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

- A main canal, generally carries water directly from the river and therefore it carries heavy supplies and is not used for direct irrigation except in exceptional circumstances. Main canal acts as a water carrier to feed supplies to branch canals and major distributaries.

**BRANCH CANAL:**

- There are the branches of main canal in either direction taking off at regular intervals. In general, branch canals are usually feeder canals or feeder branches for major or minor distributaries.
• **MAJOR DISTRIBUTARIES:**
  
  These take off from a branch canal. They may also sometimes take off from the main canal but their discharge is generally lesser than branch canals. They are real irrigation channels, in sense, that they supply water for irrigation to fields through outlet, provided along them.

• **MINOR DISTRIBUTARIES:**
  
  These take off from major distributaries, but sometimes, they take off from branch canal, also. They supply water to water courses through outlets provided along them and carry lesser discharge than major distributary.
• **WATER COURSES:**
• It is a small channel which ultimately feeds the water to the irrigation field.
• **CONTOUR CANAL:**
  A channel flowing along the survey contour.

• **RIDGE CANAL:**
  A channel flowing in filling or at a higher level than the surrounding fields on either direction.

• **CANAL in cut:**
DESIGN OF IRRIGATION CHANNELS

Design of irrigation channels consists in finding its various elements e.g.

i. Bed Width (B).

ii. Full supply depth (D).

iii. Bed slope (S).

iv. Side slopes.

v. Mean velocity

vi. Embankment top width.

vii. Extent of free board.

The guiding principles for the design of various elements are as shown below:

SIDE SLOPES:

These should be such as can withstand the pressure of water. These should be stable. The average values are as below:
• For light sand \( 2H : 1V \) to \( 3H : 1V \)
• For sandy loam or \( 1.5H : 1V \) to \( 3H : 2V \)
• Soft Clay
• For hard clay \( 1H : 1V \).

**FREE BOARD:**

Free board is the vertical distance between full supply level (F.S.L.) and the tops of channel banks. It is about 6" for water courses and 1 ’ to 4’ or even more for other big canals.

As a general rule

\[
\text{Free board} = \left( \frac{\text{F.S.D.}}{10} + 1 \right) \text{ ft.}
\]

Where F.S.D is full supply depth.
• **INSIDE BERMS**: This is kept for future widening of the canal. The width may be 2-4 times the FSD

• **CANAL BANKS**:
  - Slope of the banks depends on the nature of the material. It should be flatter on the water side.

• **SPOIL BANKS**:
  - When the excavation is more than the embankments, the extra earth is dumped in the form of a bank parallel to the canal banks, of height equal to that of the canal banks. Such a bank is called as spoil banks.
• NON SILTING NON SCOURING VELOCITY
  • The velocity of flow that prevents from deposition of silts and scouring action is known as the non silting and non scouring velocity.
  • High velocities in natural river or canals may result erosion/scouring, but low velocities would cause deposition of silt in the bed, and cross-section may be reduced. Hence flow is affected.

• TYPES OF CHANNELS
  • 1. **In Filling**
  • When the bed level of the channel is higher or the bed of the channel is made higher than the adjoining ground level by filling.
• 2. **In Cutting**
  • When the bed of the channel is in depression or made lower than the adjoining ground level by excavation.

• 3. **Partially in cutting and partially in filling**
  • When the site of the channel is such that certain places require filling and certain places require cutting. In this case B.L is below G.L & FSL is above G.L.
• DESIGN OF UNLINED CHANNELS

• Irrigation engineering involves the construction of thousands of miles of canals and to line them all would be quite uneconomic. Hence the design of an unlined channel

• which will remain stable is an important challenge for the hydraulic and irrigation engineer. Besides the difficulty that unlined sections erode easily, the water entering the canal already carries a heavy sediment load; which is likely to be deposited. This has to be given due consideration.

• In brief, the solution of the problem consists in determining (1) depth, (2) bed width, (3) side slope and (4) longitudinal slope of the channel so as to produce a non-silting and non-scouring velocity for the given discharge and sediment load.
The various formulas used for the design are:

1. **Manning's formula**

   (i.) \[ V \, (\text{FPS}) = \left(\frac{1.49}{n}\right) R^{\frac{2}{3}} S^{\frac{1}{2}} \]

   (ii.) \[ V \, (\text{SI}) = \left(\frac{1}{n}\right) R^{\frac{2}{3}} S^{\frac{1}{2}} \]

2. **Chezy's formula**

   \[ V = \sqrt{C R S} \]
3. Kutters formula

\[
(41.6 + 0.00281/S + 1.811/N) \\
C = \frac{1+(41.6 + 0.00281/S)N}{\sqrt{R}}
\]

Kennedy's Silt Theory

Kennedy worked at the upper Bari Doab Canal in 1865-1885 and his research work was published in Civil Engineers, Institution in London. He started his studies in 1865 and gave an empirical formula in 1895 i.e.,

\[
V_c = 0.84 D^{0.64}
\]
• When the same formula was applied in Sindh, then the constant 0.84 was not found to be correct. Therefore a general formula was proposed as:

• \[ V_c = m D^n \]

• Where \( V_c \) = Critical velocity i.e. non silting and non scouring velocity.

• \( m \) = Constant which depends on the nature and the charge (Parts/million) of the silt. It has a greater value for coarser silt.

• \( m = 0.84 \) for some of the Punjab canals and

• \( m = 0.63 \) for some of the Sindh canals.

• \( n = \) Constant, and its value is 0.64 approximately. The critical velocity for earthen canals is approximately taken as 3.5 ft/sec.
• Bed slope of 1 ft/miles is considered good enough for Punjab and Sindh canals to get good results i.e. 1’ in 5000’.
• Side slope of the canal can be taken as 1/2H : 1V.
• For design, we assume certain depth of flow for known discharge for given ’m’ value and we find ’V’ from \( V =0.84 \text{ Do.}^{0.62} \)
• Then from side slope and bed slope, with B & D We proceed through chezy’s formula to get V actual.
• If Vact >V - it is scouring velocity
• If Vact < V - it is silting velocity
• So we make trails to get Vact very close to ’V’
• **Problem:** Design an irrigation channel in alluvium soil in Punjab according to Kennedy's theory with the following given data: $Q = 500$ cfs, $N = 0.0225$, $S = 1$ in 5000

• **PROBLEM:**

• Design a channel as per Kennedy's theory to carry a discharge for 60 cusecs with longitudinal slope 1 ft/canal mile. $N = 0.0225$ and $m = 1$.

• **PROBLEM:**

• Design an irrigation channel according to Kennedy's theory for $Q = 500$ cfs, $m = 1$, $N = 0.0225$. 
PROBLEM No. 1: Design an irrigation channel as per Kennedy's theory for the data given below:

\[ \begin{align*} 
Q &= 500 \text{ cfs} \\
N &= 0.0225 \\
S &= 1 \text{ in 5000 m} = 1 \\
The soil is alluvium soil in Punjab. 
\end{align*} \]

Solution:

**(TRIAL 1)**

**Step 1:** \[ V_c = 0.84 m D^{0.64} \]

Let \[ D = 6' \]

\[ V_c = 0.84 (1)(6)^{0.64} = 2.64 \text{ ft/s} \]

**Step 2:**

\[ \begin{align*} 
A_c &= Q/V_c = 500/2.64 = 189.09 \text{ ft}^2 \\
B &= (A_c - 0.5D^2)/D = 28.52 \text{ ft} \\
P &= B + 2D\sqrt{1+n^2} = 41.93 \text{ ft} \\
R &= A/P = 189.09/41.93 = 4.51 \text{ ft} \\
\end{align*} \]

**Step 3:**

Using \[ C = \frac{41.6 + 0.00281/s + 1.811/N}{1 + (41.6 + 0.00281/s)N/\sqrt{R}} \]
we get \[ C = 85.63 \]

Thus \[ V = C \sqrt{RS} = 85.63 \times \sqrt{(4.51)(Y/5000)} \]

\[ = 2.57 \text{ ft/s} \]

**Result:**

\[ V = 2.57 \text{ ft/s} < V_c = 2.64 \text{ ft/s} \]

\[ \therefore \text{there is possibility of silting.} \]

**(Trial 2)**

**Step 1:** Let \[ D = 5.50 \text{ ft} \Rightarrow V_c = 2.50 \text{ ft/s} \]

**Step 2:**

\[ A_c = 200 \text{ ft}^2 \quad B = 33.61 \text{ ft} \]

\[ P = 45.91 \text{ ft} \quad R = 4.36 \text{ ft} \quad C = 85.09 \]

\[ V = C \sqrt{RS} = 2.51 \text{ ft/s}. \]

\[ V = V_c, \therefore \text{ OK. Use } D = 5.50, B = 33.61 \]
**Problem No. 2:** Design an irrigation channel as per Kennedy's theory for the soil in Punjab, with the data:

- \( Q = 60 \text{ cfs} \)
- \( N = 0.0225 \)
- \( m = 1 \)
- \( S = 1/5000 \)
- \( n = \sqrt{2} \)

**Step 1:** Considering figure of Prob. 1 and using same procedure:

Let \( D = 2' \) => \( V_c = 0.84(1)(2)^{0.64} \)

\[ V_c = 1.31 \text{ ft/s} \]

**Step 2:**

\[ A_c = \frac{Q}{V_c} = 45.60 \text{ ft}^2 \]

\[ B = \frac{(A_c - 0.5D^2)}{D} = 21.90 \text{ ft} \]

\[ P = B + 20 \sqrt{1+n^2} = 26.37 \text{ ft} \]

\[ R = A/P = 1.74 \text{ ft} \]
Step 3: \( C = 69.32 \) 
\[ V = C \sqrt{R} \Rightarrow V = 1.30 \text{ ft/s} \]
\[ V = V_c, \text{ \( \therefore \) ok} \]

Use \( B = 21.90 \), \( D = 2 \) \( \text{Ans.} \)

**Problem No. 3:** Design an irrigation channel as per Kennedy's theory for the data given (for soil in Punjab) below:

- \( Q = 5000 \text{ cfs} \)
- \( N = 0.0225 \)
- \( m = 1 \)
- \( S = 1/5000 \)

**Step 1:** Let \( D = 14' \)

\[ V_c = 0.84mD^{0.64} \]
\[ = 4.53 \text{ ft/s} \]

**Step 2:**

\[ A_c = \frac{Q}{V_c} = 1099.43 \text{ ft} \]

\[ B = \frac{(A_c - 0.5D^2)}{D} = 71.53 \text{ ft} \]
(iii)  

\[ P = B + 2D(\sqrt{1 + \mu^2}) = 102.83 \text{ ft.} \]

\[ R = \frac{A}{p} = \frac{1099.43}{102.83} = 10.69 \text{ ft.} \]

Step 3: \( C = 98.42 \).

\[ V = C \sqrt{R} = 4.55 \text{ ft/s.} \]

\[ V = V_c, \text{ i.e., } \text{circular} \]

\[ (\text{Ans.}) \]

\[ D = 14', \quad B = 71.53' \]

PROBLEM No. 4: Design an irrigation channel in alluvial soil according to Lacey's theory, given the following data:

\( Q = 500 \text{ cfs.} \quad f = 1.0 \)

Step 1: \[ P = 2.67 \sqrt{Q} = 2.67 \cdot \sqrt{500} = 59.70 \text{ ft.} \]

\[ S = \frac{f^{5/3}}{1844} \cdot \frac{1}{5G} = \frac{(1)^{5/3}}{1844} \cdot \frac{1}{500}^{1/3}. \]
MODIFICATION AND IMPROVEMENT TO KENNEDY'S THEORY

E.S. Lindley and F.W. Woods suggested modification and improvements to Kennedy's theory. Lindley performed experiments on the lower Chenab canal and developed the following relationships, between the velocity, bed width and depth.

\[ V = 0.95 \, D^{0.57} \]
\[ V = 0.59 \, B^{0.355} \]
\[ B = 3.80 \, D^{1.61} \]
• Where $V =$ non-silting, non-scouring velocity and $B$ and $D$ are the bed width and depth of channel respectively.

• According to Lindley all the variables in the channel, i.e. slope, bed with and depth, sediment charge are all fixed by nature.

• F.W. Woods analysed Lindley’s data and agreed with him that the stable channel carrying the sediment charge must have a fixed bed width, depth and slope. He developed the following equation:
- \( D = B^{(0.434)} \)
- \( V = 1.434 \log_{10} B \)
- \( S = \quad 1 \)
- \( S = \quad \frac{2 \log_{10} Q \times 1000}{-} \)
LACEY'S SILT THEORY

• One of the drawbacks in Kennedy’s theory is that the silt supporting power depends only on the bed width. Kennedy assumed that the silt was supported by the eddies caused by the bed only but the eddies are also caused by the sides too. Therefore wetted perimeter should have been used instead of bed width only. The other drawback is that:
• the velocity of water was assumed to be directly proportional to some power of depth ‘D’.
• As Kennedy did not mention any particular bed slope or any particular B/D ratio, hence if we analyse the problem without fixing bed slope of the channel or B/D ratio, a channel carrying 60 cfs, discharge designed as per Kennedy’s theory may have the following alternatives:
• S      B      D      B/D
•      --     --     --     ---
• 1. in 5000  21.5’  2.0   10.75
• 2. in 4000  11.0’  2.9   3.80
• 3. in 2000  4.0’   4.3’  0.93
• Lindley and wood also improved the kennedy’s theory but their approach was also same. However they considered B/D ratio.
• In 1929 Lacy put forward his theory. He made a systematic study of the observed data and derived some empirical relations and gave the concept of ’Regime’ theory for unlined channels.
He proposed the following conditions (Regime conditions of unlined channels for zero net erosion or deposition over a hydrological cycle i.e.):

1. Discharge should be constant.
2. Loose granular alluvium material which can be scoured out as easily as it is deposited should be of same characteristics.
3. Silt grade and silt charge are constant.

Obviously all the above requirements are unlikely to be fulfilled in nature, and therefore regime conditions may not be obtained. Lacey therefore classified the regime conditions as
• **True Regime:**
  
  • This is obtained when the above conditions are fulfilled. This happens most often in sandy rivers in alluvial plains which have lateral freedom and by meandering adjust their length and slope, which is determined solely by discharge and silt grade.
  
  • Artificial channels with no freedom of lateral movement can never achieve true regime. They may achieve initial regime but only rarely final regime.
- **Initial Regime:**
- Channels excavated in the first instance with defective slopes and with narrow dimensions, can by immediately throwing down incoherent silt on the bed, increase their slopes and by the generation of increased velocity achieve a non-silting equilibrium which may be termed initial regime. Such channels are subject to lateral restraint in that the scouring of the banks is not allowed. They attain a working stability and therefore neither silt nor scour but they are not in final regime. Their slopes and velocities are higher and the cross-section narrower than they would have been if the sides were not rigid.
• **Final Regime:**
  • If the continuous action of the current eventually overcomes the resistance of the sides and sets up a condition where the channel adjusts its perimeter, depth and slope according to discharge and silt grade, final regime conditions are said to have been achieved.

• **Lacy's Equations:**
  • Lacy worked on canals which were at the final regime conditions. He gave various formulae to design bed slope, bed width and other parameters of channels. In this case no hit and trial method is used.
The equations given by Lacey are

\[ V = 1.1547 \sqrt{f R} \]

Where \( V \) = non-silting, non-scouring velocity, \( R \) = Hydraulic mean depth, \( f \) = Lacey’s silt factor. If the particle sizes are increasing, the value of ’f’ will also increase and vice versa. The values of ’f’ for various materials was given by Lacy. (see page 114, in Book by Iqbal Ali).
• if \( m \) = Critical velocity ratio (CVR) and it is equal to the square root of Lacey’s silt factor i.e. \( f = m^2 \)

• \( P = \frac{8}{3} \sqrt{Q} = 2.67 \sqrt{Q} \)

• Where \( P = \) wetted perimeter and \( Q = \) discharge

\[ S = \frac{f^{(5/3)}}{(1844 Q^{(1/6)})} \] where \( S = \) Bed slope

• \( V = (1.346/Na) \ R^{(3/4)} S^{(1/2)} \) where \( Na \) the roughness of material = 0.0225cf

• The other formulae for design are the modified form of Manning’s Formula
• Manning’s formula:

\[ V = \left( \frac{1.36}{Na} \right) R^{(3/4)} S^{(1/2)} \]

• Where \( Na = 0.0225 \ (f)^{(1/4)} \)

• **PROBLEM**: Design an irrigation channel in alluvial soil according to Lacey’s theory, given the following data: \( Q = 500 \) cfs, and Lacey’s silt factor, \( f=1.00 \) (for Punjab)
PROBLEM No. 4: Design an irrigation channel in alluvial soil according to Lacey’s theory, given the following data:

\[ Q = 500 \text{ cfs}, \quad f = 1.0 \]

**Step 1:**

\[ P = 2.67 \sqrt[3]{Q} = 2.67 \sqrt[3]{500} = 59.70 \text{ ft.} \]

\[ S = \frac{f^{5/3}}{1844 \times Q^{1/4}} = \frac{(1)^{5/3}}{1844 \times (500)^{1/4}}. \]
Step 2:
\[ V = 1.1547 \sqrt{FR} = 1.1547 \sqrt{R} \quad (1) \]
also \[ V = \frac{1.346}{N_a} R^{3/4} S^{1/2} \]
where \( N_a = 6.0225 \) (f)
\[ \therefore V = \frac{1.346}{0.0225} R^{3/4} (0.00019)^{1/2} = 0.83 R^{3/4} \quad (2) \]

Equating eq. (1) & (2), we get:
\[ R = 3.746 \text{ ft.} \]

Step 3:
Thus
\[ A = R \times P = 3.746 \times 59.70 \]
\[ = 223.63 \text{ ft}^2 \]
From figure

\[ A = (B + 0.5D)D \]

\[ 223.63 = (B + 0.5D)D \]  \hspace{1cm} (3)

also \[ P = B + 2D \sqrt{1 + \left( \frac{1}{4} \right)^2} \]

\[ 59.7 = B + 2.236D \]  \hspace{1cm} (4)

Eq. (4) \[ \Rightarrow B = 59.70 - 2.236D \]  \hspace{1cm} (5)

Put (5) in (3)., and on simplification, we get

\[ D = 4.28 \text{ ft} \] \hspace{1cm} \text{Ans.} \]

Eq. from (5) \[ B = 50.14 \text{ ft} \]

\textbf{Problem No. 5:- Design an irrigation channel as per Lacey's theory for the data:-}
\[ Q = 60 \text{ cfs.} \quad S = 1/5000 \]

**Step 1.** First of all let us find \( f \).

\[ S = f^{5/3} / 1844Q^{1/2} \]

\[ f = \left(1844.5 \cdot Q^{1/2}\right)^{3/5} = 0.828 = 0.83 \]

**Step 2.**

\[ P = 2.67 \sqrt{Q} = 20.68 \text{ ft.} \]

\[ V = 1.1547 \sqrt{fR} = 1.052 \sqrt{R} \quad - (1) \]

also

\[ V = \frac{1.346}{0.0225 (0.83)^{1/4}} R^{3/4} \left(\frac{1}{5000}\right)^{1/2} \]

\[ V = 0.90 R^{3/4}. \quad - (2) \]

Equating (1) & (2), we get \( R = 1.865 \text{ ft.} \).

**Step 3.** Thus \( A = R \times P = 38.57 \text{ ft}^2 \).

Considering the above figure
(i) \((B + 0.5D)D = 38.57\text{ ft}^2\)  

(ii) \(B + 2.236D = 20.68\text{ ft}\)

\[\Rightarrow B = (20.68 - 2.236D)\]  

Put (5) in (3).

\[\Rightarrow D = 2.32\text{ ft}\]  

and (5) \(\Rightarrow B = 15.50\text{ ft}\)  

\[\text{Ans.}\]
• **PROBLEM-2:** Design an unlined channel as per Lacey’s theory to carry a discharge of 60 Cfs. with longitudinal slope of 1 ft/canal mile.

• **PROBLEM-3:** Design an unlined channel as per Lacey’s theory to carry a discharge of 5000 Cfs. with longitudinal slope of 1 ft/canal mile.
• Q=5000 cfs, S=1/5000,
• P= 2.67 \((5000)^{1/2}\)=188.8 ft
• f=((1844S(Q)^{0.6})=1.288
• V=1.1547(fR)^{0.5} =1.311R^{0.5} \quad (1)
• V=(1.346/Na)R^{3/4}S^{1/2} \quad Na=0.0225(f)^{0.25}
  =0.024
• V=0.794R^{3/4} \quad (2)
• Equating Eq. 1 and 2 0.794R^{3/4} = 1.311R^{0.5}
• R=7.427 \quad A=RxP=7.427\times188.8=1402.21\text{ft}^2
• $A = 1402.21 = (B + 2D(1 + (0.5)^2))^{0.5}$ \hspace{1cm} (3)
• $P = 188.8 = B + 2(0.5DxD/2))^{0.5}$ \hspace{1cm} (4)

• Solving 3 and 4
• $D = 2.51$ ft
• $B = 15.08$ ft
Comparison of Kennedy’s and Lacey’s Theories

The concept of silt transportation is same in both the cases, both agree that the silt is carried by the vertical eddies generated due to friction of the flowing water against rough surface of canal. Kennedy considered a trapezoidal channel section and, therefore, he neglected eddies generated from the sides. For this reason, Kennedy's critical velocity formula was derived only in terms of depth of flow(y).
• Lacey considered that the entire wetted perimeter of the channel contributes to the generation of silt supporting eddies. He, thus, used hydraulic mean radius (R) as a variable in his regime velocity formulas instead of depth (y).

• Kennedy stated all the channels to be in state of regime provided they did not silt or scour. But Lacey differentiated between two regime conditions, i.e. initial regime and final regime.
• According to Lacey, grain size of material forming the channel is an important factor, and should need much more attention than what was given to it by Kennedy. He connected grain size(d) with his silt factor(f).

• Kennedy used Kutter's formula for determining actual generated channel velocity. The value of Kutter's rugosity coefficient(n) is again a guess work. Lacey, on the other hand, has produced a general regime flow, after analyzing huge data on regime channels.
• Kennedy has not given any importance to bed width and depth ratio. Lacey has connected wetted perimeter (P) as well as area (A) of the channel with discharge, thus, establishing a fixed relationship between bed width and depth.

• Kennedy did not fix regime slopes for his channels, although, his diagrams indicate that steeper slopes are required for smaller channels and flatter slopes are required for larger channels. Lacey, on the other hand, has fixed the regime slope, connecting it with discharge.
• **SILT CONTROLLING WORKS**

• A lot of silt is carried by the rivers every year. The river Sutlej transports 35 million tons of sediments per year, while the Indus carries a total load of 440 million tons per year at Tarbela, with a mean annual discharge of 93 MAF, and the estimate for the Jhelum is 70 million tons per year. The Warsak reservoir on the river Kabul built in 1960, had an initial live storage of 23,000 acre feet which in the last ten years has been reduced to a residual minimum of 10,000 acre feet. Tarbela reservoir with an initial live storage of 9.3 million acre feet when completed in 1975, will reduce to one million acre feet in fifty years whereas the Mangla reservoir will lose 30 percent of its live storage in the same period.
Unlined canals can get choked or silted by sediment brought by the rivers, and diverted into the canals along with the water. Special works may be constructed to control sediments entering into the canals or sediments being carried away by water. These works are called as silt controlling works. These works are:

1. Proper channel design
2. Works in the river
   a. Guide wall and silt pocket
   b. Silt excluder
3. Works in the canal (silt ejector)
• **SILT EXCLUDERS:**

• The basic idea behind the design is that the lower layers of the flowing water carry a higher concentration of silt, and therefore if the upper layers of the water only can be skimmed into the canal, all the rolling bed silt and the silt in the lower layers is excluded. This is achieved by a silt excluder. This is a diaphragm slab supported on a number of tunnels. Tunnels are placed parallel to the head regulator and discharge d/s through the under-sluices. The water above the silt excluder slab containing less silt is then diverted into the canal. The following points should be kept in mind while designing a silt excluder.

• 1. The tunnel discharge through the under-sluice is recommended to be 20% of the canal discharge.
• 2. The silt excluder should cover only two bays of the under-sluice as this was found to be more efficient in the model studies of Kalabagh barrage than a silt excluder covering four bays.
• 3. The approach channel need not be lined.
• 4. The divide wall should be 1.2. to 1.4 times the head regulator length.
• 5. The top of the silt excluder slab should be flushed with the head regulator crest, i.e. the clear height of the tunnels would be 1/3 the depth of the water minus the slab thickness.
• 6. The roof slab should be designed to carry a full water load in case the tunnels are empty.
7. The first tunnel should cover all the head length.

The discharge through the tunnels will depend upon the head measured above the centre line of the tunnel. Tunnels can be treated as box culverts.

9. The velocity in tunnels should be 6 ft/sec to 10 ft/sec.
Silt Control Devices

Silt Excluder

Fig. 9.20. Silt Excluder.
SILT EJECTOR

- It employees the same principle of sediment removal as the silt excluder except that it is placed in the bed of the canal and is located about 1000 yards d/s of the head regulator. (see page fig).
• It consists of a horizontal slab a little above the canal bed, which separates out the bottom layers. Under the slab there are tunnels to eject heavy silt laden bottom water in an escape channel. For designing of silt ejector the following points should be kept in mind:

• 1. It should be located about 1000 yards d/s of the head regulator.

• 2. The bed width of the canal is divided into a number of tunnels. These tunnels curve to right or left and pass under the canal bank to terminate in a regulator, which is provided with gates to regulate the discharge
Fig: Plan of Silt Ejector
3. The height of the tunnel should be 20 to 25% of the design depth of water in canal.

4. The top slab of the tunnels usually project 1.5 ft to 2 ft U/S at the entrance.

5. 20% of the canal discharge is usually diverted into the ejector. This means that 20% additional discharge over and above the canal design discharge is allowed to enter the canal at the head regulator.
• 6. The method of calculating the discharge is the same as that for the silt excluder.
• 7. Normally a minimum head of at least 2.5 ft is required to operate the ejector.
• 8. A velocity of 8 ft to 10 ft/sec through the tunnel is adequate to move sand size sediment.
CANAL HEAD REGULATOR

Canal head regulator is a structure constructed at the head of canal. In the case of main canal, it consist of a number of sluices or openings between piers controlled by gates which are operated from the overhead gate bridge on the top as the weirs. Along with the service bridge above the pier we may also have a road bridge for traffic:

The objects of a canal head regulator are:

1. To regulate the supply entering the canal.
2. To control the amount of silt entry into the canal.
3. To control river floods entering the canal.
Canal Head Regulator
Fig: Alignment of a canal head regulator
• The head regulator of a distributary or minor canal is similar to that of main canal, so far as the working principle is concerned. It consists of a number of arches to carry a road bridge for traffic and also in some cases a gate bridge for the manipulation of gates which move between piers.

• **The function** of the regulator is to pass into the channel, the required quantity of water as and when necessary. Its design should be such as to allow into the channel its due share of silt in suspension.
• The sill of the regulator is usually fixed slightly above the bed level of the parent channel on its upstream side. The spans are controlled by vertical lift gates or radial gates. The regulator is generally a flumed one because it is economical. However, it involves transitions, energy dissipation arrangement etc.
• **CANAL FALLS**

➤ **Introduction:**

i. Canal fall or drop:

• A structure designed to secure lowering of the water surface in a canal and to dissipate safely the surplus energy so liberated, which otherwise scour the bed and banks of the canal

Necessity:

• Velocity in a canal is a function of the slope of the canal. There is a limit for the velocity, so that the canal bed can neither be scoured nor silted up. Hence there is a limiting surface slope in the canal.

• The slope of the country, where the canal system has to run will naturally be steeper than the surface slope the canal system that has to come up. To bring the velocity within the permissible velocities, falls or drops are introduced at suitable locations.

• Falls are combined with regulators, bridges, and escapes.
CANAL FALL

- F.S.L.
- Canal Bed
- Natural Surface
- Cut
- Fill
- Sudden Drop or Fall
- Cistern
- W.T.
• To avoid this, the bed of the canal is given a sudden drop or fall at a suitable place, so that it may run partially in excavation (cutting) and partly in filling, depending upon the command areas. The structure built to safeguard the drop is called a canal fall structure.

• REQUIREMENTS OF CANAL FALLS

• The basic requirements for the canal falls are:
  1. Suitable Energy Dissipation arrangement:
• In the case of vertical falls there should be a proper cistern to take the impact of the falling water. In addition to this, the cistern bed may be roughened by one of the many devices described. In case of glacis and modern weir-type falls, energy dissipation is achieved by the formation of the hydraulic jump at the toe of the glacis plus friction blocks.

• 2. If there has to be a bridge, then a suitable amount of fluming (constriction) is to be provided for economy and for this a suitable u/s and d/s transition has to be given.
• 3. The crest must be designed to maintain normal supply depths in the canal u/s.
• 4. To increase the seepage path a pucca floor is provided on both sides (i.e. u/s and d/s side) and then the stone pitching is also done to avoid scouring due to water.
• 5. The initial cost of construction and the cost of maintenance should be as low as possible.

**LOCATION OF FALLS:**
• The following factors are considered for the location of falls:
1. For main and branch canals which do not directly irrigate, the site is determined on the basis of economy in earth works. All the excavated earth is utilized in making up banks. The depth of excavation is known as the balancing depth of excavation. A suitable site will be where, the depth of excavation becomes minimum.
• 2. For the distributaries and minors, falls may be located d/s of the outlets as this helps in increasing the command area, and in improving the efficiency of outlets.

• 3. The site should be selected keeping in mind the requirements for a road crossing, as a bridge combined with a fall.
Development of Falls/Drops

The ancient people always tried to avoid falls by aligning canals along zig-zag route in order to increase the length of the canal and thus dissipate the excess energy head in friction. The Eastern Yamuna Canal constructed by Mughal Emperors had no falls, and the canal, followed a sinuous path. The falls were first constructed by the British in India in the nineteenth century. The development of falls, since then, took place gradually. Among the earlier type of falls are: Ogee falls, rapids and stepped falls. Later, notch falls, vertical falls and glacis type falls were developed.
1. Ogee fall

- The Ogee fall was first constructed by Sir Proby Cautley on the Ganga Canal. This type of fall has gradual convex and concave curves, with an aim to provide a smooth transition and to reduce disturbance and impact. This preserved the energy (without dissipating it). Due to this, the Ogee fall had the following defects:
  
  (i) There was considerable draw down effect on the u/s resulting in bed erosion.
  
  (ii) Due to smooth transition, the kinetic energy was preserved till sufficient depth was scoured out below the fall to ensure the formation of the hydraulic jump.
Ogee Fall
2. Rapid fall

- Rapid falls were provided on Western Yamuna Canal and were designed by Lieut. R.F. Croften. Such a fall consists of a glacis sloping at 1 vertical to 10 to 20 horizontal. The long glacis assured the formation of hydraulic jump. Hence, the fall worked admirably. However, there was very high cost of construction.
3. Stepped fall

- Stepped fall was a next development of the rapid fall. One such type was provided at the tail, of main canal escape of Sarda canal. The cost of this fall was also too high.
4. Notch fall

- Soon after the development of stepped fall, the efficiency of vertical impact on the floor for energy dissipation came to be recognized. The vertical fall came in the field along with the cistern. However, with greater discharges, vertical fall gave trouble. Hence, these were superseded for a time by the notch fall. The trapezoidal notch fall was first designed by Ried in 1864. The fall consists of one or more trapezoidal notches in a high crested wall. A flat circular lip projects downstream of each notch to disperse water. The notches were designed to maintain the normal water depth in the u/s channel at any two discharge values.
Notch fall
5. Vertical drop fall

- Consists of vertical drop wall constructed with masonry work.
- Water flows over the crest of wall.
- Concrete floor provided on downstream side to control scouring effect. Curtain wall provided on upstream and downstream side.
- This type falls provided at Sarda canal, UP. Hence known as Sarda fall.
Vertical drop fall
6. Glacis type fall

- The efficiency of the hydraulic jump as a very potent means of destroying the energy of canal falls is used in glacis falls. The glacis type of fall utilizes the standing wave phenomenon for dissipation of energy. The glacis fall may be (i) straight glacis type, or (ii) parabolic glacis type, commonly known as the Montague type. The straight glacis fall may be with baffle platform and baffle wall. In such a case, the formation of jump takes place on the baffle platform. This type was first developed by Inglis and is called Inglis fall.
6. Glacis Type Fall
CROSS-DRAINAGE WORKS

• Cross drainage works are structures which make canal is possible crossing natural stream (e.g., drainage or river) or roads and railways are possible to cross valleys.
• In an irrigation project, the crossing of the canals with such obstacle cannot be avoided. They are generally very costly.
Types of cross drainage works

Type I
Irrigation canal passes over the drainage – Aqueduct – Siphon aqueduct.

Type II
Drainage passes over the irrigation canal – Super passage – Siphon super passage.

Type III
Drainage and canal intersection each other of the same level – Level crossing – Inlet and outlet
N: NATURAL STREAM
C: CANAL

(SUPER PASSAGE)
• TYPES OF CROSS-DRAINAGE WORKS
• The cross-drainage work is classified as:
  1. Aqueduct
  2. Siphon aqueduct
  3. Super passage
  4. Siphon super-passage
  5. Level crossing
  6. Drainage inlet and outlet
• (fig)
• 1. **Aqueduct**
1. Aqueduct

- When the canal bed is above the maximum level of the water surface in the natural drain, then the canal is carried over the drain in an R.C.C flume or RCC pipe or a steel pipe (depending on the canal discharge) supported on piers. The flow in the aqueduct and the drain is an open channel flow. In the aqueduct, since the canal is flummed, a well-designed transition at the entry and exit is provided.
2. **Siphon Aqueduct**

When the maximum water level is above the bed of the canal, then the canal is carried unflumed, over the drain while the bed of the drain is lowered and the drain passes underneath the canal through R.C.C. barrels or square section or R.C.C pipes. While the flow in the canal is open channels flow, the flow in the drain through the barrel
3. **Super Passage**

When the FSL in the canal is below the bed of the drain, then the canal continues to flow unflumed and the natural drain water is carried in a R.C.C flume over the canal. This is the opposite to an aqueduct. However an elaborate transition in the case of the drainage is not necessary.
4. **Siphon Super Passage**

This is the opposite to a siphon-aqueduct. When the bed of the natural drain is below the FSL of the canal, the canal water is siphoned below the bed of the drain through R.C.C. pipes or barrels or just by dropping the bed of the canal such that the water levels u/s and d/s in the canal are touching the sides of the R.C.C. flume carrying in drain water, so that the flow in the canal is under pressure.
• In some cases two canals cross each other, instead of a drain and a canal. In such cases the structure is named after one of the canals and will be called aqueduct or siphon only as the case may be.

• 5. **Level Crossing**

• When the beds of the drain and the canal are almost at the same level, then the waters of the two are allowed to mix and the canal supplies are regulated through a regulator.
6. **Drainage Inlet**

When the volume of natural drainage is very small then instead of providing a structure to carry the water across the canal, the surface runoff due to rainfall is allowed to flow into the canal at a suitable place. Such a situation arises when the general direction of flow of the canal is perpendicular to the ground slope. Thus the surface runoff gets obstructed and should be allowed in the canal. The arrangement is economical since such surface runoff occurs only a few times during the year, and the discharge is not high.
7. **Drainage Outlet**

This is the same type of structure as an inlet built on the opposite bank of the canal slightly d/s of the drainage inlet. It discharges the extra water that has entered the canal through the inlet into an escape channel. The crest of the weir is kept slightly above the FSL so that if when extra water enters the canal it automatically spills over the weir. Energy dissipation arrangements are provided along the bed of the escape channel which leads the water away from the canal. This is also called a canal escape.

In addition to the drainage water, any extra supplies that may come into the canal are disposed off by the canal escape.
• **CANAL LINING**

• For canals we means only the unlined earthen canals. These unlined canals are compacted to some extent but the seepage cannot be prevented. Therefore to prevent the quantity of seepage the canals are lined. Different materials are used for the lining.

• Canal lining is an expensive business but in certain cases it is unavoidable. This is required for the following purposes.
• ADVANTAGE OF CANAL LINING
• 1. When the track of land through which the canal is passing is highly pervious or permeable.
• 2. When the loss of water through seepage is very high
• 3. In order to check the process of water logging.
• 4. For the defence purposes, i.e. when the canal is to serve as a line of defence and a very high velocity is desired in the canal.
• 5. In case of head race and tail race of a hydroelectric power station situated on a canal fall.
The lining of canals has the following advantages:

1. Reduction of seepage losses, resulting in a saving of water which can be utilized for additional irrigation.

2. Reduction of percolation to the ground water reservoir, leading to prevention of water logging.

3. Reduction of maintenance cost and the possibility of breaking due to stability of the section

• 5. By lining a channel, its cross-section is reduced, which means a saving in the earth work and the land to be acquired.

• 6. The risk of pilferage by cultivators is greatly reduced.

• 7. Salts will not be absorbed.
WEIRS AND BARRAGES:

• A hydraulic structure constructed across a river is known as a **dam**, a **weir** or a **barrage**.

• A **weir** is a structurally safe streamlined wall, built at a suitable site across a river. It is a low head-structure for the purpose of diverting river water.

• A **barrage** is gated weir
Barrage-Cross-Section
Barrage components

1. **U/s Flexible protection:**
2. **Concrete blocks**, to raise the level of the supply so that it can command the area to be irrigated.
3. **Stone-filling**, to gain command economically when canal has to pass in expensive cutting
4. **U/S Apron (Pucca floor)** to reduce the fluctuations of the level of the river.
5. **U/S Sheet pile**: These are situated at the U/S end of the U/S concrete floor. The piles are driven into the soil beyond the maximum possible scour that may occur. The functions are:
   - **i.** To protect the barrage structure from the scour.
   - **ii.** To reduce the uplift pressure on the barrage floor.
   - **iii.** To hold the sand compacted and densified between two sheet piles in order to increase the bearing capacity when the barrage floor is designed as a raft. In Qadirabad and Chasma, upto 2 tons/ sq.ft. of bearing capacity has been obtained.
Components

• 6: *Intermediate Sheet Piles*: These are situated at the end of U/S and D/S glacis. These serves as the In the event of U/S or D/S sheet piles collapsing due to advancing scour or undermining, then these sheet piles give protection
9.D/S Apron: The d/s floor is made of concrete and is constructed so as to contain the hydraulic jump. Thus it takes care of turbulence which would otherwise cause erosion. It is also provided with friction blocks of a suitable shape and at a distances determined by the hydraulic model experiments in order to increase friction and destroy residual kinetic energy.
• **10. D/S sheet piles:** D/S sheet piles are placed at the end of the d/s concrete floor and their main function is to check the exit gradient. Their depth should be greater than the maximum possible scour.

• **11. Inverted Filter:** An inverted filter is provided between the d/s sheet piles and the flexible protection. It would typically consist of 6" sand, 9" Coarse sand, 9” gravel. The filter material may vary with the size of the particles forming the river bed. It is protected by placing over it concrete blocks of sufficient wt. and size (ray 4’x2.75 ’x4’ used in the Kalabagh barrage).
Silts (or jhiries) are left between the blocks to allow the water to escape. The silts are filled with sand. The length of the filter should be \( 2 \times \frac{d}{s} \) depth of the sheet piles. Its primary function is to check the escape of fine soil particles in the seepage water. In the case of scour, it provides adequate cover for the d/s sheet piles against the steeping of the exit gradient.
12. Concrete blocks - as discussed in (II)

- **13. Stone filling.** Function is same as described in (3).

- **14. D/S Flexible Protection:** A flexible apron is placed d/s of the filter and consists of boulders large enough not to be washed away by the highest likely water velocity. The protection provided is such as to cover one & half times depth of scour on the U/S side and 2 x depth of scour on the d/s side at a slope of 3 : 1.
Barrage plan

- Plank Wall
- Main Weir
- Under Sluice Section
- Fish Ladder
- Divide Wall
- Silt Pocket
- Canal Head Regulator
- Upstream Right Guide Bank
- Flow
• **Undersluices:** A number of bays at the extreme ends of the barrage adjacent to the canal regulator will have a lower crest level than the rest of the bays. The main function is to draw water by the formation of a deep channel in lower river flow and, secondly, to help control the flow of silt into the canal by reducing the water velocity by the formation of deep channel in front of the canal. Accumulated silt can be washed away easily by opening the undersluice gates to high velocity currents generated by lower crest levels or a high differential head.
Undersluices
Divide Wall:

- The divide wall separates undersluice bays from the normal bays. Its length on the U/S side has to be sufficient to keep the heavy turbulence away from the protection of the sluices. Similarly, in the d/s side it should extend to cover the hydraulic jump and the resulting turbulence.
- The main functions are:
  - a. *To separate the undersluices from the normal bays to avoid the heavy turbulence which would otherwise occur due to a differential head in the two sections. This helps by creating a still pond in front of the canal off take thereby allowing better site control.*
• **Divide wall**

*To generate a parallel flow and thereby avoid damage to the flexible protection area of the undersluice portion.*

• **Fish Ladder:**

It is constructed along the divide wall. It is a device designed to allow fish to negotiate the artificial barrier in either direction. In the fish ladder the optimum velocity is 6 to 8 H/Sec, This can be obtained by creating a spatially varied flow as in the fish ladder at Marala, Qadirabad and Chashma barrages.
Divide Wall

Fish Ladders
• **Guide Banks:**
  Guide banks are earthen embankment with stone pitching to guide the river through the barrage.

• **Marginal Bunds:** Marginal Bunds are flood embankments in continuation of the guide banks designed to contain the floods within the flood plain of the river. Both height and length vary according to the back-water effect produced by the barrage.
Fig: Typical layout of diversion head-works
PURPOSE OF THE BARRAGE

• 1. To divert the required quantity of water from the river to the canals.
• 2. To raise the area proposed for irrigation by gravity flow.
• 3. To allow proper silt control.
• 4. To provide permanent Headworks for the canals in order to protect them during floods by providing for complete closure.
• 5. To provide better regulation than weir.
SITE SELECTION OF BARRAGE

• The following consideration should be kept in mind when deciding on the site for a barrage.
  • 1. The site must have a good command over the area to be irrigated, and must also be not too far distant to avoid long feeder channels.
  • 2. The width of the river at the site should preferably be the minimum with a well defined and stable river approach.
• 3. A good land approach to the site will reduce the expense of transportation and, therefore, the cost of the barrage.
• 4. Easy diversion of the river after construction.
• 5. Existence of central approach of the river to the barrage after diversion. This is essential for proper silt control.
• 6. If it is intended to convert the existing inundation canals into perennial canals, site selection is limited by the position of the head regulator and the alignment of the existing inundation canals.

**perennial canals**: They are linked to dams and barrages to provide water throughout the year and they irrigate a vast area.

**Inundation canals**: long canals taken off from large rivers are called inundation canals. They receive water when the river is high enough and especially when it is in flood

• 7. A rock foundation is the best but in alluvial plains the bed is invariably sandy.
SOME DEFINITIONS

• **Khadir:**
  • The Khadir is the flood plain of the river.
  • A flood plain is an area of land that is prone to flooding. People realize it is prone to flooding because it has flooded in the past due to a river or stream overflowing its banks.

• **Khadir Axis:**
  • The Khadir axis is a line passing through the centre of the river course, between the two high banks upto the backwater effect.
• **Weir Axis:**
  • The weir axis is a line along which the crest of the weir is laid.

• **River Axis:**
  • The river axis is a line parallel to the Khadir axis at the centre of the weir axis between the abutments.

• **Headworks Axis:**
  • The Headworks axis is a line perpendicular to the weir axis at the centre of the weir abutments.
• Retrogression is a very important phenomenon which occurs after the construction of weir or barrage in a river flowing through alluvial soil. As a result of back-water effects and the increase in depths, the velocity of the water decreases resulting in the deposition of the sediment load. Therefore, the water overflowing the barrage having less silt, picks up silt from the d/s bed. This results in the lowering of the d/s river bed for a few miles. This is known as retrogression.
Retrogression may occur for the first few years and bed levels often recover their previous level. The phenomenon is temporary because the river regime ice. its slope, adapts to the new conditions of flow created by the barrage within a few years and then the water flowing over the weir has a normal silt load. The d/s cycle than reverses due to the greater depth, silt is then deposited and the d/s retrogressed bed recovers to the point of equilibrium. Retrogression value is minimum for a flood discharge and maximum for a low discharge. The values vary from 2 feet to 8.5 feet.

ACCRETION: Accretion is the reverse of retrogression and normally occurs u/s although may also occur d/s after the retrogression cycle is complete. There is no accurate method of calculating the values of retrogression and accretion but the values that have been recorded at various barrages may serve as guidelines.
Barrage plan
In any hydraulic structure on permeable foundations, water flow from a region of high level (high pressure) to the region of low level (low pressure), beneath and around the structure. 

**Percolation** is the flow of water under the ground surface due to an applied differential head.

**Percolation length** (creep length) is the length to dissipate the total hydraulic pressure on the structure.

**Undermining** (Piping) is to carry away (wash) soil particles with flowing water below the ground surface causing collapse or failure of the above structure.
• The quantity of seepage and the uplift force depends on:
  • i. Differential head    ii. Characteristics of sub-soil
  • iii. Geometry of structure
• Hydraulic gradient = \( \frac{H}{L} \) Where ’L’ is the distance travelled by a particle.
• Exit gradient means the hydraulic gradient at the exit. Similarly, the entrance gradient is the hydraulic gradient at entrance.
• The critical path for a particle will be just along the boundary of structure where ’I’ is minimum, and is called Creep Length, and at this path the exit gradient (le) will be maximum
• The critical value of “le” = 1. If “le” is more than 1 then the piping phenomenon will take place, and the structure may collapse.

• So le should be less than 1 and we can make le less than 1 by increasing the value of ’L’. The creep length may be increased by providing the sheet piles.

• **The problem** Consists therefore in controlling the seepage force so that it cannot carry away the foundation materials. The various theories to solve these problems are discussed below:
1. **Bligh's Creep Theory**

According to Bligh’s Theory, the percolating water follows the outline of the base of the foundation of the hydraulic structure. In other words, water creeps along the bottom contour of the structure. The length of the path thus traversed by water is called the length of the creep. Further, it is assumed in this theory, that the loss of head is proportional to the length of the creep. If $H_L$ is the total head loss between the upstream and the downstream, and $L$ is the length of creep, then the loss of head per unit of creep length (i.e. $H_L/L$) is called the hydraulic gradient. Further, Bligh makes no distinction between horizontal and vertical creep.

- Consider a section a shown in Fig. Let $H_L$ be the difference of water levels between upstream and downstream ends. Water will seep along the bottom contour as shown by arrows. It starts percolating at $A$ and emerges at $B$. The total length of creep is given by
  - $L = d_1 + d_1 + L_1 + d_2 + d_2 + L_2 + d_3 + d_3$
  - $= (L_1 + L_2) + 2(d_1 + d_2 + d_3)$
  - $= b + 2(d_1 + d_2 + d_3)$
Fig-1: Bligh’s Creep
Head loss per unit length or hydraulic gradient = \[ \frac{H_L}{b+2c_1+d_2+d_3} = \frac{H_L}{L} \]

Head losses equal to \( \left( \frac{H_L}{L} \times 2d_1 \right), \left( \frac{H_L}{L} \times 2d_2 \right), \left( \frac{H_L}{L} \times 2d_3 \right) \); will occur respectively, in the planes of three vertical cut offs. The hydraulic gradient line (H.G. Line) can then be drawn as shown in figure above.

### i. Safety against piping or undermining:

According to Bligh, the safety against piping can be ensured by providing sufficient creep length, given by \( L = C.H_L \), where \( C \) is the Bligh’s Coefficient for the soil. Different values of \( C \) for different types of soils are tabulated in Table – 1

<table>
<thead>
<tr>
<th>SL No.</th>
<th>Type of Soil</th>
<th>Value of C</th>
<th>Safe Hydraulic gradient should be less than</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Fine micaceous sand</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 mixed with boulder and gravel, and for loam soil</td>
<td>5 to 9</td>
<td>1/5 to 1/9</td>
</tr>
<tr>
<td>4</td>
<td>Light sand and mud</td>
<td>8</td>
<td>1/8</td>
</tr>
</tbody>
</table>

The hydraulic gradient i.e. \( H_L/L \) is then equal to \( 1/C \). Hence, it may be stated that the hydraulic gradient must be kept under a safe limit in order to ensure safety against piping.
• **Safety against uplift pressure:**

• The ordinates of the H.G line above the bottom of the floor represent the residual uplift water head at each point. Say for example, if at any point, the ordinate of H.G line above the bottom of the floor is 1 m, then 1 m head of water will act as uplift at that point. If h’ meters is this ordinate, then water pressure equal to h’ meters will act at this point, and has to be counterbalanced by the weight of the floor of thickness say t.
- Uplift pressure = $\gamma w \times h'$ [where $\gamma w$ is the unit weight of water]
- Downward pressure = $(\gamma w \times G).t$ [Where $G$ is the specific gravity of the floor material]
- For equilibrium, $\gamma w \times h' = \gamma w \times G. \ t$
  \[h' = G \times t\]

Subtracting $t$ on both sides, we get
\[
(h' - t) = (G \times t - t) = t (G - 1)
\]
\[
t = \left(\frac{h' - t}{G - 1}\right) = \left(\frac{h}{G - 1}\right)
\]
- Where, $h' - t = h = $ Ordinate of the H.G line above the top of the floor
• **Lane’s Weighted Creep Theory**

Bligh, in his theory, had calculated the length of the creep, by simply adding the horizontal creep length and the vertical creep length, thereby making no distinction between the two creeps. However, Lane, on the basis of his analysis carried out on about 200 dams all over the world, stipulated that the horizontal creep is less effective in reducing uplift (or in causing loss of head) than the vertical creep. He, therefore, suggested a weightage factor of $1/3$ for the horizontal creep, as against $1.0$ for the vertical creep.

• Thus in Fig–1, the total Lane’s creep length ($L$) is given by
\[ L_l = (d_1 + d_1) + \frac{1}{3} L_1 + (d_2 + d_2) + \frac{1}{3} L_2 + (d_3 + d_3) \]
\[ = \frac{1}{3} (L_1 + L_2) + 2(d_1 + d_2 + d_3) \]
\[ = \frac{1}{3} b + 2(d_1 + d_2 + d_3) \]

To ensure safety against piping, according to this theory, the creep length \( L \) must not be less than \( C_1 H_L \), where \( H_L \) is the head causing flow, and \( C_1 \) is Lane’s creep coefficient given in table 2

<table>
<thead>
<tr>
<th>SL No.</th>
<th>Type of Soil</th>
<th>Value of Lane’s Coefficient ( C_1 )</th>
<th>Safe Lane’s Hydraulic gradient should be less than</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Very fine sand or silt</td>
<td>8.5</td>
<td>1/8.5</td>
</tr>
<tr>
<td>2</td>
<td>Fine sand</td>
<td>7.0</td>
<td>1/7</td>
</tr>
<tr>
<td>3</td>
<td>Coarse sand</td>
<td>5.0</td>
<td>1/5</td>
</tr>
<tr>
<td>4</td>
<td>Gravel and sand</td>
<td>3.5 to 3.0</td>
<td>1/3.5 to 1/3</td>
</tr>
<tr>
<td>5</td>
<td>Boulders, gravels and sand</td>
<td>2.5 to 3.0</td>
<td>1/2.5 to 1/3</td>
</tr>
<tr>
<td>6</td>
<td>Clayey soils</td>
<td>3.0 to 1.6</td>
<td>1/3 to 1/1.6</td>
</tr>
</tbody>
</table>
Undermining or Piping of Hydraulic Structures

• Stop Seepage
• Provide a way for seepage flow
• Lengthen the seepage path
• Piles/cutoff wall
• U/S and Downstream Floors