 0.8h depth. If the water depth is less than 0.5-0.6 m, one reading can be done at 0.6h. Then, for each vertical the flow per unit width can be calculated according to \( q = v_{av} \times h \) (Figure 86a). These \( q_s \) are distributed over the total width (Figure 86b) and the area between the q-line and the water surface gives the total discharge. It is also possible to establish the discharge per section and to consider the sum of the discharges in the sections as the total discharge.

---

**Example 36**

A sluice gate is installed in a canal with a design water depth of 0.30 m. The canal discharges 0.0783 m³/sec. The maximum allowable rise in water level upstream of the sluice gate is 0.25 m. The width of the gate opening is 0.40 m. What should be the height \( d \) of the opening?

\( h_2 \) being 0.30 m and the allowable rise in water level upstream of the gate being 0.25 means that:

\[
h_1 = h_2 + 0.25 = 0.55 \text{ m}. \]

Substituting the above data in Equation 54 gives:

\[
0.0783 = 0.63 \times (0.40 \times d) \times (2 \times 0.81 \times (0.55 - 0.30))^{1/2} \Rightarrow d = 0.14 \text{ m}. \]
Discharge measurement in pipelines

Several types of devices can be used to measure the discharge in pipelines. This section will discuss differential pressure and rotating mechanical meters, as they are the ones commonly used.

Differential pressure flow meters
Differential pressure flow meters create a pressure difference that is proportional to the square of the discharge. The pressure difference is created by causing flow to pass through a contraction. Manometers, bourdon gauges, or pressure transducers are normally utilized to measure the pressure difference. One good example of a differential pressure flow meter is the Venturi tube (Figure 87).

Venturi tube

The pressure drop between the inlet and throat is created as water passes through the throat. In the section downstream of the throat, the gradual increase in cross-sectional area causes the velocity to decrease and the pressure to increase. The pressure drop between the Venturi inlet and the throat is related to the discharge, as follows

\[
Q = \frac{C_d^2(P_1 - P_2)^{1/2}}{[1 - (d/D)^2]^{1/2}}
\]

Equation 55
Where:
Q = Discharge (l/min),  C = Flow coefficient,  D = Diameter of upstream section (cm), d = Diameter of contraction (cm)
P1 = Pressure in upstream section (kPa),  P2 = Pressure in contraction (kPa), K = Unit constant (K is 6.66 for Q in l/min, d and D in cm, and P1 and P2 in kPa)

The flow coefficient C for a Venturi metre is 0.97.

Rotating mechanical flow meters

There are many types of rotating mechanical flow meters used in pipelines. These flow meters normally have a rotor that revolves at a speed roughly proportional to the discharge and a device for recording and displaying the discharge and total volume. The rotor may be a propeller or axial flow turbine, or a vane-wheel with the flow impinging tangentially at one or more points. Calibration tests are usually needed to accurately relate rotor revolutions to the flow. The lowest discharge that can be accurately measured by a rotating mechanical flow meter depends on the amount of bearing friction that can be tolerated while the occurrence of cavitation often establishes the largest flow rate that can be measured (see Module 5). Head loss through most rotating mechanical discharge meters is moderate.
Chapter 8

DESIGN OF WEIRS AND BARRAGES. THEORY OF SEEPAGE

Weir or Anicut
It is a solid wall of masonry or concrete constructed along the width of the river from one bank to the other to raise the water level on the upstream side so that proper supplies of water may be assured to the canals. Usually water flows over the crest of the weir but sometimes shutters 1m or more high are provided on the crest of the weir. During the dry period the shutters are raised to raise the water level and during rainy season, the shutters are dropped to allow flood waters to pass without causing damage on the upstream side.

Barrage
The function of a barrage is the same as that of a weir. In a barrage, a low height weir is constructed first and then according to design piers are constructed at regular intervals along the width of the river. The gaps between the piers are closed by means of gates, which can be lifted or lowered down mechanically or manually or both. Barrages are preferred now-a-days as they offer greater flexibility in operation and better control. In this case, water level can be raised many times more than weirs and at will, more discharge can be passed by lifting the gates, but in case of weirs, the crest is fixed and only a fixed amount of discharge can be passed.

When the foundation of a hydraulic structure is pervious, seepage will take place as long as differential head exists across the structure. Seepage of water below the structure may result in failure due to piping or pressure uplift.

BLIGH’S CREEP THEORY
Bligh stated that: creep is caused by differential head across a structure and the loss of head is proportional to the length of creep.

Length of creep: is the total distance travelled by seeping (creeping) water.
If \( H = \) total head across structure,
And \( L = \) total length of travel of seepage water (creep length),
then the head lost per unit creep length \( = \frac{H}{L} \) referred to as hydraulic gradient.

Referring to the diagram below, assume that there are three cut-offs down beneath the foundation of a weir with depths \( y_1, y_2, \) and \( y_3 \) at A B and C. Then the total seepage length (creep length)
\[ L_c = L_1 + L_2 + 2(y_1 + y_2 + y_3) \]
If \( H \) is the total head difference downstream and upstream, then the hydraulic gradient is given by:
Hydraulic gradient \( = \frac{H}{L_c} = \) head lost per unit length travelled by seepage water.
Head loss across cut-off A \( = \left( \frac{H}{L_c} \right) \times 2y_1 \)
Based on the above, Bligh suggested the following to prevent failure by seepage.

1. Safety against Piping: If sufficient creep length is provided by providing vertical cut-off, the head causing creep can be destroyed before the water exits on the downstream side and piping (undermining) would be prevented. Accordingly Bligh gave the following criteria.

\[ L_c = C \cdot H \]

where \( H \) is the head across structure; and \( C \) = a constant depending on soil type and given by Bligh below.

<table>
<thead>
<tr>
<th>No.</th>
<th>Type of soil</th>
<th>Value of C</th>
<th>Limiting value of safe hydraulic gradient</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Fine micaceous sand, alluvial soil, etc</td>
<td>15</td>
<td>1/15</td>
</tr>
<tr>
<td>2</td>
<td>Coarse grained sand</td>
<td>12</td>
<td>1/12</td>
</tr>
<tr>
<td>3</td>
<td>Sand with boulders and gravel</td>
<td>5 – 9</td>
<td>1/9 - 1/5</td>
</tr>
<tr>
<td>4</td>
<td>Light sand and mud (silt?)</td>
<td>8</td>
<td>1/8</td>
</tr>
</tbody>
</table>

**Safety against Uplift**

The ordinate at any point on the bottom of the floor to the hydraulic gradient represent the uplift at that point. This uplift must be balanced by the weight of the floor if uplift is to be prevented. Therefore the floor must be of sufficient thickness to provide the necessary resisting force to this uplift force.

If we let \( t \) be the required floor thickness to balance uplift force, then
Weight of floor = γ . SG. t
Uplift force        = γ. h
where γ = weight density of water, SG specific gravity of material of floor.
In the limiting condition,
\( \gamma \cdot SG \cdot t = \gamma \cdot h \)

or
\( t = h/SG. \)

If \( h' \) is the ordinate to the hydraulic gradient line considered from the top of the
floor, then: \( h = h' + t \)
And \( t = h/SG = (h' + t)/SG \)
Or \( t = h'/(SG-1) \)
This is the limiting thickness of the floor to withstand uplift.

**Limitations of Bligh’s Theory**

1. Bligh made no distinction between horizontal and vertical creep
2. Bligh did not explain exit gradient
3. Bligh did not make any distinction between outer and inner faces of sheet piles
4. Bligh did not explain any effect of sheet piles length and their distances on exit
   gradient.
5. Head loss is not directly proportional to creep length; Also uplift pressure
distribution is not linear.
6. Bligh did not specify the necessity of providing downstream end sheet pile.

**A. N. Khosla’s Theory**
After studying a lot of dam failure constructed based on Bligh’s theory, Khosla
came out with the following;
Seeping water below a hydraulic structure does not follow the bottom profile of
the impervious floor as stated by Bligh but each particle traces its path along a
series of streamlines.
For steady flow, seepage in a homogeneous soil obeys the Laplacian equation:

\[
\frac{d^2Q}{dx^2} + \frac{d^2Q}{dz^2} = 0
\]

where \( Q = K \cdot h \) - flow potential
\( h \) = residual head at any point within the soil and \( K \) coefficient of permeability as defined by Darcy.

The above equation represents two sets of curves STREAMLINES and EQUIPOTENTIAL (VELOCITY POTENTIAL) LINES which intercept each other orthogonally.

**Streamline**

A streamline is the path traced out by particles of water seeping through the subsoil. Every particle traces out its own path which represents the streamline for that particle.

The first streamline, immediately below the hydraulic structure follows the bottom profile of the structure and will be almost the same as Bligh’s creep path. The others streamlines will be curves as they proceed through the pervious soil. If an impervious boundary intervenes, then the last streamline near the boundary follows the boundary.

**Equipotential Lines.**

Let us consider that there is no water on the downstream side. If we assume the downstream bed as the datum, then the on the upstream side is \( h \). All the floor at the upstream side AB and every particle of water entering the subsoil on the upstream side will be having a head \( h \). This head will be gradually utilize as the particle traces out the streamline and when it emerges at the downstream end, the entire head \( h \) will have been utilized so that the head at the exit end will be
zero. At any intermediate position along the streamline, the particle will be having a head \( h \) which is the residual head; which will be dissipated by the particle while traveling the remaining path on the streamline. Thus every streamline moving from a head of \( h \) to a head of zero will pass through a residual head of \( h \). If all points on all streamline with a residual head of \( h \) are joined, we shall obtain a curve called an **equipotential line**. Thus equipotential line is a line joining points of equal residual head.

**Flow Nets**
The streamlines and equipotential lines always intercept at right angles. In a flow field, the combination of the two groups of lines form a network in the flow field called the flow net. The space enclosed between any two adjacent streamlines and equipotential lines is known as a field. Though it is possible to draw an infinite streamlines and equipotential lines, for the sake of practical convenience and easy interpretation, only a limited number is usually drawn in such a way that every field becomes an elementary square.

By drawing the flow net, all characteristics such as flow rate, velocity etc can be obtained.

**Exit and Critical Gradient**

![Diagram of flow net with exit and critical gradient](image)

Every particle of water while seeping through the sub-soil, at any position will exert a force \( f \), which will be tangential to the streamline at any point. As the streamlines bend upward, the tangential force \( f \) will be having a vertical component \( f_1 \). Also at that point, there will be a downward force \( W \) due to the submerged weight of the soil particle. Thus at that point there will be two forces on the particle; one upward vertical component of \( f \), and the other, the submerged weight. It is evident that if the soil particle is not to be dislodged, then the submerged weight must be greater than the upward vertical component of \( f \).
The upward vertical component force at any point is proportional to the water pressure gradient \( \frac{dp}{dx} \).

Hence for stability of the soil and for the prevention of erosion and piping, the seeping water when it emerges at the downstream side, at the exit position, the force \( f_1 \) should be less than the submerged weight \( W \). In other words the exit gradient at the downstream end must be safe.

If at the exit point at the downstream side, the exit gradient is such that the force \( f_1 \) is just equal to the submerged weight of the soil particle, then that gradient is called **critical gradient**. Safe exit gradients = 0.2 to 0.25 of the critical exit gradient.

Values of safe exit gradient may be taken as:
- 0.14 to 0.17 for fine sand
- 0.17 to 0.20 for coarse sand
- 0.20 to 0.25 for shingle

**Method of Independent Variable of Khosla.**

For the determination of seepage below the foundation of hydraulic structure developed the method of independent variable.

In this method, the actual profile of a weir which is complex, is divided into a number simple profiles, each of which cab be solved mathematically without much difficulty.

The most useful profile considered are:

i) A straight horizontal floor of negligible thickness provided with a sheet pile at the upstream end or a sheet pile at the downstream end.

![Diagram of a typical weir profile](image)

ii) A straight horizontal floor depressed below the bed, but without any vertical cut-off.

![Diagram of a typical weir profile](image)
ii) A straight horizontal floor of negligible thickness with a sheet pile at some intermediate point

The mathematical solution of the flow-nets of the above profiles have been given in the form of curves. From the curves, percentage pressures at various key points E, C, E₁, C₁ etc) be determined. The important points to note are:

i) Junctions of pile with the floor on either side{E, C (bottom), E₁, C₁ (top) }

ii) Bottom point of the pile (D), and

iii) Junction of the bottom corners (D, D') in case of depressed floor

The percentage pressures at the key points of a simple forms will become valid for any complex profile, provided the following corrections are effected:

i) correction for mutual interference of piles

ii) correction for the thickness of floor

iii) correction for slope of the floor.
Correction for Mutual Interference of Piles

Let \( b_1 \) = distance between the two piles 1 and 2, and
\[ D = \text{the depth of the pile line (2), the influence of which on the neighbouring pile (1)} \]
\[ \text{of depth } d \text{ must be determined} \]
\[ b = \text{total length of the impervious floor} \]
\[ c = \text{correction due to interference.} \]

The correction is applied as a percentage of the head
\[ C = 19 \frac{D}{b_x} \left( \frac{d + D}{b} \right) \]  

This correction is positive when the point is considered to be at the rear of the interfering pile and negative for points considered in the forward or flow direction with the interfering pile.

For example, correction for pressure at \( C_1 \) for pile line (1) by the interference of pile line (2) is positive as pile line (1) is to the rear of the interfering pile line (2). Similarly, correction for pressure at \( E_2 \) for pile line (2) due to the interference of pile line (1) is negative, because \( E_2 \) is in the forward or flow direction of interfering pile line (1).

The interference effect will not be present on the intermediate pile if:

1. the outer pile is equal to or longer than the intermediate pile and
ii) if the distance between the intermediate pile and outer interfering pile is less than twice the length of the outer pile

**Correction for Floor Thickness**

Standard profiles assuming the floors as having negligible thickness. Hence the values of the percentage pressures computed from the curves corresponds to the top levels (E1*, C1*) of the floor. However, the junction points of the floor and pile are at the bottom of the floor (E1, C1)

The pressures at the actual points E1 and C1 are interpolated by assuming a straight line variation in pressures from the points E1* to D1 and from D1 to C1*
The corrected pressures at E₁ should be less than the computed pressure t E₁*. Therefore the correction for the pressure at E₁ will be negative. And so also is for pressure at C₁.

**Correction for Slope of Floor.**

A correction for a sloping impervious floor is positive for the down slope in the flow direction and negative for the up slope in the direction of flow.

<table>
<thead>
<tr>
<th>No.</th>
<th>Slope = Ver:Horiz</th>
<th>Correction as % of pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1:1</td>
<td>11.2</td>
</tr>
<tr>
<td>2</td>
<td>1:2</td>
<td>6.5</td>
</tr>
<tr>
<td>3</td>
<td>1:3</td>
<td>4.5</td>
</tr>
<tr>
<td>4</td>
<td>1:4</td>
<td>3.3</td>
</tr>
<tr>
<td>5</td>
<td>1:5</td>
<td>2.8</td>
</tr>
<tr>
<td>6</td>
<td>1:6</td>
<td>2.5</td>
</tr>
<tr>
<td>7</td>
<td>1:7</td>
<td>2.3</td>
</tr>
<tr>
<td>8</td>
<td>1:8</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The correction factor must be multiplied by the horizontal length of the slope and divided by the distance between the two poles between which the sloping floor exists.

In the diagram above, correction for slope can be applied only to point E₂. As the point E₂ is terminating at the descending slope in the direction of flow, the correction will be positive. The value of correction will be:

$$C.F. \times \frac{b_s}{b_1}$$

Where C.F. = correction factor

- $b_s$ = horizontal length of sloping floor
- $b_1$ = horizontal distance between the pile lines

**Exit Gradient**

```
\[ \text{----------} \quad \text{-----} \quad \text{H} \quad \text{b} \quad \text{d} \quad \text{----------} \]
```
For the standard form consisting of a floor of a length $b$, and a vertical cut-off of depth $d$, the exit gradient at its downstream end is given by:

$$
exit \text{ gradient } G_E = (H/d) \times \frac{1}{\pi \sqrt{\lambda}}
$$

By referring to plate (17.3), for any value of $\alpha = b/d$, the corresponding value of $\frac{1}{\pi \sqrt{\lambda}}$ can be read off. When $H$ and $d$ are given, $G_E$ should be easily calculated. The value of exit gradient $G_E$ should be within safe limits as given below.

<table>
<thead>
<tr>
<th>No.</th>
<th>Type of soil</th>
<th>Safe exit Gradient</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Fine sand</td>
<td>1/7 to 1/6</td>
</tr>
<tr>
<td>2</td>
<td>Coarse sand</td>
<td>1/6 to 1/5</td>
</tr>
<tr>
<td>3</td>
<td>Shingle</td>
<td>1/5 to 1/4</td>
</tr>
</tbody>
</table>

Use of Khosla’s Pressure Curves.

First consider plate 17.1. Values of $1/\alpha = d/b$ are plotted as abscissa and percentage pressure $\varphi = P/H \times 100$ are plotted as ordinates. There are three curves, one for $Q_D$ and one for $Q_E$ for sheet piles fixed at the ends of the floor and the last one for $\varphi_D$ for depressed floor. To find the percentage pressure at the points $C_1$ and $D_1$ of the upstream pile, calculate

$$A\alpha = b/d \quad \text{and then } 1/\alpha = d/b$$

For this value of $1/\alpha$ determine the value of $\varphi_D$ from the curve. Then subtract this value of $\varphi_D$ from 100 to get $\varphi_{D1}$

$$\varphi_{D1} = 100 - \varphi_D$$

Similarly, determine the value of $\varphi_E$ for the value of $1/\alpha$. Subtract this $\varphi_E$ from 100 to get $\varphi_{C1}$

$$\varphi_{C1} = 100 - \varphi_E$$

Example: Let $b =$ total length of floor $= 60.0 \text{ m}$ and $d =$ depth of u/s pile $= 6.0$

Then $1/\alpha = d/b = 6/60 = 0.1$

For this value of $1/\alpha$, we read the values of $\varphi_D = 20$ and $\varphi_E = 28$. Then

$$\varphi_{D1} = 100 - 20 = 80$$
$$\varphi_{C1} = 100 - 28 = 72$$

These percentages pressures must be corrected for mutual interference.

Next, let us consider plate 17.2. This gives pressure curves for $\varphi_C$, $\varphi_D$ for sheet pile not at end condition. The ratios $b_1/b$ are plotted as abscissa and $\varphi_C = P_e/H x$
100 plotted as ordinates on the left portion of graph and \( \varphi_D = \frac{P_v}{H} \times 100 \) plotted on the right side of graph for different values of \( \alpha \).

To find \( \varphi_E \) for any value of \( \alpha \) and base ratio \( b_1/b \), read \( \varphi_C \) for base ratio \((1-b_1/b)\) for that value and subtract this value from 100.

Thus for finding \( \varphi_E \) for base ratio \( b_1/b = 0.3 \) and \( \alpha = 3.0 \)

\[
(1 - \frac{b_1}{b}) = 1 - 0.3 = 0.7 \quad \text{for} \quad \alpha = 3.0
\]

\[Q_c = 20\]

\[\Phi_E = 100 - Q_c = 100 - 20 = 80\%\]

\( \Phi_D \) can be calculated for values of \( b_1/b \) less than 0.5.

Similarly finding \( \varphi_D \) for \( b_1/b = 0.13 \) and \( \alpha = 2.0 \)

\[
(1 - \frac{b_1}{b}) = 0.87
\]

Corresponding to 0.87 and \( \alpha = 2.0 \) value of \( \varphi_D = 40 \)

\[\Phi_D = 100 - 40 = 60\%\]

Example. Determine the percentage pressures for various key points shown in diagram below. Calculate the exit gradient and plot the hydraulic gradient line for the pond level on the u/s side with no water on downstream side.

\[\begin{align*}
\text{Solution:} \\
1. \text{Upstream pile No. 1} \\
\text{Total length of floor} \quad b = 60 \text{ m} \\
\text{Depth of u/s pile below the top of floor} = 105 - 99 = 6.0 \text{ m} \\
\alpha = \frac{b}{d} = \frac{60}{6} = 10.0 \\
\frac{1}{\alpha} = 0.1
\end{align*}\]
From plate 17.1, \( \varphi_{C1} = 100 - \varphi_E = 100 - 28 = 72\% \)
\( \Phi_{D1} = 100-20 = 80\% \)

For these values, three corrections must be applied.

i) **Correction for mutual interference of piles, for pressure at \( C_1 \)**

\( \varphi_{C1} \) is affected by the presence of intermediate pile No. (2)

Let \( D = \text{depth of pile No. (2)} = 104.00 - 99.00 = 5.0 \text{ m} \)
\( d = \text{depth of pile No. (1)} = 104.00 - 99.00 = 5.0 \text{ m} \)
\( b_1 = \text{distance between the two piles} = 14.0 \text{ m} \)
\( b = \text{total length of floor} = 60.0 \text{ m} \)

Then the correction is:

\[
\frac{19}{b_1} \left( \frac{d + D}{b} \right) = \frac{19}{b_1} \left( \frac{5 + 5}{60} \right) = \frac{19}{b_1} \times 0.1 = 1.9\% (+ve)
\]

Point \( C_1 \) is to the rear of the next pile No. (2). Hence correction is positive.

ii) **Correction at \( C_1 \) due to thickness of floor.**

\( \varphi_{C1} \) calculated from curves corresponds to point \( C_1 \) at the top of the floor. But we want the pressure at \( C_1 \) at the bottom of the floor. Pressure at \( C_1 \) will be more than that at \( C_1 \), therefore the correction will be positive.

Pressure at \( C_1 \) = \( \varphi_{C1} = 72\% \)
Pressure at \( D_1 \) = \( \varphi_{D1} = 80\% \)
Distance from \( C_1 \) to \( D_1 \) = \( 1050.0 - 99.0 = 6.0 \text{ m} \)

Hence pressure correction at \( C_1 \) = \( \frac{(80 - 72)}{(105.0 - 99.0)} \times (105.0 - 104.0) \)

\( \frac{8/6}{1} = 1.67\% (+ve) \)

iii) **Correction due to slope at \( C_1 \)**

As \( C_1 \) is neither the starting point nor terminating point of a slope, slope correction is zero.

Hence, corrected pressure at \( C_1 \)

\( \Phi\varphi_{C1} = 72\% + 1.9\% + 1.67\% = 75.57\% \)
2. Intermediate Pile Line No. 2

\[ d = 105 - 99 = 6 \, \text{m} \]
\[ b = 60 \, \text{m} \]
\[ \alpha = b/d = 60 / 6 = 10 \]
\[ b_1 = 0.5 + 14 = 14.5 \]
\[ (1 - b_1/b) = 1 - 0.24 = 0.76 \]

For \( b_1/b = 0.76 \), and \( \alpha = 10 \), \( \varphi^C = 27\% \) and \( 100 - 27 = 73\% = \varphi^E_2 \)

For base ratio 0.24 and \( \alpha = 10 \),
\[ \Phi^C_2 = 60\% \]

For base ratio 0.76 and \( \alpha = 10 \),
\[ \Phi^D = 34\% \, \text{and} \, \Phi^D_2 = 100 - 34 = 66\% \]

Corrections for \( \varphi^E_2 \)

i) Correction for sheet pile lines at \( E_2 \).

Pile No. (1) affects the pressure at \( E_2 \) (\( \varphi^E_2 \)). As \( E_2 \) is in a forward direction with respect to pile (1), the correction will be negative.

\[ D = \text{depth of pile No.(1), the effect of which on } E_2 \text{ is considered.} = 104 - 99.00 = 5.0 \, \text{m} \]
\[ d = \text{depth of pile No. (2) which is affected} = 104 - 99.0 = 5.0 \, \text{m} \]
\[ b_1 = \text{distance between the two pile lines} = 14 \, \text{m} \]
\[ b = \text{total floor length} = 60.0 \, \text{m} \]

\[ \text{Correction} = 19 \sqrt{\frac{D}{b_1}} \left( \frac{d + D}{b} \right) \]
\[ = 19 \sqrt{\frac{5}{14} \left( \frac{5 + 5}{60} \right)} = 1.9\%(-ve) \]

ii) Correction at \( E_2 \) due to floor thickness

Correction = \( \frac{\text{observed value of } \varphi^E_2 - \text{observed } \varphi^D_2 \times \text{Thickness of floor}}{\text{Distance } E_2D_2} \)

\[ = \frac{73 - 66}{105.0 - 99.0} \times 1.0 = \frac{7}{6} \times 1.0 = 1.17 \% \, (-ve) \]

Pressure is observed at \( E_2 \) and the direction of flow is from \( E_2 \) to \( E_2 \). Hence, pressure at \( E_2 \) will be less than that at \( E_2 \). Correction is therefore negative.

iii) Correction at \( E_2 \) due to slope is zero as \( E_2 \) is neither the beginning nor the end of a slope.

Hence corrected pressure at \( E_2 = (73\% - 1.9\% - 1.17\%) = 69.93\% \)

Correction for \( \varphi^C_2 \)
i) Correction at $C_2$ due to pile interference. Pressure at $C_2$ is affected by pile line (3) and as $C_2$ is to the rear of pile line (3), correction will be positive. The amount of this correction is given by:

\[
\text{Correction} = \frac{D}{b_1} \left( \frac{D + d}{b} \right)
\]

Where $D =$ depth of pile No. 3, the effect of which is considered below the level at which the interference is desired = 104.0 – 99.0 = 10 March 2007

d = depth of pile No. (2), which is affected = 104.0 – 99.0 = 5.0m

$b_1 =$ distance between pile (2) and pile (3) = 45m

$b =$ total floor length = 60.0m

\[
\text{Correction} = \frac{19}{45} \left( \frac{10 + 5}{60} \right) = 2.28\% (+ve)
\]

ii) Correction at $C_2$ due to floor thickness. From the diagram, it can be observed that the pressure at $C_2$ is less than that at $C_2$ due to the direction of seepage flow. The amount of correction is the same as that for $E_2$ and equal to 1.17% +ve

iii) Correction at $C_2$ due to slope. $C_2$ is at the commencement of an up slope of 3:1 in the direction of flow and the correction will be negative. Correction factor 3:1 slope = 4.5 (see table of correction for slope above)

Horizontal slope length = 6m

Distance between the two pile lines (2) and (3) between which the sloping floor exists = 45.0 m

\[
\text{Correction} = \text{correction factor} \times \frac{\text{horizontal slope length}}{\text{distance between piles}} = 4.5 \times \frac{6}{45} = 0.6\% (-ve)
\]

Hence corrected pressure $\varphi_{C_2} = (60\% + 2.28\% + 1.17\% - 0.6\% = 62.85\%$

3. Downstream Pile Line.

$d =$ 102.00 – 94.00 = 8.00m

$b =$ 60m

$1/\alpha = d/b = 8/60 = 0.133$

Referring to curves of plate 17.1, we find

$\Phi_{D_3} = 22\%$

$\Phi_{E_3} = 32\%$
Note that for downstream pile at end, the values of $\phi_D$ and $\phi_E$ are taken directly from the curves for the required value of $1/\alpha$.

a) **Correction for $\phi_{E3}$**

i) Correction due to mutual interference of pile. The point $E_3$ gets affected by pile No.2. $E_3$ is in the forward direction of flow with reference to pile No. 2, correction is negative. Value of correction \[ \frac{D}{b_1} \left( \frac{D + d}{b} \right) \]

where $D =$ depth of pile No. 2 which influences point $E_3$ of pile No. 3, the depth being considered below the level at which interference is desired = 100.0 – 99.0 = 1.0m

$d =$ depth of pile No. 3 which is affected = 100.0 – 94.0 = 6.0m

$b_1 =$ distance between the two piles = 45.0m

$b =$ total floor length = 60 m

**therefore correction** \[ \frac{1.0}{45} \left( \frac{1 + 6}{60} \right) = 0.262\% (-ve) \]

iii) Correction due to floor thickness. From the direction of flow of seepage water, it is clear that pressure at $E_3$ at the bottom of the floor is less than that at $E_3'$, top of the floor which is the value got by Khosla’s curves. Hence, correction to be applied for $\phi_{E3}$ should be negative.

Value of this correction = \[ 32\% - 22\% \times 2.00 \]

= \[ (102.0 - 94.0)/8 = 2.5\% (-ve) \]

iv) Correction due to slope. Point $E_3$ is neither starting point nor the terminus of a slope. Hence, slope correction = 0

Hence, corrected pressure at $E_3 = \phi_{E3} = (32\% - 0.262\% - 2.5\%) = 29.238\%$

The corrected pressures at all the key points can be shown in the table below

<table>
<thead>
<tr>
<th>Upstream pile No. 1.</th>
<th>Intermediate pile No. 2.</th>
<th>Downstream pile No. 3.</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_{E1} = 100.0%$</td>
<td>$\phi_{E2} = 69.93%$</td>
<td>$\phi_{E3} = 29.24%$</td>
</tr>
<tr>
<td>$\phi_{D1} = 80.0%$</td>
<td>$\phi_{D2} = 66%$</td>
<td>$\phi_{D3} = 22.0%$</td>
</tr>
<tr>
<td>$\phi_{C1} = 75.57%$</td>
<td>$\phi_{C2} = 62.85%$</td>
<td>$\phi_{C3} = 0 %$</td>
</tr>
</tbody>
</table>

Exit Gradient

Pond level = 110.0

With no water on downstream, maximum head causing seepage $H = (110.0 - 102.0) = 8.0m$

Depth of downstream cut-off = $d = (102.0 - 94.0) = 8.0m$

Total floor length = $b = 60m$

$\alpha = b/d = 60/8 = 7.5$

Referring to curve of plate 17.3, for $\alpha = 7.5$, 

182
\[
\frac{1}{\pi \sqrt{\lambda}} = 0.16
\]

Hence, exit gradient \( G_E = (H/d) \times \frac{1}{\pi \sqrt{\lambda}} = (8/8) \times 0.16 = 0.16 \)

This exit gradient is safe even for fine sand.
Chapter 9

CROSS DRAINAGE WORKS

Whenever a canal intercepts a natural or artificial drainage in its passage, cross drainage works have to be provided.

**Types of Cross-Drainage Works**

Three main possibilities may occur when a canal has to cross a natural drainage:

1. When the canal bed level is higher than the bed level of the river, the cross-drainage work constructed is known as **aqueduct**
2. When the bed level of the natural drainage is higher than that of the canal, the cross-drainage work constructed is called a **super passage**
3. When the bed levels of the canal and the natural drainage are almost the same, and the waters of both the canal and natural drainage are mixed up while crossing, such cross-drainage works are known as **level crossing**.

If the bed level of the canal is not very much higher than that of the drainage, then the cross-drainage work is called **siphon aqueduct**

Similarly, if the difference between the bed level of the canal and natural drainage is not very much, the natural drainage being at a higher level, then the cross-drainage work is called **siphon super passage**.

The selection of any particular type of cross-drainage work depends on the following:

1. Bed level of canal and bed level of natural drainage;
2. Discharge of the canal and the drainage
3. The foundation conditions of the site
4. Availability of materials and labour
5. Availability of existing modes of communication.

**Aqueduct**
An aqueduct is a structure constructed at the position, where a canal has to cross a drainage at a sufficiently high level. It is very similar to a bridge but instead of a roadway or railway, canal water flows above a natural drainage. Irrigation water may be taken through a pipe over the drainage water if the section of the irrigation canal is very small. The pipe is supported in position at the ends by masonry walls and in between by supports.

There are three types of aqueducts:

Type I. In this type of aqueduct, the sides of the aqueduct are completely earthen embankment with full earthen slopes. The length of the culvert through which the discharge is passing should be adequate to accommodate the water way of the canal and also the bottom width of the side embankments. In this case the original canal section is retained and no fluming of canal section is done.

Type II.
Type II is very much like type one except that the outer section is reduced by supporting it by masonry or concrete wall.

Type III.
In this type of aqueduct, the canal banks are discontinued at the aqueduct portion and the canal water is transported in a masonry or concrete trough constructed above the drainage culvert. In this case the length of the culvert is reduced because fluming is done. This type becomes necessary where big canals have to cross big natural drainage with large catchment areas.

Selection of a suitable type. In order to select a suitable type, it is necessary to understand the following terms:
1. Culvert length: is the width of the aqueduct, measured along the drain. The culvert depends upon the shape and size of the canal section.
2. Length of aqueduct: is the length measured perpendicular to the drain. It is evidently equal to the width of the drain.
3. Bank connection: consist of masonry (concrete) wings, etc required to connect the regular section of the canal to its modified section over the aqueduct.

In the type I, since the canal is fully in earthen section, the culvert length is maximum. Hence the cost per unit length of aqueduct will be maximum. However, bank connections are not require. Hence, the selection of this type depends on the relative cost of bank connections and that of the aqueduct proper. In all cases the cost of bank connection is independent of the length of the aqueduct. Therefore, the type I is suitable when the length of the aqueduct is small and the cost of bank connections would be large in comparison to the saving from the reduction in the width of work if type III were adopted. On the contrary, in type III, the culvert length is minimum. Hence the cost per unit length of the aqueduct is minimum. However, the cost of bank connections will be additional. Therefore, type III is suitable where the length of culvert is large. Type two is suitable for intermediate conditions.

Design of Cross-Drainage Works
The following are some of the important features of design of cross-drainage works:

A. From hydraulic consideration, the following are important
   i) determination of the maximum flood discharge and the high flood level (H.F.L)
   ii). Determination of the stable waterway of drain
   iii). Contraction of the canal waterway.
   iv). Discharge and head losses through the cross drainage works
   v). Determination of uplift pressure on the roof of trough
   vi). Determination of uplift pressure on the floor.
   vii). Design of bank connections

B. Structural design.
   1) Design of cross-section of aqueduct trough.
   ii). Design of piers and abutments
   iii). Design of foundation.

   i) Maximum Flood Discharge of Drain: This is determined by any one of the known methods depending on size of catchment and other available data
   ii) Determination of the Waterway of drain: This can be determined by using Lacey’s regime perimeter equation

   \[ P_w = 4.75 Q^{\frac{1}{4}} \]

   where \( P_w \) = wetted perimeter of the drain (m)
   \( Q \) = total discharge in drain (cumecs)

   In large drains, \( P_w \) can be taken as equal to width of river, and hence equal to the waterway required.. In small drains, a contraction of up to 20% of the waterway is
permissible i.e. the length between abutments may be increased by 20% to compensate for contraction of waterway due to the width of the piers.

iii) Velocity of flow through the barrel. This may be taken between 2.0 -3.0 m/s. Lower velocities may result in silting in the barrel and higher velocities will cause abrasions of barrel surface by rolling grit.

iv) Height of Opening: The depth of flow in the barrel can be calculated as the waterway discharge and velocity are fixed. Sufficient margin (free board) should be provided between the H>F>L and the bottom of canal bed.

\[
\text{Total height of opening} = (\text{Depth of flow in barrel} + \text{clearance (freeboard)})
\]

v). Number of Spans: The total width of the culvert or total length of aqueduct already been fixed. The number of spans for the culvert should be fixed considering structural strength and economy of design. More spans may be used in case of arched culverts.

vi). Contraction of canal waterway: Canal waterway in the aqueduct portion is reduced in type III by fluming. Fluming ratio is the ratio between the restricted width of canal in the aqueduct portion and the normal width of the canal. This is taken as \(\frac{1}{2}\). However, this reduction in width should not result in the velocity in the trough exceeding 3m/s or exceeding the critical value bringing the flow to supercritical flow, which could result in the formation of hydraulic jump in trough – not desirable.

vii). Length of Contraction: This is the horizontal distance in which the normal width of the canal is gradually reduced to the contracted width in the aqueduct portion. A convergence of 2 horizontal to 1 lateral could be assumed.

viii). Length of expansion: the expansion ratio is assumed as 3 horizontal to 1 lateral. This decides the length of the expansion downstream side of the aqueduct.

ix). Bank connections: these consist of two sets of wing walls for the canal called canal wings and two sets of wing walls for the drainage called the drainage wings.

**Canal wing walls.** These are to be provided on the upstream and downstream sides of the aqueduct. They protect and retain the earth in the canal banks. The wings should be constructed on sound foundation.

The Drainage wing are provided on the upstream and downstream sides of the barrel. They protect and retain the natural slopes of the drain and increase the seepage path and control the exit gradient. The drainage wings should be taken deep into the foundation, well below the maximum scour depth. They must be taken back well into the top of the guide banks.

Example. Design a suitable cross drainage for the following particulars:

a) Drainage particulars:
   - Catchment area for the drain = 20.0 km\(^2\); value of “C” in Dicken’s formula = 20
   - Gauge reading at the site of intersection = 1.0m during maximum flood; Bed level of drainage = 100.00m

b) Channel Particulars
Full supply discharge = 20cumecs; full supply depth = 1.5m; Bed width = 10.0m; Side slopes = 1.5:1. The canal has to be provided with inspection paths of 3.0 m width on both the banks. The channel is unlined. Bed level of the channel = 103.00m. Good foundation is available at site. Missing data may be suitably assumed.

Solution:
The bed level of the canal  = 103.0m
The bed level of the drainage  = 100.0m
The High Flood Level in the drainage = 1.0m so that level of H.F.L = 101.0
The difference between canal bed level and H.F.L = 103.0 – 101.0 = 2.0m
Hence an aqueduct would have to be designed.

**Determination of flood discharge: by Dicken’s formula**

\[ Q = C \cdot M^{3/4} \]

Where \( C \) = is a constant = 20; \( M \) = catchment area = 20.0km\(^2\)

---

**TYPE I AQUEDUCT**

[Diagram of Type I Aqueduct]
Q = 20 x 20^{3/4} = 189.2 \text{cumecs} \approx 190 \text{ cumecs}

Waterway for drainage:
By Lacey’s equation, the wetted perimeter $P_w = 4.75 \frac{Q}{2}$
\[ P_w = 4.75 \times 190^{1/2} = 65.47 \text{m} \]

iv) **Smaller spans can be adopted, as the foundation is good**
Assuming a trapezoidal drain of side slopes of 1.5 : 1 (1.5 horizontal to 1.0 vertical) the bed width of drain $b = 65.47 - 2 \times 1.5 \times 1 = 62.47 \text{m}$

Provide 12 spans of 5.5m each, to give a total lineal waterway of $12 \times 5.5 = 66.0 \text{m}$

iv) **Thickness of pier at the springing level of the arch:**
This can be calculated by $t = 0.552 \times \frac{s^{1/2}}{2} = 0.552 \times (5.5)^{1/2} = 1.294 = 1.3 \text{m}$
Where $s =$ span of arc (m); $t =$ thickness (m)
A batter of 1 in 15 is given to the pier

v) **Total height of the pier**
Height of pier above the drainage bed = 1.0
The springing level of the arch will be at H.F.L.
To determine the depth of pier below the bed level of the drainage, the scour depth is worked out.
Scour depth as per Lacey’s formula
\[ R_1 = 1.35 \left( \frac{q^2}{f} \right)^{1/3} \]
Where $R_1 =$ scour depth (m), $q =$ discharge per metre width of the drain (m$^3$/s/m);
$f =$ Lacey’s silt factor, taken as 1.6
\[ R_1 = 1.35 \left\{ \frac{(190/66)^2 \times 1/1.6}{1.6} \right\}^{1/3} = 2.4 \text{ m} \]
The pier must be taken 1.5 times the scour depth below the bed of drain, which is \( 1.5 \times 2.4 = 3.6 \text{ m} \).
therefore total height of pier \( h_p = 3.6 + 1.0 = 4.6 \text{ m} \)

vi). **Total length of the aqueduct between the abutments** \( L_{AB} \).
The pier pose the obstruction to the flow of water and reduce the waterway. Hence, clear waterway must be increased. This is increased by 20% or the total thickness of all the piers, whichever is more.
   a) 20% of waterway = 20.100 \times 66.5 = 13.3 \text{ m} 
   b) Total thickness of piers = 11 \times 1.3 = 14.3 \text{ m} (12 spans, so there are 11 piers)

Hence we take 14.3 m for compensating for reduction of waterway by piers.
Therefore, total length of aqueduct between the abutments \( L_{AB} = 66 + 14.3 = 80.3 \text{ m} \)

vii) **Rise of arch**:
Rise of arch \( r = \text{span} / 5 = 5.5 / 5 = 1.1 \text{ m} \)

viii) **Radius of arch**
Considering the triangle AOB, \( AO = (R-r); \ AB = s/2 \) where \( s \) is the span and \( OB = R \)
Therefore using the Phytagoras theorem
\[ AO^2 + AB^2 = OB^2 \]
\[ (R-r)^2 + (s/2)^2 = R^2 \]
Knowing \( r = 1.1 \) and solving the equation, \( R = 4.0 \text{ m} \)

ix) **Thickness of the arch at the springing level**. By Trantwine’s formula;
\[ t_a = 0.22R + 0.11r + 0.5 = 0.22 \times 4.0 + 0.11 \times 1.1 + 0.5 = 1.5 \text{ m} \]
The water face of the abutment may be kept vertical while the back face (earth face) may be given a slope or batter of 1:5 (1 horizontal to 5 vertical).

The canal over the drain is flumed with a fluming ratio of 1:2
Hence, bed width of the canal over the drain  $b_{o/d} = 10 \times \frac{1}{2} = 5.0$ m
The section over the drain will be rectangular and is to be built in R.C.C. The height of side walls is kept 0.5 m higher than the full supply discharge F.S.D

x) Transitions
Upstream transition is done in 2:1 splay in contraction
Therefore length of contraction transition  $= (10 – 5)/2 \times 2 = 5$ m
Upstream transition is done in 3:1 splay expansion
Length of expansion  $= (10 – 5)/2 \times 3 = 7.5$ m
xi) An inspection path of 3.0 m width is provided on both sides of the aqueduct.

FLUMING OF THE CANAL
The reduction in waterway of the canal at the aqueduct is known as fluming of the canal. Fluming reduces the barrel length or the width of aqueduct, and thereby makes it economical. Fluming is only done for aqueduct type III.
The maximum amount of fluming is dictated by the velocity which can be kept in trough, because it must not exceed the critical value.
After fixing the canal section and the flumed canal section, the designing of the transition is done for making smooth entry and exit, and avoiding the formation of eddies. The slope of the upstream side should not be more than $30^\circ$ (ie 2:1 splay) and that of the downstream side must not be more than $22.5^\circ$ (i.e 1:3 splay). The normal canal section is trapezoidal, whereas flumed section is rectangular.

Design of Channel Transition
Any of the following methods may be applied for the design:
1. Chaturvedi’s method (when the water depth remains constant)
2. Hind’s method (when the water depth may or may not vary)
3. Mitra’s method (when water depth remains constant).

1. Chaturvedi’s Method: R. S. Chaturvedi (1963) proposed the following equation for the design of transition with constant water depth.

$$ x = \frac{L \cdot B_{o}^{3/2}}{B_{o}^{3/2} - B_{f}^{3/2}} \left[ 1 - \left( \frac{B_{f}}{B_{x}} \right)^{3/2} \right] \quad \text{--------------------- ( )} $$

where  $B_{x}$ = channel width at any section X-X at a distance x from the flumed section.
L = length of transition
$B_{o}$ = Bed width of the normal channel section
$B_{f}$ = bed width of the flumed section.
ii) Hind’s Method.
This method can be used when the water depth in the trough and the normal section of the canal vary.
Let $V_1$, $V_2$, $V_3$, and $V_4$ be the velocities of the canal at sections A, B, C, D. Let $y_1$, $y_2$, $y_3$, $y_4$ be the depths of canal at different sections.

**Step I.** Let the bed level and cross-section of the canal at section DD be known.
Water surface level (W.S.L.) at section DD = Bed level at section DD + $y_4$
Total energy line (T.E.L) at section DD = (water surface elevation at DD + $V_4^2/2g$)

**Step II.** The energy loss due to expansion of section between section CC and DD may be taken as equal to $0.3(V_3^2 - V_4^2)/2g$

**Step III.** Total energy line at section CC = (T.E.L. at section DD + $0.3(V_3^2 - V_4^2)/2g$
Water surface elevation at section CC, = (T.E.L. at section CC – $V_3^2/2g$
Bed level at section BB = (T.E.L at section BB – $V_2^2/2g$

**Step IV.** The channel section in the trough from section CC to BB remains constant. The only loss of head in this section is due to pipe friction, which can be computed by the Manning’s formula

$$Q = i/n \cdot A \cdot R^{2/3} \cdot (I)^{1/2}$$

Therefore T.E.L at section BB = (T.E.L at section CC + loss of head)
W.S.E at section BB = (T.E.L at section BB – $V_2^2/2g$
Bed level at section BB = (W.S.E. at section BB – $y_2$)
Step V. The loss of energy between section AA and BB due to contraction

\[ \text{T.E.L at section AA} = \left[ \text{T.E.L at section BB} + \frac{0.2(V_2^2 - V_1^2)}{2g} \right] \]

\[ \text{W.S.E at section AA} = \left[ \text{T.E.L at section AA} - \frac{V_1^2}{2g} \right] \]

Bed level at section AA = W.S.E. – y1

Step VI After calculating the T.E.L and W.S.E and bed levels of all sections, the total energy line may be drawn and the bed line also.

Step VII. The drop in the W.S.L between the two adjacent sections is due to:

i) drop in the energy line between the two sections’

ii) increase velocity head for contraction and decreased velocity head for expansion.

Step VIII) When the water surface profile has been plotted over the whole length, the velocity head can be determined by measuring the vertical distance between the T.E.L and the water surface line at any point.

The velocity head can be converted into equivalent velocity by \( V = (2gh)^{1/2} \)

Step IX. Discharge at any point,

\[ Q = A \cdot V \]

With the velocity known, and the flow also available, the dimensions of the section can easily be calculated.

If the section is rectangular, then \( A = b \times h \)

If the section is trapezoidal, \( A = (b + mh)h \), where \( m = \text{side slope} = \cot \theta; \theta = \text{angle of drain} \).

**SIPHON AQUEDUCT**

In siphon aqueduct, the difference between the bed level of the drainage and the bed level of the canal is not much so the bed level of the drainage is depressed at the site of the crossing so that there is sufficient clearance between H.F.L of the drainage and the bed of the canal. The drainage water passes below the canal through the depressed portion which makes it works like siphon.

**Design consideration for a siphon aqueduct**

Siphon aqueduct is a bit different from ordinary aqueducts. Therefore in addition to the design considerations of ordinary aqueduct, the following additional considerations become necessary:

1. **Discharge through the siphon**: The head causing flow through the siphon portion of the barrel can be obtained by Unwin’s formula as follows:

\[
h = \left(1 + f_1 + f_2 \cdot \frac{L}{R} \right) \frac{V^2}{2g} - \frac{V_a^2}{2g}
\]
where \( h \) = head causing flow of the loss of head in the barrel
\( L \) = length of barrel,
\( V \) = velocity of flow through the barrel.
\( V_a \) = approach velocity,
\( R \) = barrel radius
\( f_1 \) = entry loss coefficient of barrel; \( f_1 = 0.505 \) for unshaped mouth and \( f_1 = 0.08 \) for bell mouth.

\( f_2 = \) pipe friction loss coefficient given by \( f_2 = a(1 + b/R) \) where the values of \( a \) and \( b \) for different materials may be taken from the table below.

<table>
<thead>
<tr>
<th>Nature of barrel surface</th>
<th>Value of “a”</th>
<th>Value of “b”</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Smooth iron pipe</td>
<td>0.00497</td>
<td>0.025</td>
</tr>
<tr>
<td>2. Encrusted pipe</td>
<td>0.00996</td>
<td>0.025</td>
</tr>
<tr>
<td>3. Smooth cement plaster</td>
<td>0.00316</td>
<td>0.030</td>
</tr>
<tr>
<td>4. Brick work</td>
<td>0.00401</td>
<td>0.070</td>
</tr>
<tr>
<td>5. Rubble masonry or stone pitching</td>
<td>0.00507</td>
<td>0.250</td>
</tr>
</tbody>
</table>

After fixing the velocity through the barrel of the siphon the head ‘\( h \)’ required to generate that much velocity can be determined by equation above. As the downstream H.F.L of the drain remains unchanged, the upstream H.F.L. can be obtained by adding ‘\( h \)’ to the downstream H.F.L. ‘\( h \)’ is known as the afflux.

2. Uplift Pressure on the floor of the barrel:
Since the barrel is depressed below the bed level of the drainage, it will be below the normal water table in the surrounding area. Due to this, a static uplift pressure will be exerted on the barrel.

3. Uplift Pressure on the roof of the barrel:
When the barrel is running full, the water in the barrel exerts an upward thrust on the roof of the barrel. The pressure head on the downstream side of the barrel will be equal to the height of the water level above the roof of the barrel. The pressure head on the upstream side of the barrel will be equal to the sum of the loss of head in the barrel and the pressure head on the downstream side.

Worked Example: Design a siphon aqueduct with the following data:

i) Canal discharge = 30 cumecs
ii) Canal bed width = 22 m
iii) Water depth = 1.5 m
iv) High flood drainage discharge = 420 cumecs
v) Bed level of drainage = 92.50 m
vi) Bed level of canal = 94.50 m
vii) Ground level = 94.50 m
viii) High flood level of drainage = 94.50 m
ix) Canal bank side slope = 1.5 : 1

Solution:

The flow is large and therefore the drainage size is assumed large and therefore we choose type III aqueduct which might be economical. The canal at the aqueduct shall be flumed and taken in concrete trough.

1. Design of drainage water-way:
   Wetted perimeter of the drainage \( P = 4.75 \times (Q)^{1/2} = 4.75 \times (420)^{1/2} = 97.35 \) m
   Provide 12 spans of 6.5 each, separated by 11 No. piers of 1.5 m thick.
   - Length of clear water way = 12 x 6.5 = 78 m
   - Length occupied by piers = 11 x 1.5 = 16.5 m
   - Total length of water way = 78 + 16.5 = 94.5 m
   - Keeping velocity through siphon barrel = 2.0m/s (concrete material)
   - Height of barrel required \( H = \frac{Q}{(V \times B)} = \frac{420}{(2 \times 78)} = 2.69 \) m
   - Provide size of barrel 6.5m x 2.7 m
   - Actual velocity through barrel, \( = \frac{420}{(6.5 \times 2.7 \times 12)} = 1.99\)m/s

   Design of Canal Water-Way.
   Bed width of canal = 22m
   Let the width be reduced to 12 m in trough
   Providing a splay of 2:1 in contraction, the length of contraction transition
Providing a splay of 3 : 1 in the expansion the length of the expansion transition
= \frac{(22 - 12)}{2} \times 3 = 15 \text{ m}

Length of the flumed portion from abutment to abutment
= \text{total drainage water-way} = 94.5 \text{ m}

Design of Levels at Various Sections \{\text{refer to diagram on page (8)}\}

a) Section DD. Cross-section area \(A = B + 1.5h\)
\(h = (22 + 1.5 \times 1.5) \times 1.5 = 36.75 \text{ m}^2\)

Canal velocity = \(\frac{Q}{A} = \frac{30}{36.75} = 0.82 \text{ m/s}\)

Velocity head = \(\frac{V^2}{2g} = \frac{(0.82)^2}{2 \times 9.81} = 0.034 \text{ m}\)

Therefore Relative level (R.L) of water surface
= \text{R.L of bed + water depth} = 94.5 + 1.5 = 96.0 \text{ m}

Relative level of the Total Energy Line (T.E.L) = 96.0 + 0.034 = 96.034 \text{ m}

b) Section CC. Cross section area of canal = \(12 \times 1.5 = 18 \text{ m}^2\)

Velocity = \(\frac{Q}{A} = \frac{30}{18} = 1.67 \text{ m/s}\)

Velocity head = \(\frac{V^2}{2g} = \frac{(1.67)^2}{2 \times 9.81} = 0.142 \text{ m}\)

Loss of head in expansion from section CC to DD
= \(0.3(V_3^2 - V_4^2)/2g = 0.3(0.142 - 0.034) = 0.032\)

Therefore Level of T.E.L at section CC = T.E.L at section DD + Head loss
= 96.034 + 0.032 = 96.066 \text{ m}

R.L of water surface at CC = T.E.L – velocity head = (96.066 – 0.142) = 95.924 \text{ m}

Therefore R.L. of canal bed for constant depth,
= \text{R.L. of water surface – water depth} = (95.924 – 1.5) = 94.424 \text{ m}

c) Section BB. Hydraulic mean depth in trough
\(R = \frac{A}{P} = \frac{18}{(10 + 2 \times 1.5)} = 1.384 \text{ m}\)

From Manning’s formula, \(S = \frac{V^2n^2}{R^{4/3}}\)

Where \(V = \) velocity in the trough
\(S = \) required slope
\(n = \) roughness coefficient = 0.016
\(S = (1.67)^2(0.016)^2/(1.384)^{4/3} = 4.63 \times 10^{-4}\)

Length of flumed portion of the trough = 94.5 \text{ m}

Head loss in trough = 94.5 \times 4.63 \times 10^{-4} = 0.044 \text{ m}

Therefore R.L. of T.E.L at section BB = R.L. of T.E.L at CC + Head loss in trough = 96.066 + 0.044 = 96.11 \text{ m}

.. of water surface at BB = (96.11 – 0.142) = 95.968 \text{ m}

R.L. of bed for maintaining constant depth = (95.968 – 1.5) = 94.468 \text{ m}

d) Section AA

Loss of head in contraction transition from section AA to BB
= \(0.2(V_2^2-V_1^2)/2g\)

where \(V_1 = V_4\)

and \(V_2 = V_3\)

= \(0.2[(1.67)^2 – (0.82)^2]/2 \times 9.81 = 0.0215 \text{ m}\)

R.L of T.E.L at section AA
= R.L of T.E.L at BB + head loss = (96.11 + 0.0215) = 96.325m

R.L. of water surface

= R. L. of T.E.L – \(V_1^2/2g\) = 96.325 – (0.82)^2/2 x 9.81 = (96.325 – 0.034) = 96.291 m

Design of Contraction Transition: The design of the transition will be done on the basis of Chaturvedi’s formula

\[ x = \frac{L \cdot B_o^{3/2}}{B_o^{1/2} - B_f^{1/2}} \left[ 1 - \left( \frac{B_f}{B_x} \right)^{3/2} \right] \]

here \( B_o = 22 \) m; \( L = 10 \) m; \( B_f = 12 \) m

or

\[ x = \frac{10 \cdot (22)^{3/2}}{(22)^{3/2} - (12)^{3/2}} \left[ 1 - \left( \frac{12}{B_x} \right)^{3/2} \right] = 16.74 \left[ 1 - \left( \frac{12}{B_x} \right)^{3/2} \right] \]

The values of \( x \) for various values of \( B_x \) are calculated from the above equation, and are tabulated below. The distance \( x \) is measured from flumed section BB as shown on page 8.

<table>
<thead>
<tr>
<th>( B_x )</th>
<th>12</th>
<th>13</th>
<th>14</th>
<th>15</th>
<th>16</th>
<th>17</th>
<th>18</th>
<th>19</th>
<th>20</th>
<th>21</th>
<th>22</th>
</tr>
</thead>
<tbody>
<tr>
<td>( x )</td>
<td>0</td>
<td>1.89</td>
<td>3.38</td>
<td>4.76</td>
<td>5.86</td>
<td>6.81</td>
<td>7.62</td>
<td>8.34</td>
<td>8.96</td>
<td>9.5</td>
<td>10</td>
</tr>
</tbody>
</table>

Design of Expansion Transition

\[ x = \frac{L \cdot B_o^{3/2}}{B_o^{1/2} - B_f^{1/2}} \left[ 1 - \left( \frac{B_f}{B_x} \right)^{3/2} \right] \]

here, \( L = 15 \) m; \( B_f = 12 \); \( B_0 = 22 \)

\[ x = \frac{15 \cdot x(22)^{3/2}}{(22)^{3/2} - (12)^{3/2}} \left[ 1 - \left( \frac{12}{B_x} \right)^{3/2} \right] \]

\[ x = 25.11 \left[ 1 - \left( \frac{12}{B_x} \right)^{3/2} \right] \]

The values of \( x \), for various values of \( B_x \) are calculated in the table below.

<table>
<thead>
<tr>
<th>( B_x )</th>
<th>12</th>
<th>13</th>
<th>14</th>
<th>15</th>
<th>16</th>
<th>17</th>
<th>18</th>
<th>19</th>
<th>20</th>
<th>21</th>
<th>22</th>
</tr>
</thead>
<tbody>
<tr>
<td>( x )</td>
<td>0</td>
<td>2.84</td>
<td>5.18</td>
<td>7.14</td>
<td>8.8</td>
<td>10.21</td>
<td>11.44</td>
<td>12.5</td>
<td>13.44</td>
<td>14.26</td>
<td>15</td>
</tr>
</tbody>
</table>

Design of Trough Aqueduct:

Flumed water way of canal = 12.0m
The trough shall be divided into two equal compartments by 30 cm thick concrete wall. The road way 6.0 m wide shall be carried over one of the compartments. The height of trough = 1.5 + 0.5 m free board = 2.0 m.

The entire trough section shall be constructed in monolithic RCC by usual structural methods. The outer and flow slab of the trough may be made tentatively equal to 40 cm thick. The outer width of the trough = 2 x 6 + 2 x 0.4 + 0.30 = 13.1 m

Loss of head through Siphon Barrel. The loss of head through the siphon barrel by Unwin’s formula

\[ h = \left(1 + f_1 + \frac{f_2 L}{R} \right) \frac{V^2}{2g} \]

\[ V = \text{velocity through barrel} \ (= 1.99 \text{m/sec}) \]
\[ f_1 = \text{coefficient of loss of head at entry} \ (0.505) \]
\[ f_2 = a(1 + b/R) \quad a \text{ and } b \text{ are picked from table given above.} \]
\[ a = 0.00316 \quad \text{and } b = 0.030 \]

The hydraulic radius, \( R \), = \( A/P = (6.5 \times 2.7)/2(6.5 + 2.7) = 0.953 \text{m} \)
\[ f_2 = 0.00316(1 + 0.030/0.953) = 0.00415 \]
\[ L = 13.1 \text{m} \]

Substituting into head loss equation,
\[ H = [1 + 0.505 + 0.00415 \times 13.1/0.953](1.99^2)/2 \times 9.81 = 0.311 \text{m} \]

Downstream H. F. L. = 94.5 m,

Therefore upstream H.F.L = \( d/s \) H.F.L + head loss = 94.5 + 0.311 = 94.811

Uplift pressure on the barrel roof

R.L.of trough bottom = R.L. of canal bed – thickness of slab = 94.5 - 0.40 = 94.10 m

Loss of head at entry of barrel = 0.505 \( V^2/2g = 0.505 (1.99)^2/2 \times 9.81 = 0.10 \text{ m} \)

Therefore uplift on the roof = 94.811 - (94.1 + 0.10) = 0.611 m

Uplift pressure = 0.611 ton/m²

Dead weight of slab = 0.4x 24 = 0.96 tons/m²

As the dead weight is greater than the uplift pressure, nominal reinforcement is required in the slab.

When the water leveling the drainage is low, the trough slab is to be designed for full water load of the canal.

Uplift pressure on the floor of the barrel

a) Static Head

Barrel floor R. L.

= R.L. of bottom of trough slab – height of barrel = 94.1 – 2.7 = 91.4 m

Assuming tentative thickness of the floor = 0.90 m

R.L. of bottom of the floor = 91.4 – 0.90 = 90.5 m

Bed level of drainage = 92.5 m (given)
Assuming the subsoil water level is up to the bed level of the drain, the static uplift on the floor, will be \( = (92.5 - 90.5) = 2.0 \text{ m} \)
To be continued on 475
Chapter 10

DESIGN OF CHANNELS

Channels must be designed with non-silting and non-scouring velocity. Why?
- When channel is silted up, its carrying capacity is reduced and therefore will irrigate less area
- If channel sides and bed are eroded away, the cross-section increases and the full supply depth decreases which reduces its command area.
- Bed and side erosion also causes various types of damages to the canal structures on it as well as neighbouring areas.

Aim of channel design: is to arrive at a cross-section, which can carry the design flow without either scouring or silting problems.

Design Parameters
For the design of channels, the following data shall be available:
i) Design discharge $Q$; ii) Surface and soil properties (roughness coefficient); silt factor $f$
The design: consists of determining the following four factors:
i) Area of cross-section, $A$ of the channel
ii) Hydraulic mean depth or hydraulic radius $R$
iii) Velocity of flow, $v$
iv) Longitudinal slope of the bed, $S$.

To begin the design, the following two equations are considered (available)
a) $Q = Av$ - continuity equation
b) $V = f(n, R, S)$ – Flow equation, Manning, Chezy, Kutter.

In the above, there are four unknowns and therefore to solve them, we require two additional equations.

To get the additional two equations, the following procedures may be adopted
i) By providing a channel of best discharging section, an equation between area $A$ and hydraulic mean depth may be obtained.
ii) Based on consideration of scouring and silting, a limiting equation for velocity of flow may be obtained.
iii) Fixing the longitudinal slope based on available ground slope.
iv) Based on experience, fixing a suitable width-depth $(B/D)$ ratio not necessary that which gives best section.

KENNEDY’S THEORY (METHOD)

1) To keep silt in suspension, the silt supporting power is directly proportional to the bed width of the stream and does not depend on the wetted perimeter.
2) The limiting velocity which does not produce silting or scouring may be called critical velocity. The relation between critical velocity and depth is

$$v_c = 0.55 x D^{0.64}$$

And after modification we will arrive

$$v = 0.55 \times m \times D^{0.65} \quad \text{where} \quad m = \frac{v}{v_c}$$
v - velocity of flow in channel, m = critical velocity ratio (C.V.R.), m varies from 0.7 for fine sand to 1.3 for very coarse sand
To determine the mean velocity of the flow in the channel, Kennefdy used the Kutter’s formula: v = C (RS)^{1/2} where

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

The difficulty with Kennedy’s formula is that he did not give any formula to fix the slope S but has to be fixed based on the natural ground slope which makes it difficult to obtain the best section for the given discharge.

Design of channel by Kennedy’s Method.

The design procedure:
   i) Assume a trial value of depth, D, in metres and determine Kennedy’s velocity
      \[ v = 0.55 \times m \times D^{0.64} \]  \[ \text{--------- (K-1)} \]
   ii) Calculate the cross-sectional area, A, from the continuity: \[ A = \frac{Q}{v} \]
   iii) Calculate the bed width, b, since A and D are known. For eg for a trapezoidal channel, one selects the side slope of channel based on material of channel. Eg. Assume 1/2H:1V; then the cross-sectional area is given by:
       \[ A = bxD + 1/2xD^2 \] from this, b can be determined.
   iv) Calculate the wetted perimeter and the hydraulic radius:
       The wetted perimeter, \[ P = b + 2\times[D^2 + (D/2)^2]^{1/2} = b + D(5)^{1/2} \]
       The hydraulic radius \[ R = \frac{A}{P} = \frac{(bD + 1/2D^2)}{(b + D(5)^{1/2})} \]
   v) Calculate the actual mean velocity by using the Kutter’s formula;
      \[ v = C(RS)^{1/2} \] where C is computed by the formula above. \[ \text{----- (K-2)} \]
      The value of v in (K-2) must tarry with the v in (K-1) for the assumed depth. If the two values do not tarry, the trial D must be changed and the whole steps repeated till the two values of v are the same.
      The design is done for a given value of S.

**Worked example**

Design an irrigation channel to carry a discharge of 50m\(^3\)/s at a slope of 1/5000. Take Kutter’s \( n = 0.0225 \) and \( m = 0.9 \).

Solution: i). Assume a depth of 2.0 m and determine Kennedy’s velocity
      \[ v = 0.55mD^{0.64} = 0.55 \times 0.9 \times 2^{0.64} = 0.77 \text{m/s} \]
   ii) Determine the cross-sectional area; \[ A = \frac{Q}{v} = 50/0.77 = 65 \text{m}^2 \]
   iii) Assume side slope of 0.5H:1V
       then \[ A = bxD + 1/2D^2 \]
       by putting \[ D = 2.0, \quad b = 31.5 \text{m} \]
   iv) Determine the wetted perimeter \[ P = b + D(5)^{1/2} = 35.97 \text{m} \]
The hydraulic radius \( R = \frac{A}{P} = \frac{65}{35.97} = 1.806 \text{m} \)

The Kutter’s constant \( C = \left\{ \frac{23 + \frac{1}{0.0225} + \frac{0.00155}{\sqrt{5000}}}{1 + \left( 23 + \frac{0.00155}{\sqrt{1.806}} \right) \frac{0.0225}{ \sqrt{5000} }} \right\} = 49.63 \)

vi) The actual velocity \( v = C \left( RS \right)^{1/2} = 49.63 \left( 1.806 \times 1/5000 \right)^{1/2} = 0.94 \text{m/s} \)

The actual velocity is far greater than the Kennedy velocity. Hence we assume a new depth of 2.6 and repeat the whole procedure.

Kennedy’s velocity \( v = 0.55mD^{0.64} = 0.55 \times 0.9 \times 2.6^{0.64} = 0.91 \text{m/s} \)

ii) Determine the cross-sectional area; \( A = \frac{Q}{v} = \frac{50}{0.91} = 54.94 \text{m}^2 \)

iii) Assume side slope of 0.5H:1V

then \( A = bxD + \frac{1}{2}D^2 \)

by putting \( D = 2.6, \ b = 19.83 \text{m} \)

iv) Determine the wetted perimeter \( P = b + D \left( 5 \right)^{1/2} = 25.65 \text{m} \)

v The hydraulic radius \( R = \frac{A}{P} = \frac{54.94}{25.65} = 2.14 \text{m} \)

The Kutter’s constant \( C = \left\{ \frac{23 + \frac{1}{0.0225} + \frac{0.00155}{\sqrt{5000}}}{1 + \left( 23 + \frac{0.00155}{\sqrt{2.14}} \right) \frac{0.0225}{ \sqrt{5000} }} \right\} = 51.05 \)

The actual velocity \( v = C \left( RS \right)^{1/2} = 51.05 \left( 2.14 \times 1/5000 \right)^{1/2} = 1.06 \text{m/s} \)

This value is near to the Kennedy’s critical velocity, however more accuracy could be achieved by choosing another trial depth. The channel dimensions may be taken as

Bed width = 19.83; Depth D = 6.6

USE OF GARRETT’S DIAGRAM FOR DESIGN OF CHANNEL BY KENNEDY’S METHOD.

The computation that were done previously could be done by using Garret’s diagrams, which give a graphical solution for Kennedy’s and Kutter’s equation. Garret’s diagram has discharge plotted as abscissa. The ordinates on the left side indicate the slope and those on the right side, the depths in the channel and critical velocity \( v_c \). Bed width lines are shown as dotted.

The procedure for using the Garret’s diagrams is as follows:

A) Follow the discharge curve for a given discharge and note its point of intersection with the given slope line (which are horizontal).
B) From this point of intersection, draw a vertical line. This will intersect many bed width curves.

C) For any bed width, consider the point of intersection of the vertical line and read out the corresponding water depth D and critical velocity \( v_c \) on the right end of diagram.

D) Calculate the velocity of flow \( v \) for the assumed bed width and its corresponding water depth.

E) Determine the critical velocity ratio \( \text{C.V.R} = \frac{v}{v_c} \)

F) The value of C.V.R should be the same as one given. Otherwise, the procedure must be repeated with other values of bed widths got by the intersection of the vertical on the bed width curves for the given slope, till the value of C.V.R. is very near the given value.

Garret’s diagram are drawn for side slopes of \( 1/2:1 \). The curves can be used for any other value of roughness \( n \) by making use of the small nomogram provided at the top of curves. When any other value of \( n \) is to be made use of, see whether the value is to the left or right of the arrow shown on the nomogram and note the shift. When the vertical line is drawn through the intersection of the slope line and discharge curve, it is with respect to \( n = 0.0225 \) which corresponds to the arrow on the nomogram. The vertical line has to be shifted either to the left or to the right by the same amount as the required value of \( n \) is shifted on the nomogram, and then the intersection points on the bed width curves considered.

Example of the use of the Garret’s diagram

*Determine the channel dimensions by using Garret’s diagrams. The following data are available. Discharge in the channel \( Q = 2.5 \text{ m}^3/\text{s} \); slope \( S = 1/5000 \); \( n = 0.0225 \); critical velocity ratio \( m_c = 0.99 \).*

**Solution:**

Refer to the Garret’s diagram. For a slope 1/5000, it corresponds to 0.2 as the slope is marked in 1/1000 units.

The horizontal from slope 0.2 cuts the discharge curve for 2.5 m³/s at some point. From that intersection point, draw a vertical line to cut many bed widths curves. Let us consider the bed widths curves of \( b = 6.0 \); \( b = 7.0 \); \( b = 8.0 \); \( b = 9.0 \); \( b = 10 \). From these points of intersection, horizontal lines to the right may be drawn and the corresponding values “D” and \( v_c \) read out. The rest of the computation are done in the table below.

<table>
<thead>
<tr>
<th>No.</th>
<th>Discharge Q (m³/s)</th>
<th>Bed width B (m)</th>
<th>Channel depth D (m)</th>
<th>Area A = bD + D²/2 (m²)</th>
<th>Actual velocity ( v = Q/A ) (m/s)</th>
<th>( v/v_c ) = CVR</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.5</td>
<td>6</td>
<td>0.77</td>
<td>4.917</td>
<td>0.508</td>
<td>0.485</td>
<td>1.05</td>
</tr>
<tr>
<td>2</td>
<td>2.5</td>
<td>7</td>
<td>0.74</td>
<td>5.454</td>
<td>0.458</td>
<td>0.470</td>
<td>0.974</td>
</tr>
<tr>
<td>3</td>
<td>2.5</td>
<td>8</td>
<td>0.67</td>
<td>5.58</td>
<td>0.448</td>
<td>0.455</td>
<td>0.985</td>
</tr>
<tr>
<td>4</td>
<td>2.5</td>
<td>9</td>
<td>0.62</td>
<td>5.772</td>
<td>0.433</td>
<td>0.44</td>
<td>0.984</td>
</tr>
<tr>
<td>5</td>
<td>2.5</td>
<td>10</td>
<td>0.58</td>
<td>5.97</td>
<td>0.418</td>
<td>0.43</td>
<td>0.97</td>
</tr>
</tbody>
</table>
From the table, we see that No. 3 gives very near value to the one given. Hence channel dimensions should be: \( B = 8.0 \text{m}; D = 0.67 \text{m} \)

**SILT SUPPORTING CAPACITY OF FLOW (FROM KENNEDY’S THEORY)**

The amount of silt held in suspension according to Kennedy’s theory is proportional to the upwards force of vertical eddies, as it varies as the bed width, \( b \) and some power of the velocity of flow in the channel. This can be formulated as:

\[
Q_t \propto b v_c^n
\]

or

\[
Q_t = a \cdot b \cdot v_c^n
\]

Where \( Q_t \) = quantity of silt transported by channel; \( a = \) constant.

Now \( Q_t/Q = p \) = \% of silt in water.

Assuming a trapezoidal channel; \( Q = b \cdot D \cdot v_c \)

Therefore \( Q_t = p \cdot Q = p \cdot b \cdot D \cdot v_c \)

---(B)

Equating (A) and (B)

\[
A \cdot b \cdot v_c^{n-1} = 1/a \cdot p \cdot D
\]

Therefore \( v_c = (p/a)^{(n-1)/n} \cdot D^{1/(n-1)} \) or

\[
V_c = C \cdot D^{1/(n-1)} \text{ where } C = \text{const.} \]

---©

Compare equation © to Kennedy’s critical velocity

\[
V_c = C \cdot D^{0.64}
\]

---(D)

Equating the indices in © and (D)

\[
1/(n-1) = 0.64
\]

from whence \( n = 2.56 \approx 5/2 = 2.5 \)

therefore silt quantity \( Q_t = a \cdot b \cdot v_c^{5/2} \).
LACEY’S REGIME THEORY

If for Kennedy’s theory, a channel is said to be in regime, when there is neither scouring nor silting, then for Lacey, a different view was held. Lacey observed that even when there is neither silting nor scouring, the channel may or may not be in regime. Lacey defined two regime conditions: - i) initial regime and ii) final regime.

i) Initial Regime: refers to the state of the channel that has formed its cross-section but not formed its longitudinal slope. Such a channel appears to be in regime outwardly, as there may be no visible silting or scouring. Lacey’s regime theory does not apply to such channels.

ii) Final Regime: when a channel is constructed with an improper slope, it tries to remove the incoherent silt on its bed and increase its slope. It first forms its section and then it forms its final slope so that a stable condition is established. When the channel has finished its task of forming a stable cross-section and stable slopes, it is said to be in “final regime”.

iii) Permanent Regime: This condition exists in a channel which is protected both on its slopes by suitable protecting materials. Due to these protections, the channel cannot change its cross-section or slope. Lacey’s regime theory is not applicable to channels in permanent regime.

iv) True regime conditions: A channel will be in regime only when there is no silting or scouring. To satisfy this condition, the silt load in the channel water must be efficiently transported by the channel cross-section. There can be only one channel with a particular cross-section and slope for a particular silt load, which can produce regime conditions. For a channel to be in true regime, for any sediment load brought to it, it must adjust its cross-section and slope to be able to transport the given load.

v) However artificially constructed channels with fixed cross-section and slope can attain regime conditions when the following conditions are satisfied:

i) flow in channel is uniform
ii) discharge in the channel is constant
iii) channel is flowing through incoherent alluvium. Incoherent alluvium is that type of soil which can be scoured and deposited with equal ease. Also the material (silt) transported by the channel should be the same as the material through which the channel is flowing.
iv) The amount and type of silt in the channel (silt charge and grade) are constant.

Since all the above conditions cannot usually be satisfied by artificially constructed channel, it follows that it cannot be in “true regime”. Either in initial or final regime

SHAPE OF REGIME CHANNELS

There is always only one cross-section and one slope for a channel with a particular discharge carrying a particular grade of silt. Natural silt transporting channels assume a semi-elliptical cross-section. The coarser the silt particles, the flatter will be the semi-ellipse and the larger will be the width of water surface. As the grade of silt becomes
finer and finer, the shape of the section becomes narrower. Therefore when a channel is constructed with small cross-section, and steeper slope than necessary, scour starts and continues till final regime is established. Similarly, in a channel with a bigger cross-section and flatter slope, silting starts and continues till final regime is established.

**LACEY’S REGIME EQUATIONS**

According to Lacey, silt is kept in suspension by vertical forces, which are the vertical component of the forces generated by eddies from the bed and sides of the channel. As such the hydraulic radius \( R \) becomes a variable.

Lacey gave two equations, one of which relates velocity to the hydraulic radius and the other which relates velocity to area. Silt grade is considered important and therefore becomes a function in both regime equations.

Lacey’s regime equations:

\[
\begin{align*}
\nu &= \sqrt{\frac{2}{5}} f \cdot R \quad \text{(L-1)} \\
Af^2 &= 140.0v^5 \quad \text{(L-2)}
\end{align*}
\]

where \( A \) = area of cross-section (in \( m^2 \))

\( f \) = silt factor

\( R \) = hydraulic radius (in m)

\( \nu \) = velocity of flow (m/s)

**Perimeter – Discharge (P-Q) Relation**

From equation (L-1) \( \nu^4 = (4/25) \times f^2 \times R^2 \) or

\( f^2 = (25/4) \times (\nu^4/R^2) \quad \text{----------------(L-3)} \)

By putting (L-3) into (L-2) we obtain

\( A[(25/4) \times (\nu^4/R^2)] = 140 \nu^5 \)

Or \( (25/4R^2) \times A = 140 \nu \quad \text{----------------(L-4)} \)

By multiplying (L-4) by \( A \), we have

\( (25/4R^2) \times A^2 = 140 \nu \times A = 140Q. \)

Also \( (A^2/R^2) = P^2 \quad \text{wetted perimeter of section} \)

\( (25/4) \times P^2 = 140Q \)

hence \( P^2 = (4 \times 140/25) \times Q \)

\( P = 4.75 \times (Q)^{1/2} \quad \text{----------------(L-5)} \)

**Relation between velocity, discharge and silt factor**

From equation (L-2) \( Af^2 = 140 \nu^5 \)

By multiplying by \( \nu \), \( A \times \nu \times f^2 = 140 \nu^6 \)

\( f^2 \times Q = 140 \nu^2 \quad \text{or} \)

\( \nu = (f^2 \times Q/140)^{1/6} \quad \text{----------------(L-6)} \)

Lacey gave a relation between \( \nu \), \( R \) and \( S \) as follows

\( \nu = 10.8 \times R^{2/3}S^{1/3} \quad \text{----------------(L-7)} \)

**Regime Slope Equations**
By using equation (L-1), (L-2) and (L-7) ie the fundamental Lacey’s equations, we shall obtain

\[ S = \frac{(f^3/2)}{(4980R^{1/2})} \] ---(L-8)

\[ S = 0.000178 \frac{(f^{5/3})}{q^{1/3}} \] ---(L-9)

\[ S = \frac{(f^{5/3})}{(3340Q^{1/6})} \] ---(L-10)

**Regime – Scour Depth Relations**

From equation (L-1)

\[ v = \frac{(2/5 \cdot x \cdot R)}{x} \] 
\[ v^2 = \frac{2}{5} \cdot f \cdot R \] 
\[ R = \frac{5}{2} \left(\frac{v^2}{f}\right) \] ---(L-11)

From equation (L-6)

\[ v = \left(\frac{f^2Q}{140}\right)^{1/6} \] 
\[ v^2 = \left(\frac{f^2Q}{140}\right)^{1/3} \]  \quad \text{---------L*}

By putting L* into (L-11) we obtain

\[ R = \frac{5}{2}\left(\frac{f^2Q}{140}\right) \times \frac{1}{f} \] 
\[ R = 0.47 \left(\frac{Q}{f}\right)^{1/3} \] ---(L-12)

Similarly

\[ q = 0.21 \frac{Q^{1/2}}{} \] where q silt load, Q flow rate \quad \text{---------(L-13)}
\[ R = 1.5 \left(\frac{q^2}{f}\right)^{1/3} \] ---(L-14)

**Silt factor – Grain Relationship**

If \(m_r\) = mean diameter of the silt particle (in mm), then the silt factor \( f = 1.76x(m_r)^{1/2}\)

<table>
<thead>
<tr>
<th>No.</th>
<th>Type of material</th>
<th>Size of grain (mm)</th>
<th>Silt factor (f)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Silt</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>a) very fine</td>
<td>0.052</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>b) fine</td>
<td>0.120</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>c) medium</td>
<td>0.158</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td>d) coarse</td>
<td>0.323</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Sand</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>a) medium</td>
<td>0.505</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td>b) coarse</td>
<td>0.725</td>
<td>1.50</td>
</tr>
<tr>
<td></td>
<td>Gravel</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>a) medium</td>
<td>7.28</td>
<td>4.75</td>
</tr>
<tr>
<td></td>
<td>b) heavy</td>
<td>26.10</td>
<td>9.00</td>
</tr>
<tr>
<td></td>
<td>Boulders</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>a) small</td>
<td>50.10</td>
<td>12.00</td>
</tr>
<tr>
<td></td>
<td>b) medium</td>
<td>72.50</td>
<td>15.00</td>
</tr>
<tr>
<td></td>
<td>c) large</td>
<td>188.80</td>
<td>24.00</td>
</tr>
</tbody>
</table>
Design of Channels by Lacey’s Theory

The following formulae shall be employed:
a) Data necessary
   i) Channel discharge
   ii) the silt factor
b) Formulae to be used
   i) \( v = \left( \frac{f^2 Q}{140} \right)^{1/6} \)
   ii) \( R = \frac{5}{2} \left( \frac{v^2}{f} \right) \)
   iii) \( Q = v \cdot A \)
   iv) \( P = 4.75 \left( \frac{Q}{2} \right)^{1/2} \)
   v) \( S = \left( \frac{f^5}{3} \right) / \left( 3340 Q^{1/6} \right) \)

Procedure for design
1. Calculate the velocity from (i) above
2. Calculate the hydraulic radius \( R \) from (ii) above
3. Calculate \( A \)
4. Calculate wetted perimeter \( P \)
5. Calculate \( b \) and \( h \)
6. Calculate longitudinal slope.

Example: Design a regime channel for a discharge of 50 m³/s, with silt factor \( f = 1.0 \) using Lacey’s theory.

Solution: Given: \( Q = 50 \) m³/s, \( f = 1.0 \)

1) Calculate velocity , \( v = \left( \frac{f^2 Q}{140} \right)^{1/6} = \left\{ 50 \times (1.0)^2 / 140 \right\}^{1/6} = 0.842 \) m/s

2. Calculate the hydraulic radius, \( R = \frac{5}{2} \left( \frac{v^2}{f} \right) = \frac{5}{2} \left( \frac{0.842^2}{1.0} \right) = 1.774 \)

3. Calculate the area \( A = b \times D + \frac{1}{2} \times D^2 = \frac{Q}{v} = 50 / 0.842 = 59.38 \) m²
   therefore \( A = b \times D + 0.5 \times D^2 = 59.38 \)  (**)

4. Calculate the wetted perimeter \( P = 4.75 \left( \frac{Q}{2} \right)^{1/2} = 4.75 \left( \frac{50}{2} \right)^{1/2} = 33.6 \)
   But \( P = b + 2D \left[ 1 + (1/2)^2 \right]^{1/2} = b + (5)^{1/2}D = 33.6 \)  (***)

Solving equations (**) and (***)
\( B = 29.34 \) and \( D = 1.968 \) m

5. Slope \( S = \left( \frac{f^{5/3}}{3340 Q^{1/6}} \right) = \left( \frac{1^{5/3}}{3340(50)^{1/6}} \right) = 1/6409 \)
   \( S = 1:6409 \)

Therefore channel parameters: \( b = 29.34 \) m; \( D = 1.97 \) m; \( Q = 50 \) m³/s
\( V = 0.842 \) m/s; \( S = 1:6409 \)
Design of channel by use of Lacey’s diagram

Example: determine the channel dimensions and slope for a channel with a discharge of 53 m$^3$/s and silt factor of 1.0 using Lacey’s diagram

Solution: Given: $Q = 53 m^3/s; \ f = 1.0$

Referring to plate 16.5, there are discharge curves for 50 m$^3$/s and 55 m$^3$/s. By interpolation, the discharge curve for 53 m$^3$/s can be drawn. This curve intersects the curve for silt factor $f = 1.0$ at some point. For that point, the abscissa will be 30.0m. This gives the bed width $b = 30.0m$

The ordinate for the intersection point = 2.0m. This gives the depth.

Now to determine the regime slope, refer to plate 16.6. Get the intersection point of the curve for silt factor $f = 1.0$ and the discharge curve for $Q = 53 m^3/s$. The ordinate of the intersection point represents the slope.

The ordinate in this case = 0.15 representing a slope of 0.15 in 1000. $t$

Therefore $S = 0.15/1000 = 1/6667$

Channel parameters $Q = 53 m^3/s; \ f = 1.0; \ b = 30.0m; \ D = 2.0; \ S = 1/6667$