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As this is more than 1, the provision of canal lining is economically justifiable.

Example 16.10. *Determine the leaching requirement when the electrical conductivity of drainage water from soil was found as 20 m mho/cm at 20% reduction in the yield of a crop. Take $E_{ci} = 1.5$ m mho/cm. What will be the required depth of water to be applied if the consumptive use is 55.5 mm ?*

Solution

$$L_R = \frac{E_{ci}}{E_{cd}} \times 100\% = \frac{1.5}{20} \times 100 = 7.5\%$$

$$D_i = \frac{D_c}{(1 - L_R)} = \frac{55.5}{1 - 0.075} = \mathbf{60\ mm}$$

Example 16.11. *The saturated extract of a soil is 11 m mho/cm at 25% reduction in the yield of a crop. If the electrical conductivity of irrigation water is 1.4 m mho/cm, estimate the leaching requirement. What will be the required depth of water to be applied to the field if the consumptive use requirements of the crop is 85 mm ? Assume suitable value of electrical conductivity of leaching water.*

Solution : Given : $E_{ci} = 1.4$ m mho/cm and $E_{ce} = 11$ m mho/cm

Let us assume electrical conductivity of leaching water (E_{cd}) as twice the electrical conductivity of saturated extract of soil, i.e $E_{cd} = 2\,E_{ce}$. Hence from Eq. 16.11 (*a*)

$$L_R = \frac{E_{ci}}{E_{cd}} = \frac{E_{ci}}{2\,E_{ce}} = \frac{1.4}{2 \times 11} \times 100 = \mathbf{6.4\%}$$

Now $$L_R = \frac{D_d}{D_i} = \frac{D_i - D_C}{D_i} = \frac{D_i - 85}{D_i} \times 100\%$$

Hence $$\frac{D_i - 85}{D_i} \times 100 = 6.4$$

$\therefore$ $$6.4\,D_i = 100\,D_i - 8500$$

From which $$D_i = \mathbf{90.8\ mm}$$

16.18. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 16.12. *A trapezoidal channel with side slopes 1.5 horizontal : 1 vertical is required to carry 15 m^3/s of flow with a bed slope of 1 in 4000. If the channel is lined, the Manning's coefficient n will be 0.014 and it will be 0.028 if the channel is unlined. Calculate the average boundary shear stress if a hydraulically efficient lined channel in adopted. What percentage of earth work is saved in a lined section relative to an unlined section when hydraulically efficient section is used in both cases ? The free board can be assumed to be 0.75 m in both cases and the lining can be assumed to be upto the top of the section.* ***(Civil Services Exam. 1991)***

Solution

A *hydraulically efficient* channel is the one which has minimum perimeter for a given area of cross-section (or for a given depth D). Let B be the bed width.

For side slope of 1.5 (*H*) : 1 (*V*),

Length of sloping side $= (1.5^2 + 1^2)^{1/2} D \simeq 1.8\,D$

$\therefore$ $P = B + 2\,(1.8\,D) = B + 3.6\,D$ ...(*i*)

Also, $A = (B + 1.5\,D)\,D$ ...(*ii*)

or $$B = \frac{A}{D} - 1.5\,D$$

Hence $$P = \left(\frac{A}{D} - 1.5\,D\right) + 3.6\,D$$

$$= \frac{A}{D} + 2.1\,D \qquad ...(iii)$$

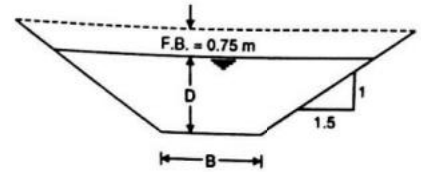

FIG. 16.9

For minimum perimeter for a given area, $\frac{dP}{dD} = 0$.

$\therefore$ $$\frac{dP}{dD} = A\,(-1\,D^{-2}) + 2.1 = 0$$

or $$\frac{A}{D^2} = 2.1 \text{ which given } A = 2.1\,D^2 \qquad ...(iv)$$

Hence from (*iii*) $$P = \frac{21\,D^2}{D} + 2.1\,D = 4.2\,D$$

$$R = \frac{A}{P} = \frac{2.1\,D^2}{4.2\,D} = \frac{D}{2} \qquad ...(v)$$

and $$B = \frac{A}{D} - 1.5\,D = \frac{2.1\,D^2}{D} - 1.5\,D = 0.6\,D \qquad ...(vi)$$

Hence for a most efficient section with 1.5 (*H*) : 1 (*V*) side slopes, we have

$$A = 2.1\,D^2,\ R = D/2 \text{ and } P = 4.2\,D$$

For lined channel : $n = 0.014$

Now $$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

$\therefore$ $$15 = \frac{1}{0.014}(2.1\,D^2)\left(\frac{D}{2}\right)^{2/3}\left(\frac{1}{4000}\right)^{1/2}$$

or $$D^{8/3} = 10.0396 \text{ from which } D = \mathbf{2.375\ m}$$

$$B = 0.6\,D = 0.6 \times 2.375 = \mathbf{1.425\ m}$$

and $$R = D/2 = 2.375/2 = 1.188 \text{ m}$$

Average boundary shear stress is given by Eq. 14.32

$$\tau = wRS = 9.81 \times 1.188 \times \frac{1}{4000} = \mathbf{2.912 \times 10^{-3}\ kN/m^2 = 2.912\ N/m^2}$$

Taking a F.B. of 0.75 m, the total depth of cutting $D = 2.375 + 0.75 = 3.125$ m

$\therefore$ $$A_1 = (B + 1.5\,D)\,D = (1.425 + 1.5 \times 3.125)\,3.125 = 19.1 \text{ m}^2$$

For unlined channel : $n = 0.028$

$\therefore$ $$15 = \frac{1}{0.028}(2.1\,D^2)\left(\frac{D}{2}\right)^{2/3}\left(\frac{1}{4000}\right)^{1/2} \text{ from which } D \simeq \mathbf{3.08\ m}$$

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$B = 0.6 \times 3.08 = \mathbf{1.848\ m}$

$R = 3.08/2 = 1.54\,\text{m}$

$\tau = 9.81 \times 1.54 \times 1/4000 = 3.777 \times 10^{-3}\ \text{kN/m}^2 = \mathbf{3.777\ N/m^2}$

$y = D + 0.75 = 3.08 + 0.75 = 3.83\ \text{m}$

$A_2 = (1.848 + 1.5 \times 3.83)\,3.83 = 29.08\ \text{m}^2$

Hence percentage saving in earthwork done due to lining

$$= \frac{A_2 - A_1}{A_2} = \frac{29.08 - 19.1}{29.08} \times 100 = \mathbf{34.32\%}$$

Example 16.13. *Design a lined canal to carry 100 m^3/s on a slope of 1 in 2500. The maximum permissible velocity is 2 m/s, n = 0.013 in Manning's formula and the side is 1.25 H : 1.0 V.* **(Engg. Services Exam. 1996)**

Solution : Given $V = 2$ m/s, $n = 0.013$.

The canal section is shown in Fig. 16.10 from which

$$A = D \times \tfrac{1}{2}[B + (B + 2.5\,D)] = D\,(B + 1.25\,D) \quad ...(1)$$

$$P = B + 2\sqrt{(1.25\,D)^2 + D^2} = B + 3.2\,D \quad ...(2)$$

But required $A = Q/V = 100/2 = 50\ \text{m}^3$. Hence from (1), we get

$$D\,(B + 1.25\,D) = 50 \quad ...(3)$$

From Manning's formula

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

$$\therefore \quad 2 = \frac{1}{0.013} R^{2/3} \left(\frac{1}{2500}\right)^{1/2}$$

$R^{2/3} = 1.3$, from which

$R = (1.3)^{3/2} = 1.482$

$$\therefore \quad P = A/R = \frac{50}{1.482} = 33.73\ \text{m}$$

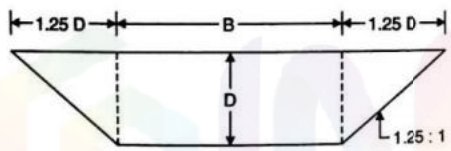

FIG. 16.10.

Hence from (2), $B + 3.2\,D = P = 33.73$ m ...(4)

Substituting the value of B from (4) into (3), we get

$D\,[33.73 - 3.2\,D + 1.25\,D] = 50$

or $1.95\,D^2 - 33.73\,D + 50 = 0$, from which $D = 1.637$ m

Hence from (4) $B = 33.73 - 3.2 \times 1.637 = 28.49$ m

Check : $A = 1.637\,(28.49 + 1.25 \times 1.637) = 50\ \text{m}^2$ Hence OK.

Hence $B = 28.49 \triangleq \mathbf{28.5\ m}$ and $D = 1.637 \triangleq \mathbf{1.64}$

Example 16.14. *A lined rectangular channel with n = 0.015 is 5 m wide and has a flow depth of 2 m with the bed slope of 1 in 1600. Retaining the rectangular shape of channel section and the same total area of lining, to what maximum extent*

can be discharge be increased without changing the bed slope ? Does the flow change from sub-critical to super-critical by this increase in discharge ? **(Gate 1988)**

Solution : Given $B = 5$ m, $D = 2$ m; $S = 1/1600$

Area of lining per m run $= B + 2\,D = 5 + 2 \times 2 = 9\ \text{m}^3/\text{m}$

Let B_1 be the new width and D_1 be the new depth.

Area of lining per m run $= B_1 + 2\,D_1$

Since area of lining is constant, $B_1 + 2\,D_1 = B + 2\,D = 9$

Hence $B_1 = 9 - 2\,D_1$ ...(1)

Now $Q = \frac{1}{n}(B_1 D_1) R_1^{2/3} S^{1/2}$

For a rectangular channel, $R_1 = D_1$

$$\therefore \quad Q = \frac{1}{n} S^{1/2} B_1 D_1^{5/3} = \frac{1}{n} S^{1/2} (9 - 2\,D_1)\,D_1^{5/3}$$

or $$Q = \frac{S^{1/2}}{n}\,[9\,D_1^{5/3} - 2\,D_1^{8/3}]$$

In order to obtain maximum discharge, differentiate Q w.r. to D_1.

$$\therefore \quad \frac{dQ}{dD_1} = 0 = \frac{S^{1/2}}{n}\left[\frac{9 \times 5}{3} D_1^{2/3} - \frac{2 \times 8}{3} D_1^{5/3}\right]$$

or $$15\,D_1^{2/3} = \frac{16}{3} D_1^{5/3} \quad \text{or} \quad D_1 = \frac{45}{16} = \mathbf{2.8125\ m}$$

Hence $B_1 = 9 - 2 \times 2.8125 = \mathbf{3.375\ m}$

Now, earlier, $$V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.015}(2)^{2/3}\left(\frac{1}{1600}\right)^{1/2} = 2.646\ \text{m/s}.$$

$$\text{Froude No.} = \frac{V}{\sqrt{g\,D}} = \frac{2.646}{\sqrt{9.81 \times 2}} = 0.597 < 1.$$

Later, $$V_1 = \frac{1}{0.015}(2.8125)^{2/3}\left(\frac{1}{1600}\right)^{1/2} = 3.32\ \text{m/s}$$

$$\therefore \quad \text{Froude No.} = \frac{3.32}{\sqrt{9.81 \times 2.8125}} = 0.632 < 1$$

Hence the flow does not change from sub-critical to super-critical.

Example 16.15. *A lined irrigation canal with trapezoidal cross-section has 5 m bed width, 2.5 m depth and 1.5 (H) : 1 (V). Longitudinal bed slope of the canal is 1 in 1000 and Manning's n = 0.16. What is the maximum carrying capacity of the canal? What area of land in hectares the canal can irrigate if the crop has 150 mm field irrigation requirement in a Kor period of 10 days.* **(Engg. Services Exam. 2001)**

Solution : For 1.5 (H) : 1 (V) side slopes, $\cot\theta = 1.5$ and $\theta = 0.588$ radian

Also, $D = 2.5$ m and $B = 5$ m

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$\therefore$ $A = BD + D^2(\theta + \cot\theta) = 5 \times 2.5 + 6.25\,(0.588 + 1.5) = 25.55\ \text{m}^2$

$P = B + 2D(\theta + \cot\theta) = 5 + 2 \times 2.5\,(0.588 + 1.5) = 15.44\ \text{m}$

$R = A/P = 25.55/15.44 = 1.655\ \text{m}$

Now, $Q = V \times A = \frac{1}{n} AR^{2/3}\sqrt{S}$

$$= \frac{1}{0.016} \times 25.55\,(1.655)^{2/3}\sqrt{1/1000} = \mathbf{70.65\ m^3/s}$$

Taking Kor period of 10 days, volume of water supply by the channel

$$= 70.65 \times 10 \times 24 \times 3600 = 61.046 \times 10^6\ \text{m}^3$$

$\therefore$ Area to be irrigated $= \frac{\text{Volume}}{\text{depth}} = \frac{61.046 \times 10^6}{0.15} = \mathbf{407 \times 10^6\ m^2}$

$= \mathbf{40700}$ **hectares**

PROBLEMS

1. What is waterlogging ? What are its ill-effects ?
2. What are causes of waterlogging ?
3. Describe various methods adopted as anti-waterlogging measures.
4. What to you understand by 'land drainage ?
5. Explain the methods of designing of various drains.
6. What is canal lining ? what are its advantages ?
7. Write the requirements of good lining material.
8. Enumerate various types of linings used for canal.
9. Design the methods of design of a lined canal.
10. Design a trapezoidal shaped concrete lined channel to carry a discharge of 40 cumecs at a slope of 10 cm/km. The side slopes of the channel are $1\frac{1}{2}:1$. The value of N may be taken as 0.015. Assume a limiting velocity as 1.5 m/sec.
11. Redesign the channel of problem 10 if instead of the limiting velocity, the limiting B/D ratio is to be kept as 8.
12. Redesign the channel of problem 10 if channel is to be triangular shaped.

17

Canal Outlets

17.1. INTRODUCTION

An outlet is a small structure which admits water from the distributing channel to a water course or field channel. Thus, an outlet is a sort of head regulator for the field channel delivering water to the irrigation fields. The responsibility of maintenance of the distributing channel and the whole canal network lies with Government, while that of the field channel lies with the farmer. The outlet is the connecting medium for the two.

Since the efficiency of a canal network mostly depends upon functioning of the outlets and other regulation works, a canal *outlet should fulfill the following requirements*:

1. The outlet should be strong, with no moving parts liable to be damaged or requiring periodic attention and maintenance.
2. It should be so designed that the farmer cannot temper with its functioning and any interference from him should be easily detectable.
3. Since a large number of outlets are fixed on a distributing channel, the most essential requirement is that it should be cheap.
4. The design should be simple so that it can be constructed or fabricated by local masons or technicians.
5. It should be possible for the outlet to work efficiently with a small working head.
6. The outlet should draw its fair share of silt carried by the distributing channel.
7. From the farmer's point of view, the outlet should give a fairly constant discharge. However, from the canal regulation point of view, the outlet should draw proportionately more or less discharge with the varying supply in the distribution channel.

17.2. TYPES OF OUTLETS

Outlets may be classified under the following there heads :

1. Non-modular Outlet
2. Semi-module or Flexible Module
3. Rigid Module.

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Non-modular Outlet : A non-modular outlet is the one in which the discharge depends upon the difference in level between the water levels in the distributing channel and the water course. The discharge through such an outlet varies in wide limits with the fluctuations of the water levels in the distributing and the field channels. The common examples under this category are : submerged pipe outlet, masonry sluice and orifices, and wooden shoots.

Semi-module or Flexible outlet. A flexible outlet or semi-module is the one in which the discharge is affected by the fluctuations in the water level of the distributing channel while the fluctuations in the water levels of the field channel do not have any effect on its discharge. The various outlets in common use that fall under this category are pipe outlet, Kennedy's gauge outlet, Crump's open flume outlet and pipe-cum open flume outlet.

Rigid Module. A rigid module is the one which maintains constant discharge, within limits, irrespective of the fluctuations in water levels in the distributing channel and/or field channel. The most common outlet that falls under this category is the Gibb's rigid module.

17.3. IMPORTANT DEFINITIONS

To understand the criteria for judging the behaviour and function of outlets, the following definitions are useful :

1. Flexibility : It is ratio of rate of change of discharge of an outlet to the rate of change of the discharge of the distributing channel.

Thus,
$$F = \frac{dq}{q} \Big/ \frac{dQ}{Q} \qquad ...(17.1)$$

where F = Flexibility
q = Discharge through the outlet
Q = Discharge of the distributing channel.

Now, for the field channel.
$$q = kH^m$$

where k = constant, m = outlet index
and H = head acting on the outlet.

$$\therefore \quad dq = mkH^{m-1}\,dH$$

$$\therefore \quad \frac{dq}{q} = \frac{mkH^{m-1}\,dH}{kH^m} = m\frac{dH}{H} \qquad ...(1)$$

Similarly, for the parent channel.
$$Q = CD^n$$

where C = constant and n = canal index
D = depth of water in the canal.

$$\therefore \quad dQ = n\,CD^{n-1}\,dD$$

$$\therefore \quad \frac{dQ}{Q} = \frac{n\,CD^{n-1}\,dD}{CD^n} = n\frac{dD}{D} \qquad ...(2)$$

Dividing (1) and (2), we get

$$F = \frac{\frac{dq}{q}}{\frac{dQ}{Q}} = \frac{m\frac{dH}{H}}{n\frac{dD}{D}} = \frac{m}{n}\frac{D}{H}\frac{dH}{dD}$$

Since any change in the water depth results in an equal change in the head causing flow, we have $dH = dD$. Thus the expression for flexibility becomes :

$$F = \frac{m}{n}\frac{D}{H} \qquad ...(17.2)$$

2. Proportionality

A *Proportional outlet* is the one on which the flexibility (F) is equal to unity. Thus, in a proportional outlet, the rate of change of its discharge is equal to the rate of change of the discharge of the distributing channel. For proportionality, putting $F = 1$ in Equation 17.2, we get the relation.

$$\frac{H}{D} = \frac{m}{n} \qquad ...(17.3)$$

The ratio H/D is known as the *setting*. In a proportional outlet therefore, setting is equal to the ratio of the outlet and the canal indices.

From the point of view of proportionality, an outlet is classified into three types:

(*a*) Proportional outlet
(*b*) Hyper-proportional outlet
(*c*) Sub-proportional outlet

These will be explained both with respect to their *flexibility* as well as *setting*.

3. Setting

Setting is the ratio of the depth of the sill or the crest level of the module below the full supply level of the distributing channel to the full supply depth of the distributing channel.

$$\text{Thus, setting} = \frac{H}{D}.$$

In the case of a proportionate outlet we have the setting equal to the ratio of outlet index to the channel index, i.e.

$$\text{Setting} = \frac{H}{D} = \frac{m}{n}.$$

For a channel of trapezoidal shape, $Q \propto D^{5/3}$, i.e. $n = 5/3$

For an orifice type outlet, $q \propto H^{1/2}$, i.e. $m = 1/2$

Hence, for such a combination, we get

$$\text{Setting} = \frac{H}{D} = \frac{m}{n} = \frac{1/2}{5/3} = 0.3$$

Hence, for the pipe outlet to be proportional, the outlet is set at 0.3 times the depth below the water surface.

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4. Hyper proportional outlet

A hyper proportional outlet is the one in which the flexibility is greater than one, i.e. the discharge in the outlet changes by a larger percentage than the percentage change in the discharge of the distributing channel. Thus, for a hyper proportional outlet

$$F > 1$$

$$\therefore \quad \frac{m}{n}\frac{D}{H} > 1$$

or

$$\frac{H}{D} < \frac{m}{n} \qquad ...(17.4)$$

Eq. 17.4 shows that the numerical value of setting (H/D) is less that the ratio m/n or in other words, an outlet is hyper proportional if the setting is higher than required for proportionality.

5. Sub-proportional outlet

A sub proportional outlet is the one in which the flexibility is lesser than one, i.e. the discharge in the outlet changes by a smaller percentage than the percentage change in the discharge of the distributing channel. Thus, for a sub-proportional outlet:

$$F < 1$$

$$\therefore \quad \frac{m}{n}\frac{D}{H} < 1$$

or

$$\frac{H}{D} > \frac{m}{n} \qquad ...(17.5)$$

Eq. 17.5 shows that the numerical value of setting H/D is greater than ratio m/n, or in other words, an outlet is sub-proportional if the setting is lower than required for proportionality.

6. Sensitivity

It is defined as the ratio of the rate of change of discharge of an outlet to the rate of change in the level of the distributing surface, referred to the normal depth of the channel. Thus,

$$S = \frac{dq/q}{dG/D} \qquad ...(17.6)$$

where S = Sensitivity of the outlet
q = Discharge through the outlet
dq = Change in the discharge of the outlet
G = Gauge reading, so set that $G = 0$ when $q = 0$
D = Depth of water in the distributing channel.

$$\therefore \quad dG = dD.$$

$$S = \frac{dq/q}{dD/D} \qquad ...(1)$$

But

$$F = \frac{dq/q}{dQ/Q}, \text{ where } \frac{dQ}{Q} = n\frac{dD}{D}$$

$$\therefore \quad F = \frac{dq}{q} \Big/ n\frac{dD}{D} \qquad ...(2)$$

Comparing (1) and (2), we get

$$S = nF \qquad ...(17.7)$$

It is evident from Eq. 17.7 that the sensitivity of a rigid module is zero.

7. Efficiency

Efficiency is the measure of a conservation of head by an outlet. It is defined as the ratio of the head recovered to the head put in. Less is the working head required for the functioning of the outlet, more will be its efficiency.

8. Drowning ratio

It is ratio between the depths of water level over crest on the downstream and upstream of the module.

9. Minimum modular head

The minimum modular head or the *minimum modular loss* is the minimum loss of head or the difference between the upstream and downstream water levels which is absolutely necessary to be maintained to enable the module to pass its design discharge.

10. Modular limits and modular range

The *modular limits* of an outlet are the upper and the lower limits of any one or more factors beyond which an outlet is incapable of acting as a module or semi-module. *Modular range* is the range between the modular limits. It is the range of various factors which a module or semi-module works as designed.

17.4. NON-MODULAR OUTLETS : PIPE OUTLET

The most common types of non-modular outlets are : submerged pipe outlet, masonry sluice and orifices, and wooden shoots. Fig. 17.1 shows a pipe outlet. The pipes vary from 10 to 30 cm in diameter, and are frequently laid on a light concrete foundation to prevent uneven settlement and consequent leakage. They are generally fixed horizontally at right angles to the direction of flow.

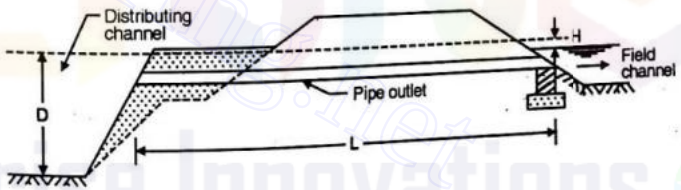

FIG. 17.1. PIPE OUTLET

The loss of head, H through the outlet is given by

H = (Entry loss) + (Frictional loss) + (Velocity head at the exit)

$$H = 0.5\frac{v^2}{2g} + \frac{4fLv^2}{2gd} + \frac{v^2}{2g} \qquad ...(17.8\,a)$$

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or
$$H = \frac{V^2}{2g}\left[1.5 + \frac{4fL}{d}\right] \quad ...(17.8)$$

From which
$$V = \sqrt{2gH}\left(\frac{d}{1.5d + 4fL}\right)^{1/2}$$

The discharge q through the pipe outlet is given by
$$q = A \times V = A\sqrt{2gH}\left(\frac{d}{1.5d + 4fL}\right)^{1/2} \quad ...(17.9)$$

or
$$q = CA\sqrt{2gH} \quad ...(17.9\ a)$$

where C = Coefficient of discharge $= \left(\frac{d}{1.5d + 4fL}\right)^{1/2}$

L = Length of the pipe (in m)

d = Diameter of the pipe (in m)

A = Area of cross-section of the piper (in m^2)

If, however, the diameter of the pipe is expressed in cm, the coefficient of discharge is given by
$$C = \frac{1}{20}\left[\frac{d}{\left(\frac{1.5d}{400f} + L\right)f}\right]^{1/2} \quad ...(17.9\ b)$$

Indian Standard (IS : 7986-1976) has given the above expression for the coefficient of discharge, which depends on friction factor, length of the pipe and size of the pipe outlet.

In all the above expressions, f is the *coefficient of fluid friction* for pipes. Its value would be 0.005 and 0.01 for clean and slightly encrusted iron pipes respectively. For earthen ware pipes its value can be taken as 0.0075.

For example, if d = 20 cm, L = 10 m and f = 0.005 we get
$$C = \frac{1}{20}\left[\frac{20}{\left(\frac{1.5 \times 20}{400 \times 0.005} + 10\right)0.005}\right]^{1/2} = 0.632$$

Similarly, if d = 25 cm, L = 5 m and f = 0.005, we get
$$C = \frac{1}{20}\left[\frac{25}{\left(\frac{1.5 \times 25}{400 \times 0.005} + 5\right)0.005}\right]^{1/2} = 0.725$$

The value of the coefficient of discharge varies from 0.61 to 0.76. Its average value, determined at CWPRS, Poona is 0.73.

It is a common practice to place the pipe at the bed of the distributing channel to enable the outlet to draw fair share of silt charge. In U.P., pipes are generally laid horizonally at about 22 cm below the water surface of the distributing channel.

The inlet and exit ends of the pipe should be fixed in masonry to prevent tampering. Fig. 17.2 shows the typical layout of a pipe out let as recommended by Indian Standard, IS : 7986-1976.

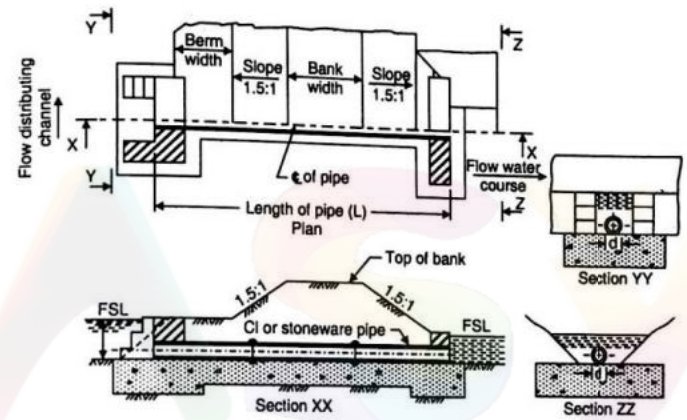

FIG. 17.2. TYPICAL LAYOUT OF PIPE OUTLET

It is apparent from Fig. 17.1 that the discharge of the outlet can be easily increased by digging the head of the water course and thereby lowering the water level in it. This will increase the head H and therefore, the discharge will be increased. On the other hand, the farmers having higher land will get lesser discharge, since the head H is decreased due to heading up of the water. As the discharging capacity of the outlet is quite uncertain, proper and equitable distribution of water is very difficult. Non-modular outlets, however, can work with very small available heads. Due to considerable disturbance at the entrance to the pipe outlet, they usually take their due share of silt. In order to increase the silt conductivity further, the upstream end of the pipe is depressed, making the pipe to slope upwards as shown in Fig. 17.3. So long as

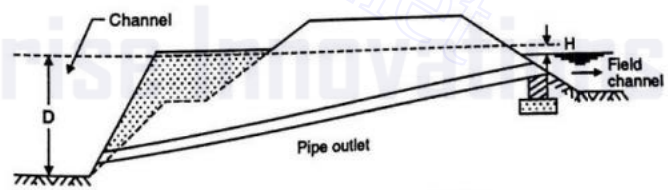

FIG. 17.3. TILTED PIPE OUTLET

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the head H remains the same, this arrangement will not change the discharge, while the silt conductivity is increased.

Example 17.1. *A submerged pipe outlet has the following data :*

F.S.L. of distributory $= 100.00$ m
F.S.L. of water course $= 99.90$ m
Length of the pipe $= 9$ m
Diameter of the pipe $= 20$ cm
Coefficient of friction $= 0.005$
Find the discharge through the outlet.

Solution

Given : $d = 20$ cm, $L = 9$ m; $f = 0.005$

Working head $H = 100.00 - 99.90 = 0.1$ m

$$\therefore \quad C = \frac{1}{20}\left[\frac{d}{\left(\frac{1.5\,d}{400\,f} + L\right)f}\right]^{1/2} = \frac{1}{20}\left[\frac{20}{\left(\frac{1.5 \times 20}{400 \times 0.005} + 9\right)0.005}\right]^{1/2} = 0.645$$

Area of cross-section of pipe $A = \frac{\pi}{4}d^2 = \frac{\pi}{4}(0.2)^2 = 0.0314 \text{ m}^2$

Hence $q = CA\sqrt{2gH} = 0.645 \times 0.0314\sqrt{2 \times 9.81 \times 0.1} = \mathbf{0.0284\ cumec}$

Example 17.2. *Design a submerged pipe outlet for the following data :*

Discharge through outlet $= 0.04$ *cumecs*
F.S.L. of distributing canal $= 100.00$ m
F.S.L. of water course $= 99.90$ m
Full supply depth of distributing canal $= 1.1$ m
Assume an average value of coefficient of discharge as 0.7.

Solution : Given $q = 0.04$ cumec and $C = 0.7$

Available head, $H = 100.00 - 99.90 = 0.10$ m

$$q = CA\sqrt{2gH}$$

$$\therefore \quad 0.04 = 0.7\,A\sqrt{2 \times 9.81 \times 0.1}$$

From which $A = 0.0408 \text{ m}^2 = \frac{\pi}{4}d^2$

$$\therefore \text{ Pipe dia.} \quad d = \sqrt{\frac{4 \times 0.0408}{\pi}} = 0.228 = 22.8 \text{ cm}$$

Hence use a pipe of **25 cm** diameter.

Example 17.3. *Redesign the submerged pipe outlet for the data of Example 17.2 if old C.I. pipe is used for which* $f = 0.01$

Solution : Given : $q = 0.04$ cumecs and $f = 0.01$

The coefficient of discharge depends upon f, L and d. Since d is not given let us assume $C = 0.7$, to start with. Then, as found in Example 17.2, we get $d = 0.228$ m $= 22.8$ cm. Using this diameter for further trial, we get

$$C = \frac{1}{20}\left[\frac{d}{\left(\frac{1.5\,d}{400\,f} + L\right)f}\right]^{1/2} = \frac{1}{20}\left[\frac{22.8}{\left(\frac{1.5 \times 22.8}{400 \times 0.01} + 9\right)0.01}\right]^{1/2} \simeq 0.57$$

In the next trial, taking $C = 0.57$, we get

$$0.04 = 0.57\,A\sqrt{2 \times 9.81 \times 0.1}, \text{ from which } A = 0.05 \text{ m}^2$$

$$\therefore \quad d = \sqrt{\frac{4 \times 0.05}{\pi}} \simeq 0.25 \text{ m}$$

Using this diameter and substituting in Eq. 17.9 (*b*), we get $C = 0.583$

$$\therefore \quad 0.04 = 0.583\,A\sqrt{2 \times 9.81 \times 0.1} \text{ from which } A = 0.049 \text{ m}^2$$

$$\therefore \quad d = \sqrt{\frac{4 \times 0.049}{\pi}} \simeq 0.25 \text{ m} = 25 \text{ cm}$$

Hence provide a pipe of **25 cm** dia.

17.5. SEMI-MODULE OR FLEXIBLE OUTLETS

In the case of a flexible outlet, the discharge is affected by the change in the water level of the distributing channel but not with the change in the water level of the field channel. The following are the common types of flexible outlets :

(*i*) Pipe outlet discharging freely in the atmosphere,
(*ii*) Kennedy's gauge outlet.
(*iii*) Open flume outlet
(*iv*) Orifice semi-modules.

(1) Pipe outlet discharging freely

If the pipe outlet is so set that it discharges freely in the atmosphere, the discharge through it becomes independent of the water level in the field channel, and hence it acts as semi-module. The discharge cannot be increased by the cultivator by digging the water course. Such outlets, therefore, worked well. Later, however, the farmers invented a method of increasing the discharge by constructing a ramp in the water-course thereby heading up the water to the top level of the pipe and drowning it.

For the outlet to be proportional, $F = 1$. Hence $\frac{H}{D} = \frac{m}{n}$. Taking $n = \frac{5}{3}$ and $m = \frac{1}{2}$, the setting is equal to 0.3. Usually a pipe outlet is set lower than this and is sub-proportional. The efficiency of the pipe outlet is high and its silt conduction is good.

(2) Kennedy's gauge outlet :

This was the earliest type of semi module, invented by R.G. Kennedy, Chief Engineer, Irrigation, Punjab, sometime in 1906. This outlet is made of cast iron and consists of three main parts (Fig. 17.4) :

1. An orifice with bellmouth entry.
2. A long expanding delivery pipe.
3. An intervening vertical air column above the throat.

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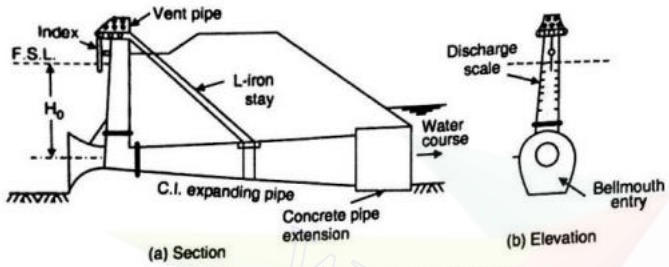

FIG. 17.4. KENNEDY'S GAUGE OUTLET

Water from the distributing channel enters the outlet through bell mouthed approach. From the narrowest diameter at the throat, the jet shoots across the air in the base of the vertical air vent pipe. The base of the air vent pipe is conical and allows free circulation of air around the jet. Thus the jet is exactly in the same condition as if it were discharging into open air, making the discharge independent of the water-level in the water course. The jet is then received by a cast iron expanding pipe, about 3 m long. At the end of the C.I. pipe, cement concrete pipe extension is usually added and the water is then discharged in the field channel.

The discharge through this semi-module is given by

$$q = Ca\sqrt{2g H_0} \qquad ...(17.10)$$

where a = Area of cross-sections of the pipe at the throat

C = Coefficient of discharge which may be as high as 0.97.

It has been found by experiments that the losses of head by friction, eddies, etc., are about $0.22\,H_0$ and that the available working head or the minimum modular head should not be less than this value.

This module has however, been superseded because its discharge could very easily be increased by closing air vents of the vent pipe. After the air vents are closed, the jet will suck the air of the chamber and its pressure drops below the atmosphere. This increases the discharge. Some safeguards like enclosing the air holes in a chamber and fixing the pipe to angle irons embedded in ground to prevent tampering were provided. This worked for sometime, but the farmer, later, found a method of plugging the vent pipe through the bell mouth entry. Due to this difficulty, and also due to high cost of its manufacture, this outlet is no longer in use.

(3) Open flume outlets :

An open flume outlet is a sort of weir with constricted throat at expanding flume on the downstream. Due to this, hyper-critical velocity is ensured and hydraulic jump is formed. The formation the hydraulic jump makes the discharge through the outlet independent of the water-level in the water course, making it a semi-module.

There are two types of open flume outlets :

(*i*) Crump's open flume outlet.

(*ii*) Punjab open flume outlet.

Crumps open flume outlet

Fig. 17.5 shows the plan and longitudinal section of Crump's open flume outlet. The upstream face is set back by a distance x equal to the width of the opening of the module. This enables the outlet to take a fair share of silt. This throat is constricted to ensure hyper-critical velocity. The length of the throat is kept equal to 2.5 H.

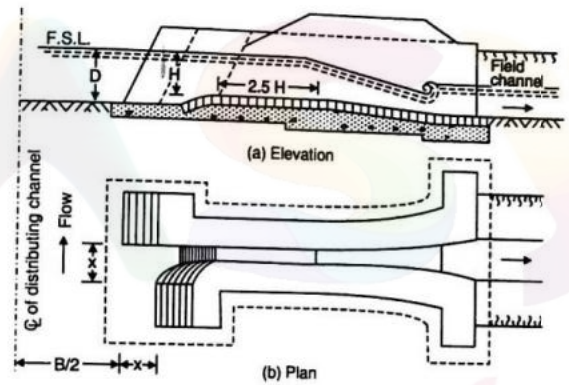

FIG. 17.5. CRUMP'S OPEN FLUME OUTLET

The discharge is given by the expression.

$$q = CB_t H^{3/2} \qquad ...(17.11)$$

where B_t = Throat width (metres)

C = Coefficient of discharge, the theoretical value of which is equal to 1.71.

q = Discharge through the outlet, in cumecs

H = Head over the crest, in metres.

Due to various losses, the actual value of C is less than 1.71 and depends upon the width of the throat, as indicated below :

Throat width	*Value of C*
0.06 to 0.09 m	1.60
0.09 to 0.12 m	1.64
over 0.12 m	1.66

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For proportionality, $F = 1$

$$\therefore \quad \frac{H}{D} = \frac{m}{n} = \frac{3/2}{5/3} = 0.9$$

Thus proportionality is obtained when setting or $H/D = 0.9$. If the outlet is set higher than this, it is hyper-proportional, and if set lower it is sub-proportional. The efficiency of the outlet varies between 80 to 90%. The minimum modular head for open flume outlets is 0.2 H or 20% of the head over the crest of the outlet with a minimum value of 6 cm. The minimum throat width is 0.06 m, to avoid the choking of the outlet by floating material. Hence high setting is adopted for small requirements of discharge, though this would result in poor silt conduction.

Punjab open flume outlet

Fig. 17.6. shows the modified form of Crump's open flume throat, commonly used in Punjab. The length of the throat is equal to $2H$. The u/s approach is made further curved.

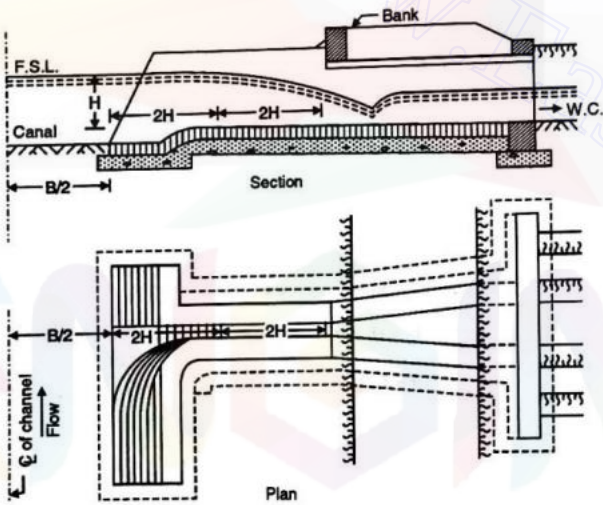
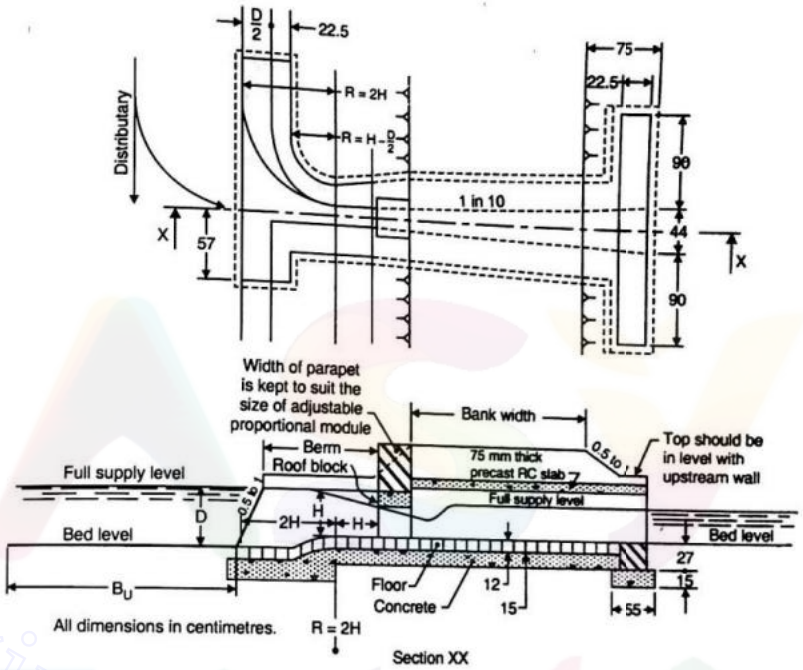


FIG. 17.6. PUNJAB OPEN FLUME OUTLET.

Open flume outlet with roof block

Fig. 17.7 shows as open flume outlet with roof block, as recommended by Indian Standard, IS : 7986-1976. The roof block is fitted over the passage of the flume at vena contracta such that it remains clear of the water surface when the outlet is discharging the full design discharge. A roof block is basically provided to over come the high flexibility of a simple open flume outlet. It is fitted at a distance H from the d/s end of the throat, in such a way that the bottom of the roof block is at a height of 0.7 H above the crest. Also, it should have a square edge at its bottom surface.



(All dimensions in cm)

FIG. 17.7. OPEN FLUME OUTLET WITH ROOF BLOCK.

(4) Orifice semi-module

An orifice semi-module consists of an orifice provided with a gradually expanding flume on the downstream side of the orifice. The flow through the orifice is hypercritical, resulting in the formation of hydraulic jump. This makes the discharge independent of the flow conditions in the water-course. There are various forms of orifice semi-module—the earliest of these being Crump's *adjustable proportional module* (commonly known as Crump's A.P.M.). Fig. 17.8 shows Crump's A.P.M. recommended by Indian Standard IS: 7986-1976.

This type is the most commonly used outlet under this class. In this type of outlet, C.I base, C.I. roof block and check plates on either side form the nucleus around which masonry is built. The roof block is fixed to the check plates by bolts which can be removed and the depth of the outlet adjusted after the masonry is dismantled. This type of outlet cannot be easily tempered with and at the same time can be conveniently adjusted at a very small cost. In Fig. 17.8, W is given by the following expression:

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$$W = k\,\frac{q}{Q}\,(B + 0.5\,D)$$

where k is given as under for various discharge Q :

Discharge Q (cumecs)	k
< 0.283	1.00
Over 0.283 and up to 1.415	1.25
Over 1.415 and up to 5.660	1.50
Over 5.660	2.0

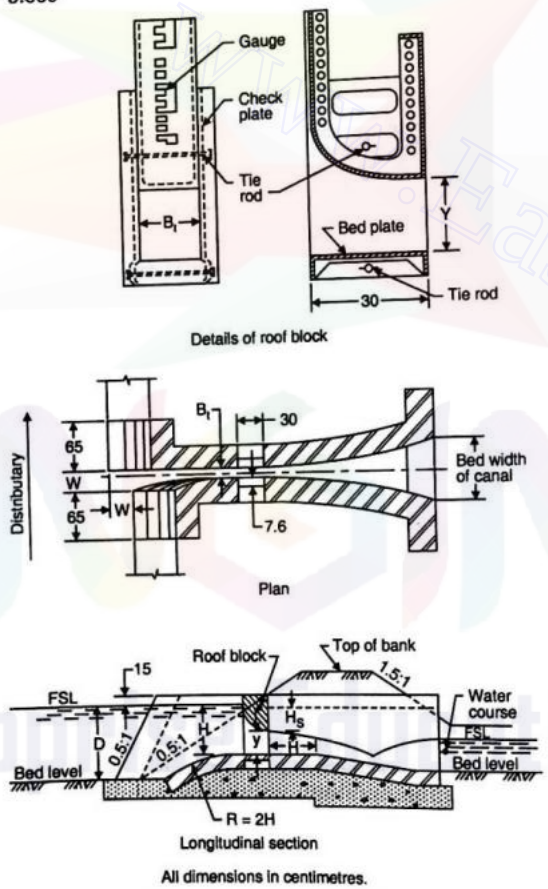

FIG. 17.8. CRUMP'S ADJUSTABLE PROPORTIONAL MODULE

The discharge through the outlet is given by

$$q = 4.03\,B_t\,y\,\sqrt{H_s} \qquad \text{...(17.12)}$$

where q = discharge in cumecs

B_t = width of throat in metres

y = Height of the smallest section in metres

H_s = Head measured from the water surface to the lowest point of the roof block in metres.

The ratio H_s/D should be between 0.375 to 0.48 for proportionate distribution of silt. Also, the ratio H_s/D should be 0.8 or less for modular working.

The above plates and the roof block are manufactured in standard sizes, such as B_t = 6.1 , 7.6 , 9.9 , 12.2 , 15.4 , 19.5 , 24.4 and 30.5 cm. These standards sizes with required opening of the orifice are used to obtain desired supply through the outlet.

The disadvantage of this outlet is that the water way is either deep and narrow which could get blocked easily or is shallow and wide in which case it does not draw its fair share of silt. Also, it is observed that channel fitted with this type of outlets get silted up very badly.

Adjustable orifice semi-module (A.O.S.M.)

It is a *modified* form of Crump's Adjustable Proportionate Module (Crump's A.P.M.) with the following modifications : (*i*) the crest of the outlet is lowered so that the setting is 8/10 or even lower, to improve silt draw (*ii*). The width B_t of the gullet (passage) is kept constant for a distance of 0.6 m after which the side walls are splayed out with a radius of 7.5 m (*iii*) There is no horizontal portion of the throat, and the glacis is given a slope of 1 in 15 until it meets the bed of the water course, and (*iv*) the roof block is shaped to a lemiscate curve with a tilt of 1 in 7.5 to ensure smooth conveyance of the flow.

The minimum modular head, as given by N.D. Gulati, is

$$\text{M.M.H.} = 0.82\,H_s - 0.5\,B_t \qquad \text{...(17.13)}$$

For proportionality.

$$\frac{H_s}{D} = \frac{m}{n} = \frac{1/2}{5/3} = 0.3$$

Hence, for the outlet to be proportional, H_s is kept equal to 0.3 times the water depth in the distributing channel. With the rise in F.S.L., the flexibility is reduced and the out let becomes sub-proportional. However, if the F.S.L. falls, the flexibility is increased and the outlet becomes hyper proportional. However in both the types of orifice semi-module, the roof block can be raised or lowered for adjustments at a very small cost by dismantling the masonry in which the roof bolts are fixed. Any tampering of the roof block by the farmers can be easily detected.

Example 17.4. *Design an open flume outlet with a discharge of 0.06 cumec across a channel having full supply depth of 0.85 m. Take available working head as 15 cm.*

Solution : Let us choose B_t = 7 cm = 0.07 m, for which C = 1.60

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Now $q = C B_t H^{3/2}$

or $0.06 = 1.6 \times 0.07 H^{3/2}$

From which $H = 0.66$ m

Now minimum modular head required for open flume outlets

$= 0.2 H = 0.2 \times 0.66 = 0.132$ m $= 13.2$ cm

This is less than the available working head of 15 cm. Hence the design is satisfactory.

$$\text{Setting of outlet} = \frac{H}{D} = \frac{0.66}{0.85} = 0.776 < 0.9$$

Hence the out let will work as hyper-proportional outlet.

17.6. RIGID MODULE

The three common types of rigid modules, having no moving parts are :

1. Gibb's rigid module.
2. Khanna's rigid module.
3. Foote module.

Gibb's module is described below.

Gibb's module

The outlet was designed by A.S. Gibb's, formerly Executive Engineer, Punjab Irrigation. Fig. 17.9 shows the plan and the section of the module. The essential feature of the outlet is an eddy chamber, semi-circular in plan, round which water flows giving rise to a free vortex flow. Water enters through an inlet pipe having bell mouth entry and is directed to the eddy chamber through a 180° rising pipe in which free vortex flow is developed. The characteristic feature of the free vortex flow is that the product of the velocity and radius is constant for all filaments (i.e. $V \times r =$ constant). Thus the water at the outer circumference of the chamber has greater radius and hence lesser velocity, resulting in rise of water level there. Thus the water surface in the eddy chamber slopes down towards the inner circumference. A series of baffles are suspended from the roof of the eddy chamber, with their lower edges sloping at the required height above the sill of the module. If the head causing flow increases, water banks up at the outer circumference of the eddy chamber and impinges against the baffles imparting an upward, rotational, direction of flow to the water, which spins round in the compartment between two successive baffles and finally drops on the oncoming stream of water, thus dissipating excess energy. This keeps discharge constant for a wide range of variation in the head. The number of baffles coming into action depends upon the variations in the head causing flow. The angle of eddy chamber varies from a semi-circle to $1\frac{1}{2}$ complete turns depending upon the discharge and the range of working required for the module.

Gibb give the following formula for the discharge through the outlet :

$$q = r_0 \sqrt{2g}\,(d_i + h_0)^{1.5} \left\{ \frac{m^2 - 1}{m^3} \log_e m + \frac{1}{m} \log_e m - \frac{m^2 - 1}{2\,m^2} \right\} \quad ...(17.14)$$

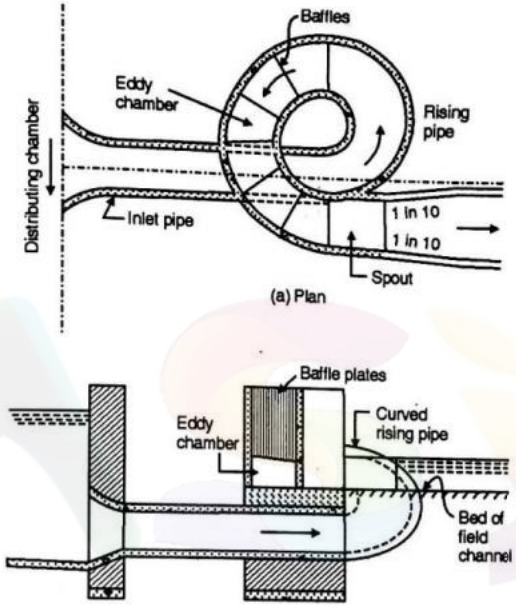

FIG. 17.9. GIBB'S MODULE

where r_0 = radius of outer semi-circle

r_i = radius of inner semi-circle.

$m = \frac{r_0}{r_i}$

d_i = depth of water at inner circumference.

h_0 = head at outer circumference.

Gibb's formula is valid only for his standard design in which $m = 2$ and $\frac{h_0}{D} = \frac{1}{7}$.

PROBLEMS

1. What is an outlet ? Write down the requirements that an outlet should fulfill.
2. What do you understand by flexibility of an outlet ? Derive an expression for the same.
3. Define proportionality of an outlet. Distinguish between a hyper-proportional outlet and a sub-proportional outlet. Find out expressions for the setting of both the types.

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4. Define sensitivity of an outlet. Find the relation between sensitivity and flexibility of an outlet.
5. Distinguish clearly between non-modular and semi-modular outlets. Give Examples.
6. What do you know about Kennedy's gauge outlet and explain the use of it.
7. Give the formulae used for calculating discharge of any three types of outlets used in irrigation canals. What is the normal setting for each type ?
8. Describe with sketch the working of Crump's open flume outlet.
9. What do you understand by a rigid module ? Describe the working of Gibb's module.

18

Canal Regulation Works

18.1. INTRODUCTION

Any structure constructed to regulate the discharge, full supply level or velocity in a canal is known as a *regulation work*. Such a structure is necessary for the efficient working and safety of an irrigation channel. The various regulation works may be categorised as under :

1. Canal fall.
2. Head regulator or head sluice.
3. Cross regulator.
4. Canal escape.
5. Canal outlet.

(A) CANAL FALLS

18.2. NECESSITY AND LOCATION OF FALLS

A fall is an irrigation structure constructed across a canal to lower down its water level and destroy the surplus energy liberated from the falling water which may otherwise scour the bed and banks of the canal.

We have seen in Chapter 14 that the canal requires a certain slope, depending upon the discharge, to overcome the frictional losses. This slope may vary from 1 in 4000 for a discharge of about 1.5 cumecs to about 1 in 8000 for a discharge of 3000 cumecs. This slope is, therefore, quite flat in comparison to the available ground slope of an average value of 5 to 20 cm per kilometre length (*i.e.*, 1 in 200 to 1 in 50). Thus the ground slope in nature is always very much steeper than the design bed slope of irrigation canal, based on the silt theories. If an irrigation canal, taking off from its head, is in cutting, it will soon meet with condition when it will be entirely in embankment.

It has been stated earlier in Chapter 15 that if the canal is in embankment, the cost of construction and maintenance is *very high* and at the same time the percolation and seepage losses are excessive. Also, there is always a danger of the adjacent area being flooded if some cut or breach takes place in the canal banks. Hence, the canal should never be in high embankment. However, the divergence between the gentle bed slope of canal and the steep ground slope throws the canal in embankment after a certain distance though it started in cutting at its head. To overcome this difficulty, *falls* are introduced at appropriate places, and the water surface of the canal is lowered. Arrangements are made to dissipate the excess energy liberated from the falling water.

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The location of a fall is decided from the following considerations :

1. For the canal which does not irrigate the area directly, the fall should be located from the considerations of economy in cost of excavation of the channel with regard to balancing depth and the cost of the falls itself.

2. For a canal irrigating the area directly, a fall may be provided at a location where the F.S.L. outstrips the ground level, but before the bed of the canal comes into filling. After the drop, the F.S.L of the canal may be below the ground level for $\frac{1}{2}$ to $\frac{1}{4}$ kilometer.

3. The location of the fall may also be decided from the consideration of the possibility of combining it with a regulator or a bridge or any other masonry work.

4. A relative economy of providing large number of small falls v/s small number of big falls should be worked out. The provision of small number of big falls results in unbalanced earth-work, but there is always some saving in the cost of the fall structure.

18.3. DEVELOPMENT OF FALLS

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 (Fig. 18.1). 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 :

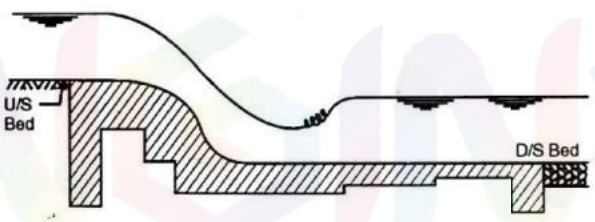

FIG. 18.1. OGEE FALL.

(*i*) There was considerable draw down effect on the u/s resulting is 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.

2. Rapid fall

Fig. 18.2 shows a *rapid fall.* Such 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. The gentle slope admitted timber traffic. Hence, the fall worked admirably. However, there was very high cost of construction.

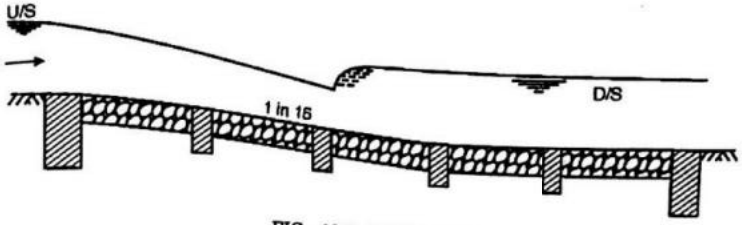

FIG. 18.2. RAPID FALL.

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.

FIG. 18.3. STEPPED FALL.

4. Notch fall

Soon after the development of stepped fall, the efficiency of vertical impact on the floor for energy dissipation came to be recognished. 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.

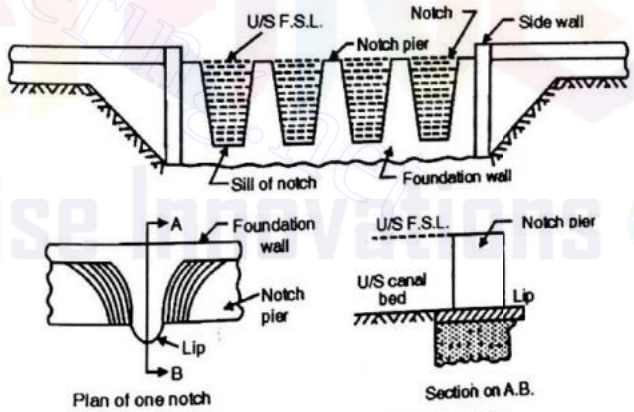

FIG. 18.4. TRAPEZOIDAL NOTCH FALL.

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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. The depth discharge relation was thus maintained with close approximation. As the channel approached the fall, there were neither drawdown nor heading up of water. The trapezoidal fall was very successful and was adopted in India for many years. It was also copied all over the World where it is still in use. There was one serious defect in these falls that they could not be used as regulators in addition.

5. **Vertical drop fall**

In the vertical drop fall, the nappe impinges clear into the water cushion below. In the earlier types of vertical falls, the dimensions of cistern were put in arbitrarily in light of experience of the designers. Another device in the form of grid was usually used in the cistern intercepting the dropping jet of water. The grid consisted of baulks of timber horizontal or inclined and spaced some centimeters apart. These were later abandoned because the timber grid got clogged and rotted and had to be replaced frequently.

The Sarda type fall developed on the Sarda Canal Project in U.P and C.D.O. type fall developed in Punjab are some of the recent types of vertical drop falls. In these falls, the high velocity jet enters the deep pool of water in the cistern and the dissipation of energy is affected by the turbulent diffusion.

6. **Glacis type fall**

The efficiency of the hydraulic jump as a very potent means of destroying the engery of canal falls was brought out clearly by the research work of the Miamy Conservancy Board of Engineers in a publication by Sharman M. Woodward on "Theory of Hydraulic Jump and Back Water Curves" (1918). The glacis type of fall utilised 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*.

Meter and non-meter falls

Meter falls are those which also measure the discharge of the canal. The non-meter falls do not measure the discharge. For a fall to act as a meter, it must have broad weir type crest so that the discharge co-efficient is constant under variable head. Generally glacis type fall is suitable as a meter. The vertical drop fall is not suitable as a meter due to the formation of partial vacuum under the nappe.

Contracted and full width falls

A fall may either be constructed of the full channel width or it may be contracted. The contracted falls, the later type, are also known as the flumed falls.

18.4. CLASSIFICATION OF FALLS

Falls may be divided into four principal classes with reference to the approach condition :

Class I. Falls designed to maintain the depth discharge relation

In this class, the normal depth discharge curve of the channel is maintained, and there is neither drawdown nor heading up of water as the channel approaches the fall. The fall under this category are : (*i*) trapezoidal notch falls, and (*ii*) low-crested rectangular notch falls. In the former the depth discharge relationship is not so well maintained as the latter. The second type of fall is liable to cause silting if the channel is run at a lower level than designed, for long periods. However, if the periods of low water is small, the silt usually gets washed off.

Class II. Falls designed to maintain a fixed supply level in the channel above the work

Such type of falls maintain a nearly fixed water surface level upstream of the work, and necessary under the following circumstances.

(*i*) When the hydraulic power station is combined with the fall.

(*ii*) When a subsidiary canal takes off some distance above the work.

The principal types of falls under this class are : (*i*) the Siphon falls (Siphon Spillways) and (*ii*) high crested weir falls. The falls under this category suffer from the defect of silting caused in the channel above the work. The crested falls are generally constructed without fluming, and hence the discharge intensity per unit length of the crest is low, making the energy dissipation simpler. It can also be used as a meter.

Class III. Falls designed to admit the variations of the surface level above the work at the will of the operator

The falls under the category are designed to admit the variations of water level on the upstream of the work according to the requirements. *They also serve as regulators.* The regulation is done with the help of (*i*) sluice gates, (*ii*) horizontal stop logs, (*iii*) vertical needles. In practice, falls of this class are confined exclusively to rectangular notches as the trapezoidal notch cannot be used for this purpose.

Class IV. Miscellaneous Types

Falls under this class are designed from special considerations without any reference to the conditions of approach. The principal types under this class are as follows :

(*i*) ***Cylinder fall or Well fall*** (Fig. 18.5)

In this type of fall, water is thrown into a well over a crest from where it escapes near its bottom. The energy is dissipated in the well in turbulences. They

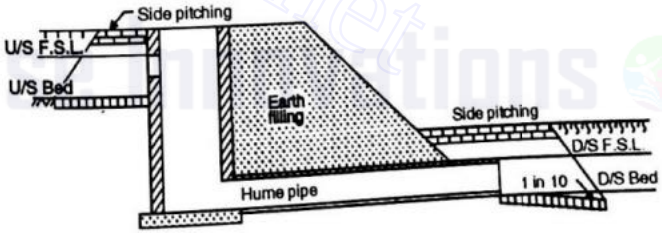

FIG. 18.5. CYLINDER FALL.

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are quite suitable and economical for low discharges and high drops, and are used at tail escapes of small channels.

(ii) Chute or rapids

They conduct the stream on open inclined trough.

(iii) Pipe falls

In this a pipe replaces chute.

18.5. CISTERN DESIGN

The *cistern* or *cistern element* is that portion of the fall down stream of the crest wall where the surplus energy of water leaving the crest is destroyed. The complete cistern element consist of (*i*) sloping glacis (if any), (*ii*) the cistern, (*iii*) roughening devices, and (*iv*) device for deflecting the high velocity jet.

The object of cistern is three fold : (*i*) to reduce the intensity of impact of the dropping jet against the downstream floor, (*ii*) to provide cushion to destroy the energy of the drop, and (*iii*) to produce reverse flow by providing a suitable end-wall to ensure an impact in the cistern.

The cistern element can be divided into the following four classes :

Class I : Cistern element in which there is impact from a stream of water falling freely under gravity

In this case, the energy is dissipated by means of impact and deflection of velocity, suddenly, from the vertical to the horizontal direction. To protect the floor from the impact of falling water, water cushion is provided by depressing the floor below the downstream bed of the channel (Fig. 18.6). Theoretical treatment to workout the required length and depth of cistern has escaped notice so far but a lot of empirical formulae have been developed by various engineers, based on their experience on such works.

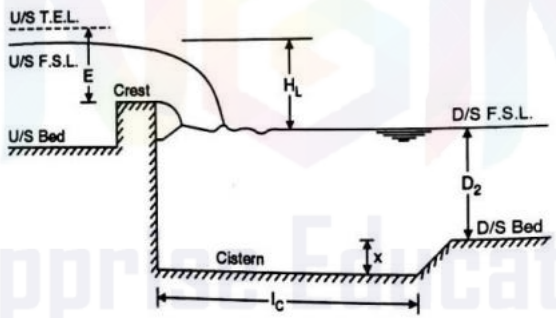

FIG. 18.6. CISTERN.

Following are some of the formulae for cistern design, in metric units :

(i) Sir Proby Cautley's recommendations

He put the depth of cisten at one half the height of the fall without any reference to the discharge intensity.

(ii) Captain Dyas formula

$$x = 0.82\, H_L^{1/2} \cdot D_2^{1/3} \quad ...(18.1)$$

where x = depth of cistern below d/s bed, in metres

H_L = height of drop in metres

D_2 = depth of water downstream in the channel.

(iii) E.L Glass formulae

Glass developed the following formulae for works of Bihar and Orissa :

$$x = 1.85\, E^{1/2} H_L^{\,1/3} - D_2 \quad ...(18.2)$$

$$l_c = x + D_2 \quad ...(18.3)$$

where $E = u/s$ total energy, above the crest

l_c = length of the cistern

(iv) Etchevery formulae

These were developed in America.

$$l_c = 3\sqrt{H_L E} \quad ...(18.4)$$

$$x = \frac{1}{6} l_c \quad ...(18.5)$$

(v) Montagu's formulae

These were developed in Punjab.

$$x = \frac{1}{2} E_{f2} \quad ...(18.6)$$

$$l_c = 4E_{f2} \quad ...(18.7)$$

where E_{f2} = energy of flow downstream, for the discharge intensity q and the fall H_L.

(vi) Blench formula

$$x = 2\,D_c + \frac{1}{4}\left(H_L - \frac{3}{8} D_c\right) - D_2 \quad ..(18.8)$$

According to him, the fall should not be designed in such a way that H_L is less than $\frac{3}{8} D_c$, where D_c is the critical depth.

(vii) U.P.Irrigation Reasearch Institute formulae

$$l_c = 5\sqrt{E\, H_L} \quad ...(18.9)$$

$$x = \frac{1}{4}(E\, H_L)^{2/3} \quad ...(18.10)$$

Class II. Cistern element for impact by a horizontal stream

In this type, the energy is dissipated by the formation of hydraulic jump. The discharge, after passing over the crest, is carried over a sloping glacis. The sloping glacis is given a reverse curvature at its lower end to turn the hypercritical jet to a horizontal direction before it impinges against the subcritical flow of lower channel in the cistern. Hydraulic jump is thus formed.

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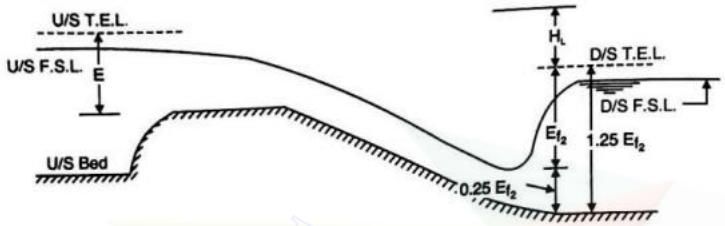

FOG. 18.7. CISTERN.

For a given drop (H_L) in the energy line and the discharge intensity (q), there will be a definite value of downstream specific energy (E_{f2}) and the downstream depth (D_2) required for the jump formation. Theoretically, the bed of the cistern should be provided at the lowest level of the jump formation. However, as an additional safety, the depth of the cistern is increased by 25% of E_{f2}. Thus the R.L. of cistern is kept to the R.L. of d/s total energy line minus 1.25 E_{f2}. In case, however, the downstream bed level is lower than the cistern level determined from the above consideration, the cistern should be provided at the d/s bed level. The length of cistern is kept equal to 5 E_{f2} for normal soil and 6 E_{f2} for sandy soils.

Class III. Cistern for impact by a stream flowing on an inclined glacis

In this case, the glacis is carried straight into cistern without giving any reverse curvature. The hydraulic jump is thus formed on the glacis itself, for the dissipation of energy. However, the falling jet of water has a vertical component while on glacis, and the formation of hydraulic jump on the glacis does not help in dissipating it. Thus the energy dissipation is not perfect, and additional roughening devices have to be provided for the complete destruction of excess energy. However, Montagu and Dr. Bose have derived equations of ideal glacis profile (a curved profile) that imparts maximum horizontal acceleration to the stream at all states of discharge. By imparting maximum horizontal acceleration, the maximum horizontal velocity is attained in any given length of work. Such a design is, therefore, conductive to economy in construction.

Class IV. Cistern without impact

In this type, hydraulic jump is not formed and, therefore energy is dissipated without any impact. The energy dissipation takes place by the provision of *roughening devices*. This case arises in case of falls with large drowning ratio or in low falls where impact is not possible. In such circumstances, the roughening devices are the only means available for energy dissipation.

Roughening Devices

The design of various roughening devices depends upon the experience, and no theoretical treatment is available. Following are some of the roughening devices used on falls :

(*i*) Baffle wall, (*ii*) Friction blocks or arrows,

(*iii*) Dentated sill,
(*v*) Biff wall,
(*iv*) Deflectors,
(*vi*) Cellular or ribbed pitching.

(*i*) ***Baffle wall*** : A baffle wall is a sort of low weir constructed at the end of the cistern to serve two purposes :

(*a*) to head up water to its upstream to such a height that hydraulic jump is formed, and

(*b*) to withstand the actual impact of the high velocity jet to dissipate the energy.

(*ii*) ***Staggered friction blocks or arrows*** : Staggered friction blocks are one of the most useful and simple devices to dissipate the energy. They consist of rectangular blocks of concrete. Their height may be upto $\frac{1}{4}$th water depth and are 1.5 to 2.0 times the height of the block. The distance between successive lines is equal to twice the height. Arrows are specially shaped friction blocks [Fig. 18.8 (1) and (2)]. Both these are built on d/s floor of the falls below the glacis or cistern with the object to divide the bottom high velocity water laterally. They just serve to reduce the bottom velocity of water leaving the pucca floor of the fall.

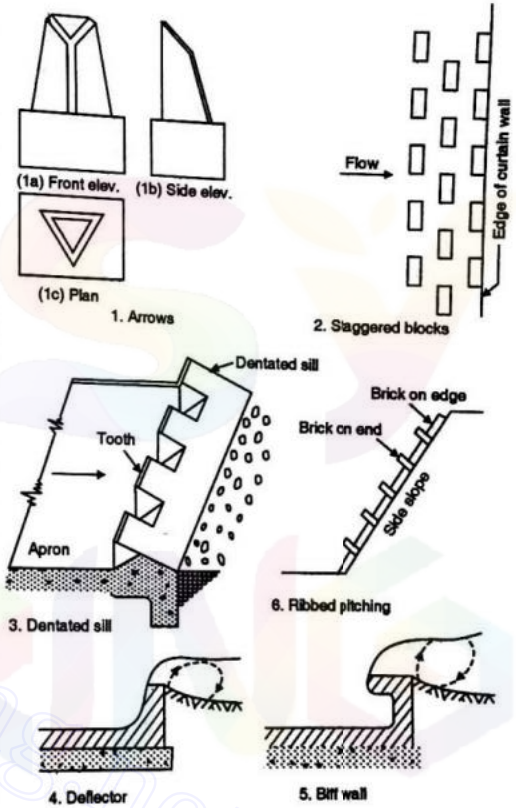

FIG. 18.8. ROUGHENING DEVICES.

Montagu gave the following empirical formula for the determination of the distance upto which roughening of bed is required by the arrows or friction blocks :

$$L = C\frac{D_2^{3/2} H_L^{1/2}}{x}$$

where x = Depth of the cistern below downstream bed, in metres
D_2 = Depth downstrem, in metres
H_L = Drop at the fall, in metres

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C = a coefficient
= 1 for vertical impact
= 2 for horizontal impact
= 4 to 6 for inclined impact
= 8 for no impact.

(iii) Dentated Sill : A dentated sill, shown in Fig. 11.8(3) is provided at the end of cistern if high velocity jet persists tc the end of the cistern. The object of the sill is to deflect up the high velocity jet from near the bed and to break it.

(iv) Deflector : A deflector [Fig. 18.8(4)] is of uniform height, unlike the dentated sill. Its object is to deflect up the high velocity jet near the bed causing a reverse roller.

(v) Biff wall : Fig. 18.8(5) shows a biff wall designed by Colyer in Punjab. It is provided at the end of cistern, causing a deep pool of water behind it in the cistern. Its object is to deflect back the water from the cistern to create super turbulence in it.

(vi) Cellular or ribbed pitching : Ribbed pitching is constructed on the sides by putting bricks flat and on edge alternatively, as shown in Fig. 18.8(6). This provides the roughening of the perimeter to destroy surplus energy down-stream of the fall.

18.6. DESIGN OF SARDA TYPE FALL

This type of fall was designed and developed for Sarda Canal System of U.P. In that area, thin veneer of sandy-clay overlies a stratum of pure sand. Hence, the main requirement was to provide a number of falls with small drops, so that the depth of cutting is kept a minimum. This fall has, therefore, been constructed for drops varying from 0.9 to 1.8 metres (3 to 6 ft.) In the earlier designs, the cistern was not depressed below the d/s floor and the d/s wings were not flared. This resulted in the erosion of banks to the d/s of the work. Extensive model experiments were then conducted at the Bahadrabad Research Station and some recommendations were made. The design criteria given below are based on these recommendations.

Fig. 18.10 shows the drawings of Sarda fall. The completd design consists of the design of the following component parts :

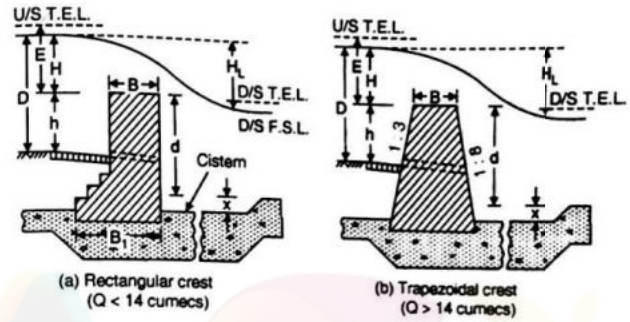
(1) Crest, (2) Cistern
(3) Impervious floor, (4) D/s protection,
(5) U/s approach.

1. Design of crest

(i) Length of crest. The length of the crest is kept equal to the bed width of the canal, and no fluming is done in this type of fall. Sometimes, however, the length of the crest is kept equal to the bed width of canal plus the water depth, to take into account the anticipated increase in discharge at a future date.

(ii) Shape of the crest and discharge formula

Two types of crests are used, as shown in Fig. 18.9. The rectangular crest is used for discharges upto 14 cumecs (500 cusecs) and trapezoidal crest is used for discharges over 14 cumecs.



(a) Rectangular crest (Q < 14 cumecs)
(b) Trapezoidal crest (Q > 14 cumecs)

FIG. 18.9. CREST DESIGN.

For the rectangular crest :

Top width of crest is given by $B = 0.55\sqrt{d}$ metres ...(18.12)

Base width is given by $B_1 = \dfrac{H+d}{\rho}$ ...(18.13)

For masonry crest, ρ may be taken equal to 2.

Discharge is given by $Q = 1.835\, LH^{3/2}\left(\dfrac{H}{B}\right)^{1/6}$ ...(18.14)

where Q = discharge in cumecs

and L = length of the crest in metres.

For a trapezoidal crest

Top width of crest is given by : $B = 0.55\sqrt{H+d}$

U/s batter = 1 : 3

D/s batter = 1 : 8

Thus the base width is determined by the better.

Discharge is given by $Q = 1.99\, LH^{3/2}\left(\dfrac{H}{B}\right)^{1/6}$ ...(18.16)

(iii) Crest level

From Eq. 18.14 or 18.16, the value of H is known.

$\therefore$ R. L. of crest = u/s F. S. L. − H

Height of crest above bed = $h = D - H$.

For falls over 1.5 m, the stability of the crest wall should be tested by actual analysis.

Brick pitching is laid on a slope of 10 : 1 of for 2 to 4 metres length u/s of the crest, and drain holes are provided in the crest at this level to drain out the u/s bed during the closure of the canal.

2. Design of cistern

The length and the depression of the cistern are given by the following equations:

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$$l_c = 5\,(E\,H_L)^{1/2} \quad ...(18.17)$$

and

$$x = \frac{1}{4}\,(E\,H_L)^{2/3} \quad ...(18.18)$$

3. Design of impervious floor

The total length of impervious floor is determined either by Bligh's theory (for small works) or by Khosla's theory. The maximum seepage head occurs when there is water on the u/s side upto the top of the crest and there is no flow to the d/s side. As indicated in Fig. 18.9, the maximum seepage head is equal to d. Out of the total impervious floor length, a minimum length (l_d), to be provided to the d/s of the crest, is given by the following expression :

$$l_d = 2\,(D + 1.2) + H_L \text{ metres} \quad ...(18.19)$$

The balance of the impervious floor length may be provided under and u/s of the crest.

The thickness of the impervious floor is determined on the principle discussed in chapter 12. However, a minimum thickness of 0.3 m to 0.4 m is provided for the floor to the u/s of the crest. For the floor to the d/s of the crest, the actual thickness depends upon the uplift pressures subject to a minimum of 0.3 to 0.4 m for small falls and 0.4 to 0.6 for large falls. The cistern and the d/s impervious floor should have a top lining of brick on edge, in lime or cement mortar, so that floor can be repaired as and when needed. A vertical cutoff of 1 to 1.5 m depth is always provided to the d/s of the impervious floor.

4. D/S protection : The d/s protection consists of (i) bed protection, (ii) side protection, and (iii) d/s wings.

(i) Bed protection : The bed protection consists of dry brick pitching about 20 cm thick resting on 10 cm ballast. Table 18.1 gives the length of the pitching and the number of curtain walls (cutoffs) to be provided

TABLE 18.1. DETAILS OF BED PITCHING.

Head over crest (metres)	*Total length of pitching on the d/s (metres)*	*Remarks*	*Curtain wall*	
			Number	*Depth (m)*
upto 0.3	3.0	Sloping at 1 in 10	1	0.30
0.30 to 0.45	$3.0 + 2\,H_L$	Horizontal up to end of masonry wings and then sloping at 1 in 10	1	0.30
0.45 to 0.60	$4.5 + 2\,H_L$		1	0.45
0.60 to 0.75	$6.0 + 2\,H_L$		1	0.60
0.75 to 0.90	$9.0 + 2\,H_L$		1	0.75
0.90 to 1.05	$13.5 + 2\,H_L$		2	0.94
1.05 to 1.20	$18.0 + 2\,H_L$		2	1.05
1.20 to 1.50	$22.5 + 2\,H_L$		3	1.35

(iii) Side protection : Side pitching, consisting of one brick on edge, is provided after the warped wings. The side pitching is curtailed at any angle of 45° from the end pitching in plan. Generally, warping of masonry wings is done from vertical to slope of 1 : 1. Hence, the side pitching is warped from a slope of 1 : 1 to $1\frac{1}{2}:1$. The pitching supported on a toe wall $1\frac{1}{2}$ brick thick and of depth equal to half the d/s water depth.

(iii) D/s wings : The d/s wings are kept vertical for a length of 5 to 8 times $\sqrt{EH_L}$ from the crest, and are then *warped* or *flared* to a slope of 1 : 1 or $1\frac{1}{2}:1$. An average splay of 1 in 2.5 to 1 in 4 for attaining the required slope is given to the top of the wings. The wings follow a circular arc, tangential at the starting point of warp, in plan.

The wing walls are designed as earth retaining structures. In the absence of elaborate stability calculations, the width of the wings at any level may be kept equal to $\frac{1}{3}$rd of the height above that level. For heavy works, actual design calculations may be made.

5. Design of u/s approach : For discharge upto 14 cumecs, the u/s wings may be splayed, straight at an angle of 45°. For greater discharges, the wings are kept segmental with radius equal to 5 to 6 times H, subtending an angle of 60° at the centre, and then are carried straight into the berm. The embedment in the berms or earth banks should be a minimum of 1 m. The foundations of the u/s wings are kept on the u/s impervious floor itself.

DESIGN EXAMPLE OF SARDA TYPE FALL

Example 18.1. *Design a Sarda type fall for the following data :*

(i) *Full supply discharge* : $\frac{u/s}{d/s} = 40$ *cumecs*

(ii) *Full supply level* : $\frac{u/s}{d/s} = \frac{218.30\ m}{216.80\ m}$

(iii) *Full supply depth* : $\frac{u/s}{d/s} = \frac{1.8\ m}{1.8\ m}$

(iv) *Bed width* : $\frac{u/s}{d/s} = \frac{26\ m}{26\ m}$

(v) *Bed level* : $\frac{u/s}{d/s} = \frac{216.50\ m}{215.00\ m}$

(vi) *Drop* : 1.5 *m*

Design the floor on Bligh's theory taking coefficient of creep = 8. Check the design by Khosla's theory and make changes if necessary. Safe exit gradient may be taken equal as 1/5.

Solution

Step 1. Calculation of *H* and *d*

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$$l_c = 5\,(E\,H_L)^{1/2} \qquad ...(18.17)$$

and

$$x = \frac{1}{4}\,(E\,H_L)^{2/3} \qquad ...(18.18)$$

3. Design of impervious floor

The total length of impervious floor is determined either by Bligh's theory (for small works) or by Khosla's theory. The maximum seepage head occurs when there is water on the u/s side upto the top of the crest and there is no flow to the d/s side. As indicated in Fig. 18.9, the maximum seepage head is equal to d. Out of the total impervious floor length, a minimum length (l_d), to be provided to the d/s of the crest, is given by the following expression :

$$l_d = 2\,(D + 1.2) + H_L \text{ metres} \qquad ...(18.19)$$

The balance of the impervious floor length may be provided under and u/s of the crest.

The thickness of the impervious floor is determined on the principle discussed in chapter 12. However, a minimum thickness of 0.3 m to 0.4 m is provided for the floor to the u/s of the crest. For the floor to the d/s of the crest, the actual thickness depends upon the uplift pressures subject to a minimum of 0.3 to 0.4 m for small falls and 0.4 to 0.6 for large falls. The cistern and the d/s impervious floor should have a top lining of brick on edge, in lime or cement mortar, so that floor can be repaired as and when needed. A vertical cutoff of 1 to 1.5 m depth is always provided to the d/s of the impervious floor.

4. D/S protection : The d/s protection consists of (i) bed protection, (ii) side protection, and (iii) d/s wings.

(i) Bed protection : The bed protection consists of dry brick pitching about 20 cm thick resting on 10 cm ballast. Table 18.1 gives the length of the pitching and the number of curtain walls (cutoffs) to be provided

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0.45 to 0.60	$4.5 + 2\,H_L$		1	0.45
0.60 to 0.75	$6.0 + 2\,H_L$		1	0.60
0.75 to 0.90	$9.0 + 2\,H_L$		1	0.75
0.90 to 1.05	$13.5 + 2\,H_L$		2	0.94
1.05 to 1.20	$18.0 + 2\,H_L$		2	1.05
1.20 to 1.50	$22.5 + 2\,H_L$		3	1.35

(iii) Side protection : Side pitching, consisting of one brick on edge, is provided after the warped wings. The side pitching is curtailed at any angle of 45° from the end pitching in plan. Generally, warping of masonry wings is done from vertical to slope of 1 : 1. Hence, the side pitching is warped from a slope of 1 : 1 to $1\frac{1}{2}$: 1. The pitching supported on a toe wall $1\frac{1}{2}$ brick thick and of depth equal to half the d/s water depth.

(iii) D/s wings : The d/s wings are kept vertical for a length of 5 to 8 times $\sqrt{EH_L}$ from the crest, and are then *warped* or *flared* to a slope of 1 : 1 or $1\frac{1}{2}$: 1. An average splay of 1 in 2.5 to 1 in 4 for attaining the required slope is given to the top of the wings. The wings follow a circular arc, tangential at the starting point of warp, in plan.

The wing walls are designed as earth retaining structures. In the absence of elaborate stability calculations, the width of the wings at any level may be kept equal to $\frac{1}{3}$rd of the height above that level. For heavy works, actual design calculations may be made.

5. Design of u/s approach : For discharge upto 14 cumecs, the u/s wings may be splayed, straight at an angle of 45°. For greater discharges, the wings are kept segmental with radius equal to 5 to 6 times H, subtending an angle of 60° at the centre, and then are carried straight into the berm. The embedment in the berms or earth banks should be a minimum of 1 m. The foundations of the u/s wings are kept on the u/s impervious floor itself.

DESIGN EXAMPLE OF SARDA TYPE FALL

Example 18.1. *Design a Sarda type fall for the following data :*

(i) *Full supply discharge* : $\frac{u/s}{d/s} = 40$ *cumecs*

(ii) *Full supply level* : $\frac{u/s}{d/s} = \frac{218.30\ m}{216.80\ m}$

(iii) *Full supply depth* : $\frac{u/s}{d/s} = \frac{1.8\ m}{1.8\ m}$

(iv) *Bed width* : $\frac{u/s}{d/s} = \frac{26\ m}{26\ m}$

(v) *Bed level* : $\frac{u/s}{d/s} = \frac{216.50\ m}{215.00\ m}$

(vi) *Drop* : 1.5 m

Design the floor on Bligh's theory taking coefficient of creep = 8. Check the design by Khosla's theory and make changes if necessary. Safe exit gradient may be taken equal as 1/5.

Solution

Step 1. Calculation of H and d

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Since the discharge is more than 14 cumecs, trapezoidal crest will be adopted.

The discharge is given by

$$Q = 1.99\, LH^{3/2}\left(\frac{H}{B}\right)^{1/6} \quad ...(1)$$

Here, L = width of canal = 26 m

Q = 40 cumecs.

$$B = 0.55\sqrt{H+d}\,.$$

But $H + d = D$ + drop in the level

$$H + d = 1.8 + 1.5 = 3.3 \text{ m} \quad ...(2)$$

$$\therefore \quad B = 0.55\sqrt{H+d} = 0.55\sqrt{3.3} = 1.0 \text{ m} \quad ...(3)$$

Substituting the value of B in (1), we get

$$40 = \frac{1.99 \times 26\, H^{(3/2+1/6)}}{(1.0)^{1/6}}$$

$$H^{5/3} = \frac{40 \times 1}{1.99 \times 26} = 0.774$$

From which $H = (0.774)^{3/5} = 0.86$ m ...(4)

Substituting this in (2) we get

$$d = 3.3 - H = 3.3 - 0.86 = 2.44 \text{ m}$$

Height of crest above bed $= D - H = 1.8 - 0.86 = 0.94$ m

Step 2. Design of crest

A trapezoidal crest will be adopted.

Top width B (found in step 1) = 1 m

d/s batter = 1 : 3 ; u/s batter = 1 : 8

Assuming canal side slope 1 : 1, velocity of approach is given by

$$V_a = \frac{40}{(27 + 1.8)\,1.8} = 0.8 \text{ m/sec}$$

Velocity head $= \frac{V_a^2}{2g} = \frac{(0.8)^2}{2 \times 9.81} = 0.032$ m

$\therefore$ u/s. T. EL. = u/s F. S. L. + velocity head = 218.30 + 0.032 = 218.332 m

R.L.of crest = u/s F. S. L. − H = 218.30 − 0.86 = 217.44 m

Value of E = u/s T. E. L. − R. L. of crest = 218.332 − 217.44 = 0.892 m

Step 3. Design of cistern

Depth of cistern $= x = \frac{1}{4}(E\,H_L)^{2/3} = \frac{1}{4}(0.892 \times 1.5)^{2/3} = 0.304$ m

Length of cistern $= l_c = 5\,(EH_L)^{1/2} = 5\,(0.892 \times 1.5)^{1/2} = 5.8$ m

Keep the length of cistern = 6 m

R. L. of bed of cistern = R. L. of d/s bed − x = 215.00 − 0.304 = 214.696 m.

Keep the cistern at R.L. 214.69 m.

$\therefore$ Final value of $x = 215.00 - 214.69 = 0.31$ m.

Step 4. Design of impervious floor

Seepage head $= H_s = d = 2.44$ m

Bligh's coefficient = 8

$\therefore$ Length of impervious floor or Creep length $= C \times H_s = 8 \times 2.44 = 19.5$ m

Provide u/s cut-off $d_1 = 1.0$ m and d/s cutoff $d_2 = 1.6$ m.

The vertical length of creep $= 2(1 + 1.6) = 5.2$ m.

$\therefore$ Length of horizontal impervious floor $= 19.5 - 5.2 = 14.3$ m

Provide 15 m length of impervious floor.

Minimum length of impervious floor to the d/s of toe of the crest wall

$$= l_d = 2(D + 1.2) + H_L = 2(1.8 + 1.2) + 1.5 = 7.5 \text{ m}$$

Provide $l_d = 8$ m. The balance of the length = 15 − 8 = 7 m is to be provided under and u/s of the crest.

Calculation for uplift pressure and thickness

Total creep length = 15 + 2(1 + 1.6) = 20.2 m

(*i*) The uplift pressure under the u/s floor will be counter-balanced by the weight of water, and hence no thickness is required. However, provide a minimum thickness of 0.4 m.

(*ii*) For other points, the maximum *vertical ordinate* between the Bligh's H.G.line and the floor level gives the uplift pressure.

Maximum unbalanced head under the toe of the crest = 1.66 m

This can also be found by calculations as under :

$$\text{Static head} = 2.44\left(1 - \frac{7 + 2 \times 1.0}{2.02}\right) + x = 2.44\,(1 - 0.446) + 0.31 = 1.66 \text{ m}$$

$$\therefore \quad \text{Thickness required} = \frac{1.66}{\rho - 1} = \frac{1.66}{2.24 - 1} = 1.34 \text{ m.}$$

Provide 1.40 m thick cement concrete floor overlaid by 0.2 m brick pitching.

(*Note :* While working out the thickness, the expression has not been multiplied by the safety factor 4/3, since the additional 0.2 m brick pitching will provide the additional safety.)

(*iii*) Provide a minimum thickness of 0.6 m overlaid by 0.2 m thick brick pitching, at the d/s end of the floor.

(*iv*) The thickness of the floor at intermediate points may be varied as per requirements of uplift pressure.

Step 5. Checking of floor thickness by Khosla's theory

Exit gradient $G_E = \frac{1}{\pi\sqrt{\lambda}}\,\frac{H_s}{d_2}$

$$\therefore \quad \frac{1}{5} = \frac{1}{\pi\sqrt{\lambda}}\,\frac{2.44}{1.60}$$

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Since the discharge is more than 14 cumecs, trapezoidal crest will be adopted.

The discharge is given by

$$Q = 1.99\, LH^{3/2}\left(\frac{H}{B}\right)^{1/6} \quad ...(1)$$

Here, L = width of canal = 26 m

Q = 40 cumecs.

$$B = 0.55\sqrt{H+d}\,.$$

But $H + d = D$ + drop in the level ...(2)

$$H + d = 1.8 + 1.5 = 3.3 \text{ m}$$

$$\therefore \quad B = 0.55\sqrt{H+d} = 0.55\sqrt{3.3} = 1.0 \text{ m} \quad ...(3)$$

Substituting the value of B in (1), we get

$$40 = \frac{1.99 \times 26\, H^{(3/2+1/6)}}{(1.0)^{1/6}}$$

$$H^{5/3} = \frac{40 \times 1}{1.99 \times 26} = 0.774$$

From which $H = (0.774)^{3/5} = 0.86$ m ...(4)

Substituting this in (2) we get

$$d = 3.3 - H = 3.3 - 0.86 = 2.44 \text{ m}$$

Height of crest above bed $= D - H = 1.8 - 0.86 = 0.94$ m

Step 2. Design of crest

A trapezoidal crest will be adopted.

Top width B (found in step 1) = 1 m

d/s batter = 1 : 3 ; u/s batter = 1 : 8

Assuming canal side slope 1 : 1, velocity of approach is given by

$$V_a = \frac{40}{(27 + 1.8)\,1.8} = 0.8 \text{ m/sec}$$

Velocity head $= \dfrac{V_a^2}{2g} = \dfrac{(0.8)^2}{2 \times 9.81} = 0.032$ m

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R.L. of crest = u/s F. S. L. − H = 218.30 − 0.86 = 217.44 m

Value of E = u/s T. E. L. − R. L. of crest = 218.332 − 217.44 = 0.892 m

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Depth of cistern $= x = \frac{1}{4}(E\,H_L)^{2/3} = \frac{1}{4}(0.892 \times 1.5)^{2/3} = 0.304$ m

Length of cistern $= l_c = 5\,(EH_L)^{1/2} = 5\,(0.892 \times 1.5)^{1/2} = 5.8$ m

Keep the length of cistern = 6 m

R. L. of bed of cistern = R. L. of d/s bed − x = 215.00 − 0.304 = 214.696 m.

Keep the cistern at R.L. 214.69 m.

$\therefore$ Final value of x = 215.00 − 214.69 = 0.31 m.

Step 4. Design of impervious floor

Seepage head $= H_s = d = 2.44$ m

Bligh's coefficient = 8

$\therefore$ Length of impervious floor or Creep length $= C \times H_s = 8 \times 2.44 = 19.5$ m

Provide u/s cut-off $d_1 = 1.0$ m and d/s cutoff $d_2 = 1.6$ m.

The vertical length of creep = 2(1 + 1.6) = 5.2 m.

$\therefore$ Length of horizontal impervious floor = 19.5 − 5.2 = 14.3 m

Provide 15 m length of impervious floor.

Minimum length of impervious floor to the d/s of toe of the crest wall

$$= l_d = 2(D + 1.2) + H_L = 2(1.8 + 1.2) + 1.5 = 7.5 \text{ m}$$

Provide $l_d = 8$ m. The balance of the length = 15 − 8 = 7 m is to be provided under and u/s of the crest.

Calculation for uplift pressure and thickness

Total creep length = 15 + 2(1 + 1.6) = 20.2 m

(*i*) The uplift pressure under the u/s floor will be counter-balanced by the weight of water, and hence no thickness is required. However, provide a minimum thickness of 0.4 m.

(*ii*) For other points, the maximum *vertical ordinate* between the Bligh's H.G.line and the floor level gives the uplift pressure.

Maximum unbalanced head under the toe of the crest = 1.66 m

This can also be found by calculations as under :

Static head $= 2.44\left(1 - \dfrac{7 + 2 \times 1.0}{2.02}\right) + x = 2.44\,(1 - 0.446) + 0.31 = 1.66$ m

$\therefore$ Thickness required $= \dfrac{1.66}{\rho - 1} = \dfrac{1.66}{2.24 - 1} = 1.34$ m.

Provide 1.40 m thick cement concrete floor overlaid by 0.2 m brick pitching.

(*Note :* While working out the thickness, the expression has not been multiplied by the safety factor 4/3, since the additional 0.2 m brick pitching will provide the additional safety.)

(*iii*) Provide a minimum thickness of 0.6 m overlaid by 0.2 m thick brick pitching, at the d/s end of the floor.

(*iv*) The thickness of the floor at intermediate points may be varied as per requirements of uplift pressure.

Step 5. Checking of floor thickness by Khosla's theory

Exit gradient $G_E = \dfrac{1}{\pi\sqrt{\lambda}}\,\dfrac{H_s}{d_2}$

$$\therefore \quad \frac{1}{5} = \frac{1}{\pi\sqrt{\lambda}}\,\frac{2.44}{1.60}$$

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$$\therefore \quad \frac{1}{\pi\sqrt{\lambda}} = \frac{1}{5} \times \frac{1.60}{2.44} = 0.131.$$

From Khosla's exit gradient curve; we get

$$\alpha = 10.5 \text{ corresponding to } \frac{1}{\pi\sqrt{\lambda}} = 0.131$$

Alternatively

$$\lambda = \left[\frac{1}{\pi(0.131)}\right]^2 = 5.891$$

$$\therefore \quad \lambda = 5.891 = \frac{1+\sqrt{1+\alpha^2}}{2}, \text{ from which } \alpha = 10.73$$

$$\therefore \quad b = \alpha \,.\, d_2 = 10.73 \times 1.6 = 17.17 \text{ m}.$$

Length actually provided by Bligh's theory = 15 m only, which is less than required. Hence, either the length is to be increased by 2.17 m or the depth of cutoff is to be increased.

Let us increase the depth of cuttoff d_2 to 1.80 m.

Keep $b = 15$ cm (same as before)

$$\therefore \quad \alpha = \frac{b}{d_2} = \frac{15}{1.80} = 8.33.$$

From the exit gradient curves, for $\alpha = 8.33$, we get

$$\frac{1}{\pi\sqrt{\lambda}} = 0.145$$

$$G_E = \frac{1}{\pi\sqrt{\lambda}} \frac{H_s}{d_2} = 0.145 \times \frac{2.44}{1.80} \simeq \frac{1}{5.1}$$

Alternatively,

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+(8.33)^2}}{2} = 4.695$$

$$\therefore \quad G_E = \frac{1}{\pi\sqrt{4.695}} \times \frac{2.44}{1.80} \simeq \frac{1}{5.02}$$

Permissible exit gradient = 1/5. Hence, safe.

Calculation of pressures

(i) U/S cutoff

$$d_1 = 1.0 \text{ m}; \; b = 15 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d_1}{b} = \frac{1}{15} = 0.0666$$

$$\Phi_{C1} = 100 - 24 = 76\,\%$$

$$\Phi_{D1} = 100 - 17 = 83\,\%$$

Assume a thickness of floor = 0.4 m at the u/s

Corrected $\Phi_{C1} = 76 + \dfrac{83-76}{1} \times 0.4 = 78.8\,\%$

(ii) D/S cutoff wall

$$d_2 = 1.8 \text{ m}; \; b = 15 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d_2}{b} = \frac{1.8}{15} = 1.12$$

$$\Phi_{E2} = 31\,\%$$

$$\Phi_{D2} = 21.5\,\%$$

Assuming a thickness of d/s floor = 0.6 m.

Corrected $\Phi_{E2} = 31 - \dfrac{31-21.5}{1.8} \times 0.6 = 27.8\,\%.$

Thus the pressure on the floor varies (linearly) from 78.8% at the u/s to 27.8% at the d/s.

Calculation of thickness

(*i*) Provide a minimum thickness of 0.4 m for the u/s floor.

(*ii*) Thickness at the d/s toe of crest :

% Pressure $= \left(27.8 + \dfrac{78.8-27.8}{15} \times 8\right) = 55\,\%$

Maximum static head $= 0.55H_s + x = 0.55 \times 2.44 + 0.31 = 1.67$ m

Thickness required $= \dfrac{1.67}{\rho - 1} = \dfrac{1.67}{2.24-1} = 1.35$ m

Thickness provided by Bligh's theory is 1.40 m.

∴ Provide 1.40 m thick cement concrete floor overlaid by 0.20 m thickness brick pitching, for a length of 3 m.

(*iii*) *At 3 m from the d/s toe of crest*

% Pressure $= 27.8 + \dfrac{78.8-27.8}{15} \times 5 = 44.8\,\%$

Maximum static head $= 0.448\,H_s + x = 0.448 \times 2.44 + 0.31 = 1.40$ m

∴ Thickness required $= \dfrac{1.40}{2.24-1} = 1.13$ m.

Hence, provide a thickness of 1.20 m cement concrete overlaid by 0.2 m thick pitching, for a length of 3 m (*i.e.*up to the end of cistern).

(*iv*) *At 6 m from the d/s toe of crest*

% Pressure $= 27.8 + \dfrac{78.8-27.8}{15} \times 2 = 34.6$

∴ Maximum static head $= 0.346 \times 2.44 = 0.842$ m

(The value of x has not been added here since the cistern ends at 6 m.)

∴ Thickness required $= \dfrac{0.842}{2.24-1} = 0.59$ m

Hence, provide a thickness of 0.6 m cement concrete overlaid by 0.2 m thick pitching, for the last 2 m length of the floor.

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Summary of floor thickness

(*i*) U/s floor : 0.4 m, (*ii*) D/s toe of crest to 3 m d/s : 1.40 m,

(*iii*) 3 m to 6 m from d/s toe : 1.20, (*iv*) Last 2 m : 0.6 m.

Step 6. Design of d/s wings

Keep the d/s wings vertical for a length of

$$6 \times \sqrt{E\,H_L} = 6\sqrt{0.892 \times 1.5} \simeq 7 \text{ m.}$$

Then the wings may be warped to 1 : 1 slope, at a splay of 1 in 3. Height of top of d/s wings above the bed = water depth + F.B. = 1.8 + 0.5 = 2.3 m

∴ Horizontal projection of this on 1 : 1 slope = 2.3 m. Using a splay of 1 : 3, length of warped wing, measured along the centre line of canal = 2.3 × 3 m = 6.9 ≈ 7 m.

The sections of the wing wall have been shown on Fig. 18.10.

Step 7. Downstream pitching

(*a*) ***Bed pitching.*** From Table 18.1, length of bed piching for $H = 0.86$ m is equal to (9 + 2 × 1.5) = 12 m. This may be provided horizontal upto the end of masonry wings and then sloping at 1 in 10. Thus length of sloping pitching = 7 m and length of horizontal pitching = 6 m.

(*b*) ***Toe-wall.*** Provide one toe wall of 1 m depth and 0.4 m width.

(*c*) ***Side pitching.*** The side protection has been provided, consisting of 0.2 m thich brick on edge. This has been warped to slope of $1\frac{1}{2}:1$. The side pitching is curtailed at an angle of 45° from the end of bed pitching in plan. Support the side pitching on a toe wall 0.4 m thick and 1 m deep.

Step 8. Energy dissipators

Energy dissipators are not provided for small discharges. However, for large discharges, additional dissipators should be provided. The dissipators consists of two rows of friction blocks in the cistern, and two rows of cube blocks just at the d/s end of impervious floor.

Critical depth $d_c = \left(\dfrac{q^2}{g}\right)^{1/3}$

Here $q = \dfrac{40}{26} = 1.538$ cumecs/m ; $d_c = \left[\dfrac{(1.538)^2}{9.81}\right]^{1/3} \simeq 0.7$ m

(*a*) ***Size and position of friction blocks***

(*i*) Length of block $2d_c = 2 \times 0.7 = 1.4$ m

(*ii*) Width of block $= d_c = 0.7$ m

(*iii*) Height of block $= d_c = 0.7$ m

(*iv*) Distance from toe of crest $= 1.5\,d_c = 1.05 \simeq 1$ m.

Provide two rows, staggered, at a distance of 1 m from the toe of the crest

(*b*) ***Cube blocks***

(*i*) Length $\dfrac{1}{10}D = \dfrac{1}{10} \times 1.8 = 0.18 \simeq 0.2$ m

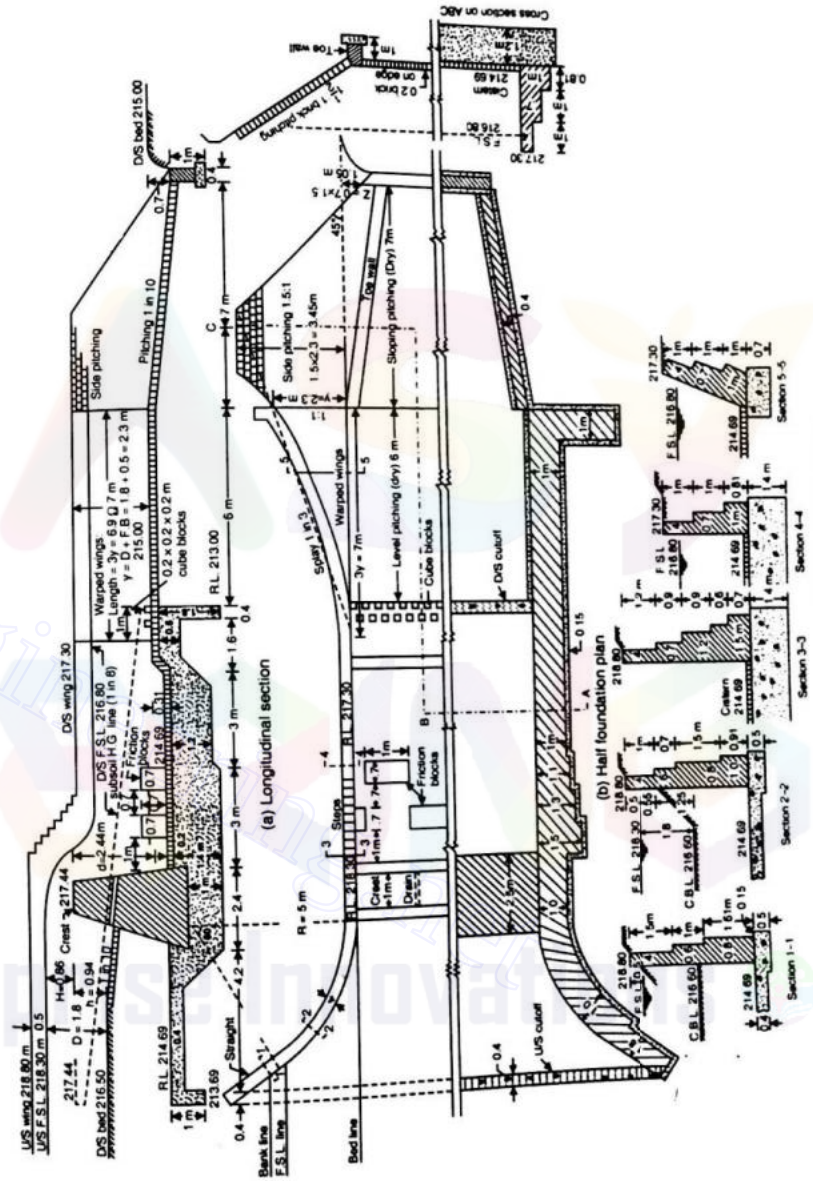

FIG. 18.10. SARDA TYPE FALL

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(ii) Width $= \frac{1}{10} D = 0.2$ m

(iii) Height $= \frac{1}{10} D = 0.2$ m.

Provide two rows, staggered, at the end of impervious floor.

Step 9. U/S approach

Radius of u/s wings $= 6H = 6 \times 0.86 \approx 5$ m.

Provide wing wall segmental with 5 m radius, subtending an angle of 60° at the centre, and the carried tangentially into the berm. The complete details of the fall are shown in Fig.18.10.

18.7. DESIGN OF STRAIGHT GLACIS FALL

(1) Crest design

(a) For non-meter falls, the u/s glacis is given a slope of $\frac{1}{2}:1$, and is joined tangentially to the crest with a radius of $E/2$, where $E =$ height of u/s T.E.L. above crest.

(b) The d/s glacis is given a slope of 2 : 1 and in joined tangentially to the cistern with a radius $= E$.

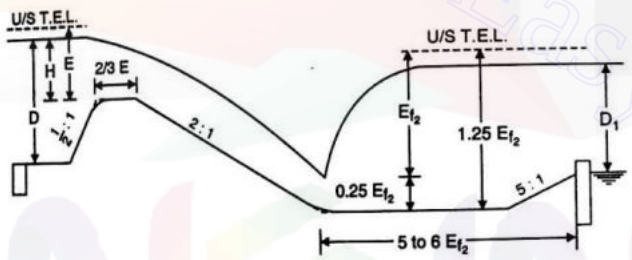

FIG. 18.11 STRAIGHT GLACIS FALL.

(c) The width of the crest is kept equal to $\frac{2}{3}E$.

(d) The glacis fall may sometimes be flumed if it is to be combined with a bridge etc. However, the minimum clear length of the crest, expressed in % of the bed width of canal, is kept according to the following recommendations :

Drop	*Clear length of crest*
Below 1.25 m	65%
1.25 to 3 m	75%
Above 3 m	85%

(e) The value of E is calculated from the discharge equation :

$$Q = 1.84\, L_t\, E^{3/2} \qquad ...(18.20)$$

where L_t = length of the crest

Q = discharge in cumecs.

If there are n number of piers, effective crest length to be substituted in Eq. 18.20 is equal to $(L_t - 0.2n)$.

(f) Crest level = u/s T. E. L. – E

u/s T. E. L. = u/s F. S. L. + velocity head.

2. Design of cistern

(a) Knowing the discharge intensity q per metre run and the drop H_L, the energy of flow (E_{f2}) below the hydraulic jump is known.

Then R. L. cistern = d/s T. E. L. – 1.25 E_{f2}.

(b) Length ef cistern = 5 to 6 E_{f2}

3. Design of impervious floor

The total length of the impervious floor is found from the considerations of permissible exit gradient. The total length of the impervious floor may be provided in the following manner :

(i) Length of cistern (ii) Horizontal length of d/s glacis

(iii) Crest width (iv) Horizontal length of u/s glacis

(v) Balance to be provided to the u/s.

4. Design of d/s protection

(a) Bed protection.

Since a deflector wall is provided at the u/s end of the floor, no bed pitching is provided.

The height of deflector wall above d/s bed, may be kept equal to $D_1/10$, where D_1 is the d/s water depth, shown in Fig. 18.11.

(b) Side protection.

Dry brick pitching of $1\frac{1}{2}$ brick thick is done upto a length $3D_1$. The pitching is supported on a toe will 0.4 m wide and $D_1/2$ deep, subjected to minimum depth of 0.5 m.

(c) ***Curtain wall.*** Depth of curtain wall = $D_1/2$; width 0.4 m.

5. Design of energy dissipators

(a) Friction blocks

Four rows of friction blocks are provided in case of flumed glacis falls. The height of each block is kept equal to $D_1/8$ and length equal to three times the height of the block. The distance between the rows is kept equal to the height of the blocks. The u/s edge of the first block may be provided at a distance of 5 times the height of the blocks from the toe of glacis.

(b) Glacis blocks.

For falls more than 2 m, one row of glacis blocks, of the same dimensions as that of friction blocks, is provided at the d/s end of glacis.

(c) Deflector wall.

A deflector wall of height $D_1/10$ and width 0.4 m is provided at the d/s of cistern. The same wall may work as the curtain wall.

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6. Design of d/s expansion

In a flumed fall, the d/s expansion starts from the toe of the glacis. A rectangular hyperbolic expansion is generally preferred.

The bed width B_x at any distance x from the d/s toe of the glacis is given by

$$b_x = \frac{B_1 \times B_2 \times L_e}{(L_e \times B_2) - (B_2 - B_1)\,x} \quad ...(18.21)$$

where L_e = total length in which the expansion is to be done. For small falls, straight 1 in 3 expansion is done.

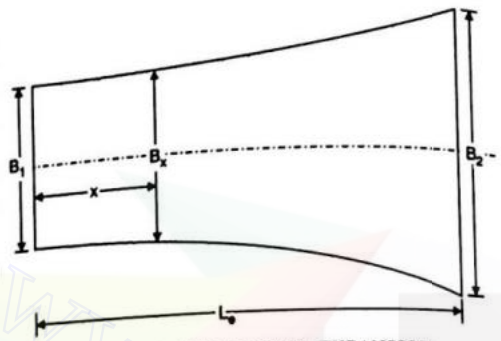

FIG. 18.12. HYPERBOLIC EXPANSION.

7. Upstream approach

For a non-meter fall, the side walls are splayed at an angle of 45° from the u/s edge of the crest and is carried straight into the berm by a length of at least 1 m.

DESIGN EXAMPLE OF UNFLUMED STRAIGHT GLACIS NON-METER FALL

Example 18.2. *Design an unflumed straight glacis non-meter fall for the data of Example 18.1.*

(i) *Full supply discharge* : $\frac{u/s}{d/s} = 40$ *cumecs*

(ii) *Full supply level* : $\frac{u/s}{d/s} = \frac{218.30}{216.80}$ *m*

(iii) *Full supply depth* : $\frac{u/s}{d/s} = \frac{1.8\ m}{1.8\ m}$

(iv) *Bed width* : $\frac{u/s}{d/s} = \frac{26\ m}{26\ m}$

(v) *Bed level* : $\frac{u/s}{d/s} = \frac{216.50}{215.00}$ *m*

(vi) *Drop* : 1.5 *m*

(vii) *Permissible exit gradient* : 1/6.

Solution

Step 1. Design of crest

Length of crest = Bed width of canal = 26 m ...(i)

Discharge is given by

$$Q = 1.84\, L_t\, E^{3/2}$$

$$40 = 1.84 \times 26 \times E^{3/2}$$

$$\therefore \quad E = \left(\frac{40}{1.84 \times 26}\right)^{2/3} = 0.89 \text{ m}$$

Velocity of approach, assuming canal side slopes of 1:1

$$= \frac{40}{(26 + 1.8)\,1.8} = 0.8 \text{ m/sec}$$

$$\therefore \text{ Velocity head } = \frac{(0.8)^2}{2 \times 9.81} \approx 0.03 \text{ m}$$

$\therefore$ u/s T. E. L. = u/s F. S. L. + velocity head = 218.30 + 0.03 = 218.33 m ...(iv)

Crest level = u/s T. E. L. − E = 218.33 − 0.89 = 217.44 m ...(v)

Width of the crest $= \frac{2}{3}E = \frac{2}{3} \times 0.89 \simeq 0.6$ m ...(vi)

Provide u/s glacis at slope $\frac{1}{2}:1$, joined tangentially to the u/s bed with radius $= \frac{1}{2} \times 0.89 \approx 0.5$ m. Keep d/s glacis at slope 2:1 joined tangentially to the cistern bed with radius $= E \simeq 0.9$ m.

Step 2. Design of cistern

$$q = \frac{Q}{L_t} = \frac{40}{26} = 1.54 \text{ cumecs/metre}$$

$$H_L = 1.5 \text{ m.}$$

Hence, from Blench curves corresponding to the value of $q = 1.54$ cumecs/metre and $H_L = 1.5$ m, we get $E_{f2} = 1.44$ m.

$\therefore$ R. L. of cistern = d/s T. E. L. − 1.25 E_{f2}

$$= (216.80 + 0.03) - (1.25 \times 1.44) = 215.03 \text{ m}$$

This is higher than the d/s bed level which is 215.00 m

Keep the R.L. of cistern at 214.50 m

Length of cistern = 6 E_{f2} = 6 × 1.44 = 8.62 m

Depth of cistern below d/s bed = 215.00 − 214.5 = 0.5 m

Provide cistern of 9 m length. This cistern will be joined to to d/s bed in a slope of 5:1.

Step 3. Design of impervious floor.

Minimum depth of u/s curtain wall d_1

$$= \frac{D_1}{3} = \frac{1.8}{3} = 0.6 \text{ m.}$$

$\therefore$ Provide 0.4 m wide and 1 m deep curtain wall.

Depth of d/s curtain wall $d_2 = \frac{D_2}{2} = \frac{1.8}{2} = 0.9$ m.

$\therefore$ Provide 0.4 m wide and 1 m deep curtain wall at the d/s. The curtain wall will project above the d/s bed by

$$\frac{D_2}{10} = \frac{1.8}{10} = 0.18 \text{ m}$$

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so as to act as a deflector.

Exit gradient. $G_E = \frac{1}{6}$

But $G_E = \frac{1}{\pi\sqrt{\lambda}} \frac{H_s}{d_2}$

where H_s = maximum seepage head = Crest level − d/s bed level

$= 217.44 - 215.00 = 2.44$ m

d_2 = depth of d/s curtain wall = 1 m.

$$\frac{1}{6} = \frac{1}{\pi\sqrt{\lambda}} \frac{2.44}{1}$$

$$\frac{1}{\pi\sqrt{\lambda}} = \frac{1}{6 \times 2.44} = 0.068$$

From Khosla's exit gradient curve we get

$$\alpha = 40 \text{ for } \frac{1}{\pi\sqrt{\lambda}} = 0.068.$$

Alternatively : $\lambda = \left[\frac{1}{\pi(0.068)}\right]^2 = 21.71$

$\therefore$ $\lambda = 21.71 = \frac{1+\sqrt{1+\alpha^2}}{2}$, from which $\alpha \simeq 42$

$b = \alpha\, d_2 = 42 \times 1 = 42$ m

This length is too excessive.

Hence, increase the depth d_2 of the d/s curtain wall to 2 m

$$\frac{1}{6} = \frac{1}{\pi\sqrt{\lambda}} \frac{2.44}{2}$$

or $\frac{1}{\pi\sqrt{\lambda}} = \frac{2}{6 \times 2.44} \approx 0.137$

Hence, for $\frac{1}{\pi\sqrt{\lambda}} = 0.137$, we get $\alpha \simeq 10$ m

Alternatively $\lambda = \left[\frac{1}{\pi(0.137)}\right]^2 = 5.43$

$\therefore$ $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 5.43$, from which $\alpha \simeq 9.8$

$\therefore$ $b = \alpha\, d_2 = 9.8 \times 2 \simeq 20$ m.

This length of 20 m will be arranged as under:

(*i*) Length of cistern = 9 m

(*ii*) Length of d/s glacis

$$d = 2\,(217.44 - 214.50) = 2 \times 2.94 = 5.88 \text{ m}.$$

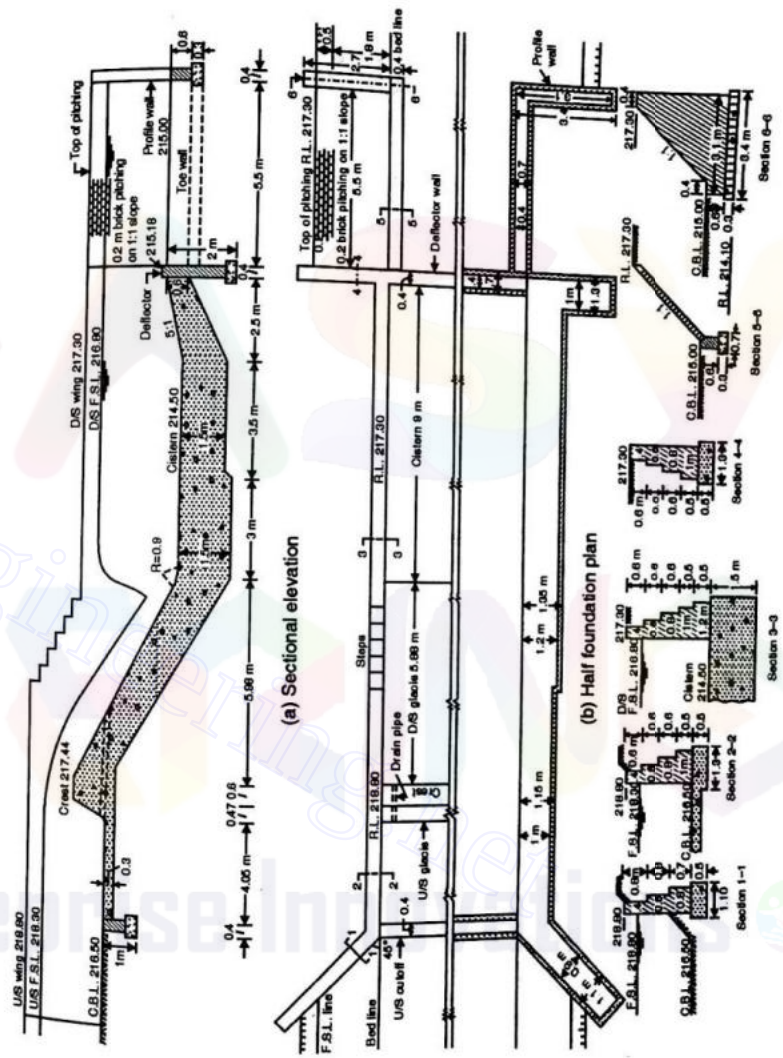

FIG. 18.13. GLACIS FALL

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(iii) Width of crest $= 0.60$ m

(iv) Length of u/s glacis $= \frac{1}{2}(217.44 - 216.50) = \frac{1}{2} \times 0.94 = 0.47$ m

Total of (i) to (iv) $= 15.95$ m

Balance to be provided to the u/s of the u/s glacis

$= 20 - 15.95 = 4.05$ m

Pressure calculations

(i) U/s curtain wall

$$d_1 = 1 \text{ m} \; ; \; b = 20 \text{ m}$$

$$\therefore \quad \frac{1}{\alpha} = \frac{d_1}{b} = \frac{1}{20} = 0.05$$

$$\Phi_{C_1} = 100 - 22 = 78\,\% \; ; \; \Phi_{D_1} = 100 - 15 = 85\,\%$$

Assume a minimum thickness of 0.3 m at the upstream.

$$\therefore \text{ Correction for thickness} = \frac{85 - 78}{1} \times 0.3 \triangleq 2\,\% \,(+)$$

Corrected $\Phi_{C_1} = 78 + 2 = 80\,\%$

(ii) D/s curtain wall

$$d_2 = 2 \text{ m} \; ; \; b = 20 \text{ m}$$

$$\therefore \quad \frac{1}{\alpha} = \frac{d_2}{b} = \frac{2}{20} = 0.1$$

$$\therefore \quad \Phi_E = 29\,\% \quad \text{and} \quad \Phi_D = 21\,\%$$

Assuming a minimum thickness of 0.5 m at the d/s

$$\therefore \text{ Correction to } \Phi_E = \frac{29 - 21}{2} \times 0.5 = 2\,\% \,(-\text{ve})$$

$$\therefore \text{ Corrected } \Phi_E = 29 - 2 = 27\,\%$$

(iii) Toe of d/s galcis

Assuming a linear variation of pressure, the pressure at the d/s of the glacis

$$= 27 + \frac{80 - 27}{20} \times 9 \triangleq 51\,\%$$

Floor thickness

(i) Provide a minimum thickness of 0.3 m at the u/s floor.

(ii) Floor thickness at the toe of glacis :

Minimum static head $= 0.51 \times 2.44 + (215.0 - 214.5) = 1.75$ m

$\therefore$ Floor thickness required

$$= \frac{1.75}{2.24 - 1} = 1.42 \text{ m}$$

Provide 1.5 m thick floor at the toe of the d/s glacis, for length of 3 m.

(iii) *Floor thickness at 3 m from the toe of glacis* :

$$\%\text{ pressure} = 27 + \frac{80 - 27}{20} \times 6 \triangleq 43\,\%$$

$\therefore$ Maximum static head $= 0.43 \times 2.44 + (215.0 - 214.5) = 1.55$ m

$$\therefore \text{ Floor thickness} = \frac{1.55}{2.24 - 1} = 1.25 \text{ m}$$

Provide a thickness of 1.30 m from 3 m to 6.5 m from the toe of the glacis.

(iv) Floor thicknes of d/s end of cistern:

% pressure at d/s curtain wall $= 27\,\%$

$$\text{Thickness} = \frac{0.27 \times 2.44}{2.24 - 1} = 0.53 \text{ m}$$

Provide a thickness of 0.6 m.

Thickness at a point 6.5 m d/s of the toe of glacis $= 1.30$ m

Thus thickness at the d/s end of the floor $= 0.6$ m.

The thickness in between may be varied linearly as shown in Fig. 18.13.

Step 4. Design of d/s protection.

(i) No bed protection is needed since the deflector wall has been provided.

(ii) Side protection : Length of side protection

$$= 3\,D_2 = 3 \times 1.8 = 5.4 \text{ m.}$$

Hence, provide 20 cm thick brick pitching in 5.5 m length beyond the impervious floor at 1 : 1 slope. The pitching will rest on a toe wall 0.4 m wide and 1.8/2 = 0.9 m deep. Provide a 0.4 m wide profile wall at the end of the pitching to protect the side pitching.

(iii) No friction blocks are required since fluming has not been done.

Step 5. Design of u/s approach.

The u/s wing walls may be splayed at 45° from the u/s end of impervious floor and extended 1.0 m into the earthen banks from the line of F.S.L.

The design details are shown in Fig. 18.13.

18.8. MONTAGU TYPE FALL

When hydraulic jump takes place on sloping glacis, the vertical component of velocity remains unaffected. Thus there is always the problem of residual energy below the jump. Hence, the profile of the glacis should be so designed that the maximum horizontal acceleration is imparted to the jet at all states of discharge. By imparting maxi-

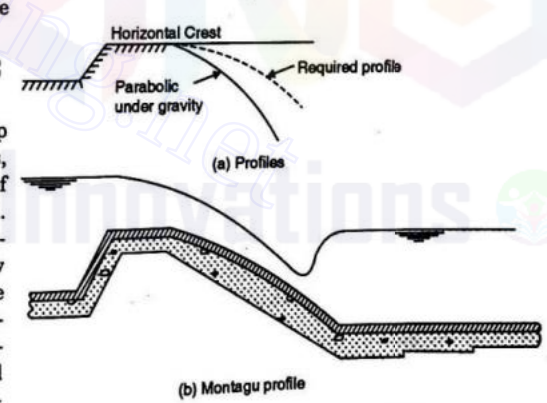

FIG. 18.14 MONTAGU TYPE FALL

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mum horizontal acceleration, the maximum horizontal velocity is attained in any given length of the work.

When a stream is moving on a horizontal bed, there is no horizontal acceleration because the reaction is normal to the bed. Also, when the stream is flowing over a parabolic bed appropriate to the initial horizontal velocity U, there will be no horizontal acceleration because these is no reaction ; the acceleration will be purely vertical under the action of gravity. In between these two, there lies such a glacis profile upon which the horizontal component of the reaction imparts a maximum horizontal acceleration.

Montagu derived analytically the following ideal glacis profile :

$$x = U\sqrt{\frac{4y}{g}} + y \qquad ...(18.22)$$

where U = initial velocity of water leaving the crest,

x = horizontal distance to any point on the profile, measured from the crest.

y = vertical distance to that point, measured below the crest.

Dr. Bose, however, disagreed with the above equation, and gave the following expression for the glacis profile:

$$x = U\sqrt{\frac{2y}{g}} + y \qquad ...(18.23)$$

Design Procedure

All falls of this type are flumed so that the width of the work is reduced. The width of the crest is kept equal to at least 2 to 2.5 times the head over the crest. The discharge is given by

$$Q = 1.7\, L_t E^{3/2}$$

where E = head over the crest.

Assuming that the jump forms at the d/s bed level, the d/s specific energy E_{f2} is known. Knowing E_{f2} and H_L , the discharge intensity q can be found from the Blench curves. Hence, $L_t = Q/q$, and can be found.

The depth of cistern, below the d/s bed, is kept equal to $\frac{1}{2}E_{f2}$ and the length of the cistern is kept four times the depth of the cistern.

The initial velocity U is found from the expression

$$U = \frac{q}{d_c}$$

where d_c is the critical depth .

Knowing U, the glacis profile can be determined. The floor is designed on Khosla's theory. The minimum depth of u/s curtain wall is kept equal to $\frac{1}{2}$ F.S.D. and the depth of d/s curtain wall is kept equal to $\frac{1}{2}$ F.S.D. However, the bottom of d/s curtain wall is taken upto the lowest level of excavation. The d/s wings are flared. Brick pitching is provided in the bed and in the sides after the concrete floor and masonry wings. Friction blocks are provided to dissipate any residual energy. The thickness of the

glacis is checked for the worst condition of the jump trough at different discharge intensities, as explained in Chapter 12 (design of sloping glacis weir).

18.9. INGLIS TYPE FALL

Designed by C.C. Inglis, the fall makes use of horizontal impact for energy dissipation. The design consists of a standard long throated weir flume followed by a glacis slope and a pavement on which a baffle is fixed to dissipate the energy. The baffle holds the jump stable on a horizontal platform. A cistern downstream of the baffle with a deflector at the d/s end of the cistern is provided. Pucca or impervious floor is provided only upto the end of the deflector. To the d/s of the deflector a second cistern in provided, which is only pitched with bricks or stones. (Fig. 18.15)

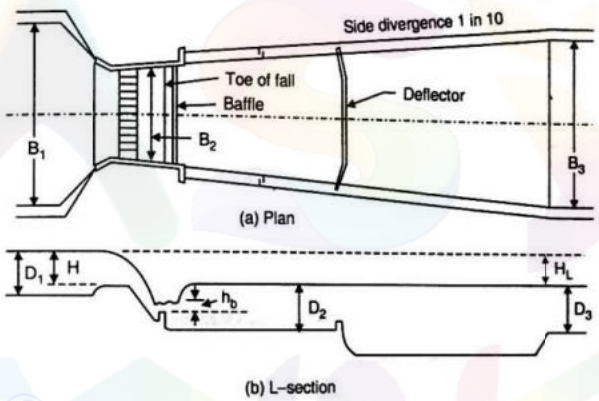

FIG. 18.15. INGLIS TYPE FALL.

The maximum dissipation of energy by a hydraulic jump occurs when the jump forms at the toe of the glacis. However, the jump may also form either on the glacis, or at the d/s of the toe. If the jump forms on the sloping glacis, the baffle should be fixed on a platform at a higher level than the canal bed level so that the natural wave will form at the toe of the platform. If the jump would form at the toe of the fall, the baffle should be fixed at pavement level. If, however, the jump would form at d/s of toe, the glacis should be extended and a cistern provided of such depth as to bring the wave to the toe and the baffle fixed on the bed of the cistern. Even though the energy is effectively dissipated by baffle near the toe of a fall, the distribution of velocities is normal ; hence, a cistern and deflector are provided. The cistern and deflector are meant to restore the normal distribution of velocities in the channel.

(B) CANAL REGULATORS

18.10. OFF-TAKE ALIGNMENT

When a distributing channel takes off from the parent channel, their off-take alignment should be very carefully designed. The best alignment of an off-take is when it makes zero angle with the parent channel, as shown in Fig 18.16(*a*). A transition

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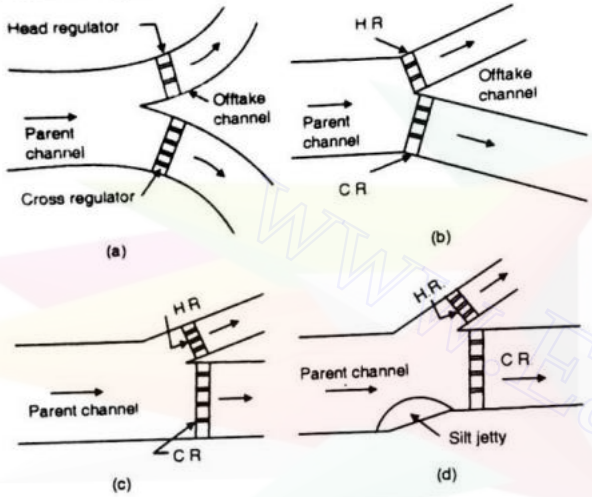
will have to be designed. If the transition curves are not given, both channels (*i.e.* parent channel and the distributing channel) should make an angle with the alignment of the parent channel upstream of the off-take Fig.18.16(*b*).



18.16. OFF-TAKE ALIGNMENT

Under the circumstances when the parent channel has to be kept straight both upstream and downstream of the off-take, the edge of the canal rather than the centre line should be considered in deciding the angle of off-take [Fig.18.16(c)]. In that circumstance, the section should not be narrowed down equally on both sides. An unbalanced off-take results in the formation of a jetty [Fig.18.16(*d*)]. The reduction in sectional area caused by the jetty would be made up by the scouring of the bed along the line of the deviated current.

18.11. HEAD REGULATORS AND CROSS-REGULATORS

Head regulator and cross-regulator regulate the supplies of the off-taking channel and the parent channel respectively. The distributary head regulator is provided at the head of the distributary and controls the supply entering the distributary. It is a necessary link between the parent channel and the distributing channel. A distributary head is a regulator, a metre of supply and a silt selective structure. A cross-regulator is provided on the main canal at the d/s of the off-take to head up the water level and to enable the off-taking channel to draw the required supply.

Functions of distributary head regulator

1. They regulate or control the supplies to the off-taking channel.
2. They serve as a meter for measuring the discharge entering into the off-taking canal.

3. They control the silt entry in the off-taking canal.
4. They help in shutting off the supplies when not needed in the off-taking canal, or when the off taking channel is required to be closed for repairs.

Functions of cross-regulator

1. The effective regulation of the whole canal system can be done with help of cross-regulator.
2. During the periods of low discharges in the parent channel, the cross-regulator raises water level of the u/s and feeds the off-take channel in rotation.
3. It helps in closing the supply to the d/s of the parent channels, for the purposes of repairs etc.
4. They help in absorbing fluctuation in various sections of the canal system, and in preventing the possibilities of breaches in the tail reaches.
5. Incidently, bridges and other communication works can be combined with it.

18.12. DESIGN OF CROSS-REGULATOR AND DISTRIBUTARY HEAD REGULATOR

1. Design of crest

The discharge is determined by the drowned weir formula :

$$Q = \frac{2}{3} C_1 L \sqrt{2g}\ [(h + h_a)^{3/2} - h_a^{3/2}] + C_2 L\, d \sqrt{2g\,(h + h_a)} \qquad ...(18.24)$$

where Q = discharge, in cumecs

L = length of water-way, in metres

h = difference in water level u/s and d/s of the channel, in metres

h_a = head due to velocity of approach

d = depth of d/s water level in the channel, measured above the crest

C_1 = constant = 0.557

C_2 = constant = 0.80.

Generally the velocity of approach is small, and may be neglected while using Eq.18.24. Knowing the discharge Q, the length of water way L can be calculated.

For the cross-regulator, the crest level is kept equal to the upstream bed level of the parent channel. For the distributary head regulator, the crest level is kept 0.3 to 1 m higher than the crest level of the cross-regulator. The crest is joined to the d/s floor with a sloping glacis of 2 : 1.

2. Design of d/s floor

The level and length of the d/s floor is determined under two flow conditions: (*i*) full supply discharge passing through both the head regulator and cross-regulator, and (*ii*) the discharge in the parent channel being insufficient, the cross regulator gate is partially opened and the off-taking channel is running full. Or, the head regulator gate is fully open.

For both of these coditions, the discharge intensity q and the head loss H_L $(= h)$ are known. Hence, the value of E_{f2} can be found from the Blench curves.

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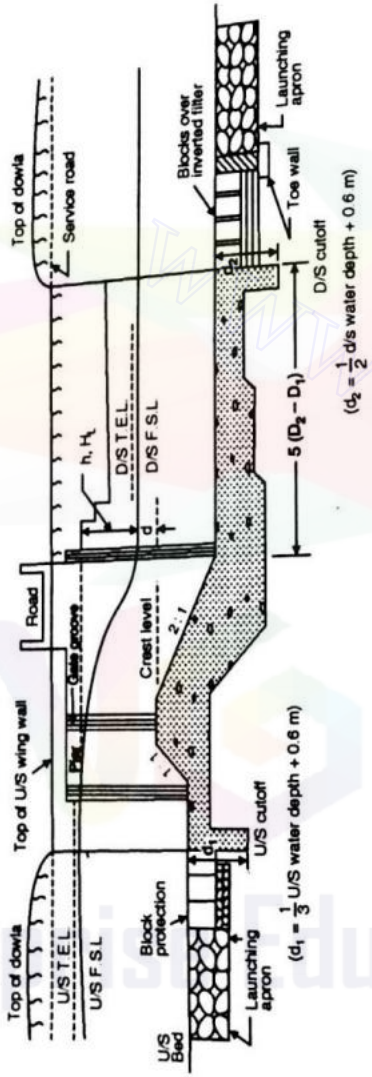

FIG. 18.17. DISTRIBUTARY HEAD REGULATOR.

$$\text{D/s floor level} = \text{d/s T. E. L.} - E_{f2} \triangleq \text{d/s F. S. L.} - E_{f2}$$

However, the d/s floor level, calculated from the above relation should never be provided higher than the d/s bed level.

Now $$E_{f1} = E_{f2} + H_L$$

Hence, the depth D_1 and D_2 corresponding to E_{f1} and E_{f2} respectively are found from specific energy curves.

Then length of d/s floor $= 5(D_2 - D_1)$

However, the d/s floor should be at least 2/3rd of the total impervious length of the floor.

3. Design of impervious floor

Total length of the impervious floor should be found from the consideration of permissible exit gradient.

The depth of u/s cutoff $d_1 = \frac{1}{3}$ u/s water depth + 0.6 m

The depth of d/s cutoff $d_2 = \frac{1}{2}$ d/s water depth + 0.6 m

Maximum static head $H_s =$ u/s F. S. L. − d/s floor level

$$G_E = \frac{1}{\pi\sqrt{\lambda}} \frac{H_s}{d_2}, \text{ from which } \frac{1}{\pi\sqrt{\lambda}} \text{ is known.}$$

Then, from the exit gradient curves, $\alpha\,(= bd_2)$ is known. Hence, the total length b of the impervious floor is known.

The floor thickness is found from the considerations of uplift pressure. A minimum thickness of 0.3 to 0.5 m is provided from the practical considerations.

4. Design of u/s and d/s protection

U/s scour depth d_1 is taken equal ($\frac{1}{3}$ u/s water depth + 0.6 m). The d/s scour depth d_2 is taken equal to ($\frac{1}{2}$ d/s water depth + 0.6 m). These scour depths are below the corresponding bed levels, and protection works are to be designed corresponding to these.

(*a*) ***U/s protection.*** The u/s protection consists of a block protection having cubic contents $= d_1$ cubic metres/m. The cubic contents of u/s launching apron is kept equal to 2.25 d_1 cubic metre/metre width of regulator.

(*b*) ***D/s protection.*** The cubic contents d of d/s inverted filter is kept equal to d_2 cubic metre/metre. The cubic contents of d/s launching apron is kept equal to 2.25 d_2 cubic metre/metre width of regulator.

DESIGN EXAMPLE OF CROSS-REGULATOR AND HEAD REGULATOR

Example 18.3. *Design a cross-regulator and the head regulator for a distributary channel taking off from the parent channel, for the following data.*

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Discharge of parent channel	:	$= 100$ *cumecs*
Discharge of distributary	:	$= 15$ *cumecs*
F.S.L. of parent channel	:	$\frac{u/s}{d/s} = \frac{218.10}{217.90}$
Bed width of parent channel	:	$\frac{u/s}{d/s} = \frac{42\ m}{38\ m}$
Depth of water in parent channel	:	$\frac{u/s}{d/s} = \frac{2.5\ m}{2.5\ m}$
F.S.L. of distributary	:	$217.10\ m$
Bed width of distributary	:	$15\ m$
Depth of water in distributary	:	$1.5\ m$
Permissible exit gradient	:	$= \frac{1}{5}$

Solution

(*a*) **DESIGN OF CROSS-REGULATOR**

Step 1. Design of crest and water-way

Crest level = u/s bed level of parent channel

$= 218.10 - 2.50 = 215.60$ m

Neglecting the velocity of approach, the discharge is given by

$$Q = \frac{2}{3} C_1 L \sqrt{2g}\ h^{3/2} + C_2 L d \sqrt{2gh}$$

Here $Q = 100$ cumecs.

$C_1 = 0.557$; $C_2 = 0.80$

$h = u/s$ F. S. L. – d/s F. S. L. $= 218.10 - 217.90 = 0.20$ m

$d =$ d/s F. S. L. – Crest level $= 217.90 - 215.60 = 2.30$ m

Substituting the values in the above expression, we get

$$100 = \frac{2}{3} \times 0.557\, L \sqrt{2 \times 9.81}\ (0.20)^{3/2} + 0.8\, L\, (2.30) \sqrt{2 \times 9.81 \times 0.20}$$

From which $L = 26.2$ m.

Provide 4 bays of 7 m each with a clear water-way = 28 m

Provide 3 piers of 1.5 m width each.

Total width of cross-regulator $= 28 + 4.5 = 32.5$ m

Step 2. Design of d/s floor

$Q = 100$ cumecs

$$q = \frac{100}{28} = 3.58 \text{ cumecs/m}$$

$H_L = h =$ u/s F. S. L. – d/s F. S. L. $= 218.10 - 217.90 = 0.20$ m

$\therefore$ $E_{f2} = 1.89$ m for $H_L = 0.20$ m and $q = 3.58$ (from Blench curves)

$\therefore$ d/s floor level = d/s F. S. L. – $E_{f2} = 217.90 - 1.89 = 216.01$ m

This is higher than the d/s bed level.

Hence, adopt the d/s floor level = 215.40 = d/s bed level.

$$E_{f1} = E_{f2} + H_L = 1.89 + 2.20 = 2.09 \text{ m}$$

Hence from specific energy curves,

$$D_1 = 0.7 \text{ m and } D_2 = 1.65 \text{ m.}$$

$\therefore$ Cistern length (i.e.length of floor to the d/s of regulator gates)

$$= 5(D_2 - D_1) = 5(1.65 - 0.7) = 4.75 \text{ m.}$$

Total length of impervious floor = 16 m (see next step)

$\frac{2}{3}$ length of impervious floor $= \frac{2}{3} \times 16 = 10.6$ m

Hence, provide length of d/s floor = 10.6 m

Step 3. Design of impervious floor

Depth of u/s cutoff $= d_1 = \frac{1}{3}$ u/s water depth + 0.6 m

$= \frac{1}{3} \times 2.5 + 0.6 = 1.43$ m.

Keep $d_1 = 1.50$ m and width of cutoff = 0.5 m

Depth of d/s cutoff $= d_2 = \frac{1}{2}$ d/s water depth + 0.6 m

$= \frac{1}{2} \times 2.5 + 0.6 = 1.85$ m

Keep $d_2 = 2$ m and width of cutoff = 0.5 m.

Worst case for under-seepage will be when full supply level is maintained in the parent channel u/s of cross-regulator, but no water to its d/s side.

Maximum static head $= H_s =$ u/s F. S. L. – d/s floor level

$= 218.10 - (217.90 - 2.5) = 2.7$ m

$$G_E = \frac{1}{\pi \sqrt{\lambda}} \frac{H_s}{d_2}$$

$$\therefore \quad \frac{1}{5} = \frac{1}{\pi \sqrt{\lambda}} \times \frac{2.7}{2}$$

$$\therefore \quad \frac{1}{\pi \sqrt{\lambda}} = \frac{2}{5 \times 2.7} = 0.148$$

From exit gradient curves, we get

$$\alpha = 8 \text{ when } \frac{1}{\pi \sqrt{\lambda}} = 0.148$$

$$\frac{b}{d_2} = 8$$

or

$$b = 8\, d_2 = 8 \times 2 = 16 \text{ m.}$$

$\frac{2}{3}$ of this length $= \frac{16 \times 2}{3} = 10.6$ m which is more than 4.75 m of d/s length suggested earlier. Hence, the total length of 16 m will be provided as under :

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(i) D/s floor length = 10.6 m

(ii) D/s glacis length with 2 : 1 slope

$= 2\,(215.60 - 215.40) = 0.4$ m

(iii) Balance to be provided upstream = 5 m.

Pressure calculations

(i) *U/s cutoff* :

$$d_1 = 1.5 \text{ m} \; ; \; b = 16 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d_1}{b} = \frac{1.5}{16} = 0.094$$

$$\Phi_{C1} = 100 - 28 = 72\ \%$$

$$\Phi_{D1} = 100 - 19 = 81\ \%$$

and Assuming a minimum floor thickness of 0.5 m at the u/s.

Corrected $\Phi_{C1} = 72 + \dfrac{81 - 72}{1.5} \times 0.5 = 75\ \%$

(ii) *D/s cutoff* $d_2 = 2$ m ; $b = 16$ m

$$\frac{1}{\alpha} = \frac{d_2}{b} = \frac{2}{16} = 0.125$$

$$\Phi_{E2} = 31\ \% \quad \text{and} \quad \Phi_{D2} = 22\ \%$$

Assuming a minimum thickness of 0.6 m at d/s,

Corrected $\Phi_{E2} = 31 - \dfrac{31 - 22}{2} \times 0.6 = 28.3\ \%$

(iii) Toe of the glacis

$$\text{Percentage pressure} = 28.3 + \frac{75 - 28.3}{16} \times 10.6 = 59.3\ \%$$

Static pressure head $= 6.593 \times 2.7 = 1.6$ m

Calculations for thickness

(i) Minimum thickness for u/s floor = 0.5 m.

(ii) Thickness of floor near d/s cutoff

$$= \frac{0.283 \times 2.7}{2.24 - 1} = 0.61 \text{ m.}$$

Hence, provide 0.70 m thick floor for the last 2.1 m length.

(iii) Thickness of floor at the toe of the glacis

$$= \frac{1.6}{2.24 - 1} = 1.29 \text{ m}$$

Provide 1.30 m thick floor at 4 m length.

(iv) Thickness of floor at 4 m from toe of glacis

$$\text{Percentage pressure} = 28.3 + \frac{75 - 28.3}{16} \times 6.6 = 47.5\ \%$$

$$\therefore \quad \text{Thickness} = \frac{0.475 \times 2.7}{2.24 - 1} = 1.03 \text{ m}$$

Hence, provide 1.10 m thick floor for the next 2 m length of the floor.

(v) Thickness of floor at 6 m from toe of glacis

$$\%\ \text{pressure} = 28.3 + \frac{75 - 28.3}{16} \times 4.6 = 41.8\ \%$$

$$\therefore \quad \text{Thickness} = \frac{0.418 \times 2.7}{2.24 - 1} \approx 0.9 \text{ m}$$

Hence, provide 0.9 m thickness for the next 2.5 m.

Summary of floor thickness

(i) 0.5 m thickness of u/s floor.

(ii) 1.30 m thick floor for a distance of 4 m from the toe of glacis.

(iii) 1.10 m thick floor for the next 2 m length.

(iv) 0.90 m thick floor for the next 2.5 m length.

(v) 0.70 m thick floor for the last 2.1 m length.

(vi) Thickness under the gates, for a distance of 1 m = 1.0 m.

Step 4. Design of us protection

Depth of u/s cutoff $= \frac{1}{3}$ F. S. D. + 0.6

or $$d_1 = \frac{1}{3} \times 2.5 + 0.6 = 1.43 \text{ m.}$$

(i) Volume of *block protection*

= 1.43 cubic metre/m

Keep the thickness of protection = 1 m

$\therefore$ Length = 1.43 m

Hence, provide 2 rows of 0.8 m × 0.8 m × 0.6 m thick concrete blocks over 0.4 m thick apron, in a length of 1.6 m.

(ii) Cubic content of *launching apron*

$$= 2.25\, d_1 = 2.25 \times 1.43 = 3.21 \text{ m}^3/\text{m.}$$

Hence provide 1 m thick and 3.5 m long launching apron.

Step 5. Design of d/s protection

Depth of d/s cutoff $= \frac{1}{2}$ F. S. D. + 0.6

or $$d_2 = \frac{1}{2} \times 2.5 + 0.6 = 1.85 \text{ m}$$

(i) Volume of *inverted filter*

$= 1.85 \text{ m}^3/\text{m.}$

Keep thickness of concrete blocks = 0.6 m kept over 0.6 m thick graded filter, making a total thickness of 1.2 m.

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Length $= \frac{1.85}{1.2} = 1.54$ m

Hence, provide 2 rows of 0.8 m × 0.8 m × 0.6 m c.c. block over 0.6 m graded filter for a length of 1.6 m.

(ii) *Lanching apron volume*

$= 2.25 \times d_2 = 2.25 \times 1.85 = 4.16 \text{ m}^3/\text{m}.$

Hence, provide 1 m thick launching apron for a length of 4.5 m.

Provide a 0.4 m thick and 1.5 m deep toe wall between the filter and the launching apron.

The design details have been shown in Fig. 18.18 (*a*).

(*b*) DESIGN OF DISTRIBUTARY HEAD REGULATOR

Step 1. Design of crest and water-way

Crest level = u/s bed level + 0.5

$= (218.10 - 2.5) + 0.5 = 216.10$ m.

Neglecting the velocity of approach, the discharge is given by

$$Q = \frac{2}{3} C_1 L \sqrt{2g}\, h^{3/2} + C_2 L d \sqrt{2gh}$$

where $Q = 15$ cumecs

$C_1 = 0.577$

$C_2 = 0.80$

h = u/s F. S. L. – d/s F. S. L. $= 218.10 - 217.10 = 1.0$ m

d = d/s F. S. L. – Crest level $= 217.10 - 216.10 = 1.0$ m.

Substituting these values in the discharge formula,

$$15 = \frac{2}{3} \times 0.577 \times L \sqrt{2 \times 9.81}\, (1.0)^{3/2} + 0.8 \times L \times 1.0 \sqrt{2 \times 9.81 \times 1.0}$$

From which $L = 2.87$ m.

This is too less in comparison to 15 m width of the offtaking channel.

Hence, provide 2 bays of 3.5 m each with 1 m thick pier in between.

∴ Over all width of regulator = 8 m.

The crest will have u/s glacis of 1 : 1 slope and d/s glacis of 2 : 1 slope.

Keep the width of crest = 1.0 m.

Step 2. Design of d/s floor

$Q = 15$ cumecs

$$q = \frac{15}{7.5} = 2 \text{ cumecs/m}$$

H_L = u/s F. S. L. – d/s F. S. L. $= 218.10 - 217.10 = 1.0$ m.

Hence, for $q = 2$ and $H_L = 1.0$, we get from Blench curves $E_{f2} = 1.58$ m

∴ R.L. of d/s floor = d/s F. S. L. – E_{f2}

$= 217.10 - 1.58 = 215.52$ m

Keep the R.L. of d/s floor = 215.50 m

Again $E_{f1} = E_{f2} + H_L = 1.58 + 1 = 2.58$ m

From specific energy curves, values of D_1 and D_2 corresponding to $E_{f1} = 2.58$ and $E_{f2} = 1.58$ m are :

$D_1 = 0.42$ m and $D_2 = 2.55$ m

∴ Length of cistern $= 5\,(D_2 - D_1)$

$= 5\,(2.55 - 0.42) = 10.5$ m

Total Length of impervious floor

$= 14$ m (from Step 3)

$\frac{2}{3}$ of impervious floor $= 14.0 \times \frac{2}{3} = 9.3$ m

Hence, provide length of cistern (i.e. floor to the d/s of glacis)

$= 10.5$ m.

Step 3. Design of impervious floor

Depth of u/s cutoff $= \frac{1}{3}$ u/s water depth = 0.6 m

$$d_1 = \frac{1}{3} \times 2.5 + 0.6 \simeq 1.5 \text{ m}$$

Hence, provide 0.5 m wide and 1.5 m deep cutoff.

Depth of d/s cutoff $= \frac{1}{2}$ d/s water depth + 0.6 m

$$d_2 = \frac{1}{2} \times 1.5 + 0.6 = 1.35 \text{ m}$$

Now, in this case, the seepage head

= u/s F. S. L. – d/s floor level

∴ $H_s = 218.10 - 215.50 = 2.60$ m.

In the case of the cross-regulator, H_s was equal to 2.70 m and d_2 was equal to 2 m. Hence, in this case also, we should provide $d_2 = 2.00$ m (as against calculated value of 1.35) otherwise the floor length will come out to be excessive (i.e. much more than 16 m length of the previous case).

Hence, provide downstream cutoff of 0.5 wide and 2 m deep.

Now $$G_E = \frac{1}{\pi\sqrt{\lambda}} \frac{H_s}{d_2}$$

$$\frac{1}{5} = \frac{1}{\pi\sqrt{\lambda}} \frac{2.6}{2.0}$$

$$\frac{1}{\pi\sqrt{\lambda}} = \frac{2}{5 \times 2.6} = 0.154$$

From exit gradient curves, value of $\alpha = 7$ for $\frac{1}{\pi\sqrt{\lambda}} = 0.154$

$$b = \alpha\, d_2 = 7 \times 2 = 14 \text{ m}.$$

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The length of 14 m will be provided as under :

1. Length below the toe of glacis $= 10.5$ m
2. Length of d/s glacis at 2 : 1 slope $= 2\,(216.10 - 215.60) = 1.2$ m
3. Width of crest $= 1$ m
4. Length of u/s glacis at 1 : 1 slope $= 216.10 - 215.60 = 0.50$ m
5. U/s floor : balance $= 14 - (10.5 + 1.2 + 1 + 0.5) = 0.8$ m.

However, provide a minimum length of 2.8 m to the u/s, making a total impervious length = 16 m. This increase of 2 m length will accommodate the piers and u/s wings.

Pressure calculations

(*i*) *U/s cutoff* $\quad d_2 = 1.5$ m ; $b = 16$ m

$$\frac{1}{\alpha} = \frac{d_1}{b} = \frac{1.5}{16} = 0.094$$

Hence, $\quad \Phi_{C1} = 100 - 28 = 72\ \%$ and $\Phi_{D1} = 100 - 19 = 81\ \%$

Assuming a minimum floor thickness of 0.5 m at u/s

Corrected $\quad \Phi_{C1} = 72 + \dfrac{81 - 72}{1.5} \times 0.5 = 75\ \%$

(*ii*) *D/s cutoff* $\quad d_2 = 1.5$ m ; $b = 16$ m

$$\frac{1}{\alpha} = \frac{d_2}{b} = \frac{2}{16} = 0.125$$

Hence, $\quad \Phi_{E2} = 31\ \%$ and $\Phi_{D2} = 22\ \%$

Assuming a minimum thickness of 0.6 m at d/s,

Corrected $\quad \Phi_{E2} = 31 - \dfrac{31 - 22}{2} \times 0.6 = 28.3\ \%$

(*iii*) *Toe of glacis*

$$\text{Pressure} = 28.3 + \frac{75 - 28.3}{16} \times 10.5 = 59.2\ \%$$

Calculations of thickness of floor

(*i*) Minimum thickness for u/s floor = 0.5 m

(*ii*) Thickness under the crest = 1 m

(*iii*) Thickness under the toe of the glacis

$$= \frac{0.592 \times 2.6}{2.24 - 1} = 1.24 \text{ m}$$

Provide a thickness of 1.30 m for a distance of 2 m from the toe of the glacis.

(*iv*) Thickness of floor at 2 m from the toe of glacis :

$$\%\ \text{pressure} = 28.3 + \frac{75 - 28.3}{16} \times 9.5 = 53\ \%$$

$$\therefore \quad \text{Thickness} = \frac{0.53 \times 2.6}{2.24 - 1} = 1.11 \text{ m}$$

Provide a thickness of 1.10 m for the next 4 m.

(*v*) Thickness of floor at 6 m from the toe of the glacis :

$$\%\ \text{pressure} = 28.3 + \frac{75 - 28.3}{16} \times 4.5 = 41.4\ \%$$

$$\therefore \quad \text{Thickness} = \frac{0.414 \times 2.6}{2.24 - 1} = 0.84 \text{ m}$$

$\therefore$ Provide a thickness of 0.90 m for the next 2.5 m.

(*vi*) *Thickness at 8.5 m from the toe*

$$\%\ \text{pressure} = 28.3 + \frac{75 - 28.3}{16} \times 2 = 34.1\ \%$$

$$\therefore \quad \text{Thickness} = \frac{0.341 \times 2.6}{2.24 - 1} = 0.71 \text{ m.}$$

Hence, provide 0.7 m thick floor for its 2 m length.

Step 4. Design of u/s protection

$$\text{U/s scour depth} = \tfrac{1}{3} \times 2.5 + 0.6 \simeq 1.5 \text{ m}$$

Hence, provide the same protection as in the case of cross regulator.

Step 5. Design of d/s protection

$$\text{D/s scour depth} = \tfrac{1}{2}\,\text{d/s water depth} + 0.6 \text{ m}$$

$$d_2 = \tfrac{1}{2} \times 1.5 + 0.6 = 1.35 \text{ m}$$

(*i*) *Inverted filter*

$$\text{Volume} = d_2 = 1.35 \text{ m}^3/\text{m}$$

Provide 0.5 m thick cement concrete blocks over 0.5 m thick graded filter.

$$\text{Length required} = \frac{1.35}{1} = 1.35 \text{ m.}$$

However, provide two rows of 0.8 m × 0.8 m × 0.5 m thick concrete blocks over 0.5 thick graded filter.

(*ii*) *Launching apron*

$$\text{Volume} = 2.25\, d_2 = 2.25 \times 1.35 = 3.04 \text{ m}^3/\text{m.}$$

Provide 1 m thick launching apron for a length of 3.5 m. Provide a masonry toe wall 0.4 m wide and 1.20 m deep between the filter and the launching apron.

The design details have been shown in Fig. 18.18(*b*).

18.13. CANAL ESCAPES

A canal escape is a structure constructed on an irrigation canal for the purpose of wasting some of its water. Depending upon the purpose, there can be three types of escapes :

(1) Canal scouring escape,

(2) Surplus escape,

and (3) Tail escape.

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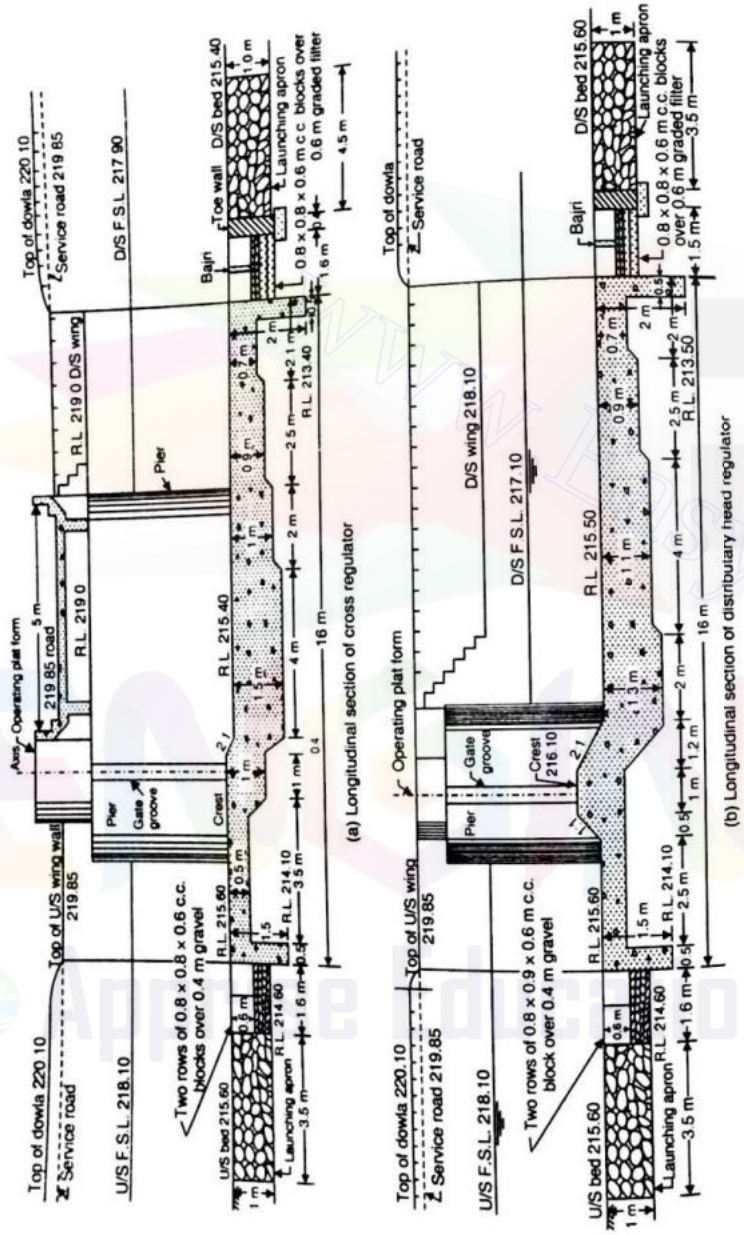

FIG. 18.18. CROSS-REGULATIOR AND DISTRIBUTARY HEAD REGULATOR

The scouring escape is constructed for the purpose of scouring off excess silt from time to time. Escapes are also constructed to dispose off excess supplies of the parent channel. Excess supplies in the canal take place either during heavy rains or due to the closure of canal outlet by the farmers. In that case, the escapes save the d/s section of the canal from overflow of banks.

Escapes are thus essential safety valves for the canals, and should be constructed at intervals. The canal leading the surplus water to a natural drain is known as *escape channel*. The capacity of escape channel should not be less than 50% of capacity of the parent channel at that point. A canal surplus escape may be weir type, with the crest of weir wall at F.S.L. of parent canal bed level [Fig.18.19(*a*)]. A tail escape [Fig.18.19(*b*)] is provided at the tail end of the canal, and is useful in maintaining the required F.S.L. at the tail end. The structure is weir type with its crest level at the required F.S.L. of canal at its tail end.

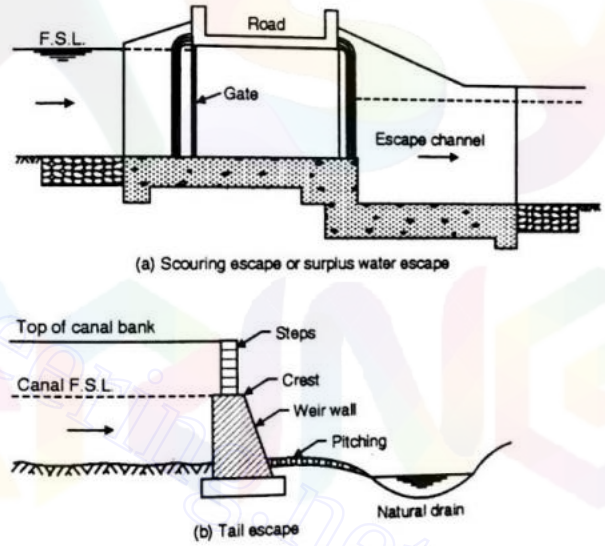

FIG. 18.19. CANAL ESCAPE.

PROBLEMS

1. What do you understand by a fall in a canal ? Why it is necessary? How do you select its location ?
2. Write a note on Notch type fall.
3. What is 'cistern element' in fall ? Give various expressions for its dimensions.
4. Explain the procedure of designing Sarda type fall.

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5. Explain the procedure of designing straight glacis fall.

6. Write a note on Montagu type fall ? How does it differ from glacis fall ?

7. Design a Sarda type fall across a canal for the following data :

(*i*) Full supply discharge $\frac{u/s}{d/s} = \frac{15}{15}$ cumecs

(*ii*) Drop $= 1.0$ m

(*iii*) Full supply level $\frac{u/s}{d/s} = \frac{101.00}{100.00}$ m

(*iv*) Full supply depth $\frac{u/s}{d/s} = \frac{1.8}{1.8}$ m

(*v*) Bed width $\frac{u/s}{d/s} = \frac{10}{10}$ m

Design the floor on Bligh's theory using coefficient of creep = 10. Sketch the longitudinal section of fall.

8. Design an unflumed straight glacis fall for the following data :

(*i*) Full supply discharge $\frac{u/s}{d/s} = 12$ cumecs

(*ii*) Drop $= 1.5$ m

(*iii*) Full supply level $\frac{u/s}{d/s} = \frac{101.50}{100.00}$ m

(*iv*) F.S. Depth $\frac{u/s}{d/s} = \frac{1.50}{1.50}$

(*v*) Bed level $\frac{d/s}{d/s} = \frac{100}{98.5}$ m

(*vi*) F.S. width $\frac{u/s}{d/s} = \frac{8.0}{8.0}$ m

Design the floor on Bligh's theory using coefficient of creep = 8.

Given : For linear discharge $q = 1.5$ cumecs/metre

and $H_L = 1.5$ m, $E_{f2} = 1.4$ m.

9. What do you understand by a head regulator ? State functions of a distributary head regulator and a cross-regulator.

10. Explain the procedure for designing the head regulator of a distributary.

11. Write a note on canal escape.

12. What are the functions of regulators and falls on main canal? How are these works made safe against failure by (a) piping (b) bed scour?

19

Cross Drainage Works

19.1. INTRODUCTION

A cross drainage work is a structure carrying the discharge of a natural stream across a canal intercepting the stream. When a canal is to be taken to the watershed, it crosses a number of natural streams in the distance between the reservoir to the watershed. Once the canal is taken to the watershed, no cross-drainage works are normally necessary except when the canal leaves the watershed for some distance forming a loop. However, when the canal is aligned as a contour canal, a number of cross-drainage works are necessary.

A cross-drainage work is generally a very costly item, and should be avoided as far as possible by (*i*) diverting one stream into another, or (*ii*) changing the alignment of the canal so that it crosses below the junction of two streams.

19.2. TYPES OF CROSS-DRAINAGE WORKS

Depending upon the relative levels and discharges, cross-drainage works may be of the following types :

(1) C.D. Works carrying canal over the drainage

In this type of C.D.work, the canal is carried over the natural drain (Fig. 19.1). The advantage of such arrangement is that the canal, running perennially, is above the ground and is open to inspection. Also, the damage done by floods is rare. However, sometimes during heavy floods the foundation can be scoured, or the waterway of the drain may be chocked with trees etc. This is the usual type of work constructed when the drain is very big in comparison to the section of the canal. The structures that fall under this type are :

1. Aqueduct 2. Syphon aqueduct.

Fig. 19.1(*b*) and (*c*) show the aqueduct and syphon aqueduct respectively. As is clear from Fig. 19.1(*b*), the H.F.L. of the drain is much below the bottom of the canal trough in the case of aqueduct so that drainage water flows freely under gravity. However, in the case of a syphon aqueduct, the H.F.L. of the drain is much higher above the canal bed, and the water runs under syphonic action through the aqueduct barrels. In this case, the water surface level of the flood is depressed when it passes under the canal trough. The bed of the drainage is also lowered.

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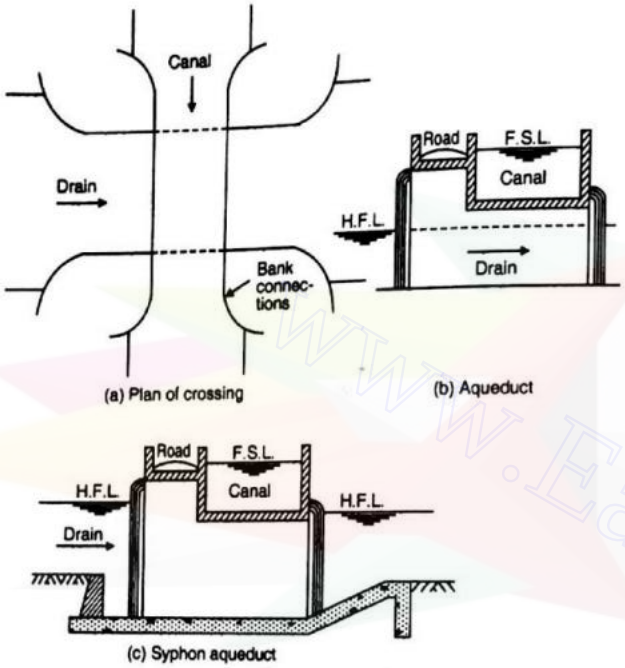

FIG. 19.1. AQUEDUCT AND SYPHON AQUEDUCT.

(II) C.D. Works carrying drainage over the canal

In this type of C.D. work, drainage is carried over the canal (Fig. 19.2). The advantage of this type is that the C.D. works themselves are less liable to damage than the earth-work of the canal.

The major disadvantage of this work is that the perennial canal is not open to inspection. Also, if the silt is deposited in the barrels of the work, it is difficult to clear it out.

The structures that fall under this type are

1. Super-passage
2. Canal syphon.

Fig. 19.2(*b*) shows a super-passage. A super-passage is similar to an aqueduct, except that in this case the drain is over the canal. The F.S.L. of the canal is lower than the underside of the trough carrying drainage water. Thus, the canal water runs under gravity. Fig. 19.2(*c*) shows a canal syphon, or simply *syphon*. In this, case, the levels are such that the F.S.L. of the canal is much above the bed level of the drainage trough, so that the canal runs under syphonic action under the trough. The canal bed is lowered and a ramp is provided at the exit so that the trouble of silting is minimised.

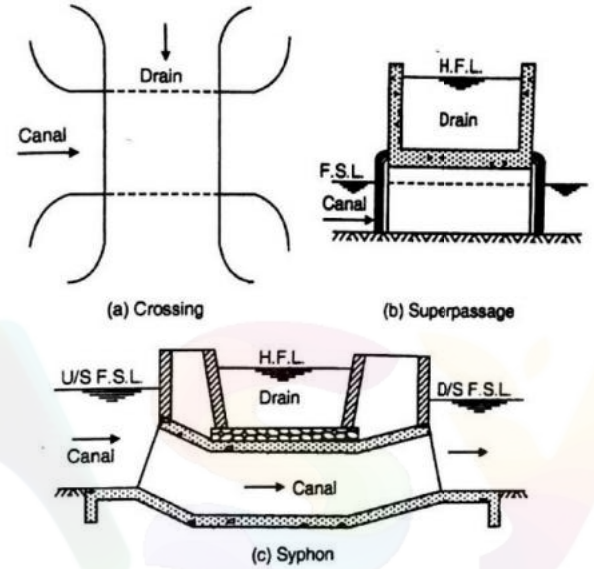

FIG. 19.2. SUPER-PASSAGE AND SYPHON.

(III) C.D. works admitting the drainage water into the canal

In this type of work, the canal water and the drainage water are permitted to intermingle with each other. The only advantage of this type of work is its low initial cost. Such type of works have the following disadvantages :

(*i*) Regulation of such a work is difficult and requires additional staff.

(*ii*) The canal has to be designed to carry the increased flood discharge of the drain.

(*iii*) The faulty regulation of gates may damage the canal.

(*iv*) There is additional expenditure of silt clearance.

Following are the structures under this type C.D. works :

1. Level crossing
2. Inlet and outlets.

Fig. 19.3(*a*) shows the schematic plan of a *level crossing* which is constructed in the circumstance when the beds of the canal and drainage are practically at the same level. In this type of work, the drainage water is passed into the canal and then taken out at the opposite bank. The work consists of (*i*) construction of a crest, with its top at the F.S.L. of the canal, at the u/s junction with the canal, (*ii*) provision of the head regulator across the drainage at its d/s junction with the canal, and (*iii*) a cross-regulator across the canal at its d/s junction with the drainage. When the drainage does not carry any water, its regulator is closed while the cross-regulator of the canal is kept fully open so that the canal flows without any interruption. During the floods, however, the drainage regulator is opened so that the flood discharge, after spilling over the crest and mixing with the canal water, passes through it to the downstream

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of the drainage. The accurate supplies in the canal are maintained by a cross-regulator. Level crossings are suitable for canals of all sizes. They are, however, specially suited to crossings of very large drains when the cost of other cross-drainage works would be very high.

A *canal inlet* [Fig. 19.3*b*] is constructed when cross-drainage flow is small, and its water may be absorbed into the canal without causing appreciable rise. However, if the canal is small, an *outlet* may be constructed to pass out the additional discharge which has entered the canal. It is not necessary that the number of inlets and outlets should be the same. There may be one outlet for two or three inlets.

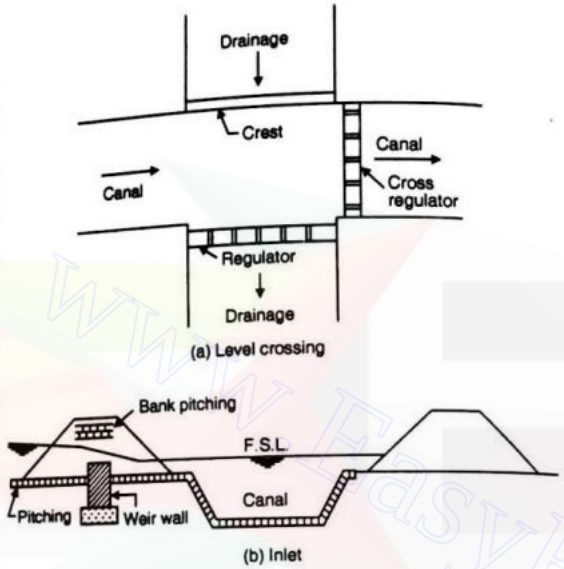

FIG. 19.3. LEVEL CROSSING AND INLET.

19.3. SELECTION OF SUITABLE TYPE OF CROSS-DRAINAGE WORK

The factors which affect the selection of the suitable type of cross-drainage works are (*i*) relative bed levels and water levels of the canal and the drainge, and (*ii*) size of the canal and the drainage. The following considerations are important :

1. When the bed level of the canal is much above the H.F.L. of the drainage, so that sufficient headway is available for floating rubbish etc., and also for the structural elements of the work, an aqueduct is the obvious choice. Similarly, if the bed level of the drain is well above the F.S.L. of the canal, super-passage is provided.

2. The necessary headway between the canal bed level and the drainage H.F.L. can be increased by shifting the crossing to the d/s of the drainage. If, however, it is not possible to change the canal alignment, or if such a shifting does not give sufficient headway between the two levels, a syphon aqueduct may be provided. Thus, in the case of sypon aqueduct, the H.F.L. of the drain is above the bed of the canal.

3. When the canal bed level is much lower, but the F.S.L. of the canal is higher than the bed level of the drainage, a canal syphon is preferred.

4. When the drainage and the canal cross each other practically at the same level, a level crossing may be preferred. This type of work is avoided as far as possible.

The considerations governing the choice between aqueduct and syphon aqueduct (or super-passage and a syphon) are : (*i*) suitable canal alignment, (*ii*) suitable soil available for bank connections and (*iii*) nature of available foundation. As indicated

earlier, the relative difference between the bed level of the canal and H.F.L. of the drainage can be suitably altered by changing the canal alignment so that the point of crossing is shifted upstream or downstrem of the drainage. For example, if the canal alignment is such that headway is not available between the H.F.L. of the drain and the bed of the canal, a syphon aqueduct is to be constructed at the crossing. But, however, if other conditions are not favourable for the construction of the syphon aqueduct, the canal alignment may be changed so that the crossing is shifted to the d/s and sufficient headway required for the construction of an aqueduct is available.

19.4. CLASSIFICATION OF AQUEDUCTS AND SYPHON AQUEDUCTS

Depending upon the nature of the sides of the aqueduct (or syphon aqueduct) it may be classified under three heads :

(*i*) Sides of the aqueduct in earthen banks, with complete earthen slopes [Fig. 19.4 (*b*)].

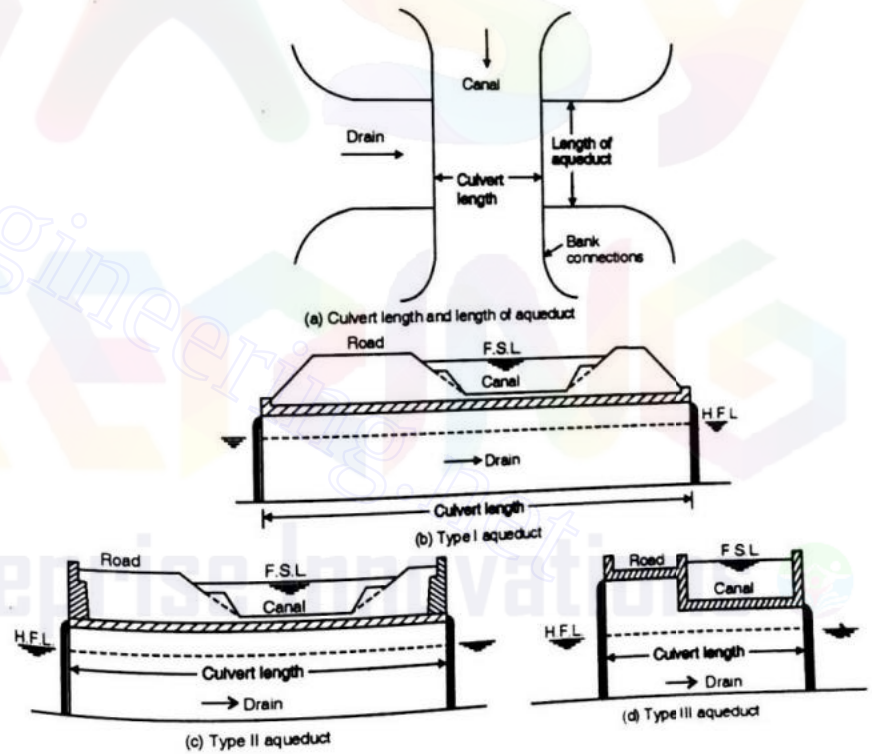

FIG. 19.4. CLASSIFICATION OF AQUEDUCTS

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(*ii*) Sides of the aqueduct in earthen banks, with outer slopes supported by masonry walls [Fig. 19.4 (*c*)].

(*iii*) Sides of the aqueduct made of concrete or masonry [Fig. 19.4 (*d*)].

In the first type, the original canal section is retained and no fluming is done. The sides of the aqueduct are kept in earthen banks with complete earthen slopes. In the third type, on the contrary, the canal section is reduced, fluming is done and the sides of the canal are made of concrete or masonry instead of earth slopes. Type two is intermediate between the two in which the sides of the aqueduct are in earthen banks but the outer section is reduced by supporting it by masonry or concrete walls.

Selection of a suitable type.

In order to select a suitable type out of the three, it is necessary to understand three terms : (*i*) culvert length, (*ii*) length of the aqueduct, and (*iii*) bank connections. *Culvert length* is the width of the aqueduct, measured along the drain. The culvert length naturally depends upon the shape and size of the canal section. The *length of the aqueduct* is the length measured perpendicular to the drain. It is evidently equal to the width of the drain. *Bank connections* consist of masonry wings etc., required to connect the regular section of the canal to its modified section over the aqueduct.

In the first type, since 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 required. Hence, the selection of this type depends upon 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. Hence, first type is suitable when the length of the aqueduct is small and the cost of bank connections would be large in comparison to the saving resulting 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 will be minimum. However, the cost of the bank connections will be additional. Therefore, type III is suitable where the length of culvert is large (*i.e.*, a big drain). Type II is suitable for intermediate conditions. To sum up, for a very large drain, type III is more suitable while for a small drain, type I is more economical. The correct way of selecting a type is to work out cost of the three types and see which is cheaper.

19.5. FEACTURES OF DESIGN OF CROSS-DRAINAGE WORKS

Following the some of the important features of design of cross drainage works.

(A) Hydraulic design :

1. Determination of maximum flood discharge and the high flood level (H.F.L.).
2. Fixation of waterway of the drain.
3. Contraction of canal waterway (for type III aqueducts)
4. Clearances and free board
5. Head loss through syphon barrels.
6. Determination of uplift pressure on the roof of trough.
7. Determination of uplift pressure on the floor.
8. Design of bank connections.

(B) Structural design

1. Design of the cross-section of the syphon barrels and the aqueduct trough.
2. Design of piers and abutments.
3. Design of foundations.

Determination of maximum flood discharge

Chapter 4 (Hydrology) deals with the various factors which affect the runoff from a catchment, and with the methods of determining the high flood discharge of a stream. For small drains, the empirical formulae may be used, while for large drains other reliable methods, such as hydrograph analysis etc. may be used.

IS Code recommendations (IS : 7784, Part I, 1975)

It is quite probable that a flood of higher magnitude than the design flood may pass through the structure posing greater danger to the stability of foundation and the structure. To take care of this unprecedented and unforeseen nature of flood intensities in cases of important structures, an adequate margin of safety is envisaged in the estimation of design discharge. For this purpose, the design discharge may be increased by the percentages given below for obtaining the *foundation* and *free board* design (Table 19.1)

TABLE 19.1

S.No.	*Catchment area*	*Increase in design discharge*
1.	Upto 500 km^2	35 percent to 25 percent decreasing with increase in area
2.	500 to 5000 km^2	25 percent to 20 percent decreasing with increase in area
3.	5000 to 25000 km^2	20 percent to 10 percent decreasing with increase in area
4.	Above 25000 km^2	Upto 10 percent

19.6. FIXATION OF WATERWAY OF THE DRAIN

The fixation of waterway of the drain is very important, as it may otherwise endanger the work. The waterway can be determined by Lacey's *P-Q* formulae :

$$P = 4.75\sqrt{Q}$$

where P = wetted perimeter, in metre

Q = total discharge in cumecs.

For large drains, the wetted perimeter may be taken equal to the width of the river, and hence equal the waterway required. However, for small drains, width may be adjusted to provide the required perimeter. A contraction upto 20% of waterway is permisible in case of small drains. In large drains, however, no extra provision is made for the space occupied by the piers. After having known the waterway, the size of the barrels is so fixed that a minimum velocity of 2 and 3 metere per second is maintained. Afflux is also limited in such a way that there is no flooding of the country side. In case of syphon aqueduct, it is desirable to leave some clear headway (min. 1 m) between the bed level of the drainage downstream and the underside of the culvert roof.

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IS Code recommendations (IS : 7784, Part I, 1975)

In plains, the drainages are generally in alluvium and water way usually providing works without rigid floor is about 60 to 80 percent of perimeter, given by Lacey formula

$$P_w = C\sqrt{Q}$$

where P_w = wetted perimeter in m

C = a coefficient varying from 4.5 to 6.3 according to local conditions, the usual value adopted being 4.8 for regime channels

Q = design discharge in m^3/s.

The wetted perimeter obtained from the above formula is the *total* waterway between the two faces of the abutment.

Maximum permissible velocities

In works with rigid floors, however, water way can be further flumed within the permissible velocity negotiated through the available ventages. Ordinarily such velocities should be limited to the values given in Table 19.2.

TABLE 19.2. MAXIMUM PERMISSIBLE VELOCITIES

S. No.	Floor	Max. permissible velocity (m/s)
1.	Steel and C.I. lined face	10
2.	Concrete face	6
3.	Stone masonry face with cement pointing	4
4.	Stone or brick masonry face with cement plaster	4
5.	Brick masonry with cement pointing	2.5
6.	Hard rock	4
7.	Murram	2 to 3
8.	Soil ; silt	1 to 2

For submountainous and mountainous terrains with flashy flows, the drainage water way is provided within the width of the existing stream. Where the slope of natural stream is quite steep, a slope flattening is usually required to bring down the velocity within the desired limits.

The minimum dimension of opening should be such as shall permit as far as possible manual clearing of deposits therein.

19.7. CLEARANCES AND FREEBOARD : IS CODE RECOMMENDATIONS

(a) Clearance for aqueduct

Clearance or headway in the case of aqueduct is the vertical distance between the u/s H.F.L. inclusive of afflux, of the drainage and the bottom surface of the trough.

Clearance for rectangular openings : The clearance will depend upon the relative levels of the canal bed and highest flood level of the drainage. The following values are suggested.

TABLE 19.3. MINIMUM VERTICAL CLEARANCE FOR RECTANGULAR OPENINGS.

S.N.	Discharge of drainage (m^3/s)	Minimum vertical clearance (mm)
1.	Below 30	600
2.	30 and above but below 300	900
3.	300 and above but below 3000	1200
4.	More than 3000	1500

If the minimum clearances specified above are not available, the safety of the superstructure shall be ensured against likely repercussions.

Clearance for arch openings : Minimum clearance measured to the crown of the arch should be normally be as recommended in Table 19.4.

TABLE 19.4. MINIMUM CLEARANCE FOR ARCH OPENINGS

S.No.	Arch opening (m)	Clearance
1.	Less than 3	Rise or 1.25 m whichever is more
2.	3 and above but less than 6	2/3 rise or 1.5 m whichever is more
3.	6 and above but less than 21	2/3 rise or 1.75 m whichever is more
4.	Above 21	2/3 rise

In the case of streams where a bed rise due to progressive silting is anticipated, the permissible clearance specified in Table 19.4 should be increased to allow for such aggradations depending upon the extent of silting.

(a) Clearance for super passage : In the case of super passage, clearance is the vertical distance between the u/s F.S.L. of the canal and the bottom surface of the drainage trough or the lowest point of the superstructure. Clearances of about 50% of those recommended for aqueducts may be provided in case of super passage.

(b) Free board

In the case of canals, free board is the difference between the F.S.L. of the canal to the top level of banks or the formation level of the guide banks. In the case of drainage, the free board is the difference between the H.F.L., including afflux, and the top of the embankment or guide banks.

The free board should not be less than 900 mm. Wherever heavy wave actions are anticipated, the free board should be increased suitably.

19.8. CONTRACTION OF CANAL WATERWAY

In type I aqueducts, the canal cross-section is not changed at the crossing, so that the culvert length is maximum. This is not economical on big drains. In such cases, type III aqueducts are adopted, where the original width of the canal is generally reduced. This requires the provision of extra transition wings for joining the flumed portion or the trough gradually to the normal section. The fluming or the contraction of the canal waterway should be done in such a way that the velocity in the trough is not more than 3 m/sec and the flow remains sub-critical without any formation

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of hydraulic jump. The approach transition wings should not be steeper than 30° (corresponding to a splay of 2 : 1) and the departure transition should not be steeper than $22\frac{1}{2}°$ (corresponding to a splay of 3 : 1). The transitions consist of curved and flared wing walls so that there is minimum loss of head and the flow is streamlined.

The transition can be designed under two differnt conditions:

(1) Design of transitions when the water depth remains constant.

(2) Design of transition when the water depth varies.

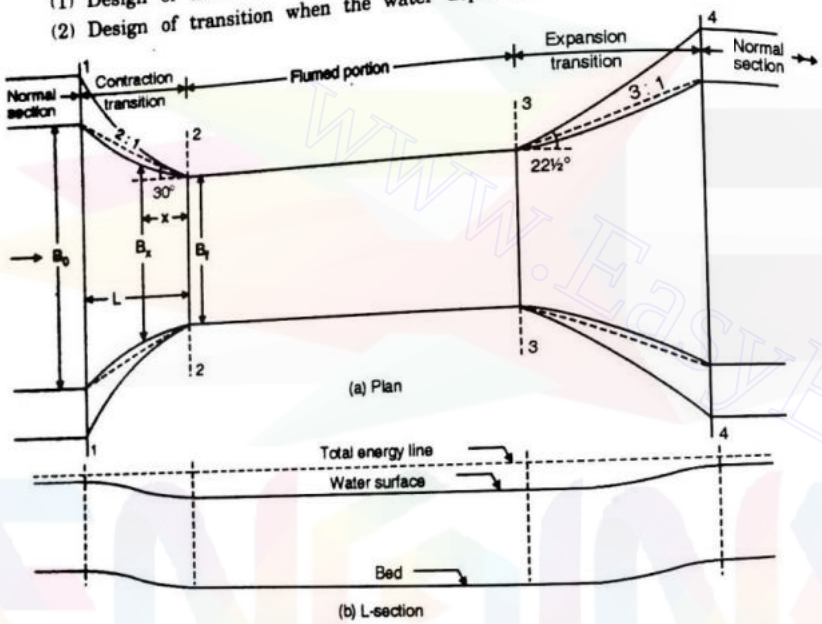

FIG. 19.5. CONTRACTION OF CANAL WATERWAY

(1) DESIGN OF TRANSITION WHEN THE WATER DEPTH REMAINS CONSTANT

(a) Mitra's hyperbolic transition

A.C. Mitra proposed hyperbolic transition for the water-depth in the trough to remain constant. Refer Fig. 19.5.

Let B_0 = original or normal width of canal section

B_f = flumed width of canal section

B_x = width of any distance x from the flumed section

Let V_0, V_f and V_x be the velocities at the corresponding sections

Let L_f = total length of transition

The transition is designed on the criterion that the rate of change of velocity per unit length of the transition is constant throughout the length of the transition. Thus

$$\frac{V_f - V_x}{x} = \frac{V_f - V_0}{L_f} \quad \text{...(i)}$$

Let Q = discharge (constant) and D = water length (constant)

$$V_f \times B_f \times D = V_x \times B_x \times D = V_0 \times B_0 \times D = Q$$

$$V_f \times B_f = V_x \times B_x = V_0 \times B_0 = \frac{Q}{D} = \text{constant} = k$$

Hence, $$V_f = \frac{k}{B_f}\ ;\ V_x = \frac{k}{B_x} \text{ and } V_0 = \frac{k}{B_0}$$

Substituting these values of velocities in (i) we get

$$\frac{\left(\frac{k}{B_f} - \frac{k}{B_x}\right)}{x} = \frac{\left(\frac{k}{B_f} - \frac{k}{B_0}\right)}{L_f}$$

or $$B_x = \frac{B_0 B_f L_f}{L_f B_0 - (B_0 - B_f)\, x} \quad \text{...(19.2)}$$

This is Mitra's hyperbolic expression for bed width at any distance from the section.

(b) Chaturvedi's semi-cubical parabolic transition

Prof. R. S. Chaturvedi proposed the following equation of transition for constant depth, on the basis of his own experiments

$$x = \frac{L_f B_0^{3/2}}{B_0^{3/2} - B_f^{3/2}} \left\{ 1 - \left(\frac{B_f}{B_x}\right)^{3/2} \right\} \quad \text{...(19.3)}$$

Choosing various convenient values of B_x, the corresponding distance x can be computed from the above expression.

(2) DESIGN OF TRANSITION WHEN THE WATER DEPTH ALSO VARIES : HIND'S METHOD

When the water depth in the trough and the normal section of the canal vary, Hind's method given below may be used.

Fig. 19.5 shows the contraction and expansion transitions. The contraction starts at section 1-1 and ends at section 2-2. The canal section remains flumed from section 2-2 to 3-3. The expansion transition starts from section 3-3 and ends at section 4-4. Let D and V, with appropriate suffixes represent depth and velocity at different sections. The design is done in the following steps :

Step 1. Let the bed level and cross-section of the canal at section 4-4 be completely known.

Let the *total energy level* at any point be abbreviated as T.E.L.

Water surface elev. at section 4 = Bed level at section 4 + D_4 and

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T.E.L. at section 4 = Water surface elev. at section 4 $+ \frac{V_4^2}{2g}$.

Step 2. Energy loss due to expansion between section 3 and 4 may be taken as $= 0.3\left(\frac{V_3^2 - V_4^2}{2g}\right)$. The friction loss between 3 and 4 is generally small and may be neglected. Since trough dimensions at 3 are known, V_3 is known.

Step 3. T.E.L. at section 3 = T. E. L. at $4 + 0.3\frac{V_3^2 - V_4^2}{2g}$

Water surface elev. at 3 = T.E.L. at $3 - \frac{V_3^2}{2g}$.

Bed level at section 3 = Water surface elev. at $3 - D_3$.

Step 4. The channel section between 3 and 2 remains constant. The only loss of head in the trough is due to friction and can be worked out from the Manning's formula :

$$Q = \frac{1}{N} A R^{2/3} S^{1/2}$$

$\therefore$ T. E. L. at 2 = T. E. L. at 3 + Head loss

Water surface elev. at 2 = T. E. L. at $2 - \frac{V_2^2}{2g}$

Bed level at 2 = Water surface elev. at $2 - D_2$.

The depth and velocity in the trough are constant throughout and hence the T.E.L., water surface line and bed line are parallel to each other from section 2 to 3.

Step 5. Energy loss due to contraction between section 1 and 2 may be taken equal to $0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$. The frictional loss may be neglected.

T.E.L. at 1 = T. E. L. at $2 + 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$

Water surface elev. at 1 = T. E. L. at $1 - \frac{V_1^2}{2g}$

Bed level at 1 = Water surface at $1 - D_1$.

Step 6. The bed level, water surface level and total energy level at all the four sections are known. The total energy line may be drawn straight between adjacent sections. The bed line is also drawn straight if the fall in level between the adjacent sections is small, and the corners can be rounded off. In case the drop in the bed is more, the sections are joined with a smooth reverse curve tangential to the bed.

Step 7. The drop in the water surface levels between any two adjacent sections will be due to (*i*) drop in the energy line between the two sections, and (*ii*) increased velocity head for contraction (or decreased velocity head for expansion). This drop is

achieved by two smooth parabolic curves, first convex and second concave upwards, meeting the water surface tangentially, as shown in Fig. 19.6.

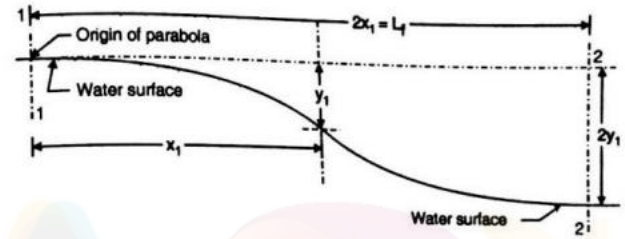

FIG. 19.6. WATER SURFACE CURVE FOR CONTRACTION TRANSITION.

Let $2x_1 = L_f =$ length in which fluming has been done.

$2y_1 =$ total difference in water levels between 1 and 2.

The distance of the middle point of transition will be x_1 and drop in water surface will be y_1. The equation of parabola, with origin at water surface of section 1, is given by

$$y = cx^2$$

When $x = x_1,\ y = y_1$

and hence $c = \frac{y}{x^2} = \frac{y_1}{x_1^2}$

Hence the equation reduces to

$$y = \frac{y_1}{x_1^2} x^2 \qquad ...(19.4)$$

This is the equation of the first parabolic curve, which can now be plotted. Similarly, the second parabola can be plotted by taking the origin at section 2.

Similarly, the expansion transition water surface between section 3 and 4 can be plotted. There will be rise in the water surface.

Step 8. After having plotted the water surface profile over the whole length, the velocity head h_a can be found by measuring the vertical distance between the T. E. L. and water surface line at any point. The velocity head can then be converted into equivalent velocity by the relationship

$$V = \sqrt{2gh_a} \qquad ...(19.5)$$

Thus the velocity at each point can be known.

Step 9. The cross-sectional area at any point, required to the given discharge is given by

$$A = \frac{Q}{V} \qquad ...(19.6)$$

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In trapezoidal channel, with water depth D, bed with B and side slopes $s:1$, the area is given by

$$A = BD + sD^2 \quad ...(19.7)$$

In flared wings, the side slopes are gradually brought to vertical from an initial slope and, therefore, the side slope s at any point can be interpolated in proportion to the length of transition undergone. Thus, at any point, A, D and s are known. Substituting these in Eq. 19.7, the bed width B at any point is known. Thus all the dimensions of the transition are fully known.

19.9. HEAD LOSS THROUGH SYPHON BARRELS

In case of canal syphon and syphon aqueducts, the following formula by Unwin is generally used to calculate the head loss :

$$h = \left(1 + f_1 + f_2 \frac{L}{R}\right)\frac{V^2}{2g} - \frac{V_a^2}{2g} \quad ...(19.8)$$

where h = head loss through the syphon, in metres

L = length of the barrel, in metres

R = hydraulic mean depth of the barrel

V = velocity of flow in metres/second

V_a = velocity of approach, and is generally neglected

f_1 = coefficient of head loss at entry

= 0.505 for unshaped mouth

and = 0.08 for bell mouth.

$f_2 = a\left(1 + \frac{b}{R}\right)$, where the values of a and b for different materials may be taken as follows :

Nature of surface of barrel	a	b
1. Smooth iron pipe	0.00497	0.025
2. Encrusted pipe	0.00996	0.025
3. Smooth cement plaster	0.00316	0.030
4. Ashlar or brickwork	0.00401	0.070
5. Rubble masonry or stone pitching	0.00507	0.250

The velocity through the syphon barrel is limited between 2 to 3 metres per second. Thus, knowing the velocity, the head required to generate the same can be found from Eq. 19.8. The H.F.L. of the drain d/s of work remains unchanged, but the H.F.L. of the u/s side increases by the amount of afflux or total head loss as calculated by Unwin's equation. The top of guide banks and other marginal bunds etc., should be governed by this raised H.F.L. The extent of bed protection in the drain to d/s of the work depends upon the value of this afflux. Thus, by permitting a higher value of afflux (and thereby higher velocity through the syphon barrel) the cross-sectional area of the syphon barrel is reduced, but there is corresponding increase in the cost due to higher guide banks, etc., and greater length of protection d/s of the work. Also in order to reduce the afflux, the entry should be smooth, by providing

bell mouth shaped piers. The inside surface of the barrel should be made as smooth as possible.

Example 19.1. *Design an expansion transition for a canal for the following data, using Mitra's method.*

Length of flume = 16 m

Width of throat = 9 m

Width of canal = 15 m

Solution : Given : $B_f = 9$ m ; $B_0 = 15$ m ; $L_f = 16$ m

Let B_x = width at any distance x from the flumed section.

Now
$$B_x = \frac{B_0 B_f L_f}{L_f B_0 - (B_0 - B_f)x}$$

$$\therefore \quad B_x = \frac{15 \times 9 \times 16}{16 \times 15 - x(16-9)} = \frac{2160}{240 - 6x}$$

Let us find the values of B_x, at $x = 2, 4, 6, 8, 10, 12, 14$, and 16 m, as tabulated below :

x (m)	2	4	6	8	10	12	14	16
B_x (m)	9.47	10	10.59	11.25	12	12.86	13.85	15

Example 19.2. *Solve Example 19.1 using Chaturvedi's method.*

Solution

$$x = \frac{L_f B_0^{3/2}}{B_0^{3/2} - B_f^{3/2}}\left\{1 - (B_f/B_x)^{3/2}\right\}$$

$$\therefore \quad x = \frac{16\,(15)^{3/2}}{15^{3/2} - 9^{3/2}}\left\{1 - \left(\frac{B_f}{B_x}\right)^{3/2}\right\} = 29.893\left[1 - (B_f/B_x)^{3/2}\right]$$

$$\therefore \quad (B_f/B_x)^{3/2} = 1 - (x - 29.893)$$

The computations are tabulated below.

x (m)	2	4	6	8	10	12	14	16
$(B_f/B_x)^{3/2}$	0.933	0.8662	0.7993	0.7324	0.6655	0.5986	0.5317	0.4648
B_f/B_x	0.9549	0.9087	0.8613	0.8125	0.7622	0.7102	0.6563	0.6000
B_x	9.43	9.90	10.45	11.08	11.81	12.67	13.71	15.000

19.10. UPLIFT PRESSURE ON THE ROOF OR TROUGH

The hydraulic gradient line can be drawn when once afflux is known by Unwin's formula. Fig. 19.7 shows the hydraulic gradient line, along with the entry loss, the frictional loss and the exit loss. The uplift pressure at any point under the roof or the trough carrying canal water will be equal to the vertical ordinate between the hydraulic gradient line and the underside of the trough at that point. Maximum uplift pressure will be at the u/s point, near the entry. The uplift pressure on the underside

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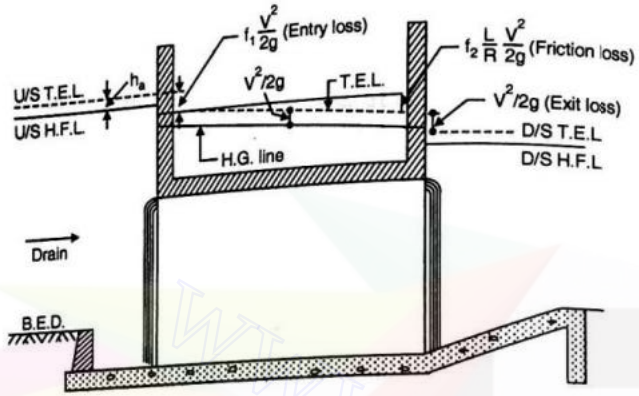

FIG. 19.7. UPLIFT PRESSURE ON THE TROUGH.

of the trough at the d/s end will be equal to the difference in the d/s water and the under side of the trough.

The trough is to be designed under two conditions of flow :

Condition I. There is high flood in the drain but no water is flowing in the canal. This case will give maximum uplift pressure which should be resisted both by gravity action as well as the bending action. The roof thickness will be excessive if the uplift is resisted by gravity action alone. The increase in thickness will result in lowering the levels of both the underside of the trough and the floor. This, in turn, increases the uplift pressure on the trough as well as the floor. The best course, therefore, would be to provide the thickness of the trough for condtion (II), and resist the remaining uplift by bending action by providing reinforcement at the top. In that case, the trough has to be anchored down to the bottom floor through piers by steel bolts etc.

Condition II. The second critical condition will be when the canal is running full, but there is no uplift from the underside. The trough is to be designed to take this water load, by providing a reinforced concrete roof with reinforcement at its bottom. A better alternative will be to provide a box culvert of reinforced concrete construction.

19.11. UPLIFT PRESSURE ON THE FLOOR OF THE AQUEDUCT

The floor of the aqueduct is subjected to uplift pressure due to two causes :

1. Uplift pressure due to seepage of water from the canal to the drainage: The maximum uplift pressure due to the seepage occurs when the canal is running full, but there is no water flowing in the drain.

2. Uplift pressure due to subsoil water table in the drainage bed. This acts when the floor is depressed below the bed of the drainage, specially in the case of syphon aqueducts. The worst condition here also will be when there is no water flowing in the drain, but the water table has risen upto the drainage bed. The uplift

pressure due to this static head will be equal to the difference in the level between the canal bed and the underside of the floor.

The determination of uplift pressure at any point under the floor due to cause (1) is highly complicated since the flow pattern is three dimensional. No approximation to a two dimensional case is possible as there is no typical plane across which the flow is practically two dimensional. As an approximate guide, Bligh's creep theory can be used to find the uplift pressure and to design the floor. For major works, this can be tested by model studies. Bligh's method is described below with reference to Fig. 19.8.

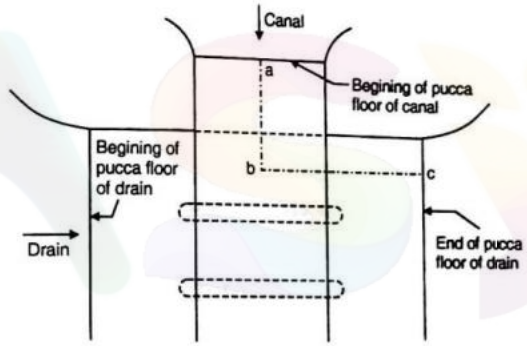

FIG. 19.8. UPLIFT PRESSURE UNDER THE FLOOR.

The flow starts from a point a, underside of the canal trough at its u/s end and then comes out at the point c under the centre of floor of the first culvert at its end. Point b is the point under the centre of the floor of first culvert.

The seepage path from a to b and from b to c can be known.

The total creep length ($\hat{L}$) will then be equal to the sum of the seepage path L_1 from a to b and the seepage path L_2 from b to c.

If $H =$ total seepage head = F.S.L. of canal − d/s bed level of the drain, the residual seepage head at the point b is given by (Bligh's assumption) :

$$H_r = \frac{H_s}{\hat{L}} \times L_2 \qquad ...(19.9)$$

The floor of the aqueduct is to be designed for the total uplift pressure, which is equal to the sum of the uplift pressure due to seepage head plus the uplift pressure due to static head. The total uplift should be partly resisted by the weight of the floor and partly by the bending action.

Methods of reducing uplift on the floor

The uplift pressure on the floor can be reduced by the following two methods:

1. By extending the impervious length of the canal bed on either side, the creep length is increased. If a concrete floor is uneconomical, a puddle apron may be provided.

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2. By providing drainage holes or relief holes in the floor of the aqueduct, the uplift pressure can be released. An inverted filter may be provided below the floor so that the subsoil particles are not lifted up. It would also be necessary to provide flap valves on the top of relief holes, so that seepage water from beneath may come up but the silt laden water may not enter the filter and choke it.

19.12. DESIGN OF BANK CONNECTIONS

The bank connections consist of :

(*i*) River Wings (*ii*) Canal Wings

(*i*) River Wings

The river wings or the drainage wings should be so designed that

(*a*) there is smooth entry and exit.

(*b*) they retain the earth behind it when no water is flowing in the drain.

(*c*) their foundation is taken below the scour level.

(*ii*) Canal Wings

The canal wings are generally warped to change the canal section from trapezoidal to rectangular in the trough. The wings are extended to the end of splay. They should be designed as retaining walls, with no water in the canal. They should be taken deep enough to give safe creep length.

19.13. DESIGN EXAMPLE OF A SYPHON AQUEDUCT

Design a syphon aqueduct for the following data :

(*i*) *Discharge of the canal : 25 cumecs.*

(*ii*) *Bed width of the canal : 20 m.*

(*iii*) *Depth of water in the canal : 1.5 m.*

(*iv*) *Bed level of the canal : 160.00 m.*

(*v*) *High flood discharge of the drainage : 400 cumecs.*

(*vi*) *High flood level of the drainage : 160.50 m.*

(*vii*) *Bed level of the drainage : 158.00 m.*

(*viii*) *General ground level : 160.00 m.*

Solution.

Since the size of the drainage is quite large, type III aqueduct will be economical, and, hence, is adopted. The canal section will be flumed and the canal water will be taken in a concrete trough.

Step 1. Design of drainage water-way

The wetted perimeter of the river is found by Lacey's *P-Q* formula :

$$P = 4.75\,Q = 4.75\sqrt{400} = 95 \text{ m.}$$

Provide 13 spans of 6 m each, separated by 12 piers each of 1.25 m thick.

Length of clear water way $= 13 \times 6 = 78$ m

Length occupied by piers $= 12 \times 1.25 = 15$ m

Total length of water way $= 78 + 15 = 93$ m

Let the velocity through syphon barrels = 2 m/sec.

$\therefore$ Height of barrels required $= \dfrac{400}{78 \times 2} = 2.56$ m

Provide rectangular barrels 6 m wide and 2.50 m high

Actual velocity through barrels $= \dfrac{400}{6 \times 2.50 \times 1.3} = 2.05$ m/sec

Step 2. Design of canal water-way :

Type III aqueduct will be adopted.

Bed width of the canal = 20 m.

Let the width be reduced to 10 m

Providing a splay of 2 : 1 in contraction the length of contraction transition

$$= \frac{20 - 10}{2} \times 2 = 10 \text{ m}$$

Providing a splay of 3 : 1 in expansion, the length of expansion transition.

$$= \frac{20 - 10}{2} \times 3 = 15 \text{ m}$$

Length of the flumed portion from abutment to abutment = 93 m.

In the transitions the side slopes of the canal section will be warped from the original slopes of $1\frac{1}{2} : 1$ to vertical.

Step 3 : Design of levels at different sections

Refer Fig. 19.5.

(*a*) *At Section 4–4*

Area of section with $1\frac{1}{2} : 1$ side slopes

$$= (B + 1.5\,D)\,D = (20 + 1.5 \times 1.5)\,1.5 = 33.4 \text{ m}^2$$

Velocity of flow $= V = \dfrac{Q}{A} = \dfrac{25}{33.4} = 0.75$ m/sec

Velocity head $= \dfrac{V^2}{2g} = \dfrac{(0.75)^2}{2 \times 9.81} = 0.029$ m

R.L. of bed = 160.00 m (given)

R.L. of water surface

$= 160.00 +$ Water depth $= 160.00 + 1.5 = 161.5$ m

R.L. of T.E.L. $= 161.5 + 0.029 = 161.529$ m

(*b*) *At Section 3-3*

Area of trough $= 10 \times 1.5 = 15$ m^2

$\therefore$ Velocity $= \dfrac{Q}{A} = \dfrac{25}{15} = 1.665$ m/sec.

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Velocity head $= \frac{V^2}{2g} = \frac{(1.665)^2}{2 \times 9.81} = 0.141$ m.

Loss of head in expansion, from Section 3-3 to 4-4

$$= 0.3 \left\{ \frac{V_3^{\,2} - V_4^{\,2}}{2g} \right\} = 0.3\,(0.141 - 0.029) = 0.034 \text{ m}$$

Elev. of T.E.L., at 3-3

= T.E.L. at 4 + Head loss $= 161.529 + 0.034 = 161.563$ m

R.L. of water surface at 3

= T.E.L. − Velocity head $= 161.563 - 0.141 = 141.422$ m.

∴ R.L. of bed, to maintain constant water depth

= R.L. of water surface − Water depth $= 161.422 - 1.5 = 159.922$ m.

(c) *At Section 2-2*

Hydraulic mean depth in the trough

$$= R = \frac{A}{P} = \frac{15}{10 + 2 \times 1.5} = 1.155$$

Now $\quad V = \frac{1}{N} R^{2/3} S^{1/2}$ (Manning's formula)

or $\quad S = \frac{V^2 N^2}{R^{4/3}}$

where V = Velocity of flow in the trough

S = Required slope

N = Rugosity coefficient = 0.016

$$\therefore \quad S = \frac{(1.665)^2\,(0.016)^2}{(1.155)^{4/3}} = 5.85 \times 10^{-4}$$

Length of trough (flumed portion) = 93 m

∴ Head loss in trough $= 93 \times 5.85 \times 10^{-4} = 0.054$ m.

∴ R.L. of T.E.L. at 2 = R.L. of T.E.L. at 3 + Head loss in trough

$= 161.563 + 0.054 = 161.617$ m

R.L. of water surface at 2 $= 161.617 - 0.141 = 161.476$ m

R.L. of bed, to maintain constant water depth

= R.L. of water surface at 2 − Water depth

$= 161.476 - 1.5 = 159.976$ m.

(d) *At Section 1-1*

Loss of head in contraction transition from section 1-1 to section 2-2

$$= 0.2\,\frac{V_2^2 - V_1^2}{2g} \quad \text{where } V_1 = V_4 \text{ and } V_2 = V_3$$

$$= 0.2 \left[\frac{(1.665)^2 - (0.75)^2}{2 \times 9.81} \right] = 0.022 \text{ m}$$

R.L. of T.E.L. at 1 = R.L. of T.E.L. at 2 + Head loss

$= 161.617 + 0.022 = 161.639$ m.

R.L. of water surface = R.L. of T.E.L. $- \frac{V_1^{\,2}}{2g}$

$$= 161.639 - \frac{(0.75)^2}{2 \times 9.81} = 161.639 - 0.029 = 161.610 \text{ m.}$$

R.L. of bed, to maintain constant water depth

$= 161.610 - 1.5 = 160.110$ m.

Step 4. Design of contraction transition

The transition will be designed on the basis of Chaturvedi's formula :

$$x = \frac{L\,B_0^{3/2}}{B_0^{3/2} - B_f^{3/2}} \left\{ 1 - \left(\frac{B_f}{B_x} \right)^{3/2} \right\}$$

where $B_0 = 20$ m ; $B_f = 10$ m ; $L = 10$ m

$$x = \frac{10\,(20)^{3/2}}{(20)^{3/2} - (10)^{3/2}} \left\{ 1 - \left(\frac{10}{B_x} \right)^{3/2} \right\}$$

or

$$x = 15.45 \left\{ 1 - \left(\frac{10}{B_x} \right)^{3/2} \right\}$$

The values of x of for various values of B_x are calculated from the above relation, and are tabulated below :

B_x	10	11	12	13	14	15	16	17	18	19	20
x	0	2.12	3.73	4.97	6.08	7.06	7.84	8.5	9.20	9.60	10

The distance x is measured from the flumed section, as shown in Fig. 19.5.

Step 5. Design of expansion transition

The expansion transition will also be designed on the basis of Chaturvedi's formula:

$$x = \frac{L\,B_0^{3/2}}{B_0^{\,3/2} - B_f^{\,3/2}} \left[1 - \left(\frac{B_f}{B_x} \right)^{3/2} \right]$$

Here $\quad L = 15 \,;\, B_f = 10 \,;\, B_0 = 20$.

$$x = \frac{15\,(20)^{3/2}}{(20)^{3/2} - (10)^{3/2}} \left\{ 1 - \left(\frac{10}{B_x} \right)^{3/2} \right\} = 23.15 \left\{ 1 - \left(\frac{10}{B_x} \right)^{3/2} \right\}$$

The value of x for various values of B_x are calculated from the above relation, and are tabulated below :

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B_x	10	11	12	13	14	15	16	17	18	19	20
x	0	3.18	5.60	7.46	9.12	10.6	11.75	12.75	13.80	14.4	15

Step 6. Design of the trough

Flumed water way of canal = 10 m. The trough carrying the canal water will be divided into two compartments, each 5 m wide separated by a 0.3 m thick partition wall. The road way, 5 m wide shall be carried over one of the compartments. The height of the trough will be kept equal to 1.5 m + 0.5 = 2 m. The entire trough section will be constructed in monolithic reinforced concrete and can be designed by the usual structural methods. The outer walls and the bottom slab of the trough may be tentatively kept equal to 0.4 m thick. Thus, the outer width of the trough (and, hence, the length of the syphon barrel) $= 10 + 0.3 + (2 \times 0.4) \approx 11.1$ m.

Step 7. Head loss through syphon barrels

The loss of head through the syphon barrels can be calculated from the Unwin's formula:

$$h = \left(1 + f_1 + f_2 \frac{L}{R}\right)\frac{V^2}{2g}$$

where V = Velocity through barrels = 2.05 m/sec.

f_1 = Coefficient of loss of head at entry = 0.505

$$f_2 = a\left(1 + \frac{b}{R}\right)$$

$a = 0.00316$ and $b = 0.030$ for cement plaster.

$$R = \frac{A}{P} = \frac{6 \times 2.5}{2(6 + 2.5)} = 0.765 \text{ m}$$

L = Length of barrel = 11.1 m.

Substituting these values, we get

$$h = \left[1 + 0.505 + 0.00316 \times \left(1 + \frac{0.030}{0.765}\right)\left(\frac{11.1}{0.765}\right)\right]\frac{(2.05)^2}{2 \times 9.81} = 0.333 \text{ m}$$

Downstream H.F.L = 160.50 (given)

∴ u/s H.F.L. = d/s H.F.L. + Loss of head

= 160.50 + 0.333 = 160.833 m.

Step 8. Uplift pressure on the roof

R.L. of bottom of trough = R.L. of canal bed − Slab thickness

= 160.00 − 0.40 = 159.60 m.

Loss of head at the entry of the barrel

$$= 0.505\frac{V^2}{2g} = 0.505\frac{(2.05)^2}{2 \times 9.81} = 0.108 \text{ m.}$$

Uplift on the roof = 160.833 − 0.108 − 159.60 = 1.125 m.

or Uplift pressure = $1.125 \times 9.81 \triangleq 11.04$ kN/m².

The trough slab is 0.4 m thick, and exerts a downward load of $0.4 \times 2.4 \times 9.81 = 9.42$ kN/m².

The balance of the uplift pressure = 11.04 − 9.42 = 1.62 kN/m² has to be resisted by the bending action of the trough slab, and for this, suitable reinforcement has to be provided at the top of the slab. Since the trough as a whole is heavier than the total upward force, no anchorage to piers is necessary. When the water level in the drainage is low, trough slab is to be designed for the full water load of the canal, and suitable reinforcement should be provided at the bottom of the slab.

Step 8. Uplift on the floor of the barrel and its design

(a) Static head

R.L. of barrel floor = R.L. of bottom of trough slab − Height of barrel

= 159.60 − 2.50 = 157.10 m.

Let the tentative thickness of the floor be equal to 0.8 m.

∴ R.L. of bottom of floor = 157.10 − 0.8 = 156.30 m

Drainage bed level = 158.00 m (given)

Assuming the subsoil level water upto the bed level of the drain, the static uplift, on the floor

= 158.00 − 156.30 = 1.70 m

(b) Seepage head

The residual seepage head at the centre of the first barrel (point B) can be calculated with reference to Fig. 19.9. The seepage line, according to Bligh's theory,

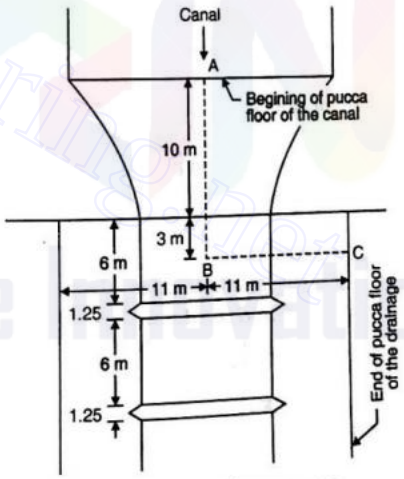

FIG. 19.9. SEEPAGE HEAD

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will follow the path *ABC*. Neglecting the floor depression or floor thickness the various lengths are as follows :

(*i*) Length of u/s transition = 10 m.

(*ii*) Half the barrel span = 3 m.

(*iii*) End of drainage floor from the centre of barrel, assuming a total length of 22 m = 11 m.

∴ Total creep length = 24 m.

Creep length upto the centre of the barrel = 13 m.

Total seepage head = Canal F.S.L. – R.L. of drainage bed

$= 161.5 - 158.00 = 3.50$ m

Residual seepage head at $B = 3.50\left(1 - \frac{31}{24}\right) = 1.6$ m

Total uplift = Static head + Seepage head

$= 1.70 + 1.6 = 3.3$ m of head of water

$= 3.3 \times 9.81 = 32.37$ kN/m^2

Provide thickness of the floor equal to 0.8 m ; the uplift resisted by the self-weight $= 0.8 \times 2.24 \times 9.81 = 17.58$ kN/m^2.

∴ Balance to be resisted by the bending action

$= 33.37 - 17.58 = 14.79$ kN/m^2.

Suitable reinforcement will have to be provided at the top of the floor to counteract the bending action.

Note. The length of the floor has been kept as 22 m on the following considerations:

(*i*)	Length of the floor under the barrel	= 11.1 m
(*ii*)	Extra floor to accommodate the piers on both sides, keeping the projection of the piers to be 0.9 m	= 1.8 m
(*iii*)	Horizontal length of the d/s ramp joining to the bed level, at a slope 1 : 5 = 5 × (158.00 – 157.10)	= 4.5 m
(*iv*)	Width of d/s cut off beyond ramp	= 0.5 m
(*v*)	Length of the extra floor to the u/s of the piers	= 4.1 m
	Total	= 22 m

The cross-section of the syphon aqueduct is shown in Fig. 19.10.

Step 10. Design of cutoff and protection works for the drainage floor.

The scour depth $R = 0.47\left(\frac{Q}{f}\right)^{1/3} = 0.47\left(\frac{400}{1}\right)^{1/3} = 3.46$ m

Depth of u/s cutoff, below H.F.L.

$= 1.5R = 1.5 \times 3.46 \simeq 5.2$ m

R.L. of bottom of u/s cutoff

= u/s. H.F.L. – 5.2 = 160.833 – 2.5 = 155.633 ≃ 155.63 m

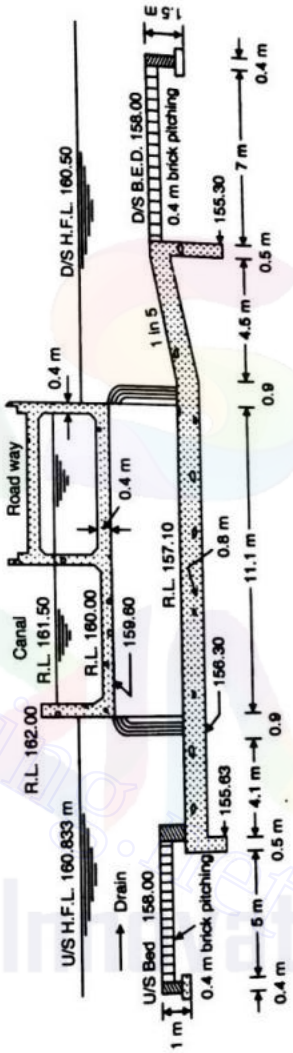

FIG. 19.10. CROSS-SECTION OF SYPHON AQUEDUCT.

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Depth of d/s cutoff, below H.F.L.

$= 1.5\,R = 1.5 \times 3.46 = 5.2$ m

R.L. of bottom of d/s cutoff

$=$ d/s H.F.L. $- 5.2 = 160.50 - 5.2 = 155.30$ m

Length of d/s protection consisting of 40 cm brick pitching

$= 2.5$ (R.L. of d/s bed – R.L. of bottom of d/s cutoff)

$= 2.5\,(158.00 - 155.30) = 6.76 \text{ m} \approx 7$ m.

The pitching may be supported by a toe wall 0.4 m wide and 1.5 m deep at its d/s end . Length of d/s protection consisting of 0.4 cm brick pitching

$= 2$ (R.L. of u/s bed – R.L. of bottom of u/s cutoff)

$= 2\,(158.00 - 155.62) = 4.76 \text{ m} \simeq 5$m.

This pitching may be supported by a toe wall 0.4 m wide and 1 m deep at its u/s end.

The cross-section of the syphon aqueduct is shown in Fig. 19.10.

PROBLEMS

1. Describe with the help of sketches various types of cross-drainage works.
2. Differentiate between (i) syphon aqueduct and canal syphon, (ii) aqueduct and super passage.
3. What do you understand by level crossing ?
4. Write a note on selection of suitable type of cross-drainage works.
5. Classify aqueducts and explain under what circumstances each one is used.
6. Explain the method of fixation of water-way of drain in an aqueduct.
7. Explain Mitra's method of hyperbolic transition. Derive the equation you use.
8. Explain the Hind's method of designing canal transitions.
9. Explain the method of determining uplift pressure on the roof of a syphon aqueduct.
10. Give Bligh's appoximate method of determining uplift pressure under the floor of a cross-drainage work.
11. What is meant by bank connections ? How are they designed ?

SECTION-IV

MISCELLANEOUS TOPICS

Chapters :

20. RIVER ENGINEERING

21. WATER POWER ENGINEERING

22. WATER RESOURCES PLANNING

23. IMPORTANT DAMS OF INDIA

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20

River Engineering

20.1. CLASSIFICATION OF RIVERS

Rivers can be divided into two parts according to the topography of river basin, namely :

(*a*) Upper reaches in the hilly region.

(*b*) Lower reaches in the alluvial plain.

Rivers in the hilly region are characterised by the steepness of their slope, the swiftness of flow, the occurrence of land slides and the formation of rapids along their courses.

Alluvial rivers are characterised by the fact that the alluvia, on which the rivers flow, are built by rivers themselves. Rivers on alluvial plains may be broadly classified into three types :

(1) The meandering type, (2) The aggrading type,

(3) The degrading type.

All the three types may be found on a single river from its uppermost point on alluvial plain to its mouth. A particular section of the river may also be aggrading, degrading or meandering at different times depending upon the variation of sediment load and discharge. Among the three-types of rivers mentioned above, the meandering type is the full and final stage of river development while the other two are interim type.

20.2. MEANDERING

A river meandering through an alluvial plain has a series of consecutive curves of reversed order connected with short straight stretches called *crossing*. *Width of mendering belt* M_B is the transverse distance between the apex point of one curve and apex point on reverse curve.

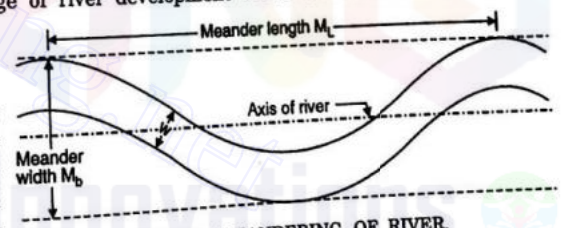

FIG 20.1 MEANDERING OF RIVER.

Length of meander M_L. It is the air distance along river between tangent point of one curve and tangent point of other curve of same order.

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Degree of Sinuosity. It is the ratio between the curved length and the straight air distance.

20.3. CAUSES OF MEANDERING

Primary cause of meandering is the excess of total charge during floods, when excess of turbulence is developed. Experiments have shown that when charge is in excess of that required for stability, a river tends to build a steeper slope by depositing the charge on the bed. This increase in slope tends to increase, in its turn, the width of the channel if the banks are not resistant. Only a slight deviation from uniform axial flow is then necessary to cause more flow towards one bank than towards the other. Additional flow is immediately attracted towards the former banks, leading to shoaling along the latter, accentuating the curvature of flow and producing, finally, meander in its wake.

The cause given in the paragraph above is due to the Inglis. According to J.S.Firedkin, meandering results from the local bank erosion and consequent over loading deposition by the river of the heavier sediments moving along the bed.

20.4. BASIC FACTORS CONTROLLING PROCESS OF MEANDERING

The basic factors controlling process of meandering are given below :

1. Valley slope . This is overall slope of terrain traversed by the stream and is measured down the axis of the valley. A change in the valley slope always produces change in the meander pattern.

2. Stream load. Composition of stream load as well as its rate of movement materially affects the meander pattern.

3. Discharge. This concerns seasonal stream flow variations as represented by the average hydrograph. There is a close relationshilp between the rate of discharge and the rate of bed load movement and, therefore, it affects meandering.

4. Bed and side resistance. Boundary resistance are characterised by the nature of materials composing the alluvium, more particularly by its resistance to erosion. Grain size, specific gravity, cohesion and roughness are important factors. Grain sizes may range from large boulders and pebbles in head reaches to fine sand and silt in lower reaches of a river. In a river with gravel and boulder bed, no definite meander pattern is observed as the river has a tendency to form interlaced channels.

20.5. GENERAL FEATURES OF MEANDERING

The general features of meandering have been worked out by various experiments on meandering rivers. The same are discussed below :

1. A fully developed meander of homogeneous valley material with a constant discharge has a definite pattern of curvature, length, width and depth of the channel is reproducible in models.

2. The width and length of meander as well as width of river vary roughly with square root of discharge.

3. The pattern of meander is altered by variation in discharge slope or bank and bed forming material.

4. Increase in discharge or slope increases the meandering feature *viz* size of bends, their length, width and degree of sinuousity.

5. Increase of sediment charge increases the slope as well as width of the river, while the depth of the river is reduced.

It can be summarised that every phase of meandering depends upon the following three closely related factors :

(*i*) The discharge and hydraulic properties of the chanel.

(*ii*) The sediment load.

(*iii*) The relative erodibility of bed and band.

Generally speaking, where a river has enough capacity to carry the incoming sediment downstream with forming large deposits, the whole or a part of it would be meandering type. The relations between M_l, M_b, W and Q as arrived at by Inglis after analysing data on Indian rivers are given in Table 20.1.

TABLE 20.1. INTER-RELATIONSHIP BETWEEN M_l, M_b , W AND Q

M_l, M_b or W (metre)	Rivers in flood plain	Incised rivers (Rivers flowing in a cross-section cut below the natural ground surface)
M_l	6.06 W	11.45.W
M_b	17.40 W	27.30 W
M_b	2.86 M_l	2.2 M_l
M_l	53.25 $\sqrt{Q}$	46 $\sqrt{Q}$
M_b	153.75 $\sqrt{Q}$	102.25 $\sqrt{Q}$
W	8.87 $\sqrt{Q}$	4.54 $\sqrt{Q}$

Q is discharge in cumecs.

20.6. THE AGGRADING TYPE OF RIVER

The aggrading type of river is building up its bed to a certain slope. This may be because of :

1. Excessive sediment entering a river with a sudden dimunition of slope on the plain.

2. Excessive sediment entering a river with the slope flattened because of some obstruction like barrage or a dam across it.

3. Extension of delta at the river mouth.

4. The sudden inrush of sediment from a tributary.

A river of aggrading type or a section of it, has usually a straight or wide reach with shoals in the middle.

20.7. DEGRADING TYPE OF RIVER

The degrading type of river is in the process of loosing its bed gradually in the form of sediment load of the stream. The reason associated with the degradation of river is sudden dimunition of sediment load due to deposition of bed upstream of a barrage or a dam. The water falling downstream of the dam scours the bed and replenishes the sediment load. For the same reason, meandering type of river develops

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temporarily into a degrading type when a loop of meander has been cut off by a straight river channel.

20.8. CUT-OFF

When the meandering of a river develops to the extreme conditions in the shape of horse-shoe bends, the land between them gradually reduces to a narrow neck which may be cut-off by natural flow during floods.

Cut-off may be *defined* as a process by which an alluvial river flowing allowing curves of bends abandons particular bend and establishing its main flow along a comparatively straight and shorter channel.

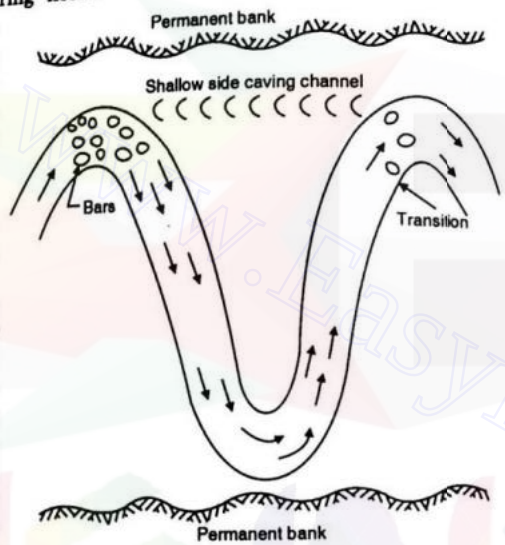

FIG. 20.2 DEVELOPMENT OF CUT-OFF

Development of cut-off

The following reasons are generally assigned for the development of cut-off.

1. ***Development of bars at inflection.*** Formation and growth of bars at inflection retard the flow through main channel and induce flow through already existing shallow side channels. The main channel flowing with reduced discharge continues to silt whereas side channel increases in size to accommodate more and more of discharge and ultimately forms into a cutoff.

2. ***Formation of drops at the junction of main channel with side channels.*** The side channel usually has a lower velocity and smaller length with the same head difference as the channel. It, therefore, joins the main channel at a local drop. This local drop cuts back into the side channel and helps in development of a cutoff.

3. ***Unfavourable cross-section and steep slope*** The unfavourable cross-section and steep slope of the side channel may generate such velocities which may erode the bed and bank of the side channel to constitute the principal course of the river.

4. ***Bank erosion.*** The river increases its curvature by erosion of concave banks. The erosion might occur to such an extent that the arms of a loop with a narrow neck cut into one another and cutoff occurs.

5. ***Duration of flood.*** The duration of flood should be sufficient so that stream may erode its bed and banks.

Effect of a cutoff

These natural cutoffs often result in violent changes in river regime, steepening of the slope, inducing erosion in the stretch above the cutoff, and lowering the waterlevel

upsteam. Owing to the reduction of channel storage above cufoff, inrushing flood may be a menace to the lower stretch. Excessive eroded material of the cutoff may damage the channel downstream. Since the regime has been affected temporarily, the river may again develop meandering to find a new equilibrium.

20.9. RIVER TRAINING

The expression *river training* implies various measures adopted on a river to direct and guide the river flow, to train and regulate the river bed or to increase the low water depth. The *purpose* of river training is to stablish the channel along a certain alignment. There may be various objects for training a river ; thses are described below :

1. High flood discharge may pass safely and quickly through the reach.
2. Sediment load including bed and suspended load may be transported efficiently.
3. To make the river course stable and reduce bank erosion to minimum.
4. To provide a sufficient draft for navigation as well as good course for it.
5. To fix direction of flow through certain defined reach.

20.10. CLASSIFICATION OF RIVER TRAINING WORKS

Keeping in view the objects of river training as stated above, river training can be classified into the following :

1. ***High water training.*** This is also called *training for discharge*. The river is trained to provide sufficient and efficient cross-sectional area for the expeditious passage of maximum flood. It concerns mainly with alignment and height of embankment for a given flood discharge.

2. ***Low water training.*** In this case the river is trained to provide sufficient depth for navigation during low stage of river. This is also called *training for depth* and is usually achieved by contraction of the width of the channel.

3. ***Mean water training.*** In this case the river is trained to correct the configuration of river bed for the efficient transport of sediment load in order to keep the channel in good shape. It can be called *training for sediment.*

Out of the various kinds of training, mean water training is the most important training. Any attempt to change the configuration of river bed in alignment or in cross-section must obviously by designed in accordance with that stage of the river at which maximum movement of sediment takes place. The river is most active during high stage which persists for a very short duration and is practically inert at low stage which persists for a very long duration. Some where between these two there is a stage at which the combined effect of forces causing sediment and the time for which such forces are maintained is maximum. The maximum bed building stage is some where in the neighbourhood of mean water. Mean water training, therefore, forms the basis on which both high and low water training are planned

20.11. TYPES OF RIVER TRAINING WORKS

The following list enumerates various types of river training and control works:

1. Guide bank system

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2. Groynes or spurs,
3. Levees or embankment,
4. Bank protection and pitched banks,
5. Pitched islands,
6. Miscellaneous methods,
   (*i*) River training without embankments
   (*ii*) Bandalling.

20.12. GUIDE BANK SYSTEM

Guide banks are made for guiding the stream near a structure so as to confine it in a reasonable width of the river. It was first designed by Bell in whose honour it is also sometimes known as *Bell's Bund*. The design was further developed by Spring and is known as guide bank.

The guide bank usually consists of a heavily built embankment in the shape of bell mouth on both sides of constricted channel. Usually only one embankment is required if the other side is a high and stable bank. The remaining side of the river *khadir* is covered by ordinary embankment.

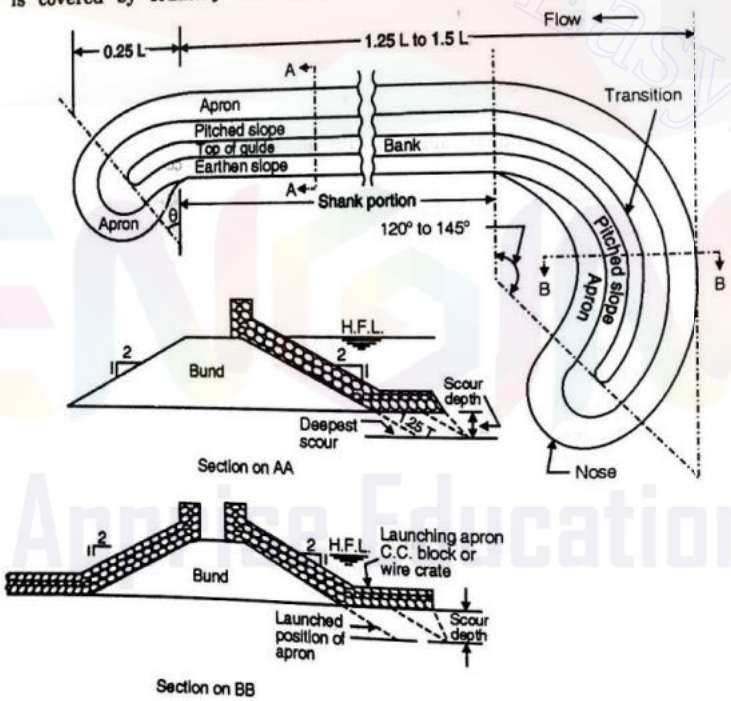

FIG. 20.3. GUIDE BANK.

Design considerations

The design of a guide bank involves the following considerations:

(*a*) The ultimate width to which an alluvial river can be constricted may be computed from the relation

$$L = 4.75\sqrt{Q}$$

where Q is estimated maximum discharge in cumecs ; L is the width of the channel in metres.

An extra allowance of 20% may be made for thickness of bridge piers etc.

(*b*) Length of upstream guide bank $= 1.25\ L$ to $1.5\ L$.

(*c*) Length of downstream guide Bank $= 0.25\ L$.

A symmetrical layout of the guide bank in plan is very essential for the successful performance of the bund. The two guide banks on both sides of stream can be made (*a*) parallel, (*b*) diverging, (*c*) converging at upper end, (*d*) single guide bank depending upon local conditions, although Bell recommended a diverging form with an area of narrowest part of upstream end equal to the effective area at bridge site. Both upstream and downstream ends of the banks are curved gently so as to lead the flow along smooth lines.

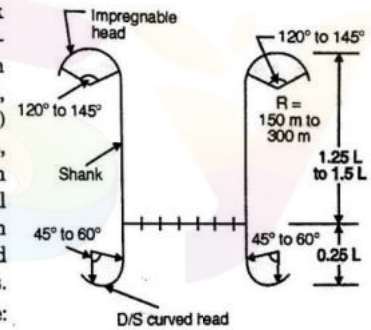

FIG. 20.4. LAYOUT OF GUIDE BANK.

(*d*) The main parts of a guide bank are:

(*i*) U/S curved head or impregnable head

(*ii*) D/S curved head

(*iii*) Shank or a straight portion which join two curved heads

(*iv*) Slope and bed protection.

(*i*) *U/S curved head*

On the u/s end the curvature has a central angle of 120° to 140° . The radius of curvature R can be provided as suggested by Spring or Gale or as per an empirical relationship. Spring suggested a value of R equal to 180 m to 250 m for rivers having velocities 2.4 metres per second to 3.1 respectively. Gales on the other hand suggested a value of 250 m for rivers having flood discharge between 7000 to 20,000 cumecs; sharper curves are permissible for discharge less then 7000 cumecs. A value of 580 m is recommended for discharge varying from 40,000 to 70,000 cumecs. The radius may be obtained by interpolation for discharge between 40,000 to 20,000 cumecs.

It has also been suggested that the value of R can be calculated from the relation

$$R = 0.45\ L.$$

(*ii*) *D/S curved head*

On the d/s the river fans out to attain the normal width. A proper shape ensures safety of approach embankment. Radius of curvature of D/S head may be kept as half of the u/s radius with a central angle of 45° to 60°.

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(iii) *Section of guide bund and material of construction*

The material of construction of guide bank is the river sand available locally. Side slope is generally kept 2 : 1 to 3 : 1. It should have sufficient top width which should not be less than 4 m so as to provide sufficient carriage way. The free board usually adopted for the design varies from 1.25 to 1.50 m. The inside slope is protected by stone pitching, the total thickness of which varies from 0.4 m to 0.6 m. The stone pitching is 1 m above H.F.L. The thickness of stone pitching (T) as recommended by Inglis is given by

$$T = 0.06\,(Q)^{1/3}$$

The thickness of the pitching should be 25% more at the impregnable head than for the rest of the bund.

The rear side of the shank portion is not pitched, but is generally coated with 0.3 m to 0.6 m earth for encouraging vegetation growth so as to make it resistant against wind and rain.

(iv) *Slope and toe protection*

The toe of the slope is protected by the so called "*falling apron*" or "*launching apron*". The quantity of stone in the launching apron should be sufficient to cover a scoured face fully after the apron fans out while launching at a slope of 2 : 1 with a thickness of 1.25 T. It is generally laid in a width equal to 1.5 D where D is the depth of maximum anticipated scour below the bed, and has a thickness of 1.9 T. The maximum anticipated scour below H.F.L. is taken as $C.R$ where R is the Lacey's normal scour depth given by the relation

$$R = 0.47\left(\frac{Q}{f}\right)^{1/3}$$

where Q is the discharge and f is the silt factor. The value of C is generally assumed as 2.25 at the nose of the guide banks, 1.50 at the transition from nose to the straight portion and 1.25 in the straight reaches of the guide bund. Inglis and Joglekar gave the following values for maximum scour depth :

(i) Maximum scour depth downstream of bridge :

$$D_s = 1.9\left(\frac{Q}{f}\right)^{1/3}$$

(ii) Maximum scour depth around bridge piers :

$$D_s = 0.95\left(\frac{Q}{f}\right)^{1/3}$$

(iii) Maximum scour depth at nose of guide banks of a large radius :

$$D_s = 1.3\left(\frac{Q}{f}\right)^{1/3}$$

The usual types of guide banks provided in India are as given below (Table 20.2) :

TABLE 20.2. TYPES OF GUIDE BANKS

S.No.	Type of Bank	Place
1.	Guide Bank parallel but not perpendicular to Weir	Rasul Weir on Jhelum
2.	Parallel guide banks with curved heads	Punjnad Weir on Sutlej
3.	Bottle neck guide banks FIG. 20.5	Sulemani Weir on Sutlej Khanki Weir on Chenab
4.	Splayed guide banks	Islam Weir on Sutlej
5.	Parallel straight guide bank	Ferozepore Weir on Sutlej

20.13. GROYNES OR SPURS

Groynes are structures constructed transverse to the river flow and extend from the bank into river upto a limit. These groynes are also known as spurs, dikes and transverse dikes.

Type of Groynes : Groynes can be classified in the following ways :

1. ***Classification according to the material of construction***
   (a) Permeable groyne.
   (b) Solid impermeable groyne.
2. ***Classification according to its height below high water***
   (a) Submerged groyne.
   (b) Non-submerged groyne.
3. ***Classification according to the function it serves***
   (a) Attracting groyne.
   (b) Deflecting groyne.
   (c) Repelling groyne.
   (d) Sedimenting groyne.
4. ***Special types of groynes, such as***
   (a) Denehy's T-headed groyne.
   (b) Hockey type groyne.
   (c) Burma type groyne.

Choice of a groyne : Factors which influence the choice and design of groynes are

(1) Fall velocity of river.
(2) Width of water way, at high water, mean water, and low water.
(3) Depth of water way, height of flood rise and nature of flood hydrograph.
(4) Character of bed material such as shingles, boulders, sand or silt.

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(5) Available material and funds.

(6) Amount of silt carried in stream.

Permeable groynes are suitable for silt laden river. If the river has to be confined in a defined channel, generally impermeable groynes are used.

Purpose of a groyne : Groynes serve a four-fold purposes, namely :

(1) Contracts a river channel to improve its depth.

(2) Protects the river bank.

(3) Silts up the area in the vicinity by creating a slack flow.

(4) Trains the flow along a certain course.

Depending upon the purpose, groynes can be used with other training measures.

20.14. REPELLING GROYNE

The repelling groyne is constructed in such a way that it is pointing towards upstream at an angle of 10° to 30° to the line normal to the bank. The head of groyne causes the current to be deflected in a direction nearly perpendicular to itself. The current coming into contact with still water area adjeacent to the spur causes vertical eddies and deep scour. The head of groyne should be strong to resist the swirling action of the current. In case of repelling groyne, a still water pocket is formed upstream of it and suspended load brought down by the river gets deposited in the pocket. Rapid success of the groynes depends upon how quickly still water pockets between groynes are filled with sediment, which can be accomplished only when they are sufficiently long.

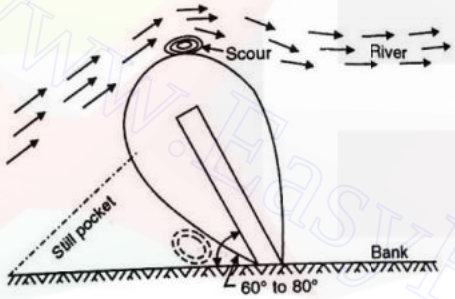

FIG. 20.6. REPELLING GROYNE.

20.15. DEFLECTING GROYNES

A deflecting groyne has a much shorter length than a repelling groyne and it is generally taken in a river perpendicular to the bank it only deflects the flow.

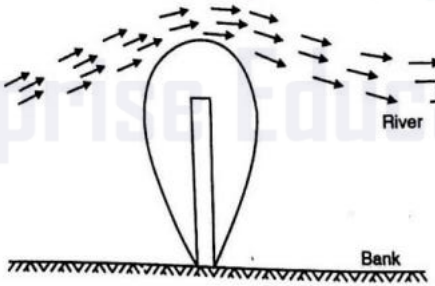

FIG. 20.7. DEFLECTING GROYNE.

20.16. ATTRACTING GROYNE

The attracting groyne is constructed in such a way that it points downstream the direction of normal flow. When a groyne points downstream, it causes scour holes to form closer to the bank than the groyne inclined at right angles ; therefore, they tend to maintain the deep current close to the bank. The attracting groyne bears the full fury of the frontal attack of the river on its upstream face and has, therefore, to be adequately strong. An equally heavy protection is not necessary on the downstream slope.

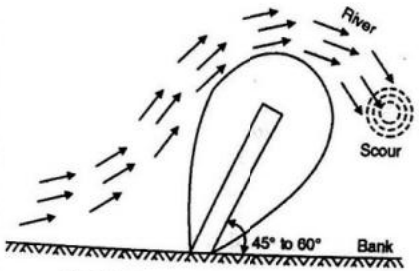

FIG. 20.8. ATTRACTING GROYNE.

20.17. LENGTH OF GROYNES

The length of groynes depends upon the magnitude of river training required. If the current is to be thrown back on the other bank, naturally longer length of groyne is required. A long groyne constructed out of easily erodible material, however, may fall. Hence, it may be worthwhile to start with a short length and extend the groynes gradually as silting between them proceeds.

20.18. SPACING OF GROYNES

It is general practice to space the groynes by 2 to 2.5 times the length of groyne. The spacing may be altered under the following conditions :

(1) The spacing can be farther apart on convex bund then on concave bund.

(2) The spacing is also farther apart for wider rivers than for narrow ones of the same discharge.

(3) Permeable spurs are spaced farther and impermeable spurs comparatively closer.

Section of Groynes

The section of groyne is similar to that shown in Fig. 20.9.

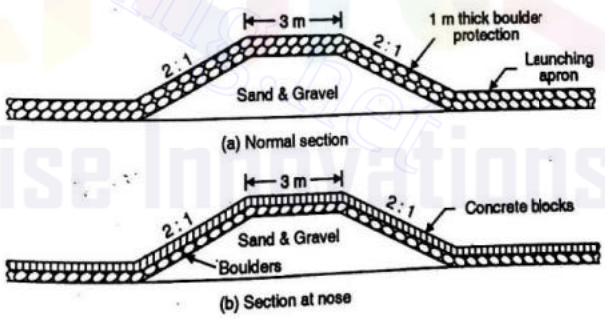

FIG. 20.9. SECTION OF GROYNE.

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20.19. IMPERMEABLE GROYNES

Impermeable or solid groynes do not permit any flow through them. They are constructed by a core of locally available material like sand, clay and gravel and heavily protected on the sides by resistant materials like stone pitching or concrete blocks. The groyne section has a side slope varying from 2 : 1 to 3 : 1 and front slope varies from 3 : 1 to 5 : 1. An apron must be provided at the bottom of the slope to prevent slipping of stones. They can be designed as any one of the repelling, deflecting or attracting groynes.

20.20. PERMEABLE GROYNES

They permit the flow of water through them. They dampen the velocity and thus reduce the erosive action of the stream. In case river water carries sediment load, the groyne cause deposition of sediment near it because of reduction of velocity.

Advantages of permeable groyne. The basic advantages associated with a permeable groyne are given below :

1. The groyne is very cheap in cost of construction.
2. Small quantity of stone is required for its construction and hence, it is useful where stone is scarce.
3. The permeable groyne has a better performance as compared to solid impermeable groynes.
4. The groyne does not change the flow abruptly, hence, there is no serious formation of eddy and scour.
5. It is more suitable for deep narrow rivers.
6. Submerged permeable groynes do not create turbulence and eddy conditions as is the case with submerged solid groynes.

Disadvantages. They are not strong to resist shock and pressures from debris of floating ice logs.

Types of permeable groyne. The permeable groyne is either a

1. Tree groyne or 2. Pile groyne.

Tree Groyne. A tree groyne consists of trees held in position by thick weir ropes anchored firmly on the bank and tied to a heavy buoy. The trees used for groynes should be very leafy with abundant branches like the babul. Roots, branches and twigs help to trap sediment by reducing the flow velocity. A tree groyne should be completed before the early floods. Tree groynes have been successfully used :

1. To divert or deflect the current which is directly threatening erosion of a band.
2. For closing a river channel and opening the other.
3. To silt up a channel of the river at its source by checking the flow in it.

Pile Groyne. These groynes are constructed with timber piles driven down the river bed upto 9 metres depth. The groynes consist of main verticals in two or three rows braced together by transverses and diagonals. The timber main verticals are spaced 2 to 3 m apart and in 2 or 3 similar rows separated 1.5 to 2 m apart. Between

the main verticals there can be two intermediates embedded at least 1.5 m below the bed. These are filled with alternate layers of brush woods and stones. Sometimes they catch silt, become sand bound and start acting as solid groyne. To safeguard it against scour in such a case it is protected by stone apron 1 m thick having a length of 3 m along shank and 6 m along the nose.

20.21. DENEHY'S GROYNES

These are special groynes first provided at Okhla barrage and then at Narora Weir. At Narora Weir the groynes are earthern embankment projecting from the marginal bund. The head of groynes has T shape of which the front perpendicular arm about 100 m in length was parallel to current. A longer portion of the head is on upstream

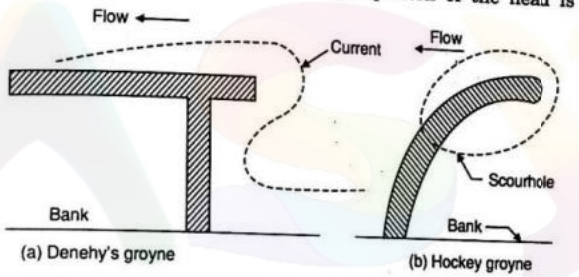

FIG. 20.10. DENEHY'S GROYNE AND HOCKEY GROYNE.

side so that it looks more like a reversed L [Fig. 20.10(*a*)]. The head has a slope on both sides and is heavily protected by stone. They are spaced 800 m apart. The T heads are on a straight line or on a regular curve along 6.4 km length upstream and 25.6 km length downstream of headworks at Narora. These groynes aim at an economy in the use of protection stone since only 100 m length of the groyne would suffice for a length of 800 m.

20.22. HOCKEY GROYNE

Amongst the various designs for groyne heads, a groyne with a curved head [Fig. 20.10(*b*)], is termed as a hockey groyne. It increases the attracting tendency of the groyne and is not likely to be helpful for bank protection.

20.23. MARGINAL BUND OR LEVEES

Marginal bund or Levee is an earthen dike constructed roughly parallel to the river rather than across the channel.

Effect of Levees on flood flows

The effect of levees on flood is sumarised in the following points :

1. Increases the intensity of flood water travel downstream.
2. Increases the water surface elevation of the river at flood.
3. Increases maximum discharge at all points downstream.
4. Increases the velocity and scouring action through leveed section.

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5. Decreases the surface slope of the stream above the leveed portion.
6. Increases the flood peak and decreases the valley storage.

Planning of levee

A levee should be planned with following points under consideration :

1. Levees should be provided on areas which are lying low and, hence, the danger from the flood is pronounced.

2. A sufficient channel must be provided in levees to transmit design flow with reasonable freeboard against wave action. The cost of levee system consists of costs of land for levees and channel plus the cost of levee construction. Hence, the channel width and levee height combination which gives minimum cost is to be determined by trial.

3. Levees should be placed on the high ground. Full advantage should be taken of ridges, which permit lower levees and better foundation condition.

4. The levee should be tied down to the high grounds.

5. The levee is roughly spaced equidistant from midstream.

Design of Levee

The design of a levee is discussed in the following steps :

(a) *Spacing and height :* Spacing and height of a levee are interdependent. More height will have levees closer together while lower height will keep them apart. A provision of free board of 1 to 1.25 m should be made over H.F.L.

(b) *Top width :* Top width is decided as per the use to which it is put to. Minimum width = 3 m.

(c) *Side slope :* The side slope provided in a levee depends upon :

(*i*) Nature of material of which levee is composed.

(*ii*) Method of construction.

(*iii*) Length of time the levee is likely to be subjected to a wave action.

The usual slop provided is as given below :

1. Slope on river side 3 : 1 to 5 : 1.
2. Land side 4 : 1 to 7 : 1.

(d) *Banquette :* A banquette is a terrace of earth added to the base of a high levees on the land side to prevent danger of slouging from seepage.

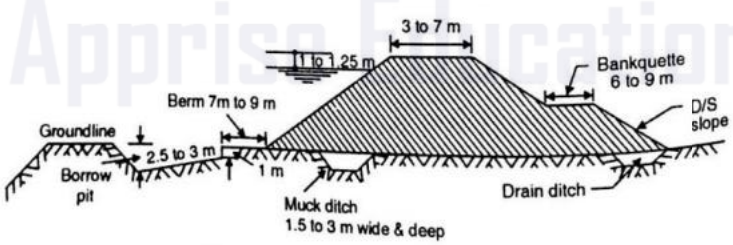

FIG. 20.11. SECTION OF A LEVEE.

(e) *Muck ditch* : They are trenches filled with most impervious material at hand, to form cutoff to check the seepage along the foundation plane.

(f) *Drain Ditch :* Any seepage through the levee is collected and disposed off through the drain ditch.

Causes of failure of a Levee. There may be many reasons of the failure of a levee, important amongst them are :

1. Caving of the stream bank may result in undercutting of the river-ward toe of the levee.
2. Seepage through foundation material at high stage of river can cause a sand boil and removal of foundation material by piping.
3. Overtopping during high floods may cause its ultimate collapse.
4. The foundation may fail due to infiltration.
5. Piping as a result of insufficient cross-section leaks and cracks due to shrinkage of soil and rat holes.

Advantages of river training by embankment

1. They are the only means of preventing the inundation by tides.
2. They are initially cheaper in the cost of construction though subsequent raising may make it more expensive.
3. The construction and maintenance is easy.
4. They can be executed in parts, but the ends should be well protected.

Disadvantages

The various disadvantages of river training by embankment are as mentioned below :

1. Embankment cause a rise in high flood level.
2. Gopher holes or channels left by decay or roots may permit the start of destructive erosion leading to eventual levee failure or crevasse.
3. They require heavy maintenance.
4. They fall suddenly and inundate the low lying areas.
5. They are susceptible to direct attack of river flow which can erode or undermine them.
6. The interior drainage of the area is enclosed by embankment which may give rise to material climate.
7. They check the deposition of silt on flood plain, thus hastens delta formation resulting in increase of flood stages and rate of aggradation of river bed.

20.24. ARTIFICIAL CUTOFF

To improve upon the flood capacity of a given reach and thereby reduce flood level in certain reaches an artifical cutoff may be introduced as a river training measure. This is generally done when the loop length exceeds $1\frac{1}{2}$ to $2\frac{1}{2}$ times the chord length. In the beginning a pilot cut is introduced. The pilot cut should carry 8 to 10% discharge in the beginning and are permitted subsequently to develop the carrying to about 40

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to 50% of the total river discharge. Its design alignment and cross section is governed by the following considerations :

1. The direction of cut should be tangential to the main direction of flow.
2. Alignment of pilot cut should preferably be made at a curvature much flatter than the curvature of river itself.
3. Entrance to cutoff should be given bellmouth shape.
4. Sometimes if the cut is not excavated to develop by scour, they should be excavated initially as a main river channel.
5. According to the regime formula by Lacey, $R\,S^2$ depends wholly upon velocity. So that to have scouring velocities, $R\,S^2$ for cutoff should be higher than the river. Since head difference is same for river and cutoff, the slope is inversely proportional to the length of channel. Hence, R/L^2 for the pilot cut should be much larger than R/L^2 for the river.
6. The pilot section should be made very deep as deeper section develop rapidly.

20.25. PITCHED ISLANDS

This consists generally of a sand core pitched with boulders along its side and slopes and protected at the toe by a falling apron. Sometimes, advantages can be taken of an existing natural island (a bela) which can be re-shaped and pitched. The pitched island (Fig. 20.12) causes redistribution of velocity and tractive force. The tractive force near a pitched island begins to increase rapidly after the construction of island, with the result that deep scour begins to form round the island gradually draws the river

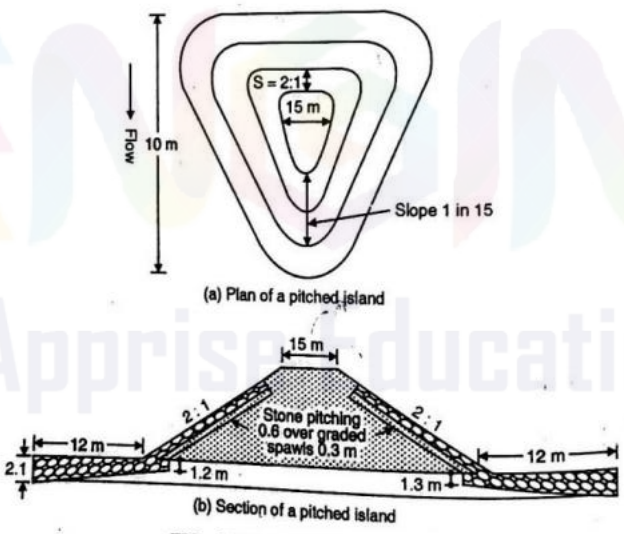

FIG. 20.12. PITCHED ISLAND.

channel into itself and ultimately holds it permanently. As deep scour occurs into its vicinity the boulder pitching and apron have to be designed for maximum scour.

$$\text{Maximum scour} = 2R = 2.70(q^2/f)^{1/3}.$$

It is used for the following purposes :

(*i*) Training of the river to have an axial flow.

(*ii*) Rectification of adverse curvature for effective sand exclusion.

(*iii*) Improving channel for navigation.

The device is not likely to be successful in shallow and wide rivers where obstruction caused by island is so small that scour cannot form. Similarly, in case of very flashy rivers the scour formed while rising is filled quickly during falling stage.

20.26. BANK PROTECTION

In a broad sense bank protection includes any protection work that aims at maintaining the stability of land against the action of water.

Purpose of Bank Protection

The purpose of bank protection is to save the banks from furies of flood. The purpose thus can be brought out in the following points :

(*i*) To check erosion of bank by current or waves.

(*ii*) To check sliding of slope due to its gradual steepening because of erosion.

(*iii*) To prevent undermining of the toe of the lower bank by currents, waves, swirls or eddies.

(*iv*) To prevent sloughing or sliding of slope when saturated with water.

(*v*) To prevent piping due to various causes.

(*vi*) To check sliding due to drawdown of the flood.

Methods of Bank Protection

The following methods of bank protection are in general practice :

1. Protection of bank by vegetal cover, either by turfing or sodding. Low growth of shrubs and willows is by far the most effective cover. The method should be used if the slope is not subjected to strong current.

2. Pavement of bank slopes by such material which can resist quick erosion. Along with pavement the slope is given a temporary covering with rush wood or lumber mattress weighed down by stones for energy purposes. The method is utilized if the current is strong.

3. When the current is very strong, protection is provided by stone revetment or various types of mattresses such as willow, lumber or asphalt or articulated concrete.

20.27. PITCHED BANK

A pitched bank may be created in part or full to attract the flow and hold the river at a desired place. The positioning of pitched bank has to be determined with the consideration of river meanders upstream and downstream. An artificial control point such as groyne upstream of pitched bank may be necessary to make the measure

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permanent. The pitched bank has a tendency to pull the river to itself, though this tendency depends upon the extent of scour which occurs at toe. The pitched bank must be protected by falling stone apron.

20.28. BANDALLING

Bandalling is an impervious system of river training. It is designed to confine the low water flow in a single channel with a view to maintain suitable depth for navigation.

A *bandal* consists of a frame-work of bamboos driven in the river bed and set 0.6 m apart with horizontal ties supported by struts placed at every 1.2 m. To this bamboo frame-work, *Jhamps* are tied with coire rope to the horizontal ties. The bamboo used on the frame-work are usually 3 to 6 m in length and *Jhamps* are made of bamboo mats from 0.75 to 1.0 m wide strengthened at the edges by strips of split bamboo. Bandals are placed at an angle 30° to 40° inclined downstream to direct an additional volume of water into the inclined downstream to direct an additional volume of water into the desired channel and to increase the velocity in it. The bandals do not stop the flow of water but merely provides an impediment to it. Their action leads to the deposition of silt in ridges parallel to and behind the bandals. Thus a channel confined betwen bandals is formed with sand banks on either side.

20.29. NANAL PLANTATION

Nanal is a plant like cane. It is 3 m to 4.5 m height of which bottom 1.5 m is about 12 and 25 mm in diameter and hollow like bamboo. Nanal grass planted along rivers will spread roots and sprouts quickly. The growth of nanal checks the velocity and induces deposition of sediment along the river bank.

Example 20.1. *Design a guide bank required for a bridge on a river having the following particulars :*

Design flood discharge	= *50000 cumecs*
Silt factor	= *1.10*
Bed level of river	= *130.00 m*
High flood level	= *140.00 m*

Also find the volume of stone required per m length of the guide bank.

Solution

$$Q = 50000 \text{ cumecs} \; ; \; f = 1.10$$

Lacey's water way $= 4.75\sqrt{50000} = 1062$ m.

Provide 20% more length to account for thickness of piers and end contractions due to piers and abutments

$= 212$ m

Gross length between banks $= L = 1274$ m

Upstream length of guide bund $= \frac{5}{4}L = \frac{5}{4} \times 1274 = 1592$ m

Downstream length of guide bund $= \frac{L}{4} = \frac{1}{4} \times 1274 = 318$ m

Radius of u/s curved head $= 0.45\,L = 573$ m.

The u/s end of the guide bank may, therefore, be curved by 145° with a radius of 573 m.

Radius of d/s curved head may be kept as 287 m with an angle of 60° at the centre.

Cross section of guide bund

The given H.F.L at the bridge site = 140.0 m

Assuming free board = 1.5 m.

Level of top of guide bund equals 141.5 m. To be more safe and making an allowance for further settlement etc., let us adopt the top level of bund as 142.00 m.

Height of bund above river bed = 142.0 – 130.0 = 12.0 m.

Keep the top width of bund as 4 m and the side slope may be kept as 2 : 1.

The stone pitching and a launching apron must be provided on the slope on water side for the entire length of the bund. However the rear side also will be pitched for curved portions of the bund.

Design of stone pitching and apron

The thickness of stone pitching is given by

$$T = 0.06\,[Q]^{1/3} = 0.06\,[50000]^{1/3} = 2.21 \text{ m.}$$

This can be kept 1.0 m above H.F.L. *i.e.* upto level 141.00 m.

Depth of scour $R = 0.47\,[Q/f]^{1/3} = 0.47\left(\frac{50000}{1.1}\right)^{1/3} = 16.77$ m.

For straight reach of guide bund,

Maximum scour $= 1.25\,R = 1.25 \times 16.77 = 20.97$ m

R.L. at maximum anticipated scour = 140 – 20.97 = 119.03 m

Depth of maximum scour D = 130 – 119.03 = 10.97 m

Length of apron = $1.5\,D = 1.5 \times 10.97$ = 16.45 m

For curvilinear transition portion of guide bund,

Maximum scour = $1.5\,R = 25.16$ m

R.L. of maximum scour = 140 – 25.16 = 114.84 m

Depth of maximum scour D = 130 – 114.84 m = 15.16 m

Length of apron = $1.5\,D = 1.5 \times 15.16$ = 22.74 m

Thickness of launching apron = $1.9\,T = 1.9 \times 2.21 = 4.2$ m

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Volume of Stones

At shank : *on slope* $= \sqrt{5}\ (141 - 130) \times T \times 1$

$= \sqrt{5} \times 11 \times 2.21 = 54.36\ m^3/m$

On apron with a slope 2 : 1

In a vertical length of D and a thickness of 1.25 T

Volume $= \sqrt{5} \times D \times 1.25\,T \times 1$

$= \sqrt{5} \times 10.97 \times 1.25 \times 2.21 = 66.76\ m^2/m$

U/S and D/S curved portions

Volume of apron $= \sqrt{5} \times 15.16 \times 1.25 \times 2.21 \times 1 = 93.65\ m^3/m$

Volume on slope = Same as for shank

$= 54.36\ m^3/m$

Thickness of launching apron

$$= \frac{\text{Volume of apron}}{\text{Width}} = \frac{93.65}{1.5 \times 15.16}$$

$= 4.12\ m \simeq 1.9\,T.$

PROBLEMS

1. Describe various types of river training and protection works.
2. What do you understand by meandering ? What are its causes?
3. Sketch a suitable cross-section of a guide banks as used in river training works. Explain the process of launching of aprons in such works.
4. Describe in brief various types of groynes used for river training. Draw a section of a groyne.
5. Explain with sketches the (i) Denehy's groyne, (ii) Bell's bund. What considerations determine the length of both ?
6. Write a note on the design of a level.
7. What do you understand by a pitched island ?

21

Water Power Engineering

21.1. GENERAL

The various sources from which energy can be had are :

(*i*) Fuels, such as coal, fuel oils and atomic fuel,
(*ii*) Flowing water in rivers and canals,
(*iii*) Wind, and
(iv) Ocean waves and tides.

The use of water power by crude devices dates back to ancient times. The primitive wheels, including the Chinese nora, or float wheel, built of bamboo, with woven paddles are still in use along with the modern development of hydraullic turbines.

India has water power potential of about 41 million kW, out of which 4.4 million kW, (i.e., about 11%) was exploited by 1966. Perhaps the first hydroelectric work in India was undertaken in 1897 near Darjeeling. The State of Mysore constructed, in 1902, the Siva Samudram hydroelectric work with an installed capacity of 4500 kW which was later increased to 42,000 kW. Tatas constructed Khapoli hydroelectric works (1914) with an installed capacity of 50,000 kW, Bhivpui hydroelectric works (1920) with a capacity 72,000 kW and Bhira scheme (1927) with an installed capcity of 110,000 kW. During the period between the two World Wars, some hydroelectric schemes were taken by various states. They include Jogindernagar hydroelectric scheme (48,000 kW), Ganga Canal power station (17,400 kW), Pykara hydroeletric scheme (38,750 kW), Mettur hydroelectic scheme (30,000 kW) and Pillivasal scheme (13,500 kW). After Second World War and before Independence, only two hydroelectric installations, i.e., Papanasam (21,000 kW) and Jog (48,000 kW) were made. After independence, India has made significant progress in Hydro power development, increasing from 0.5 million kW in 1947 to 4.4 million kW by the end of 1966. The planned capacity by type of plant, by the end of 1966 are as follows :

TABLE 21.1. INSTALLED CAPACIY BY THE TYPE OF PLANT

	1950	1955	1960-61	1965-66
Hydro	0.56	0.94	1.85	4.4
Steam	1.59	2.27	3.40	0.62
Oil	0.15	0.21	0.33	0.4
Total	2.3	3.42	5.58	11.0

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21.2. TYPES OF WATER POWER DEVELOPMENT

A water power development is essentially to utilize the available power in the fall of a river or a canal, through a portion of its course, by means of hydraulic turbines. A water power development scheme is known as *hydroelectric scheme* or briefly as *Hydel-scheme*. The hydel plants can be classified according to their function as follows:

(1) Run-Off-River Plants. (2) Storage Plants (3) Pumped Storage Plants.

1. Run-off-River Plants

Run-off-river plants are those which utilize the river flow having no pondage at its upstream. A weir or a barrage is constructed across the river, simply to raise the water level slightly. Such a scheme is essentially a low head scheme, and is adopted in the case of a perennial river which has minimum dry weather flow of such magnitude which makes the development worthwhile. Run-off-river plants are of two types :

(*i*) those that utilize the varying flow, and

(*ii*) those that operate on the minimum available discharge and work throughout the year.

In some cases, small storage or *pondage* is provided in such plants, for tiding over hourly fluctuations of load. When the available rate of flow at the site is in excess of demand, the excess water can be temporarily stored in the pond on the upsteam side of the weir. This *pondage* can then be utilized during the peak demand.

2. Storage Plants

Such a plant has a storage reservoir at its upstream. A dam is constructed across the river and water is stored during the periods of excess supply. The storage to take care of fluctuation of the river supply or that the load enhances the utility of a hydel plant. In India, most of the major plants are of this category.

3. Pumped Storage Plants

A pumped storage plant stores power in the form of potential energy of water. The scheme consists of a storage reservoir at a higher level, and a turbine-cum-pump installation. The scheme is used to generate power only during the peak hours of demand, and is meant for assisting an existing thermal plant. During the peak demand, water flows from the reservoir to the turbine and power is generated. During off-peak hours, excess power is available from the thermal plant or from any other hydro-plant. This power is utilised in pumping the water from the low reservoir (Fig.21.1) back to the storage resevoir at the higher elevation and water is thus stored. This stored water is utilised again during hours of peak demand, thus supplementing the power supply of the existing thermal power station or any other hydroplant. In such a scheme, no water is wasted.

Additional cost of pumping plant can be saved by using the newly developed machines which are designed to work as turbine as well as pump. Similarly, generator-cum-motor is provided instead of separate motor to run the pump and generator to generate electricity from the available mechanical energy of the turbine. By such a scheme, the load factor of the system is very much improved.

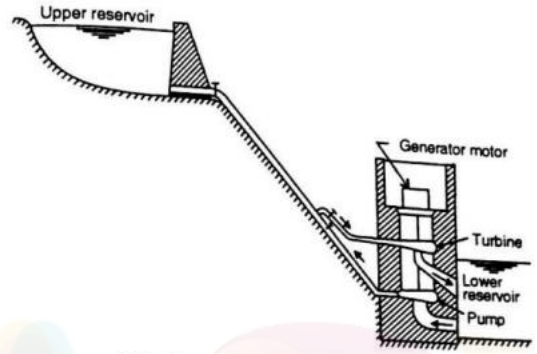

FIG. 21.1. PUMPED STORAGE PLANT.

Classification of hydroelectric schemes

Depending upon the available head, the hydel schemes may be classified as under :

(1) Low head scheme. (2) Medium head scheme. (3) High head scheme.

A *low head scheme* is the one which uses head of less than about 15 metres. It is essentially a run-off river scheme. Fig. 21.2 shows the schematic plan of such a scheme. A weir or barrage is constructed to raise the water level in the river. The power-house may be constructed in continuation with the barrage. Sometimes, the barrage is constructed some distance upstream of powerhouse and water is conveyed to the power house through an intake canal.

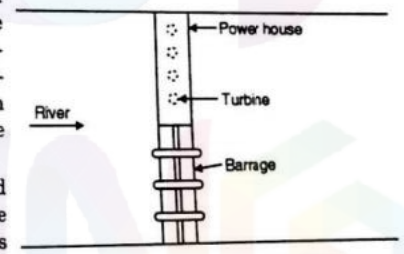

FIG. 21.2 LOW HEAD SCHEME.

A *medium head scheme* uses head between 15 to 60 metres. Such a scheme is essentially a storage scheme. A dam is constructed to store some quantity of water and also to obtain the required head. A medium head scheme has the features mid-way between the low head scheme and the high head scheme.

Conventionally, a *high head scheme* uses a head of more than 60 metres. A dam of sufficient height is constructed to store the water to ensure the supplies throughout the year, and to attain sufficient head. Fig. 21.3 shows the schematic plan for such a scheme. High head scheme using heads upto about 1800 metres have been developed.

The high heads needed for high head schemes can be obtained by one of the following methods :

1. ***Construction of high dam.*** The examples of Bhakra dam in Punjab, Rihand dam in U.P. or Hoover dam in U.S.A. fall under this method. The high dam also helps in creating sufficient storage

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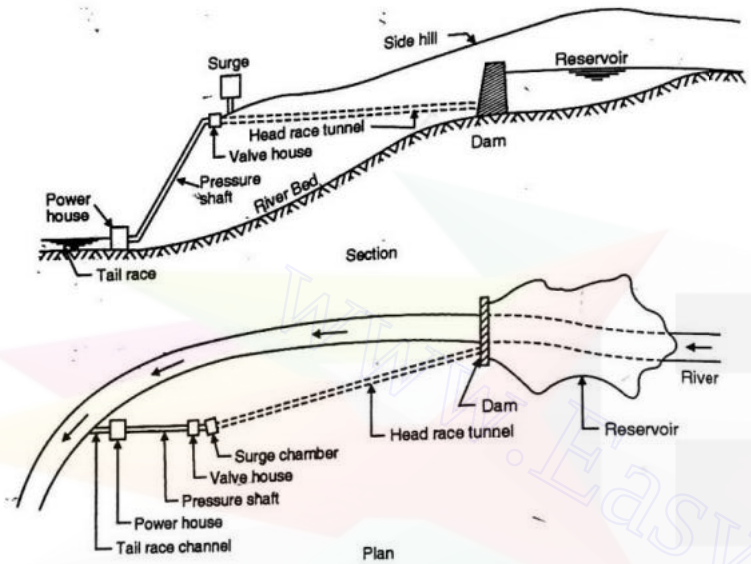

FIG. 21.3. HIGH HEAD SCHEME.

2. ***Utilisation of natural fall.*** The natural high falls may be utilised for power generation. Examples under this are Jog Falls in India and Niagara Falls in U.S.A.

3. ***Conveying water to a lower valley.*** The water may be conveyed to a lower valley, by cutting across the watershed, so that high head is available. The Koyna hydel scheme is one such example. Fig 21.4 shows the schematic section of Koyna scheme.

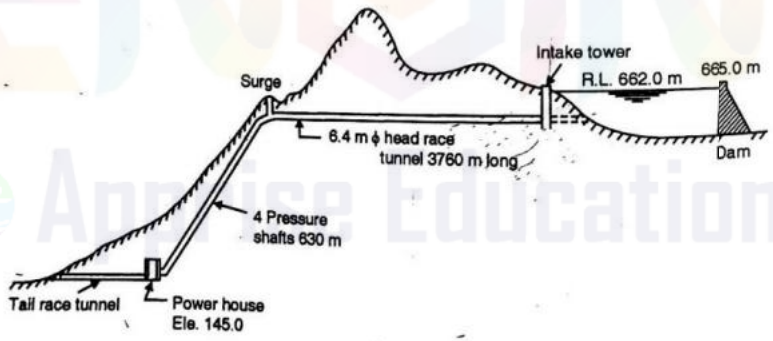

FIG. 21.4. A SCHEMATIC SECTION OF KOYNA SCHEME (TAIL RACE DEVELOPMENT STAGE III NOT SHOWN)

21.3. PRINCIPAL COMPONENTS OF HYDROELECTRIC SCHEME

A hydroelectric work comprises of the following components :

1. Forebay
2. Intake structure
3. Surge tank
4. Penstocks
5. Power-house
6. Turbines and governors
7. Generators
8. Transformers
9. Transmission lines.

21.4. FOREBAY

A forebay is an enlarged body of water just in front of the intake. The main function of the forebay is to store, temporarily, the water rejected by the plant when the load is reduced and to meet the instantaneous increased demand when the load is instantaneously increased. Thus, the forebay absorbs the short interval variations of intake of water into turbines in accordance with the fluctuating loads. When a canal leads water to the turbine, the canal itself serves as a forebay. When the penstocks directly take water from the reservoir, the resevoir acts as the foreby. Often, the canal near the power-house is enlarged to provide an effective foreby.

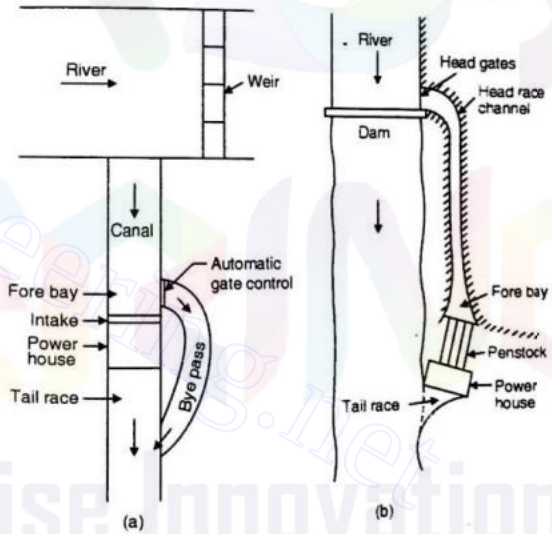

FIG. 21.5. FOREBAY.

Sometimes, a bypass is provided to convey water from the forebay to tail race when the load is reduced. For maximum efficiency, the water level in the forebay is automatically controlled within narrow limits by providing automatic gates at the head of the bypass. When the load is reduced, the governors of the turbines reduce the entry of water. This in turn, has the tendency to increase the water level in the forebay.

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However in order to keep the constant water level in the forebay, the gates at the head of the bypass are automatically operated so that the spillage into the bypass is increased. Similarly, during the increased load, more water is required. The automatic gates then move up and reduce the spillage into the bypass.

21.5. INTAKE STRUCTURE

The water is conveyed from the forebay to the penstocks through the intake structure. Following are the accessories of an intake structure

1. *Trash rack* : There rack prevents the debris getting into the penstock. See § 11.13 and Fig. 11.45 for details.
2. *Rakes* to clear the trash rack.
3. Ice removal equipment.
4. Penstock closing gates with hoists.

21.6. PENSTOCKS

Water from the storage reservoir is carried through penstocks or canal to the power-house. Penstocks are the pipes of large diameter, usually made of steel in various forms, reinforced concrete or wood stave, which carry water under pressure from the storage reservoir to the turbine. Penstocks may be subjected to water hammer pressure due to fluctuations in the load. Short length penstocks are designed to take this extra pressure. However, in the case of long penstocks, surge tank is provided to reduce the water hammer.

The intake of the penstock at the forebay must be at a level low enough to provide an adequate water seal under all conditions, particularly at low water, so that the entry of air is prevented. If there is too little depth of cover, whirpools will tend to form and carry into the penstock and to the wheels, tending to lessen the power output. In addition to proper depth of water seal, the entrance to the penstock should be flared to avoid any loss of head by contraction. Sharp bends cause loss of head and require special anchorages, and should be avoided.

An air vent, and stand pipe connecting the top of the penstock with the open air, should be provided below the gate. The air vent permits the air to enter the penstock when the head gates are closed and water drawnoff through the wheel. In the absence of this, dangerous collapse pressure may be exerted on the penstocks. The water in the vent pipe should not get frozen, otherwise it will prevent entrance of air. Before opening the main head gate, the penstock, if empty, should be filled through a bypass so that the surge of water may not damage the turbine.

21.7. SURGE TANK

When the load on the turbine is reduced, the governor automatically closes the inlet gates partially, to reduce the inflow to the turbines. This retards the water flowing in the penstocks, leading to water hammer in the penstocks. The excessive inertia pressure so caused may burst the penstock. A surge tank may be provided to reduce the water hammer pressures. The functions of surge tank are as follows :

1. The surge tank furnishes a reservoir surface very near to the discharge end of the penstock to dampen the water hammer pressure.

2. It temporarily stores water when the load is reduced until such time that the velocity has fallen to the steady value.
3. It temporarily supplies more water when the load is increased until such time that the velocity has increased to a fresh steady value.
4. It calms down effectively and rapidly the surging in the water level.

An elementary surge tank consists of cylindrical open-top storage reservoir connected by a vertical branch pipe to the penstock at a point as close to the turbine as possible (Fig. 21.6). Surge tanks are almost always built high enough so that the water cannot over flow, even with a full load change. Where topography permits, the surge tank may be placed on the surface of the ground, above the penstocks line, at the point where the latter drops rapidly to the power house. The minimum height of a surge tank is determined by the restriction that in no case must the water level in it drops to such a point that air is drown into the penstock.

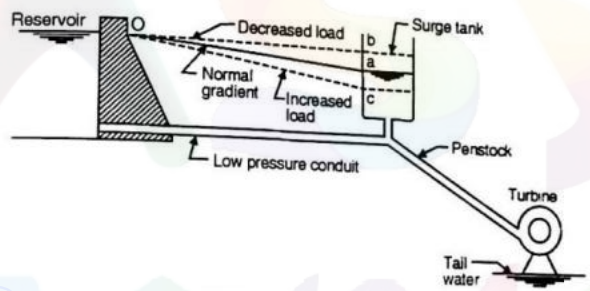

FIG. 21.6. SURGE TANK.

Under normal load condition, a normal hydraulic gradient *oa* (Fig.21.6) exists. When load is reduced, water level in the surge tanks rises and the hydraulic gradient is flattened. This decelerates the flow from the reservoir to the surge tank. When the load increases, more water is drawn and the water level to the surge tank drops. This makes the hydraulic gradient steeper, thus accelerating the flow from the reservoir to the surge tank. Other types of surge tanks are : (*i*) Surge shaft with expansion chamber, (*ii*) Throttled or restricted orifice surge tank, (*iii*) Differential surge tank and (*iv*) Multiple surge tank. A differential surge tank has a central riser pipe having small ports or holes at its lower end. The main advantage of this type is that for some stabilising effects its capactiy may be less than an elementary surge tank. This is so because in a differential surge tank retarding and accelerating heads are developed more promptly than in the simple surge tank in which the head build up gradually as the tank fills.

21.8. TURBINES

Hydraulic turbines are the machines which convert hydraulic energy into mechanical energy. The mechanical energy devloped by a turbine is used in running an electric generator which is directly coupled to the shaft of the turbine. The generator thus

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develops electric power which is also sometimes known as *hydro-electric power*. In general, a water turbine consists of a wheel called *runner* which is provided with specially designed blades or buckets. The water possessing large hydraulic energy when strikes the runner and causes it to rotate.

Hydraulic turbines may be classified under two heads :

(*i*) Impulse or velocity turbines. (*ii*) Reaction or pressure turbines.

1. Impulse turbine

In the case of impulse turbine, all the available potential energy or head is converted into kinetic energy or velocity head by passsing it through a contracting nozzle or by guide vanes before it strikes the buckets of the turbine. The wheel revolves free in air and water is in contact with only a part of the wheel at a time. The pressure of water all along is atmospheric. In order to prevent splashing and to guide the water discharged from the buckets to the tail race, a casing is provided. Thus, an impulse turbine has the following characteristic features :

(*i*) The wheel passages are not completely filled.

(*ii*) The water acting on the wheel vanes is under atmospheric pressure.

(*iii*) The water is supplied at a few points at the periphery of the wheel (usually one point, but occasionally two or more points).

(*iv*) Energy applied to the wheel is wholly kinetic.

An impulse turbine is essentially a low-speed wheel and is used for relative high heads, ordinarily beginning in the neighbourhood of 150 m and so far being limited to about 1000 m, though a head of over 1770 m has been utilized by impulse wheels in Switzerland. Examples of the impulse turbines are: Pelton wheel, Turgo-impulse wheel, Girard turbine, Banki turbine, Jonval turbine etc. Out of these, the Pelton wheel is predominantly used at present.

2. Reaction Turbine

In the case of reaction turbine, only a part of the available potential energy is converted into velocity head, at the entrance to the runner, and the balance that forms a substantial portion remains as a pressure head. The pressure at the inlet to the turbine is much higher than the pressure at the outlet, and it varies throughout the passage of water through the turbine. Major part of the power is derived from the difference in pressure acting on front and back of runner blades and only a minor part from the dynamic action of velocity. The water being under pressure, the entire flow from head race to tail race takes place in a closed conduit system and the atmospheric air has no access at any point. To summarize, a reaction turbine has the following characteristic features :

(*i*) The wheel passages are completely filled with water.

(*ii*) The water action on the wheel vanes is under pressure greater than atmospheric.

(*iii*) The water enters all around the periphery of the wheel.

(*iv*) Energy in the form of both pressure and kinetic is utilized by the wheel.

The examples for reaction turbines are : Fourneyron turbine, Tompson turbine, Propeller turbine, Francis turbine, Kaplan turbine etc. Out of these, Francis and Kaplan turbines are predominantly used at present.

The turbines may also be classified according to the direction of flow of water in the runner, as follows :

(*i*) Tangential flow turbine. (*ii*) Radial flow turbine

(*iii*) Axial flow turbine (*iv*) Mixed flow turbine.

In a *tangential flow*, the water strikes the runner along the tangent to the part of rotation of the runner, such as in the Pelton wheel. In a *radial flow*, the water flows along the radial direction and remains in a plane normal to the axis of rotation, as it passes through the runner. The radial flow may be *inward* (i.e. water entering the outer circumference and flowing radially inward) such as in the Francis turbine, Thomson turbine, or Girard turbine, or the flow may be outward (i.e. water entering at the centre and flow radially outward), such as in Fourneyron turbine or Boyden turbine. In an *axial flow* turbine or *parallel flow* turbine, water enters and leaves the runner along the direction parallel to the axis of the turbine shaft. Examples of this type of flow are in Jonval turbine, Girard turbine, propeller turbine and Kaplan turbine. In the *mixed flow turbine*, water enters the runner at the outer periphery in the radial direction and leaves it at the centre in the direction parallel to the axis of the shaft. Modern Francis turbine is one such example.

Specific speed

The turbines may also be classified according to their specific speed. The specific speed of a turbine is the speed of a geometrically similar turbine that would develope one metric horse power under a head of one metre. Table 24.2 shows the specific speeds of various turbines.

TABLE 21.2

S.No.	Head (meters)	Type of turbine	Specific speed
1	300 or more	Pelton wheel (single jet)	10 to 35
2	150 to 300	Pelten or Francis	35 to 100
2(a)	200 to 300	Pelton wheel (Multiple juets)	35 to 55
2(b)	150 to 200	Francis turbine	60 to 100
3	60 to 150	Francis turbine	100 to 220
4	Less than 60	Francis, Kaplan or Propeller	220 to 1000
4(a)	30 to 60	Francis turbine	220 to 300
4(b)	16 to 30	Kalpain turbine	300 to 1000
4(c)	Less than 16	Kaplan or Propeller (for no load variation)	300 to 1000

21.9. SELECTION OF SUITABLE TYPE OF TURBINE

The selection of the suitable type of turbine depends primarily upon the available head and the quantity of water required. On this basis, turbines may be classified as follows :

1. Low head turbine 2. Medium head turbine

3. High head turbine.

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Low head turbines are those which are capable of working under heads less than 30 m. Examples of such turbines are : propeller turbine and Kaplan turbine. These turbines use large quantity of water. *Medium head turbines* are capable of working under the heads ranging from about 30 to 150 m. The example under this type is that of Modern Francis turbine. *High head turbines* are those which are capable of working under very high head ranging from 150 to over 1000 metres. Impulse turbines are high head turbines. These turbines require relatively less quantity of water. Table 21.2. gives the range of head and specific speed for which each type of turbine is most suitable. As far as possible, a turbine with highest permissible specific speed should be chosen, which will not only be cheapest in itself but its relatively small size and high rotational speed will reduce the size of the generation as well as the power-house. But the specific speed cannot be increased indefinitely because higher specific speed turbine is generally more liable to cavitation.

Part Load Operation

The turbines may be required to work with considerable load variations. As the load deviates from the normal working load the efficiency would also vary. At part load, the performance of Kaplan and Pelton turbines are better in comparison to that of Francis and Propeller turbines. The variability of load will influence the choice of type of turbine if the head lies between 150 m to 300 m, or lies below 30 m. For higher range of heads, Pelton wheel is preferable for part load operation in comparison to Francis turbine, though former involves higher initial cost. For heads below 30 m, Kaplan turbine is preferable for part load operation in comparison to Propeller trubine.

In addition to the above points, there are certain other points to be considered for the selection of right type of a turbine. The *rotational speed* of the turbine should be such that it will give the synchronous speed of the generator. The *overall cost* which includes inital cost and the running cost should also be considered. *Cavitation* affects the installation of a reaction turbine. The *disposition of turbines shaft* may be vertical which is better for large size reaction turbines or horizontal which is suitable for large size impulse turbine. In the 150 to 300 m range of head adopt either to the Pelton wheel or the Francis turbine ; the presence of large amounts of sand in water would rule out the Francis type, since its runner will not be able to withstand the erosive action.

21.10. THE POWER-HOUSE

A power-house of a hydel scheme serves as a protective covering for the hydraulic and electrical equipment. The power-house is the conspicuous and vital part of a hydroelectric development. It should, therefore, be of satisfactory appearance, harmonizing with the surroundings. The following items of equipment should be provided for the layout of the power-house :

1. ***Hydraulic Equipment***
   - (*i*) Turbines
   - (*ii*) Gates or gate valves
   - (*iii*) Relief valves for penstock settings
   - (*iv*) Governors

   - (*v*) Flow measurement equipment.
2. ***Electrical Equipment***
   - (*i*) Generators :
     - Air ducts
   - (*ii*) Exciters
   - (*iii*) Transformers :
     - Pumps and cooling system
     - Connections
     - Runs and platforms
   - (*iv*) Switching equipment :
     - Low tension buses
     - Switch board panels
     - Switch board equipment and instruments
     - Oil switches
   - (*v*) High tension system :
     - Buses
     - Oil circuit breakers
     - Lightening arresters
     - Outgoing connections
   - (*vi*) Auxiliaries :
     - Storage batteries
     - Station lighing.
3. ***Miscellaneous Equipment*** :
   - (*i*) Crane
   - (*ii*) Shop
   - (*iii*) Office rooms
   - (*iv*) Lavatory, etc.

It is becoming more and more the practice to place the transformers and high tension equipment outdoors near the power-house which results in a material saving in cost. A power-house has two main parts : (*i*) super-structure and (*ii*) sub-structure. A super-structure is that portion of power house which is above the ground level. A big-size power station may be a multi-storeyed building having the main machine hall at the ground floor. Sometimes, in a mountaineous country, underground power-house may be constructed. The sub-structure of the power-house mainly consists of a foundation block of concrete immediately below the machine hall. The concrete block has a number of spaces and passages for fixing the turbine scrolls, their draft tubes and power cables.

General arrangement of hydroelectric units in the power-house

In the case of vertical single runner water-wheel unit of reaction turbines and direct connected generators, the most convenient and economical arrangement is to place the units in a line parallel to the length of the power-house. In the case of horizontal units of reaction turbines, the best arrangement is to place each unit on a line normal to the axis of length of the power-house. In both the cases the spacing between the

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units is controlled by the flume of scroll casing, draft tube, tail race or generators dimensions. For horizontal setting of impulse turbines, the best arrangement is to place the wheel shaft parallel to the big axis of the power-house. The spacing of units in this case is determined by their dimensions and necessary clearance rather than by penstock, flume or tail race widths. For vertical settings of impulse-wheels, the units should lie with centres on a line parallel to the axis of length of the house, both penstock and tail race channel running in the same direction.

The above arrangements require a rectangular building with main floor occupied by the turbines and their accessories, the governors, oil pumps, exciters etc. The switch board and operating platform are generally placed on a mezzanine or second floor. A travelling crane is an important part of the power-house equipment. It will span the width of the house and be arranged to travel for its entire length. In fixing the elevation of the crane rail above the floor, it is essential that sufficient head room be provided for lifting and carrying along any of the various machine parts.

21.11. SCROLL CASING, DRAFT TUBE AND TAIL RACE

1. Scroll Casing

Scroll casing surrounds the turbine runner and the guide mechanism. The water from the penstock is delivered to the scroll casing. The spiral casing is spiral shaped to avoid the formation of eddies and to distribute water evenly around the guide vanes. The casing is made of cast steel, plate steel or concrete, depending upon the head. The main purpose of the scroll casing is to maintain nearly uniform velocity at the entry to the guide vanes. The use of the concrete scroll casing is necessarily confined to low heads, usually not over 20 m. Plate-steel scroll casing is used for head upto 110 metres.

2. Draft-Tube

A draft-tube is a pipe or passage of gradually increasing cross-sectional area, which connects the runner exit to the tail race. It serves the following purposes :

(1) It permits a negative or suction head to be established at the runner exit, thus making it possible the placing of wheel and connecting machinery at a level above that of water in the tail race under high-water flow-conditions of river, without loss of head.

(2) It reduces the high velocity of the water discharged by the turbine by gradually enlarging the cross-section and thus converts the kinetic energy of the water leaving the runner, into mechanical energy of the wheel. Thus it acts as a recuperator of pressure energy.

Fig. 21.7 shows the various types of draft tubes in common use. The conical tube or vertical tapered pipe [Fig. 21.7(*a*)] expanding at the rate of 4 to 8 degrees is made of steel plate and is the most efficient of all the types of draft tubes. It is the most efficient in reducing exit losses, but with large units this would mean excessive excavation and sub-structure cost. Consequently, for large units, the elbow tube [Fig.21.7(*b*)] has become conventional as it affords benefit in cost of excavation. The hydraucone draft tube [Fig.21.7 (*c*)] patented in 1917 by W.N. White utilizes the so-called 'hydraucone action' of jet of water impinging an a plate and being deflected

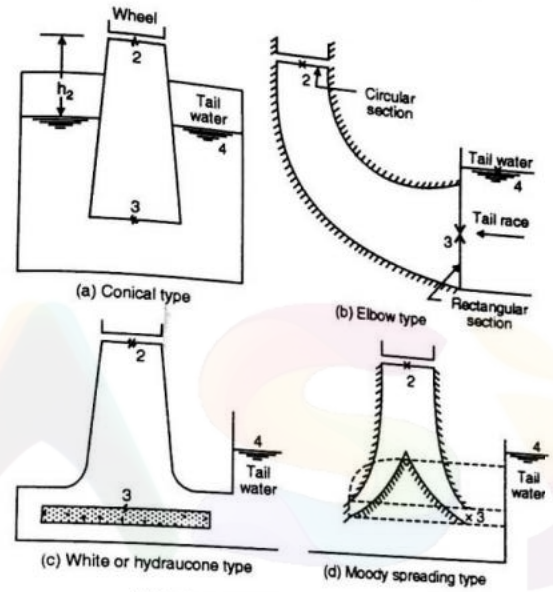

FIG. 21.7. TYPES OF DRAFT TUBES.

there from in all directions. The Moody or spreading draft tube [Fig.21.7 (*d*)] obtains substantially the same effect as the hydraucone by an actual cone constructed symmetrically in tube. The hydraucone and Moody draft tubes also permit the better utilization of the 'whirl component' of water entering the draft tube. *For impulse turbines, no draft tubes are provided* and the head from nozzle to tail race is lost.

Applying Bernoulli's equation between point 2 (at the entrance of the draft tube) and point 4 (at tail race level). we get

$$\frac{V_2^2}{2g} + \frac{p_2}{w} + h_2 = \frac{V_4^2}{2g} + 0 + 0 + h_f + k\frac{V_3^2}{2g} \quad ...(1)$$

where h_f represents friction losses in the draft tube, and k is approximately unity. Transposing and noting that $\frac{V_4^2}{2g}$ is usually small and may be neglected, we get

$$\frac{p_2}{w} = -h_2 - \left(\frac{V_2^2}{2g} - \frac{V_3^2}{2g}\right) + h_f \quad ...(21.1)$$

and

$$h_2 = -\frac{p_2}{w} - \left(\frac{V_2^2}{2g} - \frac{V_3^2}{2g}\right) + h_f \quad ...(21.2)$$

From the above approximate analysis, it is clear that the pressure in the draft tube next to the turbine blades is negative. Except for the losses, the residul head

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(h_2) and the large proportion of velocity head $\left(\frac{V_2^2}{2g} - \frac{V_3^2}{2g}\right)$ are converted by the draft tube into a negative pressure head. Since, to avoid cavitation, the negative pressure has to be limited to such a value that the absolute pressure in the draft tube remains above the vapour pressure of water, it is evident that for a given height of draft tube there is a limiting value of V_2. Or, conversely, a given value of V_2 limits the height of draft tube. The height h_2 of the wheel above the tail race is limited to about 8 m as an extreme.

3. Tail race

Tail race is the channel into which the draft tube discharges. Where the power-house is close to the river, the tail race may be the river itself. In other cases, an artifical race channel may connect the power-house with the river. Moody spreading tubes, and some forms of hydraucone, where there is no open channel under the power-house, the sub-structure becomes practically a mass of concrete with water passages. The depth of tail race depends upon the assumed design velocity. Usually a velocity of 0.8 to 1.5 m/sec will be suitable. The tail race channel should be maintained properly so that excessive aggradation and degradation is avoided. The excessive aggradation raises the tail water level, thus reducing the head. The excessive degradation may lower the tail water levels, and may uncover the draft tube. This may, in turn, cause unsatisfactory flow conditions, increases the losses and may damage the turbine blade by way of cavitation.

21.12. ASSESSMENT OF WATER POWER POTENTIAL

Let Q = discharge in cumecs, and

H = height of fall, in metres.

Then the rate of work done is

$$= QwH = 9.81\,QH \text{ kN-m or kilo-watt}$$

(Since 1 kN-m = 1 kW)

Let η_t be the efficiency of turbines, η_g be the efficiency of generators and η ($= \eta_t \times \eta_g$) be the overall efficiency. The electrical energy or power (P) generated by the hydro-electric plant is given by

$$P = 9.81\,\eta_t\,\eta_g\,QH = 9.81\,\eta\,QH \text{ kW} \quad ...(21.3)$$

Taking $\eta = 0.8$, $$P = 9.81 \times 0.8\,QH = 7.84\,QH \simeq 8\,QH \text{ kW} \quad ...(21.3\ a)$$

For the usual values of efficiencies, the electrical power available can be roughly estimated to be 7.5 QH to 8.5 QH kW.

Since 1 metric H.P. = 0.735 kW, the metric H.P. generated is given by

$$\text{Metric H.P.} = \frac{9.81\,\eta\,QH}{0.735} = 13.33\,\eta\,QH \quad ...(21.4)$$

Gross head, operating head and effective head

Gross head is the difference in the elevation of the point of diversion of water for a hydel scheme and the point of return of water back to the stream. *Operating head* is the simultaneous difference between the elevations of the water surface in the forebay and the tail race after making due allowance for approach and exit velocity head. The *effective or net head* is the head applied to the turbine and denotes the difference of head at the points of entry and exit of the turbine taking into account both the velocity and the pressure heads.

Primary or Continuous power and Secondary power

The *primary or firm power* is that which must be always available for carrying load or the corresponding to the minimum stream flow, with due consideration of the effects of pondage and load factor. It is the minimum continuous power available from a plant under the most adverse hydraulic conditions. *Secondary power* or *surplus power* is the power other than primary generated at the plant. The term surplus power is also sometimes used to distinguish the excess power available particularly at night, when owing to low or no-load conditions, there is no use for the power.

Installed capacity, dependable capacity

The *installed capacity* of a power station shows the total capacity of all the turbine-generator units installed in the power station. *Dependable capacity* is the load carrying ability of the power station with respect to the characteristic of the load to be supplied during a specified time interval and the period, and is determined by the power factor, capacity and the load on a station.

Load factor, utilization factor and capacity factor

The *load factor* is defined as the ratio of the average load over a certain period to the peak load occuring during the same period.

i.e. $$\text{L. F.} = \frac{\text{Average load over a certain period}}{\text{Peak load occuring during the same period}}$$

If the period chosen is one day, the load factor is known as the *daily load factor*. The *annual load factor* for a power station is the annual load factor for the entire area served by the power supplied from the power station. The load factor for industrial area will be higher than that for domestic area since the power demand by industry is more uniform than that by domestic area.

The *utilization factor* is the ratio of water actually utilized for power to that available in the river, this latter being limited as shown by the flow-duration curve and (in the higher part of the curve) the discharge capacity of the wheels. With constant head, utilization factor would also be the ratio of power utilized to that available.

Capacity factor or plant use factor or plant factor is the ratio the average output of the plant to the plant capacity *i.e.*

$$\text{Plant capacity factor} = \frac{\text{Average load output}}{\text{Installed capacity}} = \frac{\text{Average load}}{1.1\text{ (peak load)}},$$

if we take the reserve capacity as 10% of the peak load.

If the average capacity is zero, load factor on a particular day = capacity factor for that day. Thus, capacity factor will be identical with load factor when the maximum just equals the plant capacity. The capacity factor for a hydroelectric plant commonly varies from about 0.25 to 0.75, depending on load factor etc. The capacity factor does

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not exceed 0.75 for the usual plant capacity, unless in addition to high load factor and complete pondage at the plant, a large degree of regulation by storage is possible.

Example 21.1. *Three turbo-generators each of capacity 10000 kW have been installed at a hydel power station. During a certain period of load, the load on the plant varies from 12000 kW to 26000 kW. Calculate (i) total installed capacity, (ii) load factor (iii) plant factor and (iv) utilisation factor.*

Solution

(*i*) Total installed capacity $= 3 \times 10000 = \mathbf{30000\ kW}$

(*ii*) $$\text{Load factor} = \frac{\text{Average load}}{\text{Peak load}}$$

$$\text{Average load,} = \tfrac{1}{2}(12000 + 26000) = 19000 \text{ kW}$$

$$\text{Peak load} = 26000 \text{ kW.}$$

$$\therefore \quad \text{Load factor} = \frac{19000}{26000} = 0.7308 \triangleq \mathbf{73.1\%}$$

(*iii*) $$\text{Plant factor} = \frac{\text{Energy actually produced}}{\text{Max. energy which can be produced}}$$

Taking any time duration t hours,

$$\text{Energy actually produced} = \tfrac{1}{2}(12000 + 26000)\,t = 19000\,t \text{ kWH}$$

$$\text{Max. energy that could be produced} = 3 \times 10000 \times t = 30000\,t \text{ kWH.}$$

$$\therefore \quad \text{Plant factor} = \frac{19000\,t}{30000\,t} = 0.633 \triangleq \mathbf{63.3\%}$$

(*iv*) $$\text{Utilisation factor} = \frac{\text{Max. power developed}}{\text{Total power that can be developed}}$$

$$= \frac{26000}{30000} = 0.867 = \mathbf{86.7\%}$$

Example 21.2. *A runoff river hydroelectric power station is proposed across a river at a site where a net head of 30 m is available on the turbine. The river carries a sustained minimum flow of 40 cumecs in dry weather and behind the power station, sufficient pondage is provided to supply daily peak load of demand with a load factor of 73%. Assuming the plant efficiency of 60%, determine the maximum generating capacity of the generator to be installed at the power house. If the daily load pattern indicates 20 hours average load and 4 hours peak load, determine the pondage to be provided to supply the daily demand.*

Solution

The average electric energy is given by

$$P = 9.81\,QH\eta = 9.81 \times 40 \times 30 \times 0.6 = 7063.2 \text{ kW}$$

$$\text{Load factor} = \frac{\text{Average load}}{\text{Peak load}}$$

$$\therefore \quad \text{Peak load} = (\text{Average load})/\text{L.F.} = 7063.2/0.73 = 9675.6 \text{ kW}$$

$\therefore$ Max. capacity of generators to be installed $= 9675.6$ kW

During 3 hours of peak load, the excess power to be developed from the pondage

$$= 9675.6 - 7063.2 = 2612.4 \text{ kW.}$$

The discharge required for developing the above excess power is

$$Q = \frac{P}{9.81\,H\eta} = \frac{2612.4}{9.81 \times 30 \times 0.6} \triangleq 14.8 \text{ cumecs.}$$

$$\therefore \quad \text{Pondage required} = (14.8 \times 4 \times 3600) \triangleq \mathbf{21.3 \times 10^4\ m^3}$$

Example 21.3. *A run-off-river plant has a installed capacity of 15000 kW and operates at 30% load factor when it serves as a peak load station. What should be the minimum discharge in the stream so that it may serve as a peak load station? The plant efficiency may be taken as 82% when working under a head of 25 m. Also calculate the maximum load factor of the plant when the discharge in the stream is 32 m^3/s.*

Solution

Average power developed when the plant operates as peak load plant, with a L.F. of 30%

$$= 15000 \times 0.3 = 4500 \text{ kW}$$

$\therefore$ Discharge Q required to generate the above power is

$$Q = \frac{P}{9.81\,H\eta} = \frac{4500}{9.81 \times 25 \times 0.82} = \mathbf{22.38\ m^3/s}$$

If the stream has a flow of 32 m^3/s, power developed is

$$P = 9.81 \times 32 \times 25 \times 0.82 = 6435.36 \text{ kW}$$

$$\therefore \quad \text{Maximum load factor} = \frac{6435.36}{15000} = 0.429 = \mathbf{42.9\%}$$

PROBLEMS

1. Describe various types of hydel schemes.
2. Classify the hydel schemes according to available head. Draw a sketch to show a general layout and section of a high head scheme.
3. Write a note on pumped storage plants.
4. Distinguish clearly between run-off river plants and storage plants.
5. Enumerate principle components of a hydroelectric scheme.
6. Write notes on (*i*) Forebay, (*ii*) Intake structure, (*iii*) Penstock.
7. What is surge tank ? Discuss its function and working.
8. Write notes on : (*i*) Scroll casing, (*ii*) Draft-tube, (*iii*) Tailrace.
9. Write a note on the selection of suitable type of turbine for a hydroelectric scheme.
10. How do you assess the water potential of a hydroelectric scheme?

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22

Water Resources Planning

22.1. INTRODUCTION

Water resources are planned and developed either to use or to control ground and surface water flows. In the process of development, important questions that may arise are :

1. How much water is needed?
2. How much water is available?
3. How are the requirements satisfied by the supplies?
4. How is the used up water disposed of.

A developmental activity for the use of water comprises studying the availability and demand of water and contemplating a project that can meet the expected needs from available supplies by means of engineering works and non structural measures.

The development of water resources involves the conception, planning, designing, construction and operation of facilities to control and utilise water, with the *national objective* of *improving the quality of the life of the people*. If should be recognized that planning and design studies provide only the conceptual framework of a water development project. The complete process of development includes the construction, operation and management of the project as well.

22.2. INDIA'S WATER RESOURCES

India, with a geographical area of nearly 3.3 million square kilometers experiences extremes of climate. Normal annual rainfall varies from 100 mm in Western Rajasthan to over 11000 mm at Cherrapunji in Meghalaya. Variability of rainfall from season to season is also very high. The average annual rainfall over the country is of the order of 1170 mm depth which is nearly 4000 km^3 (where 1 km^3 = 10^9 m^3). Of the total precipitation a part goes towards increasing ground water storage, a part is lost as evapo-transpiration and the remaining appears as *surface water*.

The average flow in the river systems of the country has been estimated to be 1880 km^3. But over 90% of the annual runoff in peninsular rivers and over 80% of the annual runoff in Himalayan rivers occur during the four monsoon months of June to September. Also, the amount of water that can be actually put to beneficial use is much less. The major constraint in exploiting the available water resources is the fact that the major part of the river flows occur as flood flows during three or four

months and *consequent necessity for storing the flood flows in reservoirs for their utilisation*. is about 690 km^3 only, which is about 36% of the total.

Ground water is another important phase of water. Quantum of water which can be extracted economically from the ground water aquifers every year is generally reckoned as *ground water potential*. The Central Ground Water Board (CGWB) has under taken scientific assessment of the quantum of replenishable ground water in the country. The preliminary estimates made by the Board indicate that the utilisable ground water is about 450 km^3.

22.3. SCENARIAO OF WATER USE

Being faced with a rainfall season with few raining days, agriculture in India is heavily dependent on irrigation. The principal consumptive use of water is in irrigation. The quantum of water, being utilised for other purposes, such as that for domestic use, energy, industrial use etc etc. is far less than that used for irrigated structures.

Table 22.1 shows the projected water demands for various purposes, upto the year 2025.

TABLE 22.1 WATER DEMANDS (km^3)

Purpose	*1990*	*2000*	*2025*
1. Domestic use	25	33	52
2. Irrigation	460	630	770
3. Energy	19	27	71
4. Industrial use	15	30	120
5. Others	33	30	37
Total	552	750	1050
(a) Surface water	362	500	700
(b) Ground water	190	250	350

22.4. PURPOSES OF WATER RESOURCES DEVELOPMENT

Water resources development projects are planned to serve the following purposes:

1. ***Municipal and Industrial water supply***
   - (i) Surface water development
   - (ii) Ground Water development
   - (iii) Water desalination
2. ***Irrigation***
   - (i) Crop growth including cultivation of cash crops
   - (ii) Land reclamation
   - (iii) Salinity Control
3. ***Hydropower generation***
   - (i) Run-of-river and storage projects
   - (ii) Hydropower from pumped storage
   - (iii) Tidal power

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4. ***Flood control***
   - (*i*) Reservoirs and detention basins
   - (*ii*) Embankments or dikes
   - (*iii*) Flood diversion
   - (*iv*) River channel improvements
   - (*v*) Watershed management
   - (*vi*) Flood zoning
   - (*vii*) Flood forecasting and warning
5. ***Water transport***
   - (*i*) Navigation
   - (*ii*) Port or harbour
6. ***Soil conservation***
   - (*i*) Erosion control
   - (*ii*) Sedimentation control
   - (*iii*) Watershed management
7. ***Pollution Control***
   - (*i*) Prevention of salt water intrusion
   - (*ii*) Waste water dilution.
8. ***Scenic Preservation***
   - (*i*) Fish and wild life
   - (*ii*) Recreation
   - (*iii*) Public health and sanitation

22.5. CLASSIFICATION OF WATER RESOURCES DEVELOPMENT PROJECTS

Water resources development projects can be classified following types categories:

1. Single purpose project
2. Multipurpose project

A single purpose project is the one which serves only one purpose, such as that for irrigation, or for water power generation, or for flood control. A multipurpose project is the one which serves two or more purposes such as (*i*) storage and control of water for irrigation (*ii*) storage and diversion of water for domestic uses (*iii*) water supplies for industrial uses (*iv*) development of hydroelectric power (*v*) inland navigation (*v*) flood control (*vi*) reclamation of land (*vii*) debris control (*viii*) preservation and cultivation of aquatic life and (*ix*) recreation.

The *multipurpose development* is aimed at fulfilling the following *objectives:*

1. To enhance national economic development by *increasing the output* of goods and services.
2. To enhance the quality of the environment by *improving the quality* of the natural and cultural resources and the ecological system.
3. To enhance *social well being* through developmental activities.
4. To enhance *regional development* through increase in region's income, employment, economic base and other components of the regional objective.

22.6. FUNCTIONAL REQUIREMENTS IN MULTI-PURPOSE PROJECTS

A *multipurpose project* has uses of water for several purposs. The water stored in the reservoir has to be jointly used for these purposes, with proper coordination and compatibility.

1. Requirements for irrigation :

Water requirement for irrigation is not uniform. Maximum demand for irrigation is during winter season when Rabi crops are grown. However, if Kharif crops are grown, irrigation water will be required for the period perior to monsoon. Low rainfall years require more irrigation water than the high rainfall years. Since a storage reservoir protects the area against drought, some storage should be reserved to meet the irrigation demands during drought period.

2. Requirements for domestic supply :

The water requirement for domestic use is more or less constant, though the requirement is slightly more during summer than in winter. With increase in standards of living, the requirements of water for domestic use is gradually inreasing.

3. Requirments for industrial use :

The water requirements for industrial use is constant throughout the year. However with the steady growth of industries and power plants, the requirements of water for these are steadily increasing. Paper, petrochemicals and steel industries are some of the highly water incentive industries. The water use by thermal and nuclear power plants is also very high.

4. Requirements for hydro-electric power :

Water requirement for water-power generation may vary with the seasonal variation in the power demand which in turn may depend upon the type of area served. It should be noted that water power production is not a consumptive use of water. The water discharged through turbines may be picked-up and used for irrigation purposes.

5. Requirements for flood control :

For a reservoir used for flood control, the basic requirement is that sufficient empty space should be available for absorption of flood.

6. Requirements for navigation :

The water required to maintain a desired downstream flow for navigation has marked seasonal variation. Peak releases are required during the later period of dry season.

22.7. PROCESS OF PROJECT FORMULATION

The development process for water resources projects comprises of formulation, construction and operation of a project. It is a dynamic process, comprising of the following steps :

1. Problem identification and statement of objectives :

This includes determination of need for the project on the context of use/control of water, market demand for the product and political incentive. The planner must have a clear statement of objectives of the project, before he starts the planning. The objectives of a project depends upon the availability of funds.

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2. Solution identification :

This consists of data collection, inventory of all resources, analysis of data and developing solutions. The data so collected should be highly *reliable* for the realistic planning of a water resources project. Apart from hydrologic and topographic data of the site, data pertaining to other fields such as geological, economic and social data are required.

3. Projection for planning :

The water resources projects are not planned for present needs only, but they are also planned to cater the future needs. Hence projections of future needs compatible to the population growth should be made. However, unrealistic water use for future needs may lead overdesign and excessive investments.

4. Definifion of alternatives :

This consists of formulation of alternative systems of structural and nonstructural measures in the light of constraints on the resources.

5. Evalution of alternative projects :

This is achieved by performing economic, financial, environmental and social analyses.

6. Comparison of alternative projects :

This is done by displaying all relavent beneficial and adverse effects of each proposal.

7. Selection of a project :

This consists of picking up a project based on the sole objectives of optimum net economic benefit consistent with protecting the nation's environment.

22.8. PROJECT EVALUATION

A number of alternatives may exist, and the cost of each alternative can be worked out in steps of the previous article. The next step is to select the *unit* or *combination of units* which is *economically most efficient*. The financial analysis provides a complete picture of monetary conditions of a project. The *financial analysis* to compare project alternatives may be performed in several ways as follows :

1. Determination of *net benefits*, which is equal to the economic benefits (B) minus economic costs (C).
2. Determination of *benefit cost ratio* (B/C ratio),
3. Determination of rate of return on investment, which is equal to
   (annual benefits – annual costs) / (investment cost)
4. Determination of the internal rate of return, which is the rate of discount applied to annual costs and benefits so that the present worth of all costs equals the present worth of all benefits.

Out of these the *benefit cost ratio* is the most common indicater adopted in economic analysis.

A unit or combination of units may be (*i*) *physically independant unit* (*ii*) *economic independant unit*.

A *physically independant unit* is the one for which there is no other unit, situated either upstream or downstream which would affect the inflow to this unit or outflow from this unit. An *economically independant unit* has no economic interconnection with other units. If there are two sreams in an area, and there are two hydro power plants, one on each stream, the power plants are *physically independant units*. However, they are not economically independant units because the power produced by these two units is integrated into the same power system.

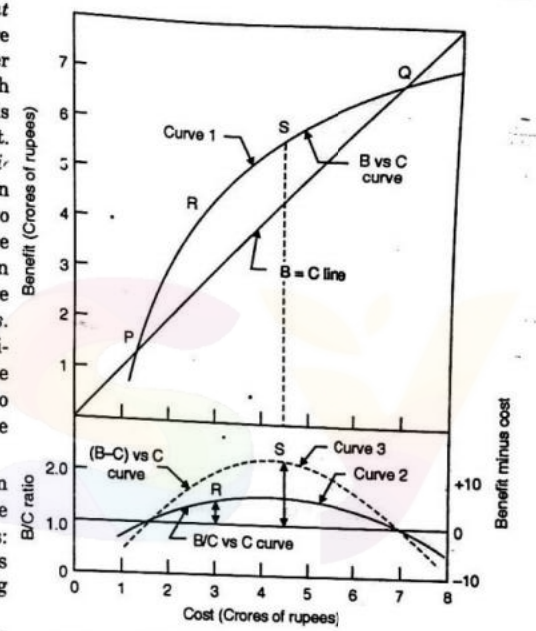

FIG 22.1 FINANCIAL ANALYSIS OF A PROJECT

The *financial analysis* can be graphically presented in the form of three alternative curves:

(*a*) Benefits (B) v/s costs (C) : Curve 1 of Fig 22.1.

(*b*) Benefit cost ratio (B/C ratio) v/s costs (C) : Curve 2 of Fig 22.1.

(*c*) Benefit minus costs ($B - C$) v/s costs (C) : Curve 3 (dotted) of Fig 22.1.

From curve 1 of Fig 22.1, the benefits are more than costs between points P and Q, and hence Benefit Cost Ratio is more than 1 during this range of cost of project. This is also reflected by curve 2. However, the B/C ratio is maximum at point R ; hence this should be the size the project. The curve of ($B - C$) v/s C has maximum *net benefit* at S. Thus, though the maximum B/C ratio is at R, increment from R to S will also be economical since the rate of return on the increment exceeds the minimum altractive rate of return. The decision on the size of the project should therefore be based on a minimum acceptable rate of return as well as on benefit cost ratio.

22.9. STRATEGIES FOR THE FUTURE

The present scenario of water use for various purposes in India and the future needs indicate that the country is fast approaching the stage when all her water resources would have been put to beneficial use and no additional source will be available to meet any further increase in demand. Stratgies should be formulated, discussed and adopted well before that critical stage is reached. The strategies can be divided into two heads :

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(a) Planning strategies
(b) Management strategies

22.10. PLANNING STRATEGIES

Water resources development planning in India has traditionally been attempted *State wise*, although it was well known that *water does not obey political administrative boundaries*. In the initial stages of water resourees development, projects were formulated to serve largely the irrigation requirements or irrigation combined with hydel power generation with some other incidental purposes. As the number of projects were relatively few, inter project considerations were relatively absent and each project were, thus, not viewed as a part of overall river basin system and integrated development and management of river basins to achieve maximum benefit out of the available water resources for various other purposes such as drinking, meeting industrial needs and navigation, was not seen as an immediate necessity. However, water is already scarce in many of the basin / sub-basins and will fast become scarce in several other basins also, since the demand for various purposes is increasing day by day. The process of planning is , therefore, becoming very complex, since several competing and sometimes conflicting demands are to be met. *Unless water resources development is planned in an integrated manner at the river basin level, optional use of the available waters in meeting the various demands cannot be achieved.*

A river basin must be the basic unit for water resources planning. The National Water Policy adopted in 1987 stresses this in clear terms when it declares : 'Resource planning in the case of water has to be done for a hydrological unit such as a drainage basin as a whole, or for a sub basin. All individual developmental projects and proposals should be formulated by the States and considered within the framework of such an overall plan for a basin or sub basin, so that the best possible combination of option can be made.' It is therefore imperative that planners and managers of water resources should start thinking in terms of river basins or sub basins as far as water resources development is concerned.

In India, there are 12 major river basins with drainage area exceeding 20000 sq. km. There are : (*i*) Indus, (ii) Ganga-Brahmputra- Meghna, (*iii*) Brahmani Baitarani, (*iv*) Mahanadi, (*v*) Godavari, (*vi*) Krishna, (*vii*) Pennar, (*viii*) Cauvery, (*ix*) Tapi, (*x*) Narmada (*xi*) Mahi, and (*xii*) Sabarmati. Subernarekha basin having a drainage area of 19296 sq. km is also an important river basin. The remaining river basins can be conveniently clubbed together into seven river systems clusters for the purpose of planning and management. *Therefore in all, India comprises twenty river basin units from the point of water management.*

River basin organisations

Treating basin as the basic unit, planning and preparation of master plans have been emphasized in the *National water Policy*. In the future, water resources planning will become so complex that without a basic master plan, it would be difficult and erroneous to hasten the decisions on proposed projects. Preparation of master plan for a river basin is a multi-disciplinary activity. Since most of the rivers of India are interstate ones, individual states can prepare master plans only for the portion of the

river basin falling within their states. There is, thus a need for setting up of multi-disciplinary units in the states for preparation of master plans and as the national water policy suggests, appropriate inter-state organisations at the basin level for the planned development and management of a river basin as a whole. *Without the establishment of river basin authorities, furture prospects in optimised development of water resources for various beneficial purposes appear bleak.*

Inter basin transfer of water

In order to remedy, atleast to a certain extent, the *regional inbalances* in the availability of water resources in the country, inter-basin transfer of water from surplus basins to deficit basins has been advocated. Transfer are already taking place in some river basins. The Navagam canal of the Sardar Sarovar project is an excellent example of such a mass transfer of water to thirsty water short river basins. Elsewhere also, large scale transfers will become inevitable in the future to meet the growing demands in the deficit basin.

The Government of India has framed National Perspectives of Water Resources Development with two broad components, viz. (*i*) *Himalayan Rivers Development* and (*ii*) *Peninsular Rivers Development* for transfer of waters from surplus basins to deficit basins. Studies have indicated that linking of peninsular rivers could be more easily attempted to meet the water requirement of peninsular regions because of limited lifts involved in such a link. The *Peninsuilar Rivers Development* again has four major parts. One is the inter linking of Mahanadi-Godavari-Krishna-Pennar-Cauvery, the second is the interlinking of west flowing rivers north of Bombay and South of Tapi, the third is the interlinking of Ken with Chambal and the fourth is the diversion of west flowing rivers to the east.

22.11. MANAGEMENT STRATEGIES

1. Basin Management

There is an intense interaction between water and land. In the river basin it will be essential to understand the linkages between land uses and soil profile on one hand and the question of water and distribution over time and space on the other hand. *Land budgeting* will have to be done at the river basin level considering the unique hydrological and soil erosion responses of various types and land uses. But the sectoral planning process being followed cannot achieve scientific land management. *Effective basin wise programmes have to be evolved considering the inter relationships of soil conservation, afforestation, land development, controlled grazing etc.* The strategy for the future should be to have forests in upland, where slopes are steep, which will minimise land slides and lead to lesser sediment transport. In *water short catchments*, selective and planned cutting of trees will decrease the interception and evapotranspiration losses leading to higher yields. Lower reaches of river basins with mild slopes and flat lands are suitable for grass lands. Developmental activities such as mining, setting up of industries, urbanisation, setting up of infrastructural facilities like roads and railway embankments, cannot be looked at in isolation but should be considered as a part of basin management.

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Environmental monitoring of the catchment area should be between as sustained activity in the context of basin management. Further, *soil conservation measures* like contour bunding, planting of vetiver grass, check dams, afforestation measures will have to be taken up in degraded catchments for rejuvenation of rivers.

In *water short basins* careful selection of crops is necessary. Water is required by crops more at their critical stages of growth. Water stress during other periods has negligible impact on yield. Additional waterings do not add proportionately more to the yield. In case of dwarf wheat, the yield level has been found to be 70 % of the optimum of 5 tons/ha ever after reducing the waterings from six to thee. The position is more favourable in case of lentils, gram, linseed, sunflower, mustard or safflower. These crops should therefore, be preferred for irrigation in water short areas. Even under rainfed conditions, millets and pulses possess higher tolerance to rainfall deficiency compared to wheat and rice and should be preferred.

2. Water Conservation

Water conservation measures comprise of the following :

(*i*) Creation of reservoirs for storing the water

(*ii*) Prevention of losses

(*iii*) Improving use efficiency

and (*iv*) Recycling and reuse of waste water.

Apart from these, conservation strategy may also include introduction of rational price structure that would reflect the true value of water.

(i) Creation of storage reservoirs :

Since bulk of the flows in most of the river systems occurs during the 3 to 4 monsoon months, storage dams are inevitable if the seasonal flows are to be conserved and put to beneficial use for irrigation or power generation. The total storage capacity created, under creation and under consideration (upto 1991) put together is only 304 km^3 where as the average annual flow in the rivers is 1880 km^3. *The major constraints for the country may perhaps be the lack of storage sites.* Bulk of the river flows are yet to be stored in Ganga Brahmaputra Meghna basin. In Ganga river system, most of the large reservoir sites lie in Nepal. Brahmputra valley has some exccellent sites which are yet to be developed. Similarly, in peninsular rivers also there is still scope for development of several large storage sites. The country has to go in for creation of storages in future — small as well as large in the basins where surface water still remain untapped. In addtion of this there are a large number of old small tanks. The capacities of these tanks are showing downward trends mainly due to *heavy siltation*. Most of these tanks need modernisation measures such as desilting, strengthening of bund, improving of surplussing arrangement and clearing of the inlet water ways/channels. It would be advantageous if these minor tanks are integrated with the canal systems of major projects. *Tank modernisation programmes should be integrated with watershed treatment programmes.*

(ii) Control of water loss through evaporation :

From the surface of the reserviors, from the open channel irrigation systems and from the irrigated farms, large quantities of water are lost by evaporation. In

the vast arid and semi arid regions of the country the annual loss through evaporation is as high as 3 metres depth of water. Large storages in deep valleys conserve water much better than small storages since large storages expose much less area for evaporation. Irrigation system particularly in water short areas should be so planned that the reservoirs are depleted well before summer sets in. *Irrigation in summer months may have to be essentially from ground water.* Research and development to evolve practical, safe and cost effective methods of evaporation control of reservoirs in the country is essential. The loss of soil moisture in irrigated areas can be controlled by *mulching of soil*, i.e. direct protection of soil by surface cover of various materials such as leaves, straw, sawdust etc.

(iii) Improving water use efficiency :

Improving water use efficiency means getting a higher output for the same quantity of input in the various processes of water use, be it irrigation or industrial manufacturing processes. In irrigation for instance, the efficiency of water use can be increased by going in for improved methods of irrigation such as *drip* and *sprinkler irrigation*. *Irrigation water use effiiciency can be increased by careful planning of conjunctive use of ground water with surface water.* Lining of canals, distributaries and water courses are effective for controlling losses in transmission systems of irrigation projects. The efficiency obtained in the field application of water needs closer examination. The industries may have to be compelled by statuatory regulations to implement inplant control measures including recycling and improved technology for reducing water consumption.

(iv) Recycling and reuse of wastewater :

If it is optimistically assumed that by 2025 AD, 50% of the effluents from the industries can be recovered by suitable treatments for reuse, this itself would amount to about 12 km^3 — enough to support a population of 33 crores or irrigate 1.5 million hectares. Waste water from urban domestic use after proper treatment could also be harnessed for irrigation and industrial use. Domestic waste water also provides additional nutritional value for agricultural crop production if necessary precautions are taken in designing and operating such irrigation systems.

(v) Restructuring of pricing policy :

It has been emphasised time and again in many forums that there is a need for a complete restructuring of the pricing system in water supply, be if for irrigation, domestic supply or industrial supply, it will discourage wasteful use. The National Water Policy recommends that water rates should be such as to convey the scarcity value of the resource to the users and to foster the motivation for economy in water use.

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23

Important Dams of India

23.1. MAJOR RIVER BASINS OF INDIA

In India, there are following 12 major river basins with drainage area exceeding 20000 sq. km :

(i) Indus
(ii) Ganga-Brahmputra-Meghna
(iii) Brahmani-Baitarani
(iv) Mahanadi
(v) Godavari
(vi) Krishna
(vii) Pennar
(viii) Cauvery
(ix) Tapi
(x) Narmada
(xi) Mahi and (xii) Sabarmati

Subernarekha basin having a drainage area of 19296 sq. km. is also an important river basin. The remaining river basins can be conveniently clubbed together into seven river system clusters for purposes of planning and management. Therefore in all, India comprises of basic twenty river basin units from the point of water management.

Table 23.1 shows water availability per capita and per hectare of culturable area in the river basins of India.

23.2. BHAKRA DAM

Bhakra Nangal Project is located near village Bhakra, Bilaspur District, Himachal Pradesh. Bhakra dam has been constructed for Irrigation and Hydro power, across river Satluj. When constructed it was the tallest concrete gravity dam of the World, surpassing the existing height of 726' of Hoover dam. However, presently, the tallest dam of the World is the Grand Dixence Dam (284 m high) in Switzerland.

The dam, reservoir and the power house have the following data :

River ..... Satluj
Area of catchment ..... 56,876 km^2
Annual runoff in the catchment ..... 14,803,430,000 m^3 to 20,971,526,000 m^3
Rainfall ..... 70 cm
Snow ..... 239 cm
Design flood discharge ..... 11,327 m^3/s
Year of commencement of construction work ... 1948
Year of completion 1963

TABLE 23.1. WATER AVAILABILITY PER CAPITA AND PER HECTARE OF CULTURABLE AREA IN THE RIVER BASINS OF INDIA

Sl. No	River Basin	Drainage Area (km^2)	Average annual runoff (Mm^3)	Population (1986) (million)	Culturable area (Th.ha)	Average annual runoff per capita (m^3)	Average Annual runoff per ha of culturable area (m^3)
1	2	3	4	5	6	7	8
1	Indus	321289	73305	36.58	9638	2015	7600
2.	Ganga-Brahamputra-Meghna						
	(a) Ganga	861404	525023	301.37	60161	1742	8727
	(b) Brahamputra	194413	537240	25.51	12146	21060	44232
	(c) Barak & others	78150	59800	8.00	1114	7475	53680
3.	Subernarekha	29196	10794	7.94	1898	1359	5687
4.	Brahamani-Baitarani	51822	36227	10.35	3201	3500	11317
5.	Mahanadi	141589	66879	24.45	7994	2735	8366
6.	Godavri	312812	118982	47.34	18931	2513	6285
7.	Krishna	258948	67790	47.09	20299	1440	3339
8.	Pennar	55213	6858	8.91	3551	770	1931
9.	East flowing rivers between Mahanadi and Pennar	74354	16948	13.41	4328	1264	3916
10.	Cauvery	87900	21358	27.46	5797	778	3784
11.	East flowing rivers between Pennar and Kanyakumari	100139	17725	36.39	6876	487	2678
12.	West flowing rivers from Tadri to Kanyakumari	112117	198854	53.70	6279	3703	3170
13.	West flowing rivers from Tapi to Tadri						
14.	Tapi	65145	18389	11.75	4536	1565	4054
15.	Narmada	98796	41273	14.92	5901	2766	6994
16.	Mahi	34842	11829	7.13	2210	1659	5352
17.	Sabarmati	21674	4079	6.60	1548	618	2635
18.	West flowing rivers of Kutch and Saurashtra	321851	5098	30.84	23447	165	217
19.	Area of inland drainage in Rajasthan desert	-	-	-	-	-	-
20.	Minor rivers draining to Bangladesh and Burma	-	31000	-	-	-	-

Dam

Type ..... Staight concrete gravity dam
Bed rock ..... Sandstone interspersed with claystone

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Maximim height above the lowest point of foundation	226 m
Length at the top of the dam	518 m
Total volume content	4,130,000 m^3
Spillway	
Type	Centrally located overflow spillway
Type and number of gates	4 Nos. Radial gates of size (15.24 m × 14.47 m)
Maximum discharge capacity	8,372 m^3/s
Reservoir	
Area at F.R.L.	166 km^2
Gross storage capacity	9,621,000,000 m^3
Effective storage capacity	7,191,000,000 m^3
Availabe drawdown	69.5 m
Power Plant	
Type	Vertical radial-axial Francis turbine
Hydraulic head	Maximum – 158 m Minimum – 80 m
Maximum discharge	102 m^3/s
Installed capacity	1204 MW
Generating units	
Type	Vertical radial axial Francis
Number	16 Nos. (5 + 5 + 3 + 3)
Capacity	1204 MW (600 + 450 + 77 + 77)
Firm	407 MW
Secondary	148 MW

Plate 1 shows the photographic view of Bhakra dam.

Fig. 23.1 shows the maximum spillway section.

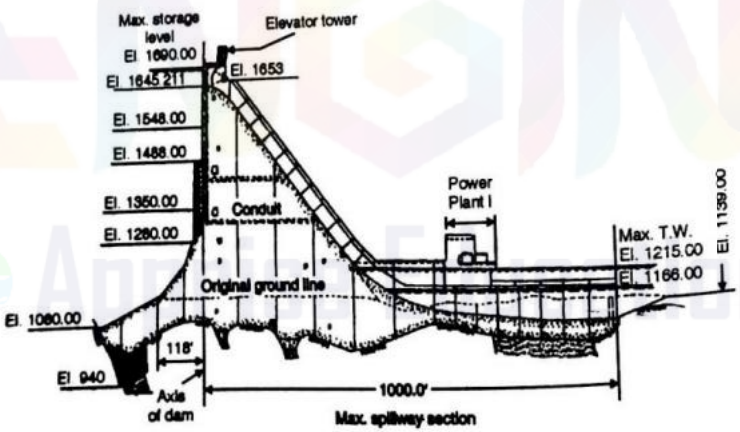

FIG. 23.1 MAXIMUM SPILLWAY SECTION OF BHAKRA DAM.

23.3. KOYNA DAM

Koyna dam is situated at Deshmukhwadi, Koyna, is Satara District of Maharashtra. It is constructed on Koyna river, and is for hydro-power generation. The salient data about the project are as follows :

Area of catchment	891 km^2
Rainfall	389.4 cm
Design flood discharge	7,854 m^3/s
Year of commencement of construction work	1956
Year of completion	1961
Dam	
Type	Gravity
Bed rock	Deccan trap
Maximum height above the lowest point of foundation	103 m
Length at the top of the dam	808 m
Total volume content	1,555,000 m^3
Spillway	
Type	Controlled Spillway
Type and number of gates	Tainter gates (12.5 m × 7.62 m)
Maximum discharge capacity	3,823 m^3/s
Reservoir	
Area at F.R.L.	115,35 km^2
Gross storage capacity	2,796,500,000 m^3
Effective storage capacity	2,662,000,000 m^3
Power Plant	
Type	Underground Power House
Hydraulic head	Stage I–475 m. State II–430 m
Installed capacity	540 MW

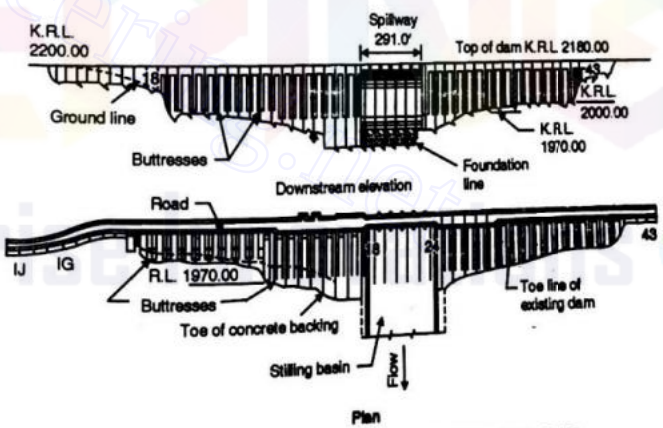

FIG. 23.2. PLAN AND D/S ELEVATION OF KOYNA DAM

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Generating units
- Type ..... Pelton
- Number ..... 8 Nos.
- Capacity ..... 4 Nos. - 75 MW
  ..... 4 Nos - 60 MW

Plate 2 shows the photgraphic view of Koyna dam. Fig. 23.2 shows the plan and down stream elevation of the dam. Fig. 23.3 shows the non-overflow section and overflow section of the dam.

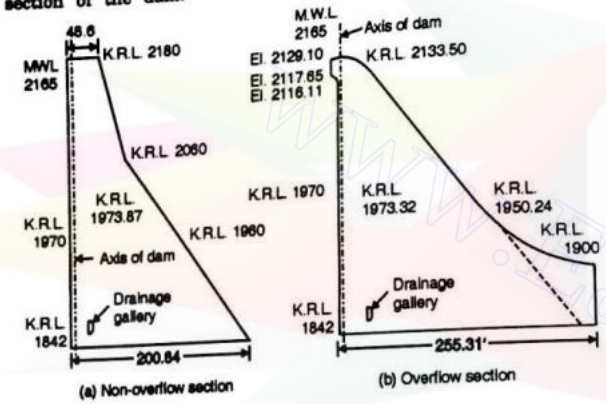

FIG. 23.3 KOYNA DAM.

23.4. NAGARJUN SAGAR DAM

Nagarjun Sagar Dam is located near Nandikonda village Dist. Nalgonda, Andhra Pradesh. It is a combined irrigation and hydro power project, constructed across Krishna river. The salient data of the project are given below.

- Area of catchment ..... 215,192 km^2
- Mean annual runoff in the catchment ..... 42,360,000,000 m^3
- Rainfall ..... 89 cm
- Design flood discharge ..... 53,450 m^3/s
- Year of commencement of construction work 1957
- Year of completion 1974

Dam
- Type ..... Earth-cum-gravity
- Bed rock ..... Archaean granite-gneiss
- Maximum height above the lowest point of foundation ..... 125 m
- Length at the top of the dam
  - (i) Non overflow
    - (a) Earth dam ..... 3,415 m
    - (b) Masonry dam ..... 978 m
  - (ii) Spillway ..... 472 m
- Total volume content
  - (i) Concrete and masonry ..... 5,610,000 m^3
  - (ii) Earth ..... 2,350,000 m^3

Spillway
- Type ..... Ogee Spillway
- Type and number of gates ..... Radial gates–13.72 m×13.41 m–26 Nos.
- Maximum discharge capacity ..... 53,450 m^3/s

Reservoir
- Area at F.R.L. ..... 285 km^2
- Gross storage capacity ..... 11,550,000,000 m^3
- Effective storage capacity ..... 6,940,000,000 m^3
- Available drawdown ..... 24.38 m

Power Plant

Type	Conventional	Pumped Storage
	Francis vertical	Francis vertical
Hydraulic head	95 m	95 m
Installed capacity	110 MW	4×110 MW=440 MW

Generating units

Number	1	4
Capacity	110 MW	440 MW

Plate 3 shows Nagarjun Sagar dam. Fig. 23.4 (a) shows the non-overflow section of gravity dam, while Fig. 23.4 (b) shows the overflow section. Fig. 23.5 (a) shows left earth dam section while Fig. 23.5 (b) shows right earth dam section.

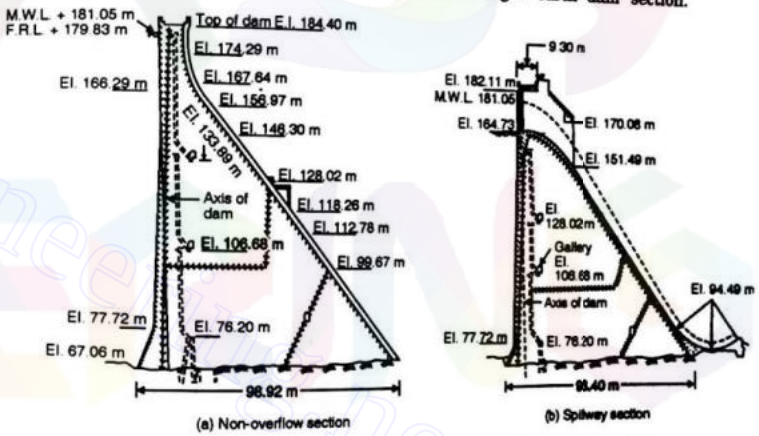

FIG. 23.4. NAGARJUN SAGAR DAM : GRAVITY DAM SECTIONS.

23.5. GANDHI SAGAR DAM

Gandhi Sagar dam is situated near Chaurasigarh Fort, Dist. Mandsaur, Madhya Pradesh. It is an irrigation and hydropower project, constructed across river Chambal. The salient data about the project are as follows.

- Area of catchment ..... 23,140 km^3
- Rainfall ..... 86 cm
- Design flood discharge ..... 21,240 m^3/s
- Year of commencement of construction work 1954

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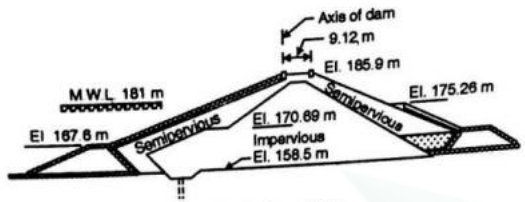
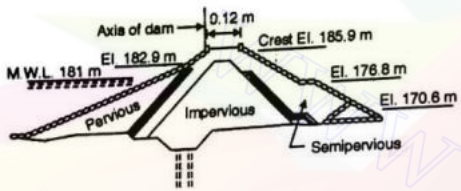


(a) Left earth dam section



(b) Right earth dam section

FIG. 23.5. NAGARJUN SAGAR DAM : EARTH DAM SECTIONS

Year of completion	1960
Dam	
Type	Masonry dam (Straight gravity stone masonry)
Maximum height above the lowest point of foundation	64 m
Length at the top of the dam	
(*i*) Non overflow	260 m
(*ii*) spillway	254 m
Total volume content	
(*i*) concrete	169,920 m^3
(*ii*) Masonry	560,000 m^3
Spillway	
Type	Gated
Type and number of gates	Vertical gates–10 Nos
Maximum discharge capacity	21,240 m^3/s
Reservoir	
Area at F.R.L.	723 km^3
Gross storage capacity	7,746,000,000
Effective storage capacity	6,911,000,000 m^3
Power Plant	
Hydraulic head	45 m
Installed capacity	115 MW
Generating units	
Number	5
Capacity	23 MW each
Firm	80 MW at 60% load factor

Plate 4 shows the view of Gandhi Sagar dam. Fig. 23.6 shows the plan and downstream elevation of the dam. Fig. 23.7 shows the section of spillway and non-spillway portions.

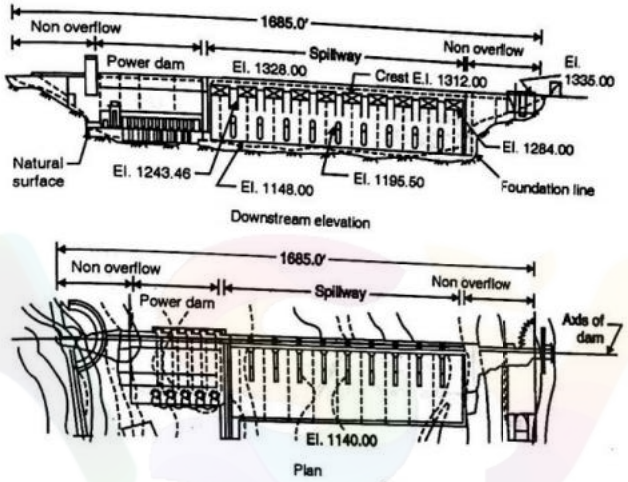


FIG. 23.6 GANDHI SAGAR DAM.

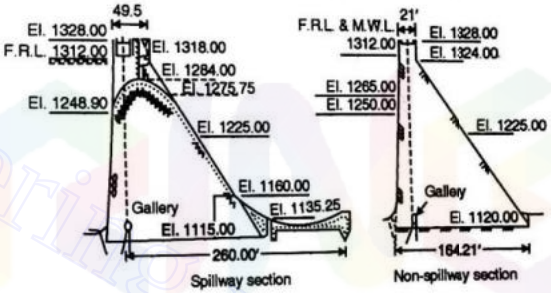


FIG. 23.7. GANDHI SAGAR DAM.

23.6. RIHAND DAM

Rihand dam has been constructed across Rihand river, at Pipri, Mirzapur district, Uttar Pradesh. It is a hydro power project. The salient data of the project are given below :

Area of catchment	13,333 km^2
Rainfall	142 cm
Design flood discharge	17,275 m^3/s
Year of commencement of construction work	1952
Year of completion	1962

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Dam

Type ..... Straight concrete gtavity

Maximum height above the lowest point of foundation ..... 91 m

Length at the top of the dam .....

(i) Non overflow ..... 744 m

(ii) Spillway ..... 190 m

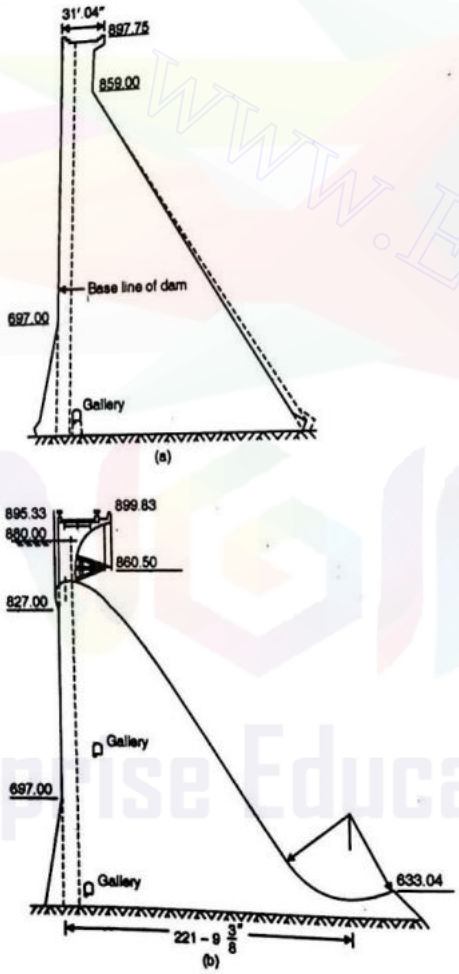

FIG. 23.8. RIHAND DAM.

Total volume content ..... 1,680,000 m^3

Spillway

Type ..... Ogee type

Type and number of gates ..... Tainter gates 13 Nos.

Maximum discharge capacity ..... 13,339 m^3/s

Reservoir

Area at F.R.L. ..... 469 dm^2

Gross storage capacity ..... 10,600,000,000 m^3

Effective storage capacity ..... 8,900,000,000 m^3

Available drawdown ..... 32 m

Power Plant

Hydraulic head ..... 44 m to 76 m

Maximum discharge ..... 510 m^3/s

Installed capacity ..... 300 MW

Generating units

Types ..... Umbrella type

Number ..... 60 Nos

Capacity ..... 50 MW each

Fig. 23.8 (*a*) shows non-overflow section while Fig. 23.8(*b*) shows the overflow section of Rihand dam. Plate 5 gives the view of the project.

23.7. LOWER BHAVANI DAM

Lower Bhavani Dam is located near Tettupallayam, Tamil Nadu. It is an irrigation dam constructed across river Bhavani which is a tributory of Cauvery. The salient data of the dam are as follows:

Area of catchment ..... 4200 km^2

Rainfall ..... 152.4 cm

Design flood dischange ..... 4040 m^3/s

Year of commencemnet of construction work 1948

Year of completion 1955

Dam

Type ..... Masonry cum earth dam

Bed rock ..... Biotite gneiss

Max. height above the lowest point of foundation ..... 62 m

Length at the top of the dam. ..... 8,677 m

(i) Non overflow ..... 8,677 m

(ii) Spillway ..... 121 m

Total volume content

(i) Concrete and masonry ..... 400,000 m^3

(ii) Earth ..... 4,560,000 m^3

Spillway

Type ..... Ogee type

Type and number of gates ..... Lift gates - 9 Nos (10.97 m ×6.10 m)

Maximum discharge capacity ..... 3,141 m^3/s

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Reservoir

Area at F.R.L. ..... 78,76 km²

Gross storage capacity ..... 928,240,000 m³

Effective storage capacity ..... 928,240,000 m³

Plate 5 gives the photograhic view of the dam. Fig. 23.9 shows the typical section of earth dam portion. Fig. 23.10 shows the maximum section of spillway.

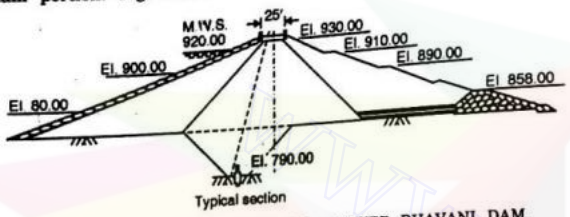

FIG. 23.9. EARTH DAM SECTION : LOWER BHAVANI DAM.

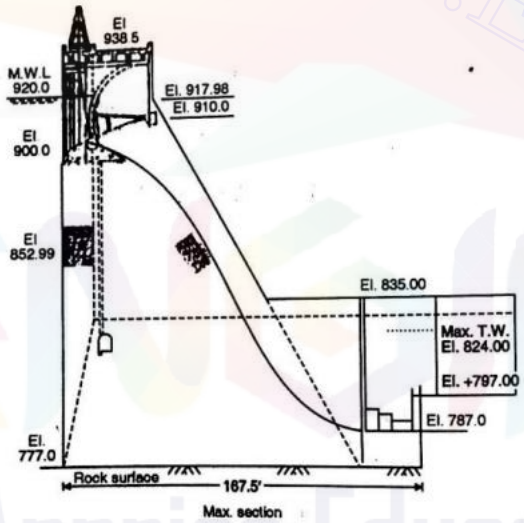

FIG. 23.10. MAX. SPILLWAY SECTION : LOWER BHAWANI DAM.

23.8. HIRAKUD DAM

Hirakud dam has been constructed at Hirakud, Sambalpur district, Orissa. It is a multipurpose project, meant for irrigation, hydropower and flood control, constructed across river Mahanadi. The salient data of the project are given below :

Area of catchment ..... 82,880 km²

Mean annual runoff in the catchment ..... 37,000,000,000 m³

Rainfall ..... 137 cm

Design flood discharge ..... 35,765 m³/s

Year of commencement of Construction work 1948

Year of completion 1957

Dam

Type ..... Composite (earth, masonry & concrete)

Bed rock ..... Granite rocks with banks of schistose rocks mainly quartz silicate & medium grained granite gneisses, mylomites slate & shales

Maximum height above the lowest point of foundation ..... 59 m

Length at the top of the dam

(*i*) Non overflow ..... 3,652 m

(*ii*) Spillway ..... 1,148 m

(*iii*) Earthen dykes ..... 20,660 m

Total volume content

(*i*) Concrete and masonry ..... 1,400,000 m³

(*ii*) Earth ..... 23,700,000 m³

Spillway

Type ..... Ogee (overflow) gated spillway

Type and number of gates ..... 34 crest bays with radial crest gates and 64 undersluices with vertical gates

Maximum discharge capacity ..... 4595 m³/s in each crest bay and 465.25 m³/s in each undersluice

Reservoir

Area at F.R.L. ..... 727 km²

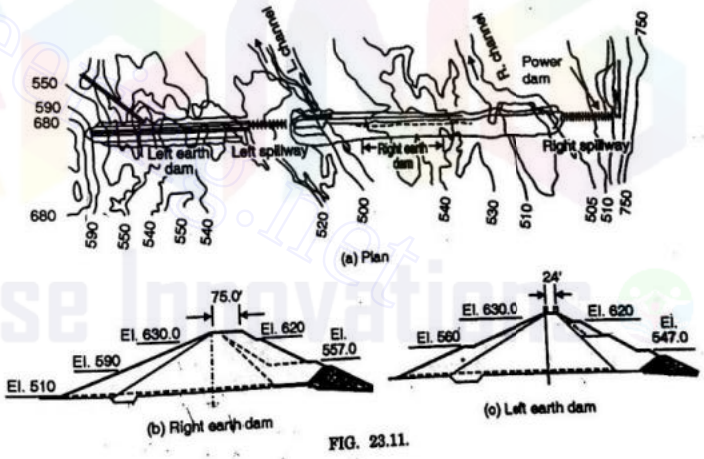

FIG. 23.11.

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Gross storage capacity ..... 8,105,000,000 m^3
Effective storage capacity ..... 5,843,000000 m
Available drawdown ..... 12.19 m

Power Plant

Hydraulic head ..... Average head 25.6 m
Maximum discharge ..... 600 m^3/s
Installed capacity ..... 270 MW

Generating units

Type ..... vertical alternator
Number ..... 9 Nos (7 Kaplan & 2 Francis)
Capacity ..... 37.5 MW-4 Nos. & 24 MW - 5 Nos.
Firm ..... 120 MW

Plate 7 shows the view of Hirakud dam. Fig. 23.11 (*a*) gives the plan of the dam showing the position of the two spillways, power dam, and the two earth dams. Fig. 23.11 (*b*) shows the section of the right earth dam while Fig. 23.11 (*c*) show the section of left earth dam.

23.9. KRISHNARAJA SAGAR DAM

Krishnaraja Sagar dam is located at Krishnaraja Sagar, Karnataka. Constructed across main Cauvery (tributaries Hemavathy and Lakshamanathirtha), it is meant for Irrigation and hydropower, apart from recreation because of location of famous Brindawan Gardens at its down stream side. The salient data about the dam are as follows :

Area of catchment ..... 10.619 km^2
Rainfall ..... 635 cm
Design flood discharge ..... 8,213 m^3/s
Year of commencement of construction work 1911
Year of completion 1932

Dam

Type ..... Masonry

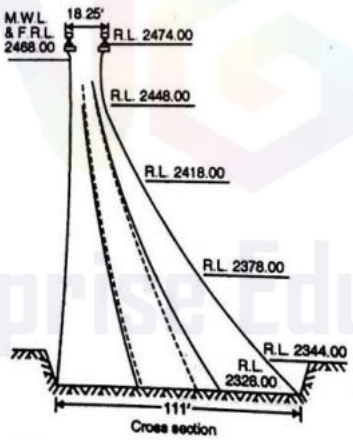

FIG. 23.12. KRISHNARAJA SAGAR DAM.

Bed rock ..... Hard rock
Maximum height above the lowest point of foundation ..... 43 m
Length at the top of the dam ..... 2,621 m
Total volume content ..... 849,600 m

Spillway

Type and number of gates ..... 152 lift gates
Maximum discharge capacity ..... 9,911 m^3/s

Reservoir

Area at F.R.L. ..... 129 km^2
Gross Storage capacity ..... 1,368,847 m^3
Effective storage capacity ..... 1,244,210,800 m^3

Plate 8 shows the view of Krishnaraja Sagar dam. Fig. 23.12 shows the section of Krishnaraja Sagar dam.

23.10. UKAI DAM

Ukai dam, constructed at Fort Songardh, Gujarat, across river Tapi is a multpurpose dam, meant for hydro-power , irrigation and flood control. The salient data about the dam are as under.

Area of catchment ..... 62,225 km^2
Mean annual runoff in the catchment ..... 17,220,000,000 m^3
Rainfall ..... 79 cm
Design flood discharge ..... 49,490 m^3/s
Year of commencement of construction work 1964
Year of completion 1972

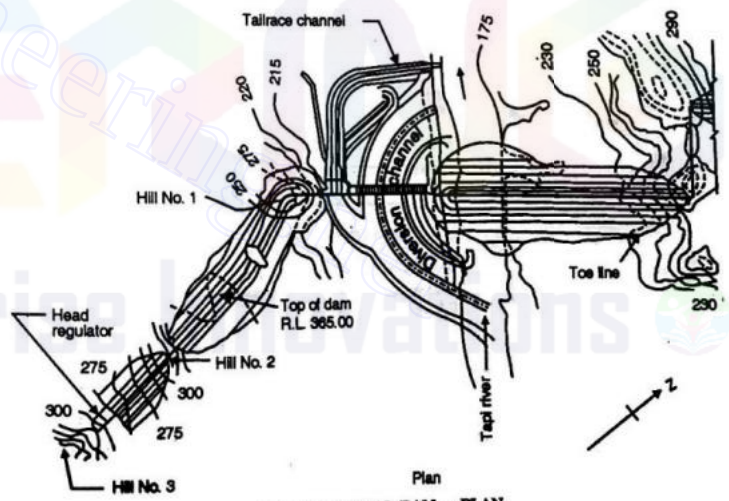

FIG. 23.13 UKAI DAM : PLAN.

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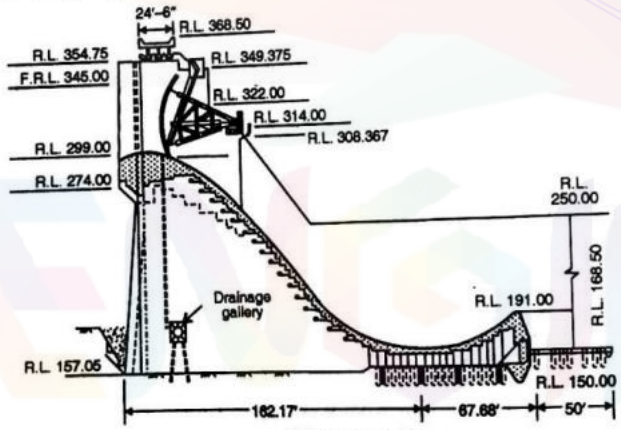
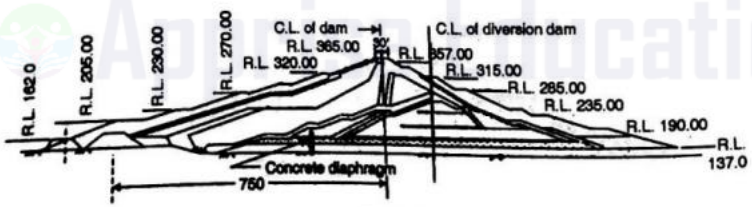
Dam
- Type ..... Earth-cum-gravity
- Bed rock ..... Basalt
- Maximum height above the lowest point of foundation ..... 81 m
- Length at the top of dam
  - (i) Non overflow ..... 4640 m
  - (ii) Spillway ..... 425 m
- Total volume content
  - (i) Concrete ..... 1,510,000 m^3
  - (ii) Earth ..... 23,670,000 m^3

Spillway
- Type ..... Gated Ogee with ski-jump
- Type and number of gates ..... Radial crest gates–22 Nos.
- Maximum discharge cappacity ..... 35,960 m^3/s

Reservoir
- Area at F.R.L. ..... 601 km^2
- Gross storage capacity ..... 8,511,000,000 m^3



Spillway cross section



Cross section

23.14. UKAI DAM

- Effective storage capacity ..... 7,092,000,000 m^3

Power Plant
- Type ..... Underground, River bed power house
- Hydraulic head ..... 57 m to 34 m
- Maximum discharge ..... 213 m^3/s
- Installed capacity ..... 300 MW

Generating units
- Type ..... Generators–Umbrella type, vertical shaft, salient pole Turbines-Vertical shaft Kaplan turbine
- Number ..... 4 Nos
- Capacity ..... 75 MW each
- Firm ..... 193 MW at 35% load factor continuously

Fig. 23.13 shows the layout plan of the dam, showing various components.

Fig. 23.14 shows the spillway cross-section and earthdam cross section.

23.11. THEIN DAM

Thein Dam is located across river Ravi, Gurdaspur district, Punjab. It is a multipurpose project, meant for irrigation, hydropower and flood control. It is an earth core rock fill dam, with salient data as follows :

- Area of catchment ..... 6087 km^2
- Mean annual runoff in the catchment ..... 8,142,000,000 m^3
- Design flood discharge ..... 21,238 m^3/s
- Year of commencement of Construction work 1978
- Year of completion ..... Under construction

Dam
- Type ..... Earth core rockill dam
- Bed rock ..... Alternate bands of sandstone and clay/silt stone
- Maximum height above the lowest point of foundation ..... 147 m
- Length at the top of the dam
  - (i) Non overflow ..... 661 m
  - (ii) Spillway ..... 217 m
- Total volume content
  - (i) Concrete and masonry ..... 1,070,000 m^3
  - (ii) Earth ..... 20,850,000 m^3

Spillway
- Type ..... Controlled gated ogee
- Type and number of gates ..... 12 Nos radial gates (15.25 m × 10.67 m)
- Maximum discharge capacity ..... 19,329 m^3/s

Reservoir
- Area at F.R.L. ..... 83 km^3
- Gross storage capacity ..... 3,670,000,000 m^3
- Effective storage capacity ..... 2,340,000,000 m^3
- Available drawdown ..... 40.23 m

Power Plant
- Hydraulic head ..... 121 m

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Maximum discharge ..... 500 m^3/s
Installed capacity ..... 480 MW

Generating units

Type ..... Vertical axis Francis reaction turbine
Number ..... 4 Nos.
Capacity ..... 120 MW each
Firm ..... 133 MW at 100% load factor or 222 MW
..... at 60% load factor

Fig. 23.15 shows the layout of various components of the project. Fig. 23.16 shows the typical earth dam section at maximum height.

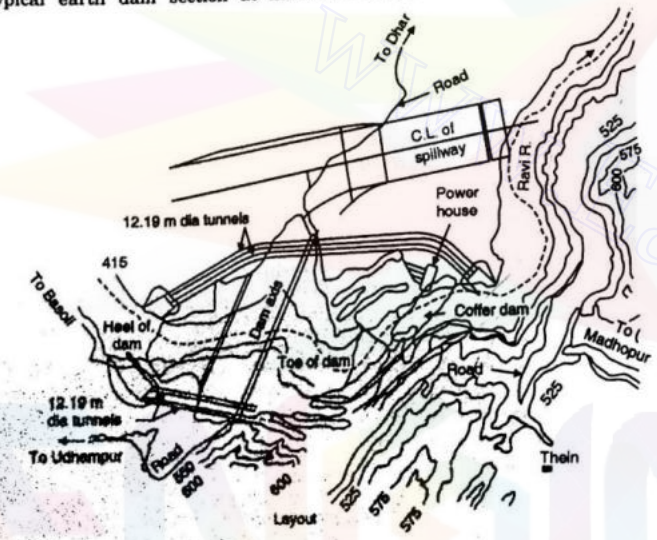

FIG. 23.15. THEIN DAM : LAYOUT

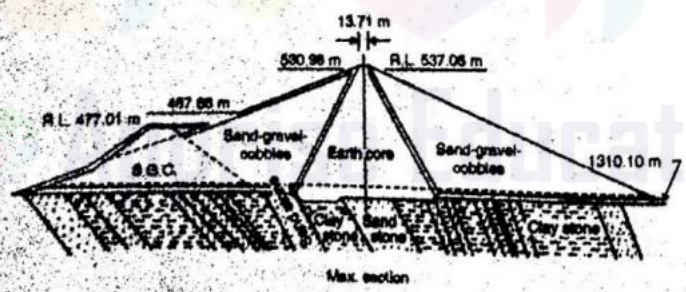

FIG. 23.16. THEIN DAM : EARTH DAM SECTION

23.12. TEHRI DAM

Tehri dam is located at new Tehri town, Tehri district, Uttar Pradesh. Constructed acros river Bhagirathi, it is a combined irrigation and hydropower project. The salient data of the project are as follows :

Area of catchment ..... 7,511 km^2 out of which 2,323 km^2 is snow bound
Rainfall ..... Min. 102 cm Max. 178 cm
Design flood discharge ..... 15,300 m^3/s

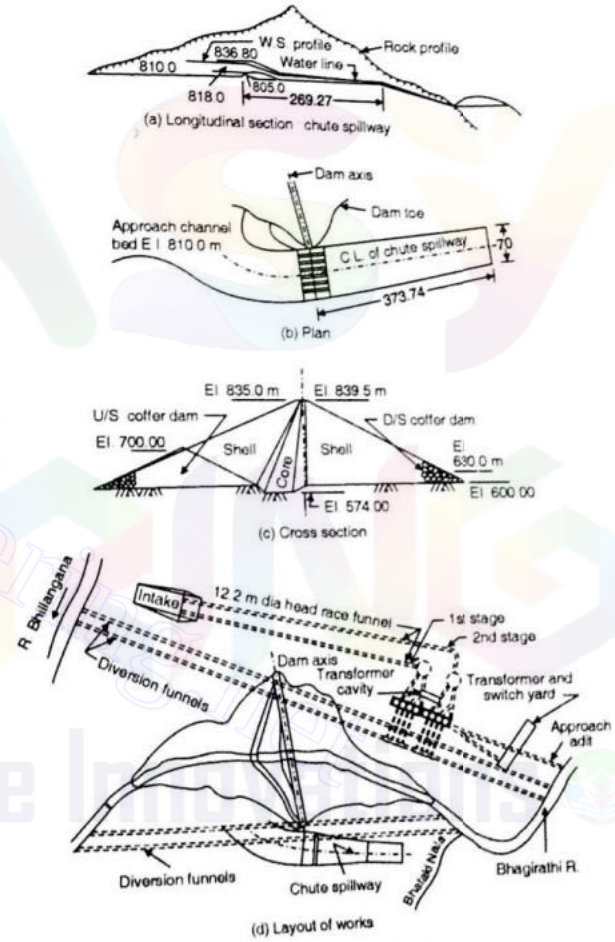

FIG. 23.17. TEHRI DAM.

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Maximum discharge	500 m^3/s
Installed capacity	480 MW
Generating units	
Type	Vertical axis Francis reaction turbine
Number	4 Nos.
Capacity	120 MW each
Firm	133 MW at 100% load factor or 222 MW at 60% load factor

Fig. 23.15 shows the layout of various components of the project. Fig. 23.16 shows the typical earth dam section at maximum height.

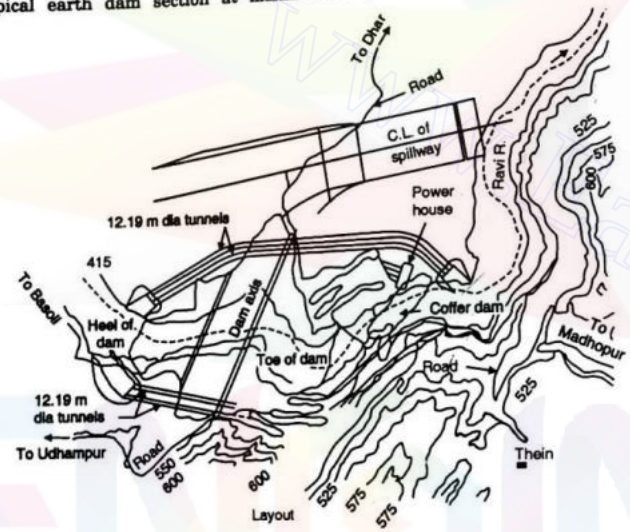

FIG. 23.15. THEIN DAM : LAYOUT

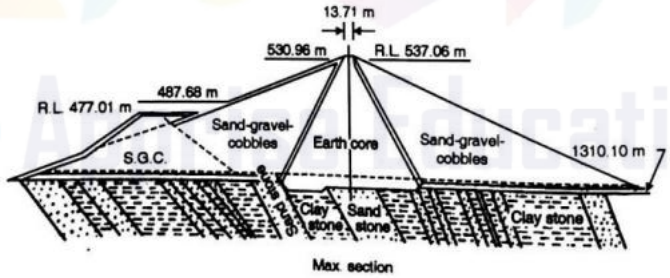

FIG. 23.16. THEIN DAM : EARTH DAM SECTION

23.12. TEHRI DAM

Tehri dam is located at new Tehri town, Tehri district, Uttar Pradesh. Constructed acros river Bhagirathi, it is a combined irrigation and hydropower project. The salient data of the project are as follows :

Area of catchment	7,511 km^2 out of which 2,323 km^2 is snow bound
Rainfall	Min. 102 cm Max. 178 cm
Design flood discharge	15,300 m^3/s

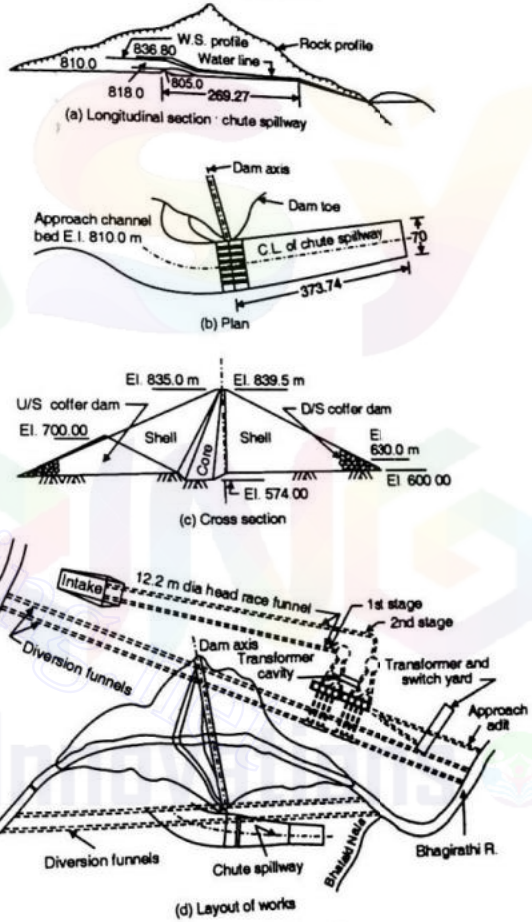

FIG. 23.17. TEHRI DAM.

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Year of commencement. of construction work .. 1978
Year of completion ..... Under Construction

Dam

Type ..... Earth and rockfill
Bed rock..... Slightly jointed phyllites of different grades
Maximum height above the lowest point of foundation..... 261 m
Length at the top of the dam..... 570 m
Total volume content
(*i*) Concrete and masonry ..... 550,000 m^3
(*ii*) Earth ..... 22,000,000 m^3

Spillway

Type ..... Open channel chute spillway
Type and number of gates..... 7 Nos. tainter gates
Maximum discharge capacity ..... 13,150 m^3/s

Reservoir

Area at F.R.L..... 42 km^2
Gross storage capacity..... 3,539,000,000 m^3
Effective storage capacity..... 2,615,000,000 m^3
Available drawdown..... 90 m

Power Plant

Type ..... Underground power house on left bank
Hydraulic head ..... 218 m
Maximum discharge..... 550 m^3/s
Installed capacity ..... 600 MW

Generating units

Type ..... Synchronous vertical umbrells type
Number..... 4 Nos.
Capacity..... 150 MW each

Fig 23.17 (*a*) shows the layout of the works. Fig. 23.17 (*b*) shows the cross-section of earth dam. Fig. 23.17 (*c*) shows the plan and logitudinal section of chute spillway.

23.13. RAMGANGA DAM

Ramganga dam, constructed across river Ramganga, at Kalagarh (Bijnor), Uttar Pradesh, is a combined irrigation and hydro-power project. It is an earth and rockfill dam, with a chute spillway. It has a surface power house designed for submerged condition, utilising a hydraulic head of 98 m. The salient data about the project are as follows :

Area of catchment..... 3,134 km^2
Rainfall ..... 155 cm
Design flood discharge..... 12,121 m^3/s
Year of commencement of construction work ... 1964
Year of completion 1978

Dam

Type ..... Earth and rockfill dam
Bed rock..... Alternate clay and sand rock bands with bands of silt stone

	Main dam	Saddle dam
Maximum height above the lowest point of foundation	128 m	71 m
Length at the top of the dam		
(*i*) Non overflow	630 m	562 m
(*ii*) Spillway	113 m	
Total volume content		
(*i*) Concrete and masonry	195,000 m^3	
(*ii*) Earth	10,818,000 m^3	
Spillway		
Type	Chute spillway	Auxiliary spillway
Type and number of gates	5 Nos tainter gates	3 Nos tainter gates
Maximum discharge capacity	8,467 m^3/s	1,926 m^3/s
Reservoir		
Area at F.R.L.	78 km^2	
Gross storage capacity	2,442,600,000 m^3	
Effective storage capacity	2,195,500,000 m^3	
Power Plant		
Type	Surface Power House, designed for submerged condition	
Hydraulic head	98 m	
Maximum discahrge	226 m^3/s	
Installed capacity	198 MW	
Generating units		
Type	Francis turbine	
Number	3 Nos	
Capacity	66 MW each	

Fig 23.18 shows a typical section of the earth and rockfill dam having a max. height of 128 m above the lowest point of foundation.

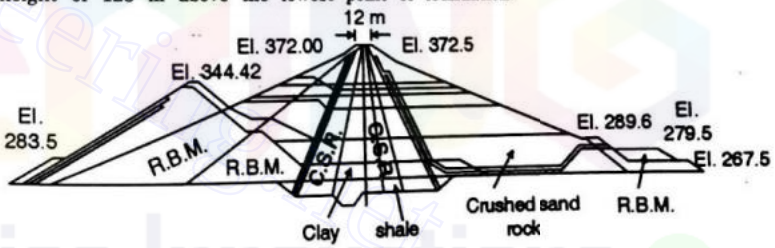

FIG. 23.18 RAMGANGA DAM

23.14. MAHI BAJAJ SAGAR DAM

Mahi Bajaj Sager dam is located in Banswara district, Rajasthan, constructed across Mahi river. It is a combined irrigation and hydro-power project, having dam sections of masonry and earth. The salient feature of the project are given below.

Area of catchment..... 6,149 km^2
Rainfall ..... 82 cm

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Design flood discharge ..... 25,595 m^3/s
Year of commencement of construction work ... 1971

Dam

Type ..... Masonry and earth
Bed rock ..... Alluvium consisting of sand gravel/cobble deposit
Maximum height above the lowest point of foundation ..... 68 m

Length at the top of the dam

(i) Non-overflow ..... 2662 m
(ii) Spillway ..... 300 m

Total volume content

(i) Concrete and masonry ..... 781,000 m^3
(ii) Earth ..... 5,324,000 m^3

Spillway

Type ..... W.E.S. Type
Type and number of gates ..... 16 radial gates (15.0 m × 12.25 m)
Maximum discharge capacity ..... 23,785 m^3/s

Reservoir

Area at F.R.L. ..... 133 km^2
Gross storage capacity ..... 2,059,000,000 m^3
Effective storage capacity ..... 1,715,000,000 m^3

Power Plant

Type ..... Two indoor type surface power houses
Hydraulic head ..... At Power House 1–40 m.
At Power House 2–84 m
Maximum discharge ..... 165 m^3/s
Installed capacity ..... 140 MW

Generating units

Type ..... Francis turbine
Number and capacity ..... Power House No 1–2 units of 25 MW each
Power House No 2–2 units of 45 MW each
Firm ..... 29 MW

Fig. 23.19 shows the typical section of earthdam

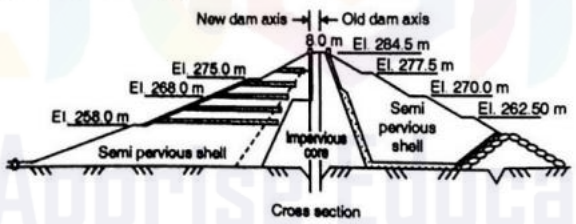

FIG. 23.19 MAHI BAJAJ SAGAR EARTH DAM

23.15. UJJANI DAM

This dam is located at Ujjani Village, Village, Sholapur district, Maharashtra. Constructed across river Bhima, it is a combined irrigation and hydro-power project. The dam is earth-cum-gravity. The salient data of the project are given below.

Area of catchment ..... 14,850 km^2
Mean annual runoff in the catchment ..... 8,712,000,000 m^3
Rainfall ..... 69 cm
Design flood discharge ..... 18,013 m^3/s
Year of commencement of construction work ... 1979

Dam

Type ..... Earth- cum-gravity
Bed rock ..... Masssive basalt
Point of foundation ..... 56 m
Length at the top of the dam
(i) Non overflow ..... 1,864 m
(ii) Spillway ..... 603 m

Total volume content
(i) Concrete and masonry ..... 680,000 m^3
(ii) Earth ..... 1,116,000 m^3

Spillway

Type ..... Controlled
Type and number of gates ..... Radial gates - 41 Nos. of size 18 m × 6.5 m
Maximum discharge capacity ..... 18,013 m^3/s

Reservoir

Area at F.R.L. ..... 337 km^2

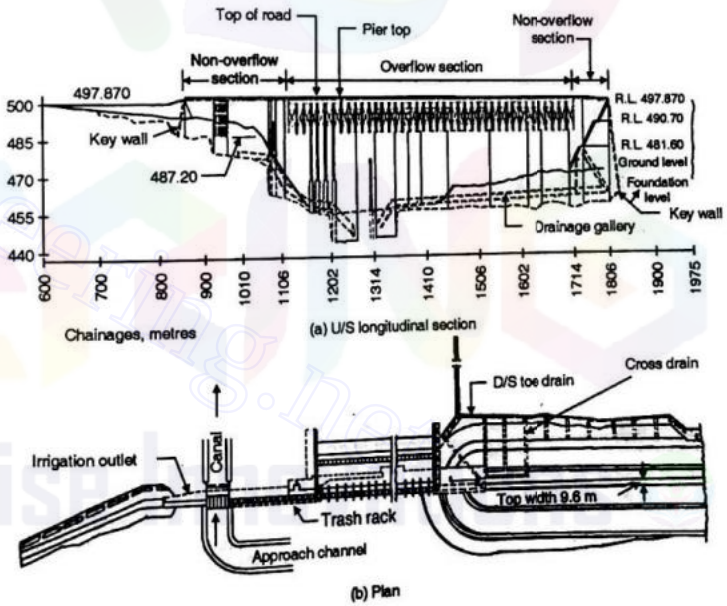

FIG. 23.20. UJJANI DAM.

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Gross storage capacity ..... 3,140,000,000 m^3
Effective storage capacity ..... 1,440,000,000 m^3
Power Plant ..... There is provision of one penstock of 3.2 m dia. in the dam. It is meant for generating 12 MW power seasonally

Fig. 23.20 shows the plan and longitndinal section of the dam. Fig. 23.21 shows typical section of earth dam while Fig 23.22 show maximum over flow section.

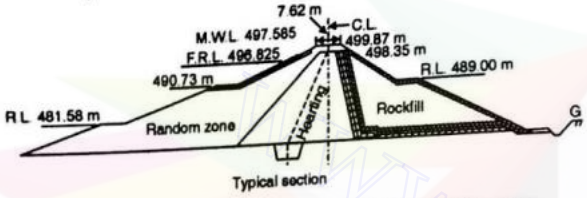

FIG. 23.21. TYPICAL EARTH DAM SECTION: UJJANI DAM.

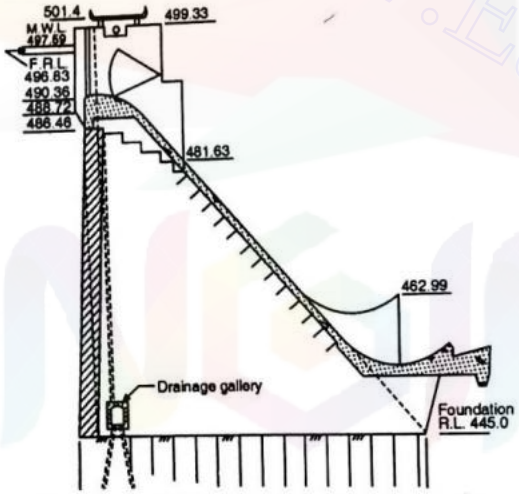

FIG. 23.22. MAX. OVERFLOW SECTION UJJANI DAM.

23.16. TUNGBHADRA DAM.

Tungbhadra dam is located near Hospet town Karnataka and has been constructed across river Tungbhadra. It is a combined irrigation and hydropower project. It has masonry dam with central spillway section, and earth dam and composite dam on the left side. The salient data of the project are as under.

Area of catchment ..... 27,853 km^2
Rainfall ..... 104 cm

Design flood discharge ..... 18,408 m^3/s
Year of commencement of construction work 1945
Year of completion 1958

Dam

Type ..... Masonry dam with central spillway section Earthem dam and composite dam on the left side
Bed rock ..... Hard rock
Maximum height above the lowest point of foundation ..... 49 m
Length at the top of the dam
(*i*) Non overflow ..... 1,748 m
(*ii*) Spillway ..... 701 m
Total volume content ..... 1,155,000 m^3,

Spillway

Type ..... Ogee type spillway
Type and number of gates ..... 33 Nos.
Maximum discharge capacity ..... 18,408 m^3/s

Reservoir

Area at F.R.L. ..... 378 km^2
Gross storage capacity ..... 3,767,000,000 m^3
Effective storage capacity ..... 3,701,000,000 m^3
Available drawdown ..... 25 m

Power plant

Generating Units
Number ..... 14 units of 9 MW each
Capacity ..... 126 MW

Fig 23.23 shows the non-spillway section and spillway section of the Tungbhadra dam.

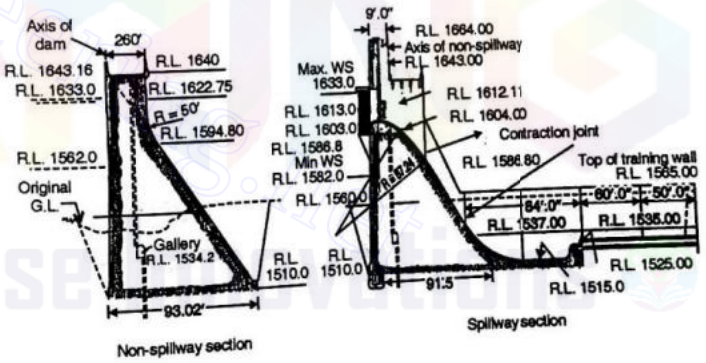

FIG 23.23. TUNGBHADRA DAM

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23.17. DANTIWADA DAM

This dam is located at Dantiwada village, Banaskatha District, Gujrat. It is mainly an irrigation dam, constructed across river Banas. It is a composite rolled filled earth dam with masonry spillway having radial gates. The salient data about the project are as under.

Area of catchment	2,862 km^2
Mean annual runoff in the catchment	447,830,000 m^3
Rainfall	94 cm
Design flood discharge	6,655 m^3/s
Year of commencement of construction work	1958
Year of completion	1965
Dam	
Type	Composite rolled filled earthen dam with masonry spillway
Bed rock	Frinpura granite
Maximum height above the lowest point of foundation	61 m
Length at the top of the dam	
(i) Non overflow	4,719 m
(ii) Spillway	162 m
Total volume content	
(i) Concrete	32,000 m^3
(ii) Earth	6,200,000 m^3/s

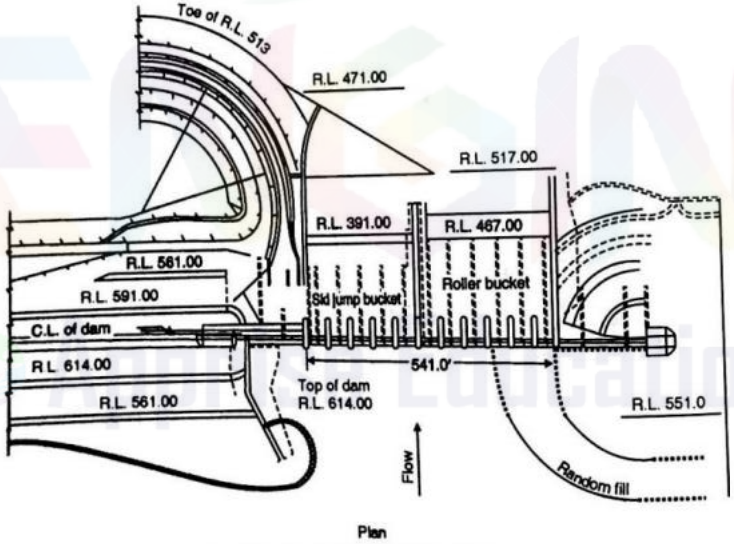

FIG. 23.24. LAYOUT OF DANTIWADA DAM.

Spillway	
Type	Masonry gated spillway
Type and number of gates	11 Nos radial gates
Maximum discharge capacity	6,665 m^3/s
Reservoir	
Area at F.R.L.	42 km^2
Gross storage capacity	464,450,000 m^3
Effective storage capacity	444,770,000 m^3

Fig. 23.24 shows the layout of various components of the dam and spillway.

Fig. 23.25 shows the typical earth dam cross-section and the elevation along the length of the dam.

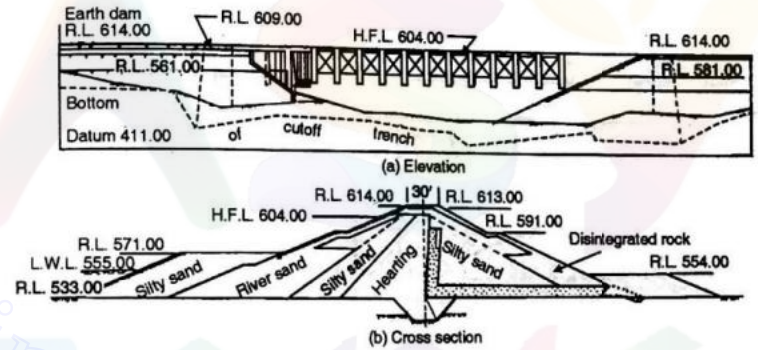

FIG. 23.25. DANTIWADA DAM.

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APPENDIX-1

Khosla's Theory

1. COMPOSITE FLOOR PROFILE

Fig. 1 shows a composite floor profile, the most general case analysed by A.N.Khosla, Dr. N.K.Bose and Dr. E.M.Taylor (1936) by method of *Schwarz-Christoffel transformation.* The flow lines and equipotential lines of the actual section are no longer ellipses and hyperbolas. By the mathematical process of transformation, the distorted flow net can be brought back to the normal elliptical and hyperbolic curves.

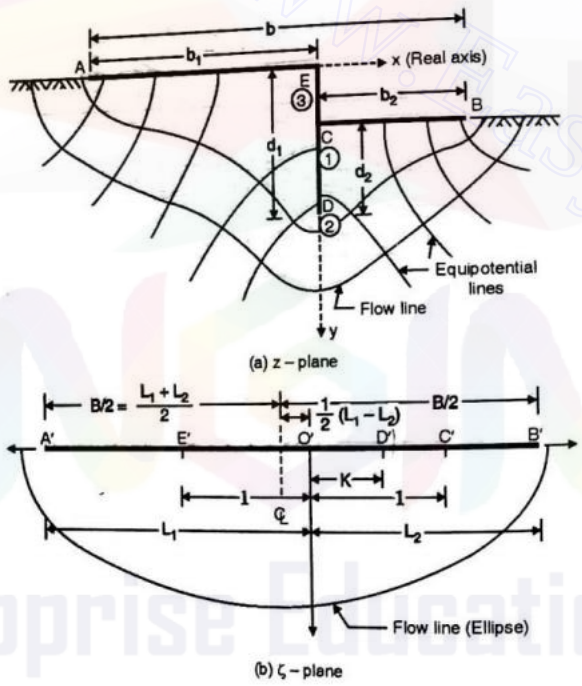

FIG. 1.

If a polygon is located in the Z-plane, than the Schwarz-Christoffel equation, that maps it conformally on to ζ-plane is given by

$$z = A \int \frac{d\zeta}{(\zeta-\zeta_1)^{\lambda_1}(\zeta-\zeta_2)^{\lambda_2}\ldots(\zeta-\zeta_n)^{\lambda_n}} \quad \ldots(1)$$

where $\pi\lambda_1, \pi\lambda_2, \ldots, \pi\lambda_n$ = change in the angles of the polygon which are required to transform it into a straight line.

$\zeta_1, \zeta_2 \ldots, \zeta_n$ = co-ordinates of its vertices.

A = complex constant.

Step 1. Transformation Equation $z = f(\zeta)$: Let us now apply this equation to transform the composite profile in Fig. 1(*a*) into straight line profile of Fig. 1(*b*). The key points A, E, D, C and B of Fig. 1(*a*) fall into points A', E' D', C', and B', respectively in Fig. 1(*b*). It will be seen that the angles at vertices E, D and C are respectively $\frac{\pi}{2}$, 2π and $\frac{\pi}{2}$, and these have been changed to π each at E'. D' and C' on transformation. The Schwarz-Christoffel formula is :

$$z = A \int \frac{d\zeta}{(\zeta-\zeta_1)^{\lambda_1}(\zeta-\zeta_2)^{\lambda_2}(\zeta-\zeta_3)^{\lambda_3}} \quad \ldots(1)$$

where $\pi\lambda_1$ = change in the angle at $C = \pi - \pi/2 = +\pi/2$

$\pi\lambda_2$ = change in the angle at $D = \pi - 2\pi = -\pi$

$\pi\lambda_3$ = change in the angle at $E = \pi - \pi/2 = +\pi/2$

Hence $\lambda_1 = \frac{1}{2}$; $\lambda_2 = -1$ and $\lambda_3 = \frac{1}{2}$.

Also, selecting the origin O' midway between C' and E', OE' and OC' will be equal, and can be selected to be equal to unity. Let $O'D' = K$, where K is an absolute number.

Then, ζ_1 = co-ordinate of $C = +1$

ζ_2 = co-ordinate of $D = K$

ζ_3 = co-ordinate of $E = -1$.

Substituting these in (1), we get

$$z = A \int \frac{d\zeta}{(\zeta-1)^{1/2}(\zeta-K)^{-1}(\zeta+1)^{1/2}}$$

or

$$z = A \int \frac{(\zeta-K)\,d\zeta}{\sqrt{(\zeta^2-1)}} \quad \ldots(2)$$

Integrating the above, we get

$$z = A\sqrt{\zeta^2-1} - AK\log_e(\zeta+\sqrt{\zeta^2-1}) + B \quad \ldots(3)$$

Step 2. Determination of constants A and B : In order to find the constant B, let us compare the co-ordinates of point C and C' in the two planes. Evidently, the co-ordinate z of $C = i(d_1 - d_2)$ while that of C', $\zeta = 1$

Substituting these in (2), we get

$$B = i(d_1 - d_2)$$

Similarly, the co-ordinates of E and E' are : $z = 0$ and $\zeta = -1$.

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Substituting these in (3), we get

$$0 = B - A K \log_e(-1)$$

or $$A K \pi i = B = i(d_1 - d_2)$$

[Since $\log_e(-1) = \pi i$]

$$\therefore \quad A K = \frac{d_1 - d_2}{\pi} \quad \text{and} \quad A = \frac{d_1 - d_2}{\pi K} \qquad ...(4)$$

Step 3. Determination of Parameter K

Comparing the co-ordinates of points D and D', we get

$$z = i d_1 \text{ and } \zeta = K$$

Substituting these in Eq. (3), we have

$$i d_1 = A\sqrt{K^2 - 1} - A K \log_e(K + \sqrt{K^2 - 1}) + i(d_1 - d_2)$$

or $$i d_1 = A i \sqrt{1 - K^2} - A K \log_e(K + i\sqrt{1 - K^2}) + i(d_1 - d_2)$$

Putting $K = \cos\theta$, we get

$$i d_1 = A i \sin\theta - A K \log_e(\cos\theta + i \sin\theta) + i(d_1 - d_2)$$

or $$i d_1 = A i \sin\theta - A K i \theta + i(d_1 - d_2)$$

$$d_2 = A \sin\theta - A K \theta$$

Substituting the values of A and AK from Eq. 4, we get

$$d_2 = \frac{d_1 - d_2}{\pi K} \sin\theta - \frac{d_1 - d_2}{\pi}\theta$$

or $$d_2 = \frac{d_1 - d_2}{\pi} \tan\theta - \frac{d_1 - d_2}{\pi}\theta$$

Hence, $$\frac{d_2}{d_1 - d_2}\pi = \tan\theta - \theta$$

or $$\pi\delta = \tan\theta - \theta = \frac{\sqrt{1 - K^2}}{K} - \cos^{-1} K \qquad ...(5)$$

where $$\delta = \frac{d_2}{d_1 - d_2}$$

Thus, from Eq. 5 the values of K for various values of δ can be determined.

Step 4. Final Transformation Equation : Substituting the values of A and AK in Eq. 3, we get the final transformation formula

$$z = \frac{d_1 - d_2}{\pi K}\sqrt{\zeta^2 - 1} - \frac{d_1 - d_2}{\pi}\log_e(\zeta + \sqrt{\zeta^2 - 1}) + i(d_1 - d_2) \qquad ...(I)$$

where K is now a known parameter.

Step 5. Determination of co-ordinates of A' and B'

Equation (I) is applicable to all points on the profile of the floor. Let the ζ co-ordinates of point A' and B' be $-L_1$ and $+L_2$ respectively. The z co-ordinates of the corresponding points A and B are $-b_1$ and $[b_2 + i(d_1 - d_2)]$ respectively. Substituting these co-ordinates, separately in Eq.(I), we get

$$-b_1 = \frac{d_1 - d_2}{\pi K}\sqrt{L_1^2 - 1} - \frac{d_1 - d_2}{\pi}\log_e(-L_1 + \sqrt{L_1^2 - 1}) + i(d_1 - d_2)$$

or $$-b_1 = \frac{d_1 - d_2}{\pi K}\sqrt{L_1^2 - 1} - \frac{d_1 - d_2}{\pi}\log_e(L_1 - \sqrt{L_1^2 - 1}) \qquad ...(6)$$

and $$b_2 + i(d_1 - d_2) = \frac{d_1 - d_2}{\pi K}\sqrt{L_2^2 - 1} - \frac{d_1 - d_2}{\pi}\log_e(L_2 + \sqrt{L_2^2 - 1}) + i(d_1 - d_2)$$

or $$b_2 = \frac{d_1 - d_2}{\pi K}\sqrt{L_2^2 - 1} - \frac{d_1 - d_2}{\pi}\log_e(L_2 + \sqrt{L_2^2 - 1}) \qquad ...(7)$$

Assuming $L_1 = \cosh\gamma_1$ and $L_2 = \cosh\gamma_2$ and substituting these in the above, we get

$$-b_1 = \frac{d_1 - d_2}{\pi K}\sinh\gamma_1 + \frac{d_1 - d_2}{\pi}\gamma_1$$

or $$\sinh\gamma_1 + K\gamma_1 = -\frac{\pi K b_1}{d_1 - d_2}$$

and $$\sinh\gamma_2 - K\gamma_2 = \frac{\pi K b_2}{d_1 - d_2}$$

Again let $\delta_1 = \dfrac{b_1}{d_1 - d_2}$ and $\delta_2 = \dfrac{b_2}{d_1 - d_2}$. Substituting in the above, we get

$$\sinh\gamma_1 + K\gamma_1 = -\pi K\delta_1 \qquad ...(8)$$

$$\sinh\gamma_2 - K\gamma_2 = \pi K\delta_2 \qquad ...(9)$$

Thus, from Eqs.(8) and (9), γ_1 and γ_2 can be calculated. Hence, $L_1 = \cosh\gamma_1$ and $L_2 = \cosh\gamma_2$ can be known. Thus, corresponding to given set of values of floor dimensions b_1, b_2, d_1 and d_2 we can first get K, and then L_1 and L_2.

Step 6. Pressure distribution below the floor : Now, in order to get the pressure distribution under the floor, let us consider Eq. I. In § 12.8, we have seen that for the case of a horizontal floor (Fig. 12.9)

$$x + iy = z = \frac{b}{2}\cosh w = \frac{b}{2}\cosh(u + iv).$$

Applying this to the *transformed floor* of Fig. 1(*b*) where $\frac{b}{2}$ = half the width of the floor $= \frac{1}{2}(L_1 + L_2)$ and $z = \zeta + \frac{1}{2}(L_1 - L_2)$, we get

$$\zeta + \tfrac{1}{2}(L_1 - L_2) = \tfrac{1}{2}(L_1 + L_2)\cosh w$$

Putting $\dfrac{L_1 + L_2}{2} = \lambda$ and $\dfrac{L_1 - L_2}{2} = \lambda_1$, we get

$$\zeta = \lambda\cosh w - \lambda_1 \qquad ...(II)$$

Substituting this in Eq. (1), we get a general equation for the flow net diagram as under :

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$$z = \frac{d_1 - d_2}{\pi K}\sqrt{(\lambda \cosh w - \lambda_1)^2 - 1} - \frac{d_1 - d_2}{\pi}\log_e\left[|\lambda \cosh w - \lambda_1| + \sqrt{(\lambda \cosh w - \lambda_1)^2 - 1}\right] + i(d_1 - d_2)$$

For the first stream line which hugs to the base, we have $u = 0$ and hence $w = u + iv = iv$.

Also $z = x + iy$. Hence

$$x + iy = \frac{d_1 - d_2}{\pi K}\sqrt{(\lambda \cos v - \lambda_1)^2 - 1} - \frac{d_1 - d_2}{\pi}\log_e\left\{(\lambda \cos v - \lambda_1) + \sqrt{(\lambda \cos v - \lambda_1)^2 - 1}\right\} + i(d_1 - d_2) \quad ...(III)$$

From Eq. (III), we can get the pressure function v for any point whose co-ordinate is $(x + iy)$. We shall apply this equation to different regions of the floor of Fig. 1(*a*).

Step 7. Pressure Distribution for Length *AE*

In this length, $y = 0$. Also, introducing a new variable defined by the equation

$$\lambda \cos v - \lambda_1 = -\cosh \beta_1' \quad ...(10)$$

we get, from Eq. (III),

$$x = \frac{d_1 - d_2}{\pi K}\sinh \beta_1' + \frac{d_1 - d_2}{\pi}\log_e[-\cosh \beta_1' + \sinh \beta_1'] + i(d_1 - d_2)$$

or

$$x = \frac{d_1 - d_2}{\pi K}\sinh \beta_1' + \frac{d_1 - d_2}{\pi}\beta_1' - \frac{d_1 - d_2}{\pi}\log_e(-1) + i(d_1 - d_2)$$

or

$$x = \frac{d_1 - d_2}{\pi K}\sinh \beta_1' + \frac{d_1 - d_2}{\pi}\beta_1'$$

Hence $\frac{x}{d_1 - d_2}\pi K = = \delta_x \pi K = \sinh \beta_1' + K B_1'$ ...(11)

From this equation, β_1' can be calculated. Hence from Eq. (10), the pressure function v is found to be

$$v = \cos^{-1}\left(\frac{\lambda_1 - \cosh \beta_1'}{\lambda}\right) = \pi\frac{P}{H}$$

$$P = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1 - \cosh \beta_1'}{\lambda} \quad ...[IV\ (a)]$$

$$\Phi = \frac{P}{H} = \frac{1}{\pi}\cos^{-1}\frac{\lambda_1 - \cosh \beta_1'}{\lambda} \quad ...[IV\ (b)]$$

This gives pressure distribution under *AE*

Step 8. Pressure distribution for Length *CB* : Introducing a new variable β_2' defined by the equation $\lambda \cos v - \lambda_1 = \cosh \beta_2'$, we get from Eq. (III)

$$\sinh \beta_2' - K\beta_2' = \pi K\frac{x}{d_1 - d_2} = \delta_x \pi K \quad ...(12)$$

From this equation β_2' can be known. Hence the pressure distribution is given by

$$v = \cos^{-1}\frac{\lambda_1 + \cosh \beta_2'}{\lambda} = \pi\frac{P}{H}$$

Hence $$P = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1 + \cosh \beta_2'}{\lambda} \quad ...[V(a)]$$

$$\Phi = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + \cosh \beta_2'}{\lambda}\right) \quad ...[V(b)]$$

Combining Eqs. [IV(*a*)] and [V(*a*)] in one common form, we get

$$P = \frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1 \mp \cosh \beta'_{1,2}}{\lambda}\right) \quad ...(VI)$$

where − sign is used for upstream and + sign for the downstream part of the floor.

Step 9. Pressure distribution along *ED* and *DC* : For the pressure distribution along the u/s and d/s faces of the pile, we have $x = 0$. Hence from the Eq. (III)

$$iy = \frac{d_1 - d_2}{\pi K}\sqrt{(\lambda \cos v - \lambda_1)^2 - 1} - \frac{d_1 - d_2}{\pi}\log_e\left[[\lambda \cos v - \lambda_1] + \sqrt{(\lambda \cos v - \lambda_1)^2 - 1}\right] + i(d_1 - d_2)$$

Introducing a new variable β defined by the equation

$\lambda \cos v - \lambda_1 = \cos \beta$, we get

$$iy = \frac{d_1 - d_2}{\pi K} i \sin \beta - \frac{d_1 - d_2}{\pi} i\beta + i(d_1 - d_2)$$

or $$\left(\frac{y}{d_1 - d_2} - 1\right)\pi K = \sin \beta - K\beta \quad ...(13)$$

From which β can be calculated.

Hence $$v = \cos^{-1}\left[\frac{\lambda_1 + \cos \beta}{\lambda}\right] = \pi\frac{P}{H}$$

$\therefore$ $$P = \frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1 + \cos \beta}{\lambda}\right) \quad ...(VII)$$

Step 10. Pressure at key points

(i) Pressure at E

For $E'A'$, $\zeta = \lambda \cos v - \lambda_1$ (From Eq. II)

At E', $\zeta = -1$

Hence $-1 = \lambda \cos v - \lambda_1$

or $$v = \pi\frac{P}{H} = \cos^{-1}\frac{\lambda_1 - 1}{\lambda}$$

or $$P_E = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1 - 1}{\lambda} \quad ...[VIII(a)]$$

(ii) Pressure at C

For $C'B'$, $\zeta = \lambda \cos v - \lambda_1$ (From Eq. II)

At C', $\zeta = +1$

$\therefore$ $+1 = \lambda \cos v - \lambda_1$

or $$v = \pi\frac{P}{H} = \cos^{-1}\frac{\lambda_1 + 1}{\lambda}$$

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$$P_C = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1+1}{\lambda} \qquad \text{...(VIII(b))}$$

(iii) Pressure at D

At point D' $\qquad \zeta = \lambda\cos v - \lambda_1 = K$

or
$$P_D = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1+K}{\lambda} \qquad \text{[VIII(c)]}$$

2. SPECIAL CASE : $b_2 = 0$

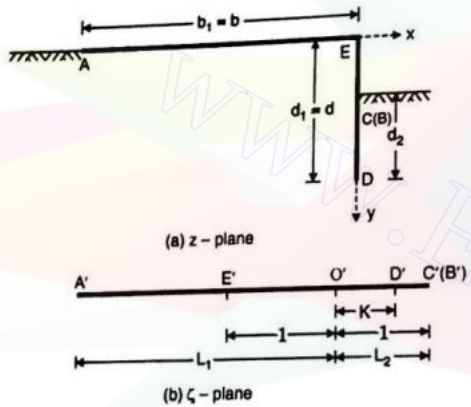

FIG. 2

Let us now take the case when $b_2 = 0$ and $b_1 = b$ as shown in Fig. 2.

$$\therefore \qquad \delta_2 = \frac{b_2}{d_1 - d_2} = 0$$

Hence from Eq. (9),

$$\sinh\gamma_2 - K\gamma_2 = \pi K\delta_2 = 0$$

or
$$\gamma_2 = 0$$

$$\therefore \qquad L_2 = \cosh 0 = 1$$

This could also be found from Fig. 2.

Now,
$$\lambda = \frac{L_1+1}{2}$$

$$\lambda_1 = \frac{L_1-1}{2} = \lambda - 1$$

$$\lambda_1 - 1 = \frac{L_1-3}{2}$$

$$\lambda_1 + 1 = \frac{L_1+1}{2} = \lambda$$

Hence from Eq.(VIII),
$$P_E = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda_1-1}{\lambda}\right] = \frac{H}{\pi}\cos^{-1}\frac{L_1-3}{L_1+1} \qquad \text{...[IX (a)]}$$

$$P_C = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda_1+1}{\lambda}\right] = \frac{H}{\lambda}\cos^{-1}(1)$$

or
$$P_C = 0 \qquad \text{...[IX (b)]}$$

$$P_D = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda_1+K}{\lambda}\right] = \frac{H}{\pi}\cos^{-1}\left[\frac{2K+L_1-1}{L_1+1}\right] \qquad \text{...[IX (c)]}$$

Exit Gradient.

Along the sheet pile, we get from Eq.13,

$$\left[\frac{y}{d_1-d_2} - 1\right]\pi K = \sin\beta - K\beta$$

or
$$y = \frac{d_1-d_2}{\pi K}\sin\beta - \frac{d_1-d_2}{\pi}\beta + (d_1-d_2)$$

To find the exit gradient at C, differentiate the above with respect to y :

$$\frac{dy}{dy} = 1 = \frac{d_1-d_2}{\pi K}\cos\beta\frac{d\beta}{dy} - \frac{d_1-d_2}{\pi}\frac{d\beta}{dy}$$

But $\qquad \lambda\cos v - \lambda_1 = \cos\beta$

$$\therefore \qquad \lambda\sin v\frac{dv}{dy} = \sin\beta\frac{d\beta}{dy}$$

or
$$\frac{d\beta}{dy} = \lambda\frac{\sin v}{\sin\beta}\cdot\frac{dv}{dy} \qquad \text{...(14)}$$

Substituting this, we get

$$1 = \frac{d_1-d_2}{\pi K}\cos\beta\cdot\frac{\lambda\sin v}{\sin\beta}\frac{dv}{dy} - \frac{d_1-d_2}{\pi}\lambda\frac{\sin v}{\sin\beta}\cdot\frac{dv}{dy} \qquad \text{...(15)}$$

At the exit end $v = 0$

$$\therefore \qquad \cos\beta = \lambda\cos v - \lambda_1 = \lambda - \lambda_1 = 1.$$

$$\beta = 0$$

Hence when $\quad v = 0\ ,\ \beta = 0$

or
$$\frac{\sin v}{\sin\beta} = \frac{0}{0}$$

It is, therefore, essential to determine the limit of $\left(\frac{\sin v}{\sin\beta}\right)_{v=0,\beta=0}$

$$\text{Lt}\left(\frac{\sin v}{\sin\beta}\right)_{v=0,\beta=0} = \text{Lt}\left(\frac{\frac{d}{dy}(\sin v)}{\frac{d}{dy}(\sin\beta)}\right)_{v=0,\beta=0} = \text{Lt}\left(\frac{\cos v\cdot\frac{dv}{dy}}{\cos\beta\frac{d\beta}{dy}}\right)_{v=0,\beta=0}$$

From (14)

$$= \text{Lt}\left\{\frac{\cos v}{\cos\beta}\cdot\frac{\sin\beta}{\lambda\sin v}\right\}_{v=0,\beta=0}$$

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$$= \text{Lt}\left(\frac{\sin\beta}{\lambda \sin v}\right)_{v=0,\beta=0}$$

$$\therefore \quad \left[\text{Lt}\left(\frac{\sin v}{\sin\beta}\right)\right]^2_{v=0,\beta=0} = \frac{1}{\lambda}$$

or
$$\text{Lt}\left(\frac{\sin v}{\sin\beta}\right)_{v=0,\beta=0} = \frac{1}{\sqrt{\lambda}}$$

Substituting this in 15, we get

$$1 = \frac{d_1 - d_2}{\pi K} \cdot \frac{\lambda}{\sqrt{\lambda}} \left[\frac{dv}{dy}\right]_{v=0} - \frac{d_1 - d_2}{\pi} \frac{\lambda}{\sqrt{\lambda}} \left(\frac{dv}{dy}\right)_{v=0}$$

$$\therefore \quad \left[\frac{dv}{dy}\right]_{v=0} = \frac{\pi K}{(d_1 - d_2)\sqrt{\lambda}} \cdot \frac{1}{1-K}$$

Now
$$\left[\frac{dP}{dy}\right]_{E,dt} = G_E = \frac{H}{\pi}\left[\frac{dv}{dy}\right]_{v=0} = \frac{H}{\pi} \cdot \frac{\pi K}{(d_1 - d_2)\sqrt{\lambda}} \cdot \frac{1}{1-K}$$

$$G_E = \frac{H}{d_2} \cdot \frac{d_2}{d_1 - d_2} \cdot \frac{K}{1-K} \cdot \frac{1}{\sqrt{\lambda}} \qquad \text{...(X)}$$

End Sheet pile with $\quad d_1 = d_2 = d$

For case where $d_1 = d_2$ we have, $\quad K = 0$

$$\therefore \quad \frac{K}{d_1 - d_2} = \frac{0}{0}$$

Let us find the value of the above limit. From Eq. (V),

$$\tan\theta - \theta = \pi\delta = \frac{\pi d_2}{d_1 - d_2} \; ; \quad \text{also} \quad \cos\theta = K$$

Hence multiplying left side by $\cos\theta$ and right side by K, we get

$$\sin\theta - \theta\cos\theta = \pi d_2 \frac{K}{d_1 - d_2}$$

For $\quad K = 0, \; \theta = \pi/2$

Hence the above equation becomes

$$1 - 0 = \pi d_2 \left[\frac{K}{d_1 - d_2}\right]_{\text{Lt } d_1 = d_2}$$

$$\therefore \quad \text{Lt}\left[\frac{K}{d_1 - d_2}\right]_{d_1 = d_2 = d} = \frac{1}{\pi d_2} = \frac{1}{\pi d} \qquad \text{...(15)}$$

Hence from Eq. X,
$$G_E = \left(\frac{H}{1-K}\right)\left(\frac{K}{d_1 - d_2}\right) \cdot \frac{1}{\sqrt{\lambda}} = \frac{H}{1} \cdot \frac{1}{\pi d} \cdot \frac{1}{\sqrt{\lambda}}$$

or
$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}} \qquad \text{...(XI)}$$

This is very important equation for exit gradient.

When floor is absent (*i.e.* $b = 0$), $\lambda = 1$

$$\therefore \quad G_E = \frac{H}{\pi d} \qquad \text{...(XII)}$$

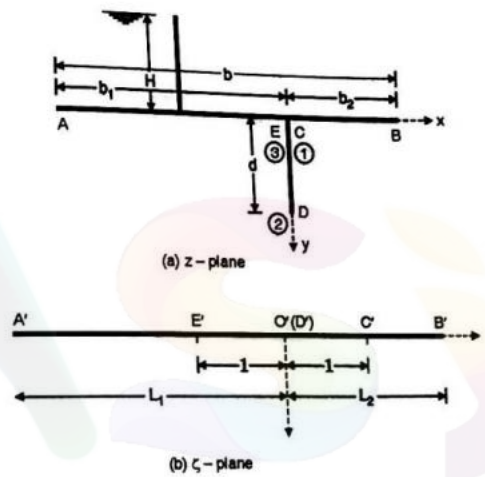

FIG. 3.

3. ECCENTRIC SHEET PILE : INDEPENDENT SOLUTION

The Schwarz-Christoffel transformation equation is :

$$z = A\int \frac{d\zeta}{(\zeta-\zeta_1)^{\lambda_1}(\zeta-\zeta_2)^{\lambda_2}(\zeta-\zeta_3)^{\lambda_3}} = A\int \frac{d\zeta}{(\zeta-1)^{1/2}(\zeta-0)^{-1}(\zeta+1)^{1/2}} = A\int \frac{\zeta d\zeta}{\sqrt{\zeta^2-1}}$$

or
$$z = A\sqrt{\zeta^2 - 1} + B \qquad \text{...(16)}$$

In order to determine constant B, consider co-ordinates of points C and C'

$$z = 0 \text{ and } \zeta = 1.$$

Hence from Eq. (16), $\quad B = 0$

In order to determine constant A, consider co-ordinates of D and D'

$$z = id \quad \text{and} \quad \zeta = 0$$

Hence from Eq. (16), $\quad id = A\sqrt{-A} = iA$

or $\quad A = d$

Hence the transformation equation becomes,

$$z = d\sqrt{\zeta^2 - 1} \qquad \text{...(17)}$$

Now, for the horizontal floor in ζ plane, we have

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$$\zeta + \frac{1}{2}(L_1 - L_2) = \frac{1}{2}(L_1 + L_2)\cosh w$$

Putting $\frac{L_1 + L_2}{2} = \lambda$ and $\frac{L_1 - L_2}{2} = \lambda_1$

we get $\zeta = \lambda \cosh w - \lambda_1$

For the floor profile, $u = 0$, hence $\cosh w = \cosh iv = \cos v$

$$\therefore \quad \zeta = \lambda \cos v - \lambda_1 \qquad ...(18)$$

Substituting this in Eq. (17), we get

$$z = d\sqrt{(\lambda \cos v - \lambda_1)^2 - 1} \qquad ...(19)$$

From the above equation, pressure distribution below the floor can be found.

For finding co-ordinates of A' and B', we get from Eq. (17),

$$-b_1 = d\sqrt{L_1^2 - 1}$$

and $$b_2 = d\sqrt{L_2^2 - 1}$$

$$\therefore \quad L_1 = \sqrt{\left(\frac{b_1}{d}\right)^2 + 1} = \sqrt{1 + \alpha_1^2} \qquad ...[20(a)]$$

and $$L_2 = \sqrt{\left(\frac{b_2}{d}\right)^2 + 1} = \sqrt{1 + \alpha_2^2} \qquad ...[20(b)]$$

where $\alpha_1 = \frac{b_1}{d}$ and $\alpha_2 = \frac{b_2}{d}$

Hence $$\lambda = \frac{L_1 + L_2}{2} = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2} \qquad ...[21(a)]$$

and $$\lambda_1 = \frac{L_1 - L_2}{2} = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2} \qquad ...[21(b)]$$

For the floor, $y = 0$. Hence from Eq. (19),

$$z = x + iy = x = d\sqrt{(\lambda \cos v - \lambda_1)^2 - 1}$$

or $$\lambda \cos v - \lambda_1 = \pm\sqrt{\frac{d^2 + x^2}{d^2}}$$

$$\cos v = \frac{\lambda_1 d \pm \sqrt{d^2 + x^2}}{\lambda d}$$

$$\therefore \quad v = \pi\frac{P}{H} = \cos^{-1}\frac{\lambda_1 d \pm \sqrt{d^2 + x^2}}{\lambda d}$$

or $$P = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1 d \pm \sqrt{d^2 + x^2}}{\lambda d} \qquad ...(22)$$

where $-$ve sign is to be used for u/s floor and $+$ve sign is to be used for d/s floor.

For point E, $x = 0$

$$P_E = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda_1 - 1}{\lambda}\right] \qquad ...[23(a)]$$

For point C, $x = 0$

$$P_C = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda_1 + 1}{\lambda}\right] \qquad ...[23(b)]$$

For pressure along the sheet pile, put $x = 0$ in Eq.(19),

$$z = x + iy = iy = d\sqrt{(\lambda \cos v - \lambda_1)^2 - 1}$$

or $$\lambda \cos v - \lambda_1 = \pm\sqrt{\frac{d^2 - y^2}{d^2}}$$

$$\therefore \quad \cos v = \frac{\lambda_1 d \pm \sqrt{d^2 - y^2}}{\lambda d}$$

$$\therefore \quad v = \pi\frac{P}{H} = \cos^{-1}\left(\frac{\lambda_1 d \pm \sqrt{d^2 - y^2}}{\lambda d}\right)$$

or $$P = \frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1 d \pm \sqrt{d^2 - y^2}}{\lambda d}\right)$$

From point D, $y = d$

$$\therefore \quad P_D = \frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right) \qquad ...[23(c)]$$

The curves of Fig. 12.13 are based on Eqs. 23(a), (b), (c).

4. FLOOR WITH END SHEET PILE

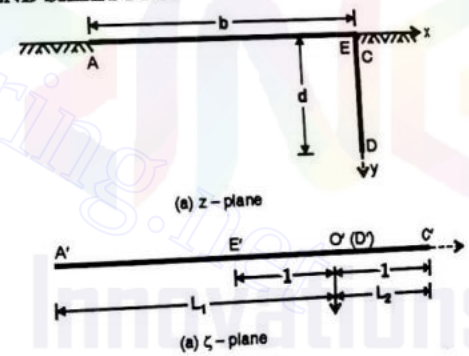

FIG. 4

Schwarz-Christoffel transformation equation becomes :

$$z = A\int\frac{d\zeta}{(\zeta - 1)^{1/2}(\zeta - 0)^{-1}(\zeta + 1)^{1/2}} = A\int\frac{\zeta\, d\zeta}{\sqrt{\zeta^2 - 1}}$$

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or $$z = A\sqrt{\zeta^2 - 1} + B$$

For point C, $z = 0$ and $\zeta = 1$; $\therefore$ $B = 0$

For point D, $z = id$ and $\zeta = 0$

$\therefore$ $$id = A\sqrt{-1} = Ai$$

Hence $$A = d.$$

Thus, the transformation equation becomes :

$$z = d\sqrt{\zeta^2 - 1} \quad ...(24)$$

From Fig. 4, $L_2 = 1$. For L_1, consider co-ordinates of A and A'

$\therefore$ $$-b_1 = d\sqrt{L_1^2 - 1} \quad \text{(From 24).}$$

$\therefore$ $$L_1 = \sqrt{\left(\frac{b}{d}\right)^2 + 1} = \sqrt{1 + \alpha^2}$$

where $$\alpha = \frac{b}{d}$$

Now for the horizontal floor of ζ-plane

$$\zeta + \frac{1}{2}(L_1 - L_2) = \frac{1}{2}(L_1 + L_2)\cosh w$$

$$\lambda = \frac{L_1 + L_2}{2} = \frac{L_1 + 1}{2} = \frac{\sqrt{1+\alpha^2} + 1}{2} \quad ...[26(a)]$$

and $$\lambda_1 = \frac{L_1 - L_2}{2} = \frac{L_1 - 1}{2} = \frac{(L_1 + 1) - 2}{2} = \lambda - 1 = \frac{\sqrt{1+\alpha^2} - 1}{2} \quad ...[26(b)]$$

$$\zeta = \lambda \cosh w - \lambda_1$$

For the floor profile, $u = 0$. Hence $\cosh w = \cosh iv = \cos v$.

$$\zeta = \lambda \cos v - \lambda_1$$

Substituting this in Eq. (24),

$$z = d\sqrt{(\lambda \cos v - \lambda_1)^2 - 1} \quad ...(27)$$

For the floor, $y = 0$, and hence $z = x$,

$$x = d\sqrt{(\lambda \cos v - \lambda_1)^2 - 1} \quad ...[27(a)]$$

$$\lambda \cos v - \lambda_1 = \pm\sqrt{\frac{d^2 + x^2}{d^2}}$$

From which $$v = \pi\frac{P}{H} = \cos^{-1}\frac{\lambda_1 d \pm \sqrt{d^2 + x^2}}{\lambda d}$$

(where negative sign is used for u/s floor)

$$P = \frac{H}{\pi} = \cos^{-1}\frac{\lambda_1 d \pm \sqrt{d^2 + x^2}}{\lambda d} \quad ...(28)$$

For point E, $x = 0$

$$P_E = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1 d - d}{\lambda d} = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1 - 1}{\lambda}$$

or $$P_E = \frac{H}{\pi}\cos^{-1}\frac{\lambda - 2}{\lambda} \quad \text{(since } \lambda_1 = \lambda - 1) \quad ...[29(a)]$$

For point C, $x = 0$

$$P_C = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1 d + d}{\lambda d} = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1 + 1}{\lambda} = \frac{H}{\pi}\cos^{-1}\frac{\lambda}{\lambda} = 0 \quad ...[29(b)]$$

For the sheet pile, $x = 0$. Hence from Eq. 27, we get

$$iy = d\sqrt{\lambda \cos v - \lambda_1)^2 - 1} \quad ...[27(b)]$$

or $$\lambda \cos v - \lambda_1 = \pm\sqrt{\frac{d^2 - y^2}{d^2}}$$

$\therefore$ $$v = \pi\frac{P}{H} = \cos^{-1}\frac{\lambda_1 d \pm \sqrt{d^2 - y^2}}{\lambda d}$$

or $$P = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1 d \pm \sqrt{d^2 - y^2}}{\lambda d}$$

From point D, $y = d$

$$P_D = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1 d}{\lambda d} = \frac{H}{\pi}\cos^{-1}\frac{\lambda - 1}{\lambda} \quad ...[29(c)]$$

The curves of Fig. 12.14 are based on Eqs. 29 (a) and (c).

Exit Gradient.

For finding exit gradient, consider Eq. 27(b)

$$iy = d\sqrt{\lambda \cos v - \lambda_1)^2 - 1} \quad ...[30(a)]$$

Let $$\lambda \cos v - \lambda_1 = \cos\beta$$

$\therefore$ $$iy = d\sqrt{\cos^2\beta - 1} = id \sin\beta$$

$\therefore$ $$y = d \sin\beta$$

Differentiating it with respect to y ;

$$1 = d \cos\beta\frac{d\beta}{dy} \quad ...[30(b)]$$

But from Eq. 30(a), $$\lambda \sin v\frac{dv}{dy} = \sin\beta\frac{d\beta}{dy} \quad ...[30(c)]$$

Substituting the value of $\frac{d\beta}{dy}$ in Eq.30(b).

$$1 = d \cos\beta\frac{\lambda \sin v}{\sin\beta}\cdot\frac{dv}{dy} \quad ...[30(d)]$$

At the exit end, $v = 0$. Hence from Eq. 30(a)

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$$\cos\beta = \lambda\cos\upsilon - \lambda_1 = \lambda\cos 0 - \lambda_1 = \lambda - \lambda_1 = 1$$

$$\beta = 0 \text{ when } \upsilon = 0$$

$$\therefore \quad \frac{\sin\upsilon}{\sin\beta} = \frac{0}{0}$$

Let us determine the limit of $\left(\frac{\sin\upsilon}{\sin\beta}\right)_{\upsilon=0,\beta=0}$

$$\text{Lt}\left(\frac{\sin\upsilon}{\sin\beta}\right)_{\upsilon=0,\beta=0} = \text{Lt}\left(\frac{\frac{d}{dy}\sin\upsilon}{\frac{d}{dy}\sin\beta}\right)_{\upsilon=0,\beta=0} = \text{Lt}\left(\frac{\cos\upsilon\frac{d\upsilon}{dy}}{\cos\beta\frac{d\beta}{dy}}\right)_{\upsilon=0,\beta=0}$$

But $\quad \frac{d\upsilon}{dy} \Big/ \frac{d\beta}{dy} = \frac{\sin\beta}{\lambda\sin\upsilon}$ $\quad$ From Eq. [30(c)]

$$\therefore \quad \text{Lt}\left(\frac{\sin\upsilon}{\sin\beta}\right)_{\upsilon=0,\beta=0} = \text{Lt}\left(\frac{\sin\beta}{\lambda\sin\upsilon}\right)_{\upsilon=0,\beta=0}$$

Hence $\quad \text{Lt}\left(\frac{\sin\upsilon}{\sin\beta}\right)_{\upsilon=0,\beta=0} = \frac{1}{\sqrt{\lambda}}$ $\quad$ ...(31)

Substituting the value in Eq. 30(*d*), we get, for $\upsilon = 0$ and $\beta = 0$

$$1 = d\frac{\lambda}{\sqrt{\lambda}}\left(\frac{d\upsilon}{dy}\right)_{exit}$$

or $\quad \left(\frac{d\upsilon}{dy}\right)_{exit} = \frac{1}{d\sqrt{\lambda}}$ $\quad$ ...(32)

Now $\quad \left(\frac{dP}{dy}\right)_{exit} = G_E = \frac{H}{\pi}\left(\frac{d\upsilon}{dy}\right)_{exit} = \frac{H}{\pi}\cdot\frac{1}{d\sqrt{\lambda}}$

$$G_E = \frac{H}{d}\cdot\frac{1}{\pi\sqrt{\lambda}} \quad ...(33)$$

This is the required expression for exit gradient. In Fig. 12.13 (*b*), the exit gradient curve is based on the above equation.

Example 1. *A horizontal impervious floor of length 20 m is provided with a cutoff of 4 m depth at its downstream end (Fig. 5.). Determine analytically the uplift pressure at point E and D, and also the exit gradient, if the head causing seepage is 2 m.*

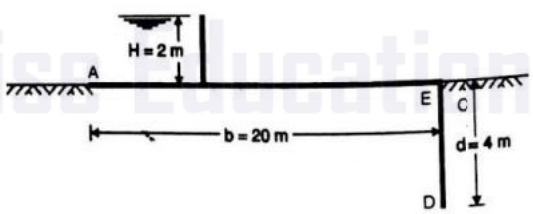

FIG. 5

Solution. (Fig. 5) : Given : $b = 20$ m ; $d = 4$ m; $H = 2$ m.

The pressure at point E is given by

$$P_E = \frac{H}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$$

where $\quad \lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$, and $\alpha = b/d = \frac{20}{4} = 5$

$$\therefore \quad \lambda = \frac{1+\sqrt{1+25}}{2} = 3.05$$

$$\therefore \quad P_E = \frac{2}{\pi}\cos^{-1}\left(\frac{3.05-2}{3.05}\right) = \frac{2}{\pi}(1.2195) = \mathbf{0.78\ m}$$

Also $\quad P_D = \frac{H}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = \frac{2}{\pi}\cos^{-1}\left(\frac{3.05-1}{3.05}\right) = \frac{2}{\pi}(0.8337) = \mathbf{0.53\ m}$

and $\quad G_E = \frac{H}{d}\cdot\frac{1}{\pi\sqrt{\lambda}} = \frac{2}{4}\cdot\frac{1}{\pi}\cdot\frac{1}{\sqrt{3.05}} \simeq \frac{1}{11}$

Example 2. *A horizontal impervious apron of length 10 m is provided with a cutoff of 4 m depth, placed eccentrically as shown. Determine analytically the uplift pressure at points E, D and C.*

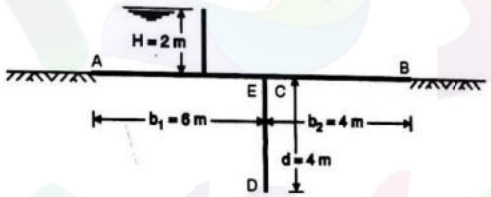

FIG. 6.

Solution. Given $\quad b_1 = 6$ m ; $b_2 = 4$ m ; $d = 4$ m ; $b = 10$ m

$$\alpha_1 = \frac{b_1}{d} = \frac{6}{4} = 1.5 ; \quad \alpha_2 = \frac{b_2}{d} = \frac{4}{4} = 1$$

$$\lambda = \frac{\sqrt{1+\alpha_1^2}+\sqrt{1+\alpha_2^2}}{2} = \frac{\sqrt{1+(1.5)^2}+\sqrt{1+(1)^2}}{2} = 1.61$$

$$\lambda_1 = \frac{\sqrt{1+\alpha_1^2}-\sqrt{1+\alpha_2^2}}{2} = \frac{\sqrt{1+(1.5)^2}-\sqrt{1+(1)^2}}{2} = 0.194$$

$$P_E = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1-1}{\lambda} = \frac{2}{\pi}\cos^{-1}\left(\frac{0.194-1}{1.61}\right) = \frac{2}{\pi}(2.095) = \mathbf{1.33\ m}$$

$$P_C = \frac{H}{\pi}\cos^{-1}\frac{\lambda_1+1}{\lambda} = \frac{2}{\pi}\cos^{-1}\left(\frac{0.194+1}{1.61}\right) = \frac{2}{\pi}(0.735) = \mathbf{0.47\ m}$$

$$P_D = \frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right) = \frac{2}{\pi}\cos^{-1}\left(\frac{0.194}{1.61}\right) = \frac{2}{\pi}(1.45) = \mathbf{0.92\ m.}$$

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Example 3. *For the impervious floor shown in Fig. 7, determine the uplift pressure at points C_1 and E_2.*

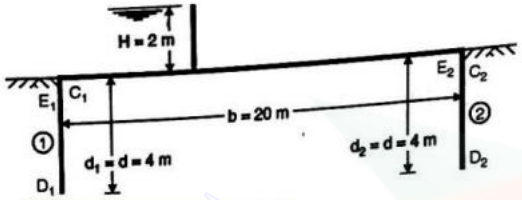

FIG. 7.

Solution : (*a*) ***Pile No. 2.***

$$\alpha = \frac{b}{d} = \frac{20}{4} = 5.$$

Neglecting presence of upstream pile,

$$P_{E2} = \frac{H}{\pi} \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

where $$\lambda = \frac{1 + \sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+25}}{2} = 3.05$$

$$\therefore \quad P_{E2} = \frac{2}{\pi}\cos^{-1}\left(\frac{3.05-2}{3.05}\right) = \frac{2}{\pi}(1.2195) = 0.78 \text{ m.}$$

The correction due to interference of pile No. 1 is (Eq. 12.12)

$$C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

where $$D = d = 4 \text{ m}; b = b' = 20 \text{ m.}$$

$$\therefore \quad C = 19\sqrt{\frac{4}{20}}\left(\frac{4+4}{20}\right) = 3.4\% \quad \text{(substractive)}$$

$\therefore$ Corrected $P_{E2} = 0.78 - 0.034 \times 2 = \mathbf{0.71}$ **m.**

(*b*) ***Pile No. 1.***

Both piles 1 and 2 are of same depth. By the principle of reversibility of flow,

$$P_{C1} = H - P_{E2} = 2 - 0.78 = 1.22 \text{ m.}$$

Also, correction due to interference of pile No. 2. is

$$C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = 19\sqrt{\frac{4}{40}}\left(\frac{4+4}{20}\right) = 3.4\ \%$$

$$= 0.034 \times 2 = 0.068 \text{ m (additive)}$$

Corrected $P_{C1} = 1.22 + 0.068 \simeq 1.29$ m.

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