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COMPOSITE PROFILE OF FLOOR : KHOSLA'S SOLUTION :

Fig. 12.11(*a*) shows a composite profile of floor or apron, the most general case analysed by A.N. Khosla[1], Dr. N.K. Bose and Dr. E.M. Taylor by the method of Schwarz-Christoffel transformation. Fig. 12.11(*b*) shows the transformed section. By the mathematical process of transformation, the distorted flow net can be brought back to the normal elliptical and hyperbolic curves.

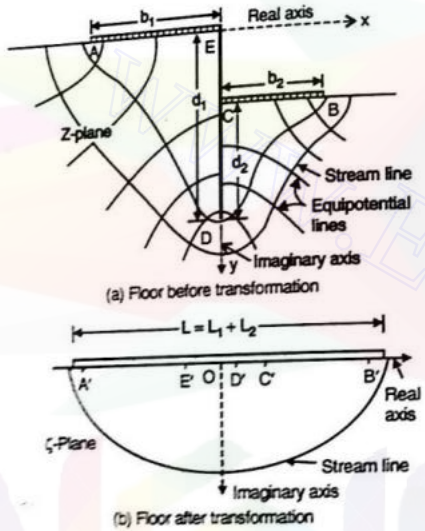

FIG. 12.11. KHOSLA'S CASE OF COMPOSITE PROFILE.

The complete analytical solution, using Schwarz-Chritoffel transformation, is given in article 1 of Appendix-1, at the end of the book.

SPECIFIC CASES.

We shall now consider the following specific cases of the general form shown in Fig. 12.11.

(*i*) Straight horizontal floor of negligible thickness with pile either at the upstream end or at the downstream end [Fig. 12.12 (*a*, *b*)].

(*ii*) Straight horizontal floor of negligible thickness with pile at some intermediate point [Fig. 12.12(*c*)].

(*iii*) Straight horizontal floor depressed below the bed, but with no cutoff [Fig. 12.12(*d*)].

1. "Design of Weirs on Permeable Foundations", by A.N. Khosla, Dr. N.K. Bose and Dr. E.M. Taylor, Central Board of Irrigation, India, Publication No. 12, 1936.

Case 1 : Pile at some intermediate point (Fig. 12.12 *c*)

These cases were analysed by Khosla and his associates with the help of Schwarz-Christoffel transformation. An independent analytical solution, for the floor with eccentric or intermediate pile (Fig. 12.12 *c*) is given in article 3 of Appendix-1 at the end of the book. The uplift pressures, P_E, P_D and P_C at the three key points *E*, *D* and *C* are given by the following equations.

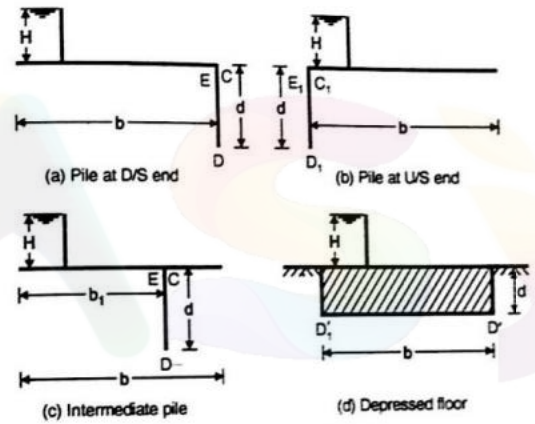

FIG. 12.12. SPECIFIC CASES.

$$P_E = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda_1 - 1}{\lambda}\right] \text{ or } \Phi_E = \frac{P_E}{H}\times 100 = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda_1 - 1}{\lambda}\right] \quad ...(12.6\ a)$$

$$P_D = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda_1}{\lambda}\right] \text{ or } \Phi_D = \frac{P_D}{H}\times 100 = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda_1}{\lambda}\right] \quad ...(12.6\ b)$$

$$P_C = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda_1 + 1}{\lambda}\right] \text{ or } \Phi_C = \frac{P_C}{H}\times 100 = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda_1 + 1}{\lambda}\right] \quad ...(12.6\ c)$$

where $$\lambda = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2} \quad ...(12.6\ d)$$

$$\lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2} \quad ...(12.6\ e)$$

$$\alpha_1 = b_1/d \quad \text{and} \quad \alpha_2 = b_2/d \quad ...(12.6\ f)$$

Fig. 12.13 gives the values of uplift pressures Φ_E, Φ_D and Φ_C, expressed as percentage of head *H*, at the three key points *E*, *D* and *C*. The procedure for finding these pressures is explained on the Fig itself, which is self explanatory.

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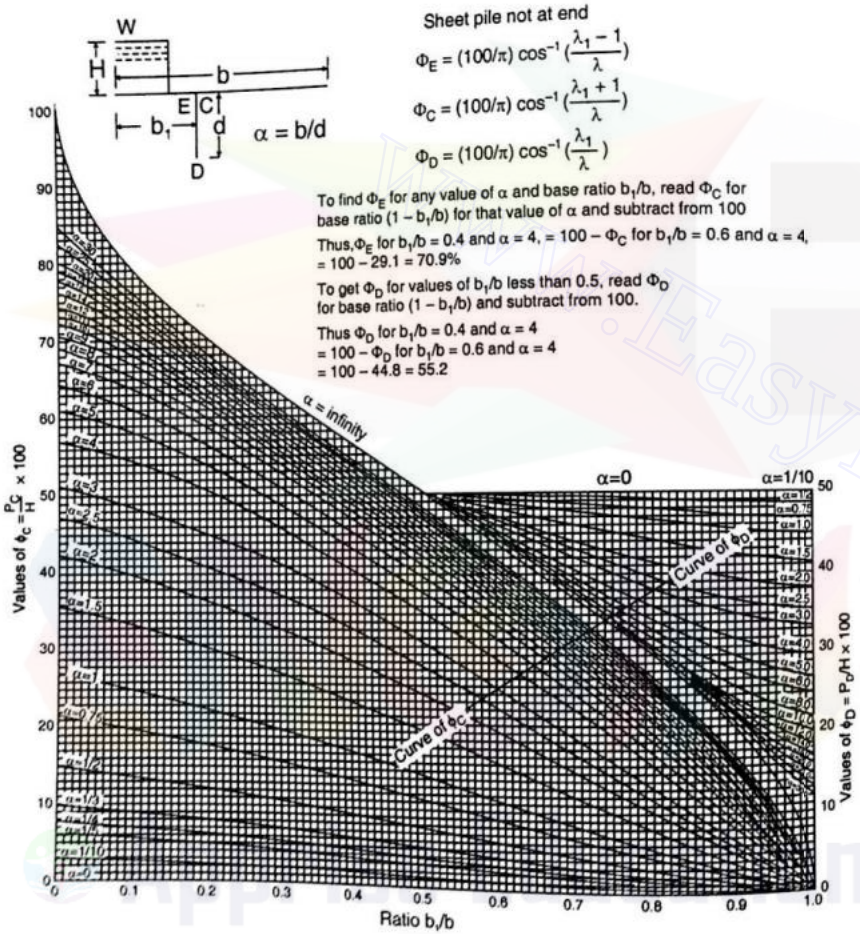

FIG. 12.13. KHOSLA'S CURVES FOR INTERMEDIATE SHEET PILE

Case 2. Pile at downstream end (Fig. 12.12 *a*)

Article 4 of Appendix -1 gives an independent solution for the case of an impervious floor with pile (or cutoff) at the downstream end (Fig. 12.12 *a* and Fig. 4 of Appendix-1). The uplift pressure at the key points *E*, *D* and *C* are given by the following equations

$$P_E = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda - 2}{\lambda}\right] \text{ or } \Phi_E = \frac{P_E}{H} \times 100 = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda - 2}{\lambda}\right] \quad ...(12.7\ a)$$

$$P_D = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda - 1}{\lambda}\right] \text{ or } \Phi_D = \frac{P_D}{H} \times 100 = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda - 1}{\lambda}\right] \quad ...(12.7\ b)$$

$$P_C = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda}{\lambda}\right] = 0 \text{ or } \Phi_C = \frac{P_C}{H} \times 100 = 0 \quad ...(12.7\ c)$$

where $$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} \text{ and } \alpha = b/d \quad ...(12.7\ d)$$

Fig. 12.14 gives the curves for finding out the values of Φ_E and Φ_D for the case of cutoff at down stream end. This Fig. also gives the curve for finding out the uplift pressure Φ_D' at the point *D'* situated at the downstream end of a *depressed floor*, having finite thickness (or depression) *d*.

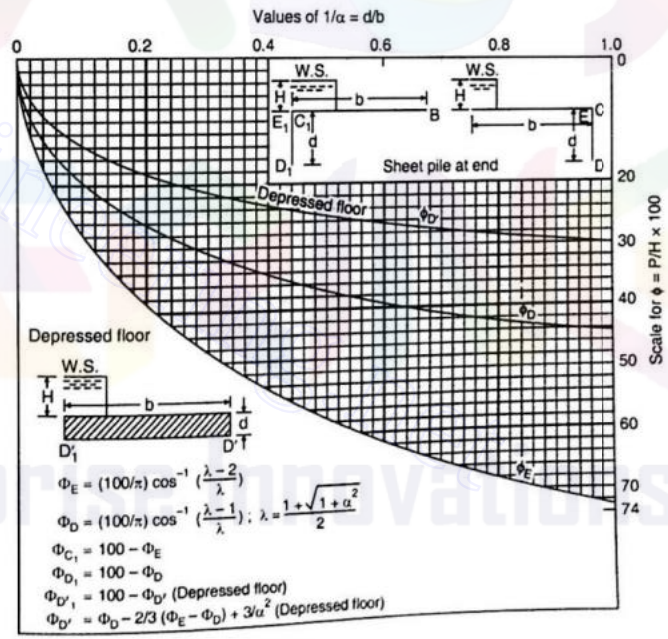

FIG. 12.14. KHOSLA'S CURVES FOR CUTOFF AT D/S END.

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For illustration, let us take the case of a horizontal floor of length 20 m with a pile of length 4 m at the downstream (d/s) end.

Then $\alpha = \frac{b}{d} = \frac{20}{4} = 5$ and $\frac{1}{\alpha} = \frac{1}{5} = 0.2$.

Hence from the curves (Fig. 12.14) for this case, when $1/\alpha = 0.2$

we get $\Phi_D = 27\%$ or $P_D = 0.27\,H$

and $\Phi_E = 40\%$ or $P_E = 0.40\,H$

Case 3 Pile at the upstream end (Fig. 12.12 *b*)

If, however, the pile is provided at the upstream end (and not at the downstream end), the pressure at the key points E_1, D_1 and C_1 are given by the following equation.

$$P_{E1} = H \qquad \text{...(12.8 a)}$$

$$P_{D1} = \frac{H}{\pi}\cos^{-1}\left[\frac{1-\lambda}{\lambda}\right] \text{ or } \Phi_{D1} = \frac{100}{\pi}\cos^{-1}\left[\frac{1-\lambda}{\lambda}\right] \qquad \text{...(12.8 b)}$$

$$P_{C1} = \frac{H}{\pi}\cos^{-1}\left[\frac{2-\lambda}{\lambda}\right] \text{ or } \Phi_{C1} = \frac{100}{\pi}\cos^{-1}\left[\frac{2-\lambda}{\lambda}\right] \qquad \text{...(12.8 c)}$$

where $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$ and $\alpha = b/d$

However, Khosla did not provide separate curves for Φ_{D1} and Φ_{C1} but recommended to use the principle of reversibility of flow along with curves of Φ_C and Φ_D of Fig. 12.14 (Sheet pile at the downstream end). The principle of reversibility of flow gives the following relations for Φ_{C1} and Φ_{D1} :

$$\Phi_{C1} = 100 - \Phi_E \text{ and } \Phi_{D1} = 100 - \Phi_D$$

Let us take the example of an impervious floor of 20 m length with upstream cutoff of 4 m depth, so that $\alpha = b/d = 20/4 = 5$. The pressures at key points C_1 and D_1 are calculated by first finding Φ_D and Φ_E (by assuming as if the pile is at the d/s end) and then using the above relations.

Thus for $1/\alpha = 0.2$, we have $\Phi_D = 27\%$ and $\Phi_E = 40\%$

$\therefore$ $\Phi_{C1} = 100 - \Phi_E = 100 - 40 = 60\%$ of H

and $\Phi_{D1} = 100 - \Phi_D = 100 - 27 = 73\%$ of H

Exit gradient

For the case of horizontal impervious floor with cutoff at the down stream end (Fig. 12.12 *a*), the *exit gradient* (G_E) is given by the following expression :

$$G_E = \frac{H}{d}\cdot\frac{1}{\pi\sqrt{\lambda}} \qquad \text{...(12.9)}$$

where $\lambda = (1+\sqrt{1+\alpha^2})/2$ and $\alpha = b/d$

The proof of the above expression for G_E is given is article 4 to Appendix-1 at the end of the book. Fig. 12.15 gives the curves for variation of λ versus α and variation of the factor $1/\pi\sqrt{\lambda}$ versus α, with the help of which, exit gradient for the given values of parameters b, d and H can be found.

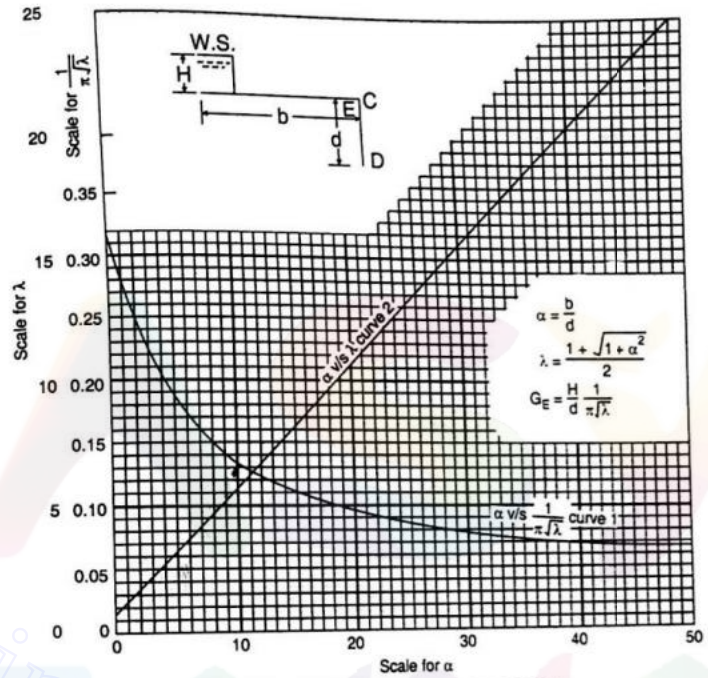

FIG. 12.15. CURVES FOR EXIT GRADIENT

For example, if $b = 20$ m, $d = 4$ m, and $H = 5$ m, we have $\alpha = b/d = 20/4 = 5$. Hence from the exit gradient curve, for $\alpha = 5$, we get $1/\pi\sqrt{\lambda} \simeq 0.185$. Substituting these values in Eq. 12.9, we get

$$G_E = \frac{H}{d}\cdot\frac{1}{\pi\sqrt{\lambda}} = \frac{5}{4}\times 0.185 \simeq 0.23$$

Alternatively, G_E can be computed analytically by first computing λ. Thus

$$\lambda = (1+\sqrt{1+\alpha^2})/2 = (1+\sqrt{1+5^2})/2 = 3.0495$$

$$\therefore \quad G_E = \frac{H}{d}\cdot\frac{1}{\pi\sqrt{\lambda}} = \frac{5}{4}\times\frac{1}{\pi\sqrt{3.0495}} = 0.228$$

Depressed floor A depressed floor is a straight horizontal apron or floor of finite thickness (or depression) d penetrating into the foundation. Khosla also solved this case empirically and gave a curve for uplift pressure Φ_D' at point D'. This curve is shown in Fig. 12.14. This curve is based on the following empirical expression :

$$\Phi_D' = \Phi_D - \frac{2}{3}(\Phi_E - \Phi_D) + 3\,\alpha^2 \qquad \text{...(12.10)}$$

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where α, Φ_E and Φ_D are the corresponding values for an equivalent horizontal apron of negligible thickness of length b with d/s cutoff of depth d.

For example, let us take the case of a depressed floor, having thickness $d = 4$ m and length $b = 10$ m. Then $\alpha = b/d = 10/4 = 2.5$ and $1/\alpha = 1/2.5 = 0.4$. Hence we get from curve for depressed floor, $\Phi_D' = 23.5\%$.

From the principle of reversibility of floor,

$$\Phi_{D_1}' = 100 - \Phi_D' = 100 - 23.5 = 76.5\%$$

The exit gradient for the case of depressed floor is given by

$$G_E = \frac{0.84}{d} \times P_D' \quad \text{...(12.11)}$$

where $$P_D' = \Phi_D' \times H \quad \text{...(12.11 a)}$$

Permissible Exit Gradient

For alluvial soils, the critical hydraulic gradient may be approximately equal to 1. The permissible hydraulic gradient can be found by adopting a suitable factor of safety of 5 to 7. The permissible exit gradient for three soils are given in Table 12.3.

TABLE 12.3. PERMISSIBLE EXIT GRADIENT

Type of soil	G_E
1. *Fine sand*	$\frac{1}{6}$ to $\frac{1}{7}$
2. *Coarse sand*	$\frac{1}{5}$ to $\frac{1}{6}$
3. *Shingle*	$\frac{1}{4}$ to $\frac{1}{5}$

METHOD OF INDEPENDENT VARIABLES

Fig. 12.12 shows some specific cases only. In actual practice, we may have a number of piles, (*i.e.* at u/s, d/s and intermediate points), and the floor also has some thickness. Khosla solved the '*actual problem*' by an empirical method known as the *method of independent variables*. This consists of breaking up a complex profile into a number of simple profiles (such as shown in Fig. 12.12), each of which is independently amenable to mathematical treatment, and then applying the corrections due to the mutual interference of pile and due to the thickness and slope of floor.

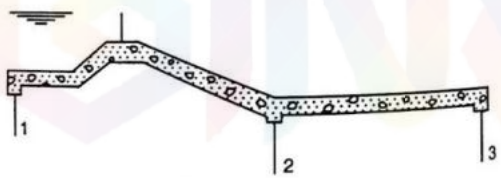

FIG. 12.16.

Thus, to analyse the flow for the floor shown in Fig. 12.16 the complex profile can be broken up into the following simple profiles, and pressures at 'key points' can be obtained :

(*i*) Straight floor of negligible thickness with pile at u/s end.

(*ii*) Straight floor of negligible thickness with pile at the intermediate point.

(*iii*) Straight floor of negligible thickness with pile at the d/s end.

The pressures obtained at the 'key points' by considering the simple profile are then corrected for the following :

(1) Correction for thickness of the floor.

(2) Correction for the mutual interference of the piles.

(3) Correction due to the sloping floor.

(1) Correction for the thickness of the floor

Fig. 12.13 and 12.14 give pressure at key points assuming thickness of the floor to be negligibly small. Thus, the pressure at the key points E and C pertain to the level at the top of the floor, while actually the junction of the pile is at the bottom (points E_1 and C_1) of the floor. The pressures at actual points E_1 and C_1 are computed by considering *linear variation* of pressure between point D and the hypothetical points E and C.

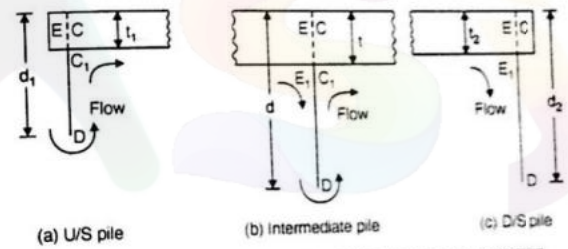

FIG. 12.17. CORRECTION FOR THE FLOOR THICKNESS.

Thus, when the *pile is at the upstream end,*

Correction for $C_1 = \frac{\Phi_D - \Phi_C}{d_1} \times t_1$ (additive)

$\therefore$ Pressure at $C_1 = \Phi_{C1} = \Phi_C + \frac{\Phi_D - \Phi_C}{d_1} \times t_1$

where $t_1 =$ floor thickness and $d_1 =$ depth of u/s pile

For the intermediate pile :

Correction for $E_1 = \frac{\Phi_E - \Phi_D}{d} \times t$ (subtractive)

Correction for $C_1 = \frac{\Phi_D - \Phi_C}{d} \times t$ (additive)

$$\therefore \quad \Phi_{E1} = \Phi_E - \frac{\Phi_E - \Phi_D}{d} \times t$$

$$\Phi_{C1} = \Phi_C + \frac{\Phi_D - \Phi_C}{d} \times t$$

Similarly, when pile is at the d/s end,

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Correction for $E_1 = \dfrac{\Phi_E - \Phi_D}{d_2} \times t_2$ (subtractive)

$$\therefore \quad \Phi_{E1} = \Phi_E - \frac{\Phi_E - \Phi_D}{d_2} \times t_2.$$

(2) Correction for the mutual interference of piles

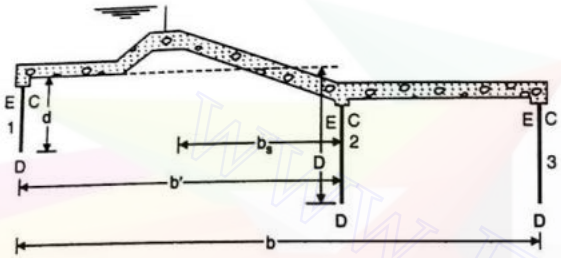

FIG. 12.18. CORRECTION OF MUTUAL INTERFERENCE OF PILES..

The correction (C) given by (Fig. 12.18) :

$$C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) \qquad ...(12.12)$$

where C = percentage correction to be applied to the pressure head.

b' = distance between the piles.

b = total length of the impervious floor.

d = length of the pile on which the effect of another piles of depth D is required to be determined.

D = Depth of pile whose effect is required to be determined on the neighbouring pile of depth d.

Both d and D are measured below the level at which interference is required to be determined, as shown in Fig. 12.18.

The correction is positive for points in the rear or back water and subtractive for points forward in the direction of flow. For example, if we want to find the interference of pile No. 2 on pile 1 (Fig. 12.18), the correction will be positive since the point C of pile 1 (where correction is to be applied) is in the rear with respect to the direction of flow. This is evident since due to the provision of pile No. 2, there will be heading up of pressures at all points to its supstream side. Similarly, if correction is to be determined at point E for pile No. 2 due to interference of pile No. 1, the correction will be subtractive since point E of pile 2 is *forward* in the direction of flow, with respect to pile 1. See example 12.2 for illustration. It should be noted, however, that the above formula does not apply to find the effect of the outer pile on the intermediate pile if the intermediate pile is equal to or smaller then the outer pile and is at a distance less than twice the length of the outer pile. The mutual interference of the

piles is *local* and is to be determined only for the face of the adjacent pile towards the interfering pile. Thus, pile No. 2 will interfere with the d/s face of pile No. 1 and u/s face of pile No. 3. Similarly, pile No. 1 will interfere with u/s face of pile No. 2. There will be no interference of pile No. 1 on the d/s face No. 2 and on the pile No. 3. Also, the mutual interference mainly extend up to a distance equal to the depth of the pile beyond which it gradually falls off till the residual effects at twice that distance is negligible in most cases.

(3) Correction for Slope

The percentage pressure under a floor sloping down or sloping up the direction of flow are respectively greater or less than those under a horizontal floor for the same base ratio. Hence the correction is *plus* for the *down slopes* and minus for the *up slopes* following the direction of flow. The corrections are given in Table 12.4 for various slopes. These corrections are to be further multiplied by the proportion of the horizontal length of the slope to the distance between the two pile lines in between which the sloping floor is located. Thus, for the slope corrections for the point E of pile No. 2 (Fig. 12.18), the correction given by the Table 12.4 are to be multiplied by a factor b_s/b'. *The slope correction is applicable only to that key point of pile line which is fixed at the beginning or the end of the slope.*

TABLE 12.4. SLOPE CORRECTION

Slope (vertical /horizontal)	*Correction (% of Pressure)*
1 in 1	11.2
1 in 2	6.5
1 in 3	4.5
1 in 4	3.3
1 in 5	2.8
1 in 6	2.5
1 in 7	2.3
1 in 8	2.0

Example 12.2. *Fig. 12.19 shows the section of a Barrage. The various dimensions and levels are in metres. Determine the uplift pressure at the key points, and the exit levels are in metres.*

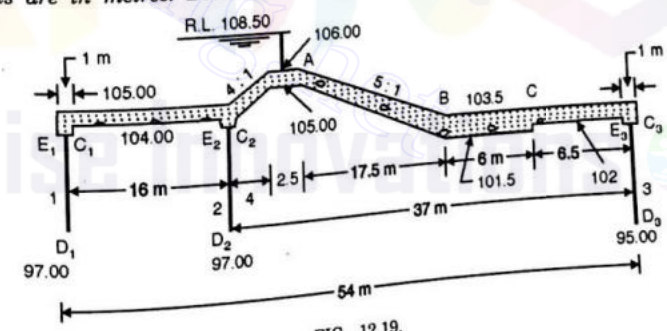

FIG. 12.19.

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gradient. Also find whether the section provided is safe against uplift and piping if it is founded on fine sand with permissible exit gradient of 1/6.

Solution

(1) First pile line

$$d = d_1 = 105.00 - 97.00 \text{ m} = 8.00 \text{ m}$$

$$b = 54 \text{ m} \; ; \; b_1 = 0.5 \text{ m}$$

$$\therefore \quad \alpha = \frac{b}{d} = \frac{54}{8} = 6.75 \; ; \quad \frac{b_1}{b} = \frac{0.5}{54} \simeq 0$$

$\therefore$ From the curves, for $\frac{b_1}{b} = 0$ and $\alpha = 6.75$

$$\Phi_{C1} = 66.5\,\% \; ; \quad \Phi_{D1} = 76\,\% \; ; \quad \Phi_{E1} = 100\,\%$$

Correction for floor thickness,

$$t = 105.00 - 104.00 = 1.00 \text{ m}$$

$$\therefore \quad \text{Correction} = \frac{\Phi_{D1} - \Phi_{C1}}{d_1} t = \frac{(76 - 66.5)}{8} \times 1 \simeq 1.2\,(+)$$

Correction for interference of pile No.2

$$C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

Here $d = 104.00 - 97.00 = 7$ m ; $b' = 16$ m ; $b = 54$ m ; $D = 104.00 - 97.00 = 7$ m

$$\therefore \quad C = 19\sqrt{\frac{7}{16}}\left(\frac{7.00 + 7.00}{54}\right) = 3.3\,\%\,(+)$$

$\therefore$ Corrected pressures are $\Phi_{C1} = 66.5 + 1.2 + 3.3 = 71\,\%$ and $\Phi_{D1} = 76\,\%$.

(2) Intermediate pile line

$$d = d_2 = 105.00 - 97.00 = 8.00 \text{ m} \; ; \; b = 54 \text{ m}; \; b_1 = 16.5 \text{ m}$$

$$\therefore \quad \alpha = \frac{b}{d} = \frac{54}{8} = 6.75 \; ; \; \frac{b_1}{b} = \frac{16.5}{54} = 0.306$$

From curves, for $\alpha = 6.75$ and $\frac{b_1}{b} = 0.306$, we get

$$\Phi_{C2} = 52\,\% \; ; \quad \Phi_{E2} = 72.5\,\% \; ; \quad \Phi_{D2} = 61.5\,\%$$

Correction for floor thickness

$$\text{Correction for } \Phi_{C2} = \frac{61.5 - 52}{8} \times (105.00 - 104.0) \simeq 1.2\,(+)$$

$$\text{Correction for } \Phi_{E2} = \frac{72.5 - 61.5}{8} \times 1 \simeq 1.4\,(-)$$

Correction for interference of pile No.1

Since pile No. 1 and 2 are of the same length, and are fixed at the same level, the correction for Φ_{E2}, due to interference of pile No. 1 is $= 3.3\,\%\,(-)$

Correction for interference of pile No. 3.

$$d = 104.0 - 97.0 = 7.0 \text{ m} \; ; \quad D = 104.0 - 95.0 = 9.0 \text{ m}; \; b' = 37 \text{ m}$$

$$\text{Correction to } \Phi_{C2} = 19\sqrt{\frac{9.0}{37}} \times \left(\frac{7+9}{54}\right) \simeq 2.7\,\%\,(+)$$

Correction due to slope of 4 : 1

Correction (from Table 12.4) $= 3.3\,\%\,(-ve)$

Correction is negative due to upward slope in the direction of flow.

Horizontal length of the slope $= 4$ m

$\therefore$ Actual correction, to be applied to $\Phi_{C2} = 3.3 \times \frac{4}{37} \simeq 0.4\,\%\,(-\text{ve})$

Hence the corrected pressures are

$$\Phi_{E2} = 72.5 - 1.4 - 3.3 = 67.8\,\%$$

$$\Phi_{C2} = 52 + 1.2 + 2.7 - 0.4 = 55.5\,\%$$

(3) Pile No. 3 at the d/s end

$$d = d_3 = 103.5 - 95 = 8.5 \text{ m}$$

$$\therefore \quad \frac{1}{\alpha} = \frac{d}{b} = \frac{8.5}{54} \simeq 0.158.$$

Hence, for curves for pile at d/s end, we get

$$\Phi_{E3} = 35\,\% \; ; \quad \Phi_{D3} = 24.2\,\%$$

$$\textbf{\textit{Correction for thickness}} = \frac{35 - 24.2}{8.5}(103.5 - 102) = 1.9\,\%\,(-\text{ve})$$

Correction for interference of pile No. 2

$$d = 102.0 - 95 = 7 \text{ m} \; ; \; D = 102.0 - 97 = 5 \text{ m}$$

$$\therefore \quad \text{Correction for } \phi_{E3} = 19\sqrt{\frac{5}{37}}\left(\frac{5+7}{52}\right) \simeq 1.5\,\%\,(-)$$

Hence the corrected pressures are :

$$\phi_{E3} = 35 - 1.9 - 1.5 = 31.6\,\%.$$

The percentage pressures and the actual pressure heads at the key points are tabulated below :

TABLE 12.5. MAXIMUM PERCOLATION HEAD = 108.5 - 103.5 = 5 m

Point	*% Pressure (ϕ)*	*Pressure head (P) m*	*Point*	*% Pressure (ϕ)*	*Pressure head (P) m*
C_1	71	3.55	C_2	55.5	2.78
E_2	67.8	3.39	E_3	31.6	1.58

The pressures at intermediate points may be computed by linear variation.

(4) Check for thickness of floor

Let us check the floor thickness at the three points A, B and C (Fig. 12.19).

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Pressure head at $A = P_{C2} - \left(\frac{P_{C2} - P_{E3}}{37} \times 6.5\right) = 2.78 - \frac{2.78 - 1.58}{37} \times 6.5 \simeq 2.57$ m

Pressure head at $B = 2.78 - \frac{2.78 - 1.58}{37} \times 24.0 = 2$ m

Pressure head at $C = 2.78 - \frac{2.78 - 1.58}{37} \times 30 = 1.81$ m

Taking the specific gravity (ρ) of the concrete $= 2.24$, we have

Thickness required at $A = \frac{P_A}{\rho - 1} = \frac{2.57}{1.24} = 2.07$ m.

Actual thickness provided at $A = 107.0 - 105 = 2$ m

Hence increase the thickness at A to 2.1 m

Thickness required at $B = \frac{P_B}{\rho - 1} = \frac{2.0}{1.24} = 1.61$ m

Actual thickness provided at $B = 103.5 - 101.5 = 2$ m

Thickness required at $C = \frac{P_C}{\rho - 1} = \frac{1.81}{1.24} = 1.46$ m

Actual thickness provided $= 103.5 - 102.0 = 1.5$ m

Hence, the floor thicknesses at B and C are adequate.

(4) **Exit gradient**

Seepage head $= H = 108.5 - 103.5 = 5$ m

Depth of cut off $= d = 103.5 - 95 = 8.5$ m

$$\alpha = \frac{b}{d} = \frac{54}{8.5} = 6.35$$

Hence, from exit gradient curve, for $\alpha = 6.35$, $\frac{1}{\pi\sqrt{\lambda}} \simeq 0.165$

Alternatively, $\lambda = (1 + \sqrt{1 + \alpha^2})/2 = (1 + \sqrt{1 + (6.35)^2})/2 = 3.716$

and $\frac{1}{\pi\sqrt{\lambda}} = \frac{1}{\pi\sqrt{3.716}} = 0.165$

$$\therefore \quad G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}} = \frac{5}{8.5} \times 0.165 = 0.097 = \frac{1}{10.3}$$

Permissible exit gradient $= 1/6$. Hence, safe.

12.9. ELEMENTS OF DESIGN FOR SURFACE FLOW

We will consider the following elements of design for surface flow :

1. Design flood discharge
2. Permissible intensity of discharge
3. Afflux
4. Pond level
5. Stable river width : Waterway
6. Protection against scour due to surface flow
7. Hydraulic jump computations and protection against it.

The last item (hydraulic jump computations) has been discussed separately in § 12.11.

1. Design flood discharge

The design discharge adopted for the design of weirs or barrages is usually the *observed* (or recorded) or *computed* maximum discharge. Table 12.6 gives the criteria for fixing maximum flood for this purpose. It should be clearly noted that passage of *smaller floods* than the design flood results in shoal formation and meandering of the river channel while passage of *higher floods* than the design flood result in encroachment upon free board of river training works.

TABLE 12.6. CRITERIA FOR FIXING MAXIMUM FLOOD DISCHARGE

Agency	*Criteria*
1. Central Water Power Commission (CWPC) of India	Standard Project Flood (SPF) or a 100 year flood whichever is higher
2. Indian Standard (IS 6966-1989)	50 to 100 years frequency flood; However, for fixing free board, 500 year frequency flood or Standard Project Flood.
3. Khosla Committee for Engineers	Maximum recorded flood for a period of greater than 50 years

In the above Table, SPF is the flood resulting from most severe combination of hydrological and meteorological factors that are considered reasonably characteristic of the region and is computed by using the Standard Project Storm (SPS). For *pick-up weirs*, a flood of 50-100 years frequency is adopted according to its importance and local conditions.

Experience shows that passage of floods higher than the design discharge *through a barrage* washes off shoals to make available full water way than that provided with a waterway for the maximum flood. Hence, for fixing waterway, a flood of more frequent recurrence interval than the maximum flood is adopted. On the contrary, a flood of long return period is adopted for the structural safety and free board of river training works.

2. Permissible intensity of discharge

Safe permissible intensity of discharge, to determine the length of waterway, depends upon (*i*) nature of river bed material, and (*ii*) allowable afflux depending on topography and economy. The average intensity of discharge per metre of water way between abutments is limited to a safe value so as to prevent scouring of river bed material. In boulder stage, the coarse bed material can withstand higher velocities and higher afflux does not create problem due to inundation of land; hence higher discharge intensity may be allowed. In contrast to this, lower discharge intensity is adopted in the alluvial reach where slopes are flat and river bed material is alluvial.

It should be clearly noted that a higher discharge intensity reduces the length of water way, thus resulting in economy, but has the defect of increasing the cost of protection works against greater scouring tendency and deepening to the downstream floor of the barrage, resulting in increased cost due to dewatering and increased thickness of floor. Table 12.7 gives the data about average discharge intensity provided in some prominent barrages and weirs in India.

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TABLE 12.7. DISCHARGE INTENSITY THROUGH SOME PROMINENT BARRAGES IN INDIA

Weir/barrage	Location	Discharge intensity q (cumecs/m)
Kalabagh	Rajasthan	22
Sarda	U.P.	28
Gandak	Bihar	32
Nangal	Punjab	34
Kosi barrage and Rishikesh barrage	U.P.	40
Veerbhadra barrage	U.P.	67

3. Afflux

Afflux is rise of water level on the upstream of the structure, after the construction of the weir/barrage. During floods, the H.F.L. on the upstream of weir/barrage is higher than that at the downstream. *Afflux* affects the area of submergence and top levels of the marginal banks and guide banks. If the afflux is large, the length of weir will be small because of high discharge intensity over the crest. However, selection of higher value of afflux will increase the cost of river training works and also increase the cost of protection works to the u/s and d/s sides because of increased scour depth. Hence a *safe value* of afflux is adopted. In alluvial sands of Punjab and Haryana, *safe afflux* usually adopted is 1 m while for U.P. it may be upto 2.25 m. IS 6933-1973 has recommended an afflux of 1 m in the case of barrages/weirs on alluvial rivers in upper and middle reaches of the river and 0.3 m in lower river reaches having flat gradients.

4. Pond level

Pond level is the water level in the pond created on the upstream side of the weir, when a part (or full) of the river discharge is being diverted to the canal through the canal head regulator. Pond level is required to be maintained throughout the year, in order to feed the full supply discharge to the offtaking canal. Its value is given by the relation :

Pond level = Full supply level (F.S.L.) of the canal + Head loss thro' regulator.

The head loss through regulator usually ranges from 1.0 to 1.5 m. Pond level is usually fixed considering water level in the winter or that during the low discharge period. Higher the pond level, higher will be the afflux.

5. Stable river width : Recommended waterway

The length of *water way* (W) provided over the weir, to pass the maximum flood discharge should be equal to the *stable river width* for that discharge. This is taken equal to Lacey's wetted perimeter (P) given by the generalised equation :

$$W \triangleq P = C\sqrt{Q} \qquad ...(12.13\ a)$$

where C is the constant in Lacey's equation, the value of which may vary between 3.6 to 6.15, depending upon the discharge and the nature of the bed material. For regime conditions, the best fit value of C may be taken equal to 4.83.

With the *restricted water way* given by the above equation, the shoaling upstream is mostly eliminated and a nearly straight and stable approach to the weir/barrage is secured.

Recommendations of IS : 6966-1989

The recommendations given by IS : 6966-1989 for waterway are as under :

(*i*) In deep and confined rivers with stable banks, the overall water-way between abutments including the thickness of piers is approximately equal to the actual river width at design flood.

(*ii*) For shallow and meandering alluvial rivers, the minimum stable width, as under is calculated from Lacey's formula :

$$W = 4.83\sqrt{Q} \qquad ...(12.13\ b)$$

The regime conditions as envisaged by Lacey can occur only at one discharge, frequency of which will vary for different climate and geological region. The overall width of the weir/barrage actually provided may be more or less than that computed theoretically, using Lacey's regime equation for wetted perimeter (Eq. 12.13 *b*). The ratio of the two is known as the *looseness factor*, the values of which for recently constructed weirs/barrages in India are as under (Table 12.8).

TABLE 12.8. VALUES OF LOOSENESS FACTOR

River stage	Looseness factor
1. Boulder	1.0 to 0.45
2. Alluvial	1.0 to 1.45

It is observed that in weirs constructed with looseness factor greater than 1.5, heavy shoals are invariably formed in the upstream with top upto crest of the weir. Due to these formations, it becomes difficult to feed the canal due to shifting of the channel, caused by shoals. However, restriction of water way (for looseness factor less than 1) removes shoal formation immediately upstream of weir, though at the cost of deep scour formed due to excessive velocities. On the contrary, provision of wider water way (looseness factor greater than 1.0) gives rise to low scour, at the cost of increased cost of construction of weir/barrage and formation of shoals at the u/s side. For weirs and barrages on rivers flowing in alluvial soils, normal practices is to provide a clear water way length of 1.1 to 1.25 times Lacey's regime perimeter obtained for *dominant discharge* (i.e. discharge corresponding to 60 to 75% of observed maximum flood or 1 in 100 years flood in wide and shallow rivers and bankful discharge in incised rivers). More frequent floods higher than the dominant discharge also help in flushing the shoals.

6. Protection against scour due to surface flow

(*a*) *Scour depth*

Due to surface flow, specially during high flood discharge, scouring takes place at upstream and down stream sides of the structure, also around guide banks and other related protection works. The scour depth (R) likely to occur in river bed consisting of cohesive and non-cohesive soils is given in Table 12.9.

The *maximum* depth of scour depends upon type of *reach* as indicated in Table 12.10, as recommended by Lacey.

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TABLE 12.9. SCOUR DEPTH

S.N.	Type of river bed material	Normal scour depth
1.	Cohesive soils (clay etc.)	To be based on actual observations
2.	Non-cohesive soils, when looseness factor is *more* than 1	$R = 0.473\left(\frac{Q}{f}\right)^{1/3}$
3.	Non-cohesive soils, when looseness factor is *less* than 1	$R = 1.35\left(\frac{q^2}{f}\right)^{1/3}$ (Lacey's formula)

TABLE 12.10. RECOMMENDED MAXIMUM SCOUR DEPTH

S.N.	Class	Type of Reach	Maximum scour depth
1.	A	Straight reach	1.25 R
2.	B	Moderate reach	1.50 R
3.	C	Severe bend	1.70 R
4.	D	Right angled bend	2.00 R

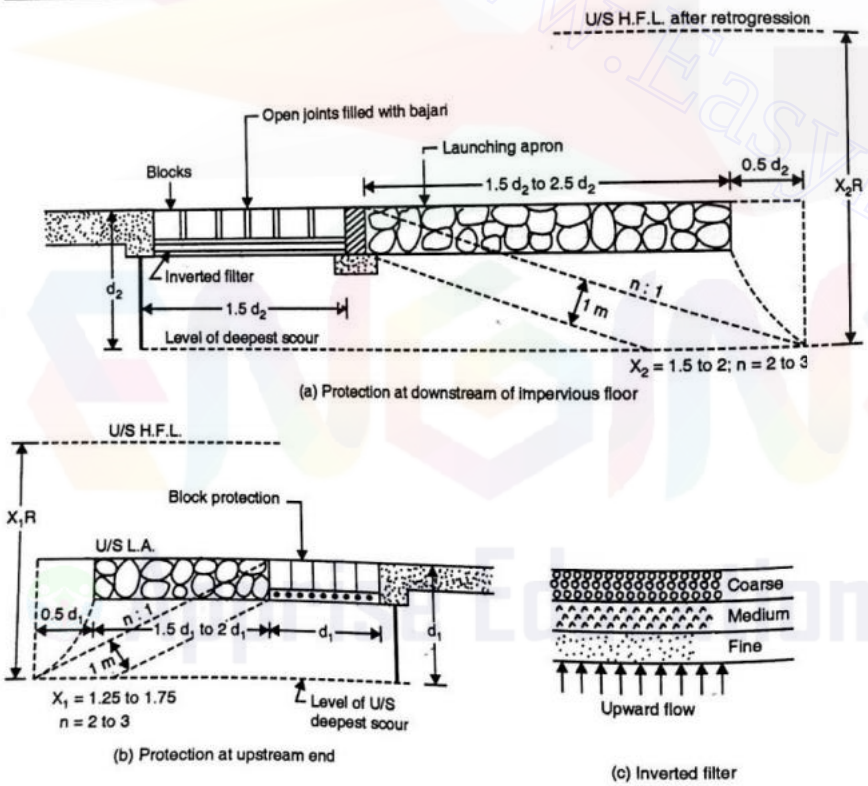

FIG. 12.20. PROVISION AGAINST SCOUR AT D/S AND U/S SIDES

The maximum scour depth indicated in Table 12.10 is measured below the high flood level (H.F.L.)

(b) Provisions against scour at upstream and down stream sides of pucca or impervious floor of weir/barrage

As seen earlier under the design of floor for under-seepage (or subsurface floor), impervious floor of certain length is provided, both at the upstream and downstream side of the main structure. The thickness of the impervious floor varies along its length. However, in order that scouring does not damage this impervious floor, resulting in the collapse of the structure, the following *provisions* are made, as indicated in Fig. 12.20 (*a*) and (*b*).

(*i*) Cutoffs of certain depths, both at the up stream and downstream ends of pucca floor

(*ii*) Provision of block protection at the upstream of the pucca floor.

(*iii*) Provision of inverted filter topped by blocks with open joints at the downstream of pucca floor.

(*iv*) Provision of pervious aprons, commonly known as *launching aprons*, both at the upstream and downstream of the block protections.

1. Upstream and downstream cutoffs

Cutoffs, either in the form of concrete (or masonry) walls or in the form of sheet-piles, are *invariably* provided both at the u/s as well as d/s ends of pucca (or impervious) floor. These are taken upto the level of *deepest scour*. The level of deepest scour is taken equal to $X_2 R$ *below* the d/s H.F.L. (after retrogression) where the value of X_2 is taken between 1.5 to 2. For sloping glacis weir, where energy dissipation is quite efficient due to the formation of hydraulic jump, X_2 is taken equal to 1.5, while X_2 is taken equal to 2 for vertical drop weirs. Similarly, at the upstream side, the level of deepest scour is taken equal to $X_1 R$ below the u/s H.F.L., where the value of X_1 varies between 1.25 to 1.75, the more commonly adopted value being 1.5.

It should be noted that a certain depth of d/s pile is also required to control the exit gradient. Hence the greater of the two depth (i.e. one based on deepest scour level and the other based on exit gradient) should be provided.

2. Block protection at the upstream end of impervious floor

Immediately to the upstream side of u/s cutoff, block protection of length d_1 is provided. The block protection consists of 0.6 to 0.9 m thick stone or concrete blocks laid on 0.3 to 0.6 m thick loosely packed gravel so that the blocks do to not settle in the softened soil below. The purpose of the upstream block protections is to protect the u/s cutoff against twisting / bending due to the u/s launching apron, when the launching apron launches into the scour holes.

3. Inverted filter at the downstream of impervious floor

In order to relieve the uplift pressure due to under seepage water, inverted filter is provided *immedialety* to the downstream of the pucca floor. It should be noted that the direction of flow of seeping water is upwards. In order that this upward flow of water does not carry fines with it, the filter is inverted, with coarser material at

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the top and finer material (slightly coarser than the base material) at the bottom, as shown in Fig. 12.20 (c). In order that thick filter is not washed away during floods, heavy stones or concrete blocks, with open joints, are placed on the top of the inverted filter. The open joints are filled with bajri (clean sand) so that upward flowing water can easily come out, thus dissipating the entire uplift pressure. The length of the filter is taken equal to $1.5\,d_2$. The filter unit may be protected at the end by toe wall extending upto 0.5 m below the bottom of the filter.

4. Launching aprons at downstream and upstream ends

At the end of d/s filter or u/s block protection, launching aprons of loose boulder or stones (of size +300 mm and weighting + 40 kg) or precast cement concrete blocks are provided to protect against the scour holes formed during high floods. The length of down stream launching apron is kept between $1.5\,d_2$ to $2.5\,d_2$, the higher value being adopted for flatter launched slopes. The slope of the launched apron depend on the grade and size of river bed material, as indicated in Table below.

TABLE 12.11. SLOPES OF LAUNCHED APRONS

S.N.	River bed material	Slope (n)
1.	River in alluvium sandy reach	Not steeper than 2:1
2.	River in very fine sand and sixty bed	Not flatter than 3:1
3.	River in boulder reach	Upto 1.5:1

Generally, for sloping glacis weirs, where energy dissipation is more efficient due to formation of hydraulic jump, the d/s launching apron length is taken equal to $1.5\,d_2$ and its launched slope is taken equal to 2 : 1. However for vertical drop weirs, the length of d/s launching apron is taken equal to $2.5\,d_2$ and its launched slope is taken equal to 3:1.

Similarly, the length of u/s launching apron is kept equal to $1.5\,d_1$ to $2\,d_1$, the higher value being adopted for flatter launched slope. Generally, the length of u/s launching apron of sloping glacis weir is taken equal to $1.5\,d_1$, with a steeper launched slope of 2 : 1. For vertical drop weirs, this length is taken equal to $2\,d_1$, with launched slope of 3 : 1.

According to Spring, the quantity of stone in launching apron should be sufficient to provide about 1 m thick cover over the launched slope of n :1 in the launched position.

Thus, if $n = 3$ and length of apron is $2.5\,d_2$, the length of d/s apron in the launched position will be equal to $\sqrt{10}\,d_2$ and the volume of stones in launched position will be equal to $(\sqrt{10}\,d_2 \times 1)\,\text{m}^3$ per metre length. Hence the thickness of apron in the horizontal position will be equal to $\sqrt{10}\,d_2/2.5\,d_2 \simeq 1.27$ m in horizontal position. Similarly, if the length of d/s launching apron is $1.5\,d_2$ and $n = 2$, the length of d/s apron in launched position will be equal to $\sqrt{5}\,d_2$ and the volume of stones in launched position will be equal to $(\sqrt{5}\,d_2 \times 1)\,\text{m}^3$ per metre length. Hence the thickness of the apron in the horizontal position will be equal to $(\sqrt{5}\,d)/1.5\,d_2 = 1.49$ m in horizontal position. Similar computations for thickness can be made for upstream launching apron.

12.10. DESIGN OF VERTICAL DROP WEIR

The complete design of a vertical drop weir consists of the design calculations for the following :

1. Hydraulic calculations for fixing various elevations.
2. Design of weir well.
3. Design of impervious aprons.
4. Design of inverted filter and d/s talus.

1. HYDRAULIC CALCULATIONS

Before starting the design calculations, the following data must be known :

(a) Maximum flood discharge (Q)
(b) H.F.L. before construction of weir
(c) D/S bed level
(d) F.S.L. of canal taking off from the river
(e) Allowable afflux
(f) Lacey's silt factor f

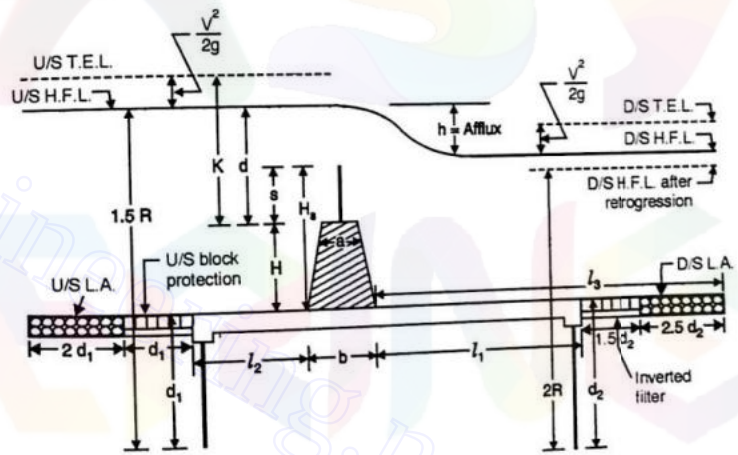

FIG. 12.21. VERTICAL DROP WEIR.

Step (i) : The *length of water way* (L) is calculated from Lacey's regime formula for wetted perimeter (P)

$$P = 4.75\,Q^{1/2} \qquad ...(12.13)$$

The length of water way (L) is usually taken between P and $1.2\,P$. However, in vertical drop weirs, L is taken equal to P. Hence

$$L = 4.75\,Q^{1/2}$$

where L = length of water way in metres
Q = discharge in cumecs.

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Step (ii) : Knowing L, the discharge q per unit width of river is calculated from relation $q = \frac{Q}{L}$ ...(12.14)

Step (iii) : The regime scour depth is calculated from Lacey's formula,

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3} \quad ...(12.15)$$

where f = Lacey's silt factor.

Step (iv) : The regime velocity and velocity head are calculated from the expressions:

$$V = \frac{q}{R} \text{ m/sec} \quad \text{and Velocity head} = \frac{V^2}{2g}.$$

Step (v) : Water levels and total energy levels are calculated as follows :

Level of d/s T.E.L. = (H.F.L. before construction) $+ \frac{V^2}{2g}$

Elev. of u/s T.E.L. = d/s T.E.L. + Afflux

Elev. of u/s H.F.L. = u/s T.E.L. $- \frac{V^2}{2g}$

Step **(vi)** : Discharge over the crest of the weir is given by

$$q = 1.70\,K^{3/2}$$

$$\therefore \quad K = \left(\frac{q}{1.70}\right)^{2/3}$$

where K = total energy head measured above the weir crest.

$\therefore$ Crest level = u/s T.E.L. $- K$

Step (vii) : Pond level = Level of top of gates
= F.S.L. of canal + Head loss through regulator.

Head loss through regulator may be taken as $\frac{1}{2}$ to 1 m.

Height of shutters = s = Level of top of gates − Crest level.

Step (viii) : Protection against scour :

Level of bottom of u/s pile = u/s H.F.L. $- 1.5\,R$

Level of bottom of d/s pile = H.F.L. after retrogation $- 2\,R$.

2. DESIGN OF WEIR WELL

The design of a vertical drop weir wall is similar to that of the overflow section of gravity dam. In the case of a dam, however, the condition of maximum stress is when the water level above the base is maximum. In the case of weir, greater the discharge the smaller is the difference of level between the head and tail water surfaces and the condition of maximum water level above the work generally connotes that of minimum head on the work. *The condition of maximum pressure on the work is that in which the head water is at the level of the crest of the weir or weir shutters and no water is passing over the weir.* However, the condition of maximum stress on the weir may be different in certain cases. Hence, it is necessary to check the stability under three states:

State 1. When the upstream water or *head water* is at crest level or at the level of crest shutter, if any (Fig.12.22(*a*)] and there is no flow.

State 2. When the water is passing over the weir crest and the weir is submerged [Fig.12.22(*b*)].

State 3. When the water is passing over the weir crest and the weir is discharging with a clear overfall [Fig.12.22(*c*)].

Preliminary section of weir wall

The top width a and the bottom width b of the weir wall are selected on the following considerations. The stability of the section is then checked under the above three states of flow.

(*i*) **Top width (*a*)**

$$a = \frac{d}{\sqrt{\rho}} \quad ...(12.16)$$

The above formula is on the considerations of the elementary profile (stress criterion) of the weir.

If we consider the width a of a horizontal joint at crest level to ensure safety against sliding, we get

$$a = \frac{d}{\mu\rho}$$

Assuming $\mu = 2/3$ as the safe value, we get

$$a = \frac{3d}{2\rho} \quad ...(12.17)$$

The practical considerations for the fixation of a are :

1. The height of the weir.
2. Nature of the drift carried by the stream.
3. The width required for the crest shutters or for the traffic of any kind.

For crest shutters of height s,

$$a = s + 1 \text{ meters} \quad ...(12.18)$$

The top width should be greater of the three values of a given by Eqs. 12.16, 12.17 and 12.18.

(*ii*) **Bottom width (*b*)**

The bottom width b can be calculated by equating the overturning moment to the resisting moments taken at the outer third point. To find the moments, we shall consider all the three states mentioned above.

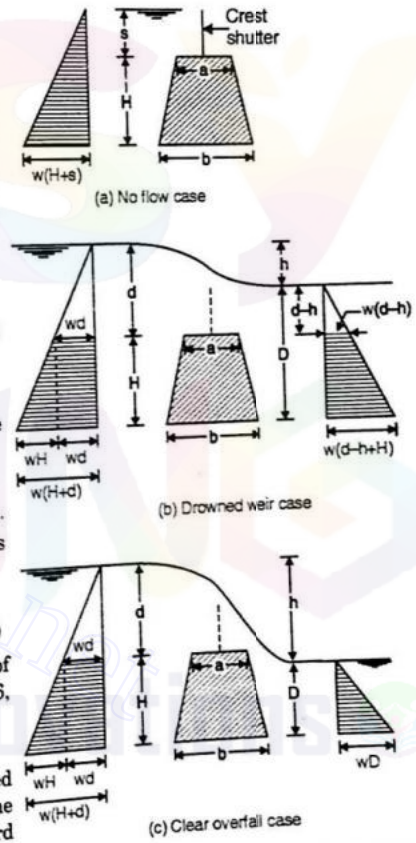

FIG. 12.22. CONDITIONS OF FLOW OVER VERTICAL DROP WEIR.

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(a) *State I : Head water at the crest level or at the level of the crest shutters.*

In this state, the head on the weir, i.e., the difference of levels of the head and tail water is a maximum. Fig. 12.22(*a*) shows the pressure diagram on the u/s face, assuming no water to the d/s side.

The overturning moment is given by

$$M_0 = \frac{w\,(H+s)^3}{6} \qquad ...(12.19)$$

The resisting moment about the outer third point of the base fulfiling the 'middle third rule' is given by

$$M_r = \frac{w}{12}\left[\left\{(\rho+1.5)H+2.5\,s\right\}b^2 + a\,(\rho H - H - s)\,b - \frac{1}{2}a^2\,(H+3s)\right] \qquad ...(12.20)$$

The above expression is valid when both upstream and downstream slopes are kept equal. If, however, the upstream face is kept vertical, the moment of resistance is increased and is given by

$$M_r = \frac{wH\rho}{6}\,(b^2 + ab - a^2) \qquad ...[12.20(a)]$$

Equating the overturning moment to the moment of resistance, the base width b can be determined for State I.

(b) *State II : When the water is passing over the weir crest and the weir is submerged.*

Fig.12.22(*b*) shows the pressure distribution diagrams for this case. The overturning moment is given by

$$M_0 = \left\{\frac{wH^3}{6} + \frac{wdH^2}{2}\right\} - \left\{\frac{wH^3}{6} - \frac{w(d-h)H^2}{2}\right\} = \frac{whH^2}{2} \qquad ...(12.21)$$

For maximum value of M_o, h is taken corresponding to the case when the weir is just submerged.

The moment of resistance, *with tail water at weir crest level,* considering *full floatation* and fulfiling the 'middle third rule' is given by

$$M_r = \frac{wH(\rho-1)}{12}\,(b^2 + ab) \qquad ...(12.22)$$

(for both faces having the same slope)

If the upstream face is vertical

$$M_r = \frac{wH(\rho-1)}{12}\,(b^2 + ab - a^2) \qquad ...[12.22(a)]$$

Equating the overturning moment to the moment of resistance, b can be determined.

(c) *State III : When the water is passing over the weir crest and the weir is discharging with free overfall case.*

Fig. 12.22 (*c*) shows the pressure diagram, from which the overturning moment is given by

$$M_0 = \frac{wH^3}{6} + \frac{wdH^2}{2} - \frac{wD^3}{6} = \frac{w}{6}\left[H^3 + 3\,dH^2 - D^3\right] \qquad ...(12.23)$$

Here there are two variables : d and D. Relation between the two must be known to find the maximum overturning moment. For finding a suitable relation between d and D, river gauging etc. are necessary for the clear overfall case.

When the weir extends the whole width of the river and the latter is of considerable width in comparison to its depth, say width not less than 30 times the depth, the value of d is roughly proportional to that of D, *i.e.*

$$d = kD$$

where k is constant, and can be known if the river discharge for any one depth D is known. Substituting in Eq. 12.23, we get

$$M_0 = \frac{w}{6}\left[H^3 + 3kDH^2 - D^3\right]$$

For maxima $\quad \dfrac{dM_0}{dD} = \dfrac{w}{6}\,(3kH^2 - 3D^2) = 0.$

$$\therefore \quad D = H\sqrt{k} \qquad ...(12.24)$$

Hence, the maximum moment is given by

$$M_0 = \frac{w}{6}\left[H^3 + 3k^{3/2}H^3 - k^{3/2}H^3\right]$$

or

$$M_0 = \frac{wH^3}{6}\left\{1 + 2k^{3/2}\right\} \qquad ...(12.25)$$

The above expression is after S.V.K. Pillai.

3. DESIGN OF IMPERVIOUS APRON

For the under-seepage, the worst condition will be when the water at the u/s is upto the crest of the shutters, and there is no tail water. Thus the percolation head will be H_S (Fig.12.21).

If the floor of the weir is designed on Bligh's creep theory, the total creep length is given by Eq. 12.2, *i.e.*

$$L = CH_S$$

If, however, the floor is to be designed on Khosla's theory, the horizontal length b of the impervious apron is found by the consideration of the permissible exit gradient G_E. From Eq.12.9. $\quad G_E = \dfrac{H_S}{d}\dfrac{1}{\pi\sqrt{\lambda}}$

In this equation G_E, H_S and d are known. Hence, $\dfrac{1}{\pi\sqrt{\lambda}}$ is calculated. From the exit gradient curve knowing $\dfrac{1}{\pi\sqrt{\lambda}}$ the corresponding value of α is known. But $\alpha = b/d$. Knowing α and d, b can be determined.

Out of the total horizontal impervious length b (or the total creep length L), the lengths l_1 , l_2, and l_3 of the floor as indicated in Fig.12.21 are fixed on the basis of Bligh's recommendations given below :

1. **Downstream Impervious Apron (l_1)** For weir crest with shutters

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$$l_1 = 2.21C\sqrt{\frac{H_s}{13}} \text{ in metric units} \quad ...(12.26)$$

For weir with no crest shutters,

$$l_1 = 2.21\,C\sqrt{\frac{H}{10}} \text{ in metric units} \quad ...(12.27)$$

2. **Upstream Impervious Apron**

$$l_2 = L - l_1 - (b + 2d_1 + 2d_2)$$

3. **Total Length of d/s Apron**

The total length l_3 of the d/s impervious floor and the d/s pervious aprons is given by the following :

For weir with crest shutters

$$l_3 = 18C\sqrt{\frac{H_s}{13} \times \frac{q}{75}} \text{ in metric units} \quad ...(12.28)$$

For weir with no crest shutters,

$$l_3 = 18C\sqrt{\frac{H}{10} \times \frac{q}{75}} \text{ in metric units.} \quad ...(12.29)$$

4. **Design of Inverted Filter and Launching Aprons**

An inverted filter is provided immediately at the end of d/s impervious apron to relieve the pressure. The inverted filter consists of 90 to 120 cm thick stones placed on graded filter 50 to 75 cm thick. If d_2 is the depth of the pile below the d/s bed, the recommended length of the inverted filter is 1.5 d_2, as shown in Fig.12.23.

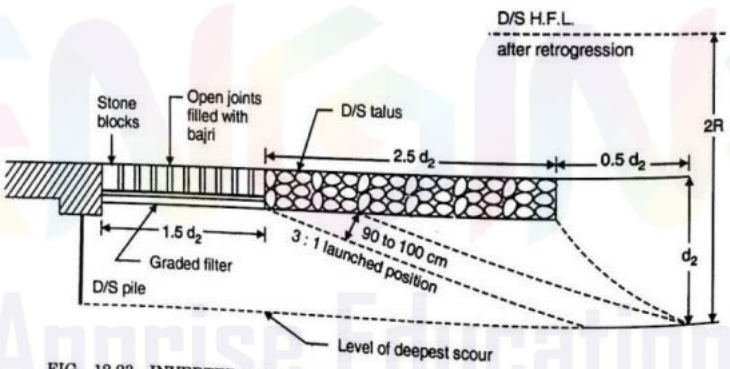

FIG. 12.23. INVERTED FILTER AND LAUNCHING APRON AT D/S END.

After the inverted filter, a length of 2.5 d_2, of launching apron or *talus* is provided. The function of the launching apron is to proctect the d/s pile from the scour holes progressing in the u/s direction. When the scour holes are formed; the stones of the talus settle in the holes and cover them. Spring recommends that in the launched position over a slope of 3 : 1 the apron must have a thickness 3 ft (90 to100 cm).

Knowing the inclined length and the thickness, the total volume of stones can be calculated, and hence the thickness in the horizontal position of length 2.5 d_2, can be easily determined.

A similar launching apron is provided to the u/s side. If d_1 is the depth of the u/s pile, the horizontal length of the launching apron is kept equal to $2d_1$. As in the previous case, the thickness in the launched slope of 3 : 1 is kept equal to 3 ft (90 to 100 cm). In between u/s launching apron and u/s impervious floor, block protection of a length d_1 is provided. The block protection consists of 0.9 to 1 m thick stones or precast blocks placed on 0.3 to 0.6 m thick loosely packed gravel.

DESIGN EXAMPLE OF A VERTICAL DROP WEIR

Example 12.3. *Design a vertical drop weir on Bligh's theory for the following site conditions:*

(a)	*Maximum flood discharge*	= *2800 cumecs*
(b)	*H.F.L. before construction*	= *285.0 m*
(c)	*Minimum water level*	= *d/s bed level = 278.0 m*
(d)	*F.S.L. of canal*	= *284.0 m*
(e)	*Allowable afflux*	= *1 m*
(f)	*Coefficient of creep*	= *12*
(g)	*Permissible exit gradient*	= *1/6*

Assume any other data not given.

Test the floor of the above weir by Khosla's theory and make changes if found necessary.

Solution (Refer Fig.12.21 for symbols used)

(A) **Hydraulic Calculations**

***Step* 1.** $Q = 2800$ cumecs

$$L = 4.75\,Q^{1/2} = 4.75\,(2800)^{1/2} \text{ metres} = 250 \text{ m}$$

$$q = \frac{Q}{L} = \frac{2800}{250} = 11.2 \text{ cumecs/m.}$$

***Step* 2.** *Regime scour depth* : Take $f = 1$

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3} = 1.35\left[\frac{(11.2)^2}{1}\right]^{1/3} = 6.75 \text{ m}$$

$$\text{Regime velocity} = \frac{q}{R} = \frac{11.2}{6.75} = 1.66 \text{ m/second}$$

$$\text{Velocity head} = \frac{V^2}{2g} = \frac{(1.66)^2}{2 \times 9.81} = 0.14 \text{ m}$$

***Step* 3.** Level of d/s T.E.L. = H.F.L. before cons. $+ \frac{V^2}{2g} = 285.00 + 0.14 = 285.14$ m

Afflux = 1 m

Level of u/s T.E.L. = *d/s* T.E.L. + Afflux = 285.14 + 1 = 286.14 m

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$\therefore$ u/s H.F.L. = u/s T.E.L. $- \frac{V^2}{2g} = 286.14 - 0.14 = 286.00$ m

Actual d/s H.F.L., allowing 0.5 m for retrogation = 285.00 − 0.5 = 284.50 m.

Step 4. $q = 1.70\,K^{3/2}$

$$K = \left(\frac{q}{1.70}\right)^{2/3} = \left(\frac{11.2}{1.70}\right)^{2/3} = 3.56 \text{ m.}$$

$\therefore$ Crest level = u/s T.E.L. $- K = 286.14 - 3.56 = 282.58$ m.

Step 5. Pond level = Level of top of gates

= F.S.L. in canal + Head loss through regulator

= 284.00 + 0.50 (say) = 284.50 m

Height of shutter = s = Level of top of gates − Crest level

= 284.50 − 282.58 = 1.92 m

Step 6. R.L. of bottom of u/s pile

= u/s H.F.L. − 1.5 R = 286 − (6.75 × 1.5) = 275.88 m.

Keep the R.L. at 276.0 m

Depth of u/s cut off = 278.0 − 276.0 = 2 m.

Hence, provide concrete cutoff of 2 m depth.

R.L. of bottom of d/s pile

= d/s H.F.L. after retrogation − 2R = 284.50 − (6.75 × 2) = 271.0 m.

Step 7. Seepage head of water

H_S = R.L. of gates crest − R.L .of bed = 284.50 − 278.0 = 6.50 m.

Height of crest H = R.L. of crest − R.L. of bed = 282.58 − 278.0 = 4.58 m.

(Check : $H_S = H + s = 4.58 + 1.92 = 6.50$ m)

(B) Design of Weir Wall

Step 8. Calculation of top width

d = u/s H.F.L. − Crest level = 286.0 − 282.58 = 3.42 m

$\therefore$ Top width $a = \frac{d}{\sqrt{\rho}} = \frac{3.42}{\sqrt{2.24}} = 2.3$ m. ...(i)

From sliding consideration, $a = \frac{3\,d}{2\,\rho} = \frac{3 \times 3.42}{2 \times 2.24} = 2.29$ m ...(ii)

From practical considerations, $a = s + 1 = 1.92 + 1 = 2.92$ m. ...(iii)

Hence, provide top width $a = 3$ m.

Step 9. Calculation of bottom width (b)

Considering state I when the water is to the top of the crest gates, with no tail water, the overturning moment is given by :

$$M_0 = \frac{w\,(H+s)^3}{6} = \frac{wH_S^3}{6} = \frac{9.81 \times (6.5)^3}{6} \simeq 449 \text{ kN-m}$$

The moment of resistance is given by

$$M_r = \frac{w}{12}\left[\left((\rho + 1.5)\,H + 2.5\,s\right) b^2 + a\,(\rho H - H - s)\,b - \frac{1}{2}\,a^2\,(H + 3s)\right]$$

Equating it to the overturning moment and substituting the various values i.e. ($H = 4.58$ m; $s = 1.92$ m; $a = 3$ m; $\rho = 2.24$), we get

$$449 = \frac{9.81}{12}\left[\left((2.24 + 1.5)\,4.58 + 2.5 \times 1.92\right) b^2 + 3\,(2.24 \times 4.58 - 4.58 - 1.92)\;b - \frac{1}{2} \times 9\,(4.58 + 3 \times 1.92)\right]$$

or $549.25 = [21.93\,b^2 + 11.286\,b - 46.53]$, from which $b = 4.96 \simeq 5$ m

Consider state II, when the water is flowing over the weir and the weir is submerged. Considering the tail water just at the crest of the weir, the overturning moment is given by

$$M_0 = \frac{wh\,H^2}{2}$$

When the tail water is at the crest, d and h will be equal. For this case, the value of d, i.e. head over the crest is given by

$$d = \left[\frac{q^2}{(\frac{2}{3}\,C)^2 \times 2\,g}\right]^{1/3} \text{ (neglecting the velocity of approach)}$$

where $C = 0.58$ and $q = 11.2$ cumecs/m

$$\therefore \quad d = h = \left[\frac{(11.2)^2}{(\frac{2}{3} \times 0.58)^2 \times 2 \times 9.81}\right]^{1/3} = 3.5 \text{ m}$$

$$M_0 = \frac{wh\,H^2}{2} = \frac{9.81 \times 3.5\,(4.58)^2}{2} = 360.1 \text{ kN-m}$$

The moment of resistance is given by

$$M_r = \frac{wH(\rho - 1)}{12}\,(b^2 + ab) = \frac{9.81 \times 4.58\,(2.24 - 1)}{12}\,(b^2 + 3b) = 4.643\,(b^2 + 3b)$$

Equating the two, we get $4.643\,(b^2 + 3b) = 360.1$

From which $b \simeq 8$ m

Adopting the greater of the two, we get $b = 8$ m

Thus the weir wall has top width of 3 m and bottom width equal to 8 m, with equal slopes of u/s and d/s faces.

(c) Design of Impervious and Pervious Aprons

Step 10. Assuming $C = 12$, total creep length is

$L = CH_S = 12 \times 6.5 = 78$ m.

Step 11. *Downstream impervious apron :*

$$l_1 = 2.21C\sqrt{\frac{H_S}{13}} = 2.21 \times 12\sqrt{\frac{6.5}{13}} = 18.8 \text{ m} \simeq 19 \text{ m.}$$

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Step 12. Upstream impervious apron (l_2) :

Depth of u/s pile below bed $= d_1 = 278 - 276 = 2$ m

Depth of d/s pile below bed $= d_2 = 278 - 271 = 7$ m.

$$l_2 = L - l_1 - (b + 2d_1 + 2d_2) = 78 - 19 - (8 + 4 + 14) = 33 \text{ m.}$$

Step 13. Total length of d/s apron

$$l_3 = 18\,C\sqrt{\frac{H_s}{13} \times \frac{q}{75}} = 18 \times 12\sqrt{\frac{6.5}{13} \times \frac{11.2}{75}} = 53.2 \text{ m} \triangleq 54 \text{ m.}$$

Step 14. Length of filter + launching apron $= l_2 - l_1 = 54 - 19 = 35$ m

Minimum length of inverted filter $= 1.5\,d_2 = 1.5 \times 7 = 10.5$ m.

Minimum horizontal length of launching apron $= 2.5\,d_2 = 2.5 \times 7 = 17.5$ m.

Total minimum length of both $= 10.5 + 17.5 = 28$ m

But we have to provide 35 m length of both.

Hence, provide the filter of 17.5 m length and launching apron of 17.5 m length.

Make filter of 1 m thick block stones placed on 60 cm thick graded filter material.

Length of launching apron on 3:1 slope $= \sqrt{10}\,d_2 = 7\sqrt{10} = 22$ m.

Keeping 1 m thickness in the launched position, volume of stones per metre length of weir $= 22\text{ m}^3$. Hence thickness in horizontal position $= 22/17.5 = 1.3$ m.

Provide 1.5 m thick launching apron.

Step 15. Upstream block protection and launching apron :

$$d_1 = 2 \text{ m}$$

Length of u/s block protection $= d_1 = 2$ m.

Provide 1 m thick block stones over 0.5 m thick graded stones and gravel.

$\therefore$ Length of u/s Talus $= 2d_1 = 4$ m.

Volume of stones on 3 : 1 launched position, with 1 m thickness

$$= (\sqrt{10}\,d_1 \times 1) = 6.28\text{ m}^3$$

$\therefore$ Thickness of apron in horizontal position

$$= \frac{6.28}{4} \triangleq 1.5 \text{ m.}$$

Step 16. Thickness of impervious floor. Provide a nominal thickness of 1 m to the u/s of the weir wall and 1.5 m below the weir wall.

Residual pressure at point A (Fig. 12.24) just at the d/s of the weir wall

$$= H_S - \frac{H_S}{L}x = 6.5 - \frac{6.5}{78}(2 \times 2 + 33 + 8)$$

or

$$H_r = 6.5 - \frac{6.5}{78} \times 45 = 6.5 - 3.9 = 2.6 \text{ m}$$

$\therefore$ Thickness

$$= \frac{4}{3}\,\frac{H_r}{\rho - 1} = \frac{4}{3} \times \frac{2.6}{2.24 - 1} = 2.8 \text{ m.}$$

Hence, provide a thickness 2.8 m from the d/s of weir wall (point A) to a point B, 6 m from it.

Total length of d/s impervious floor $= l_1 = 19$ m.

Pressure at 6 m from the d/s end of the weir wall :

$$H_r = 6.5 - \frac{6.5}{78}(45 + 6) = 2.08 \;;\quad t = \frac{4}{3}\,\frac{2.08}{1.24} = 2.25 \text{ m.}$$

Hence, provide $t = 2.30$ m from 6 to 12 m from d/s end of weir wall.

Pressure at 12 m from the end of the weir wall $= 6.5 - \frac{6.5}{78}(45 + 12) = 1.56$ m.

$$t = \frac{4}{3}\,\frac{1.56}{1.24} = 1.68 \text{ m.}$$

Hence, provide $t = 1.7$ m for the rest of the length of the floor.

(D) Check by Khosla's Theory

(i) Check for the exit gradient :

Total horizontal length of the impervious floor $= 33 + 8 + 19 = 60$ m

Depth of downstream pile $= 7$ m

$$\therefore \quad \alpha = \frac{b}{d} = \frac{60}{7} \triangleq 8.6.$$

Hence, from Khosla's exit gradient curves,

For $\alpha = 8.6$, $\frac{1}{\pi\sqrt{\lambda}} = 0.14$; $G_E = \frac{H_s}{d}\,\frac{1}{\pi\sqrt{\lambda}} = \frac{6.5}{7} \times 0.14 \triangleq \frac{1}{7.8}$.

Permissible exit gradient $= 1/6$. Hence, safe.

(ii) Check for thickness of the floor :

Pressure at points C_1 *and* D_1 *of the first pile* :

$$\frac{d_1}{b} = \frac{1}{\alpha} = \frac{2}{60} = \frac{1}{30} = 0.033$$

$$\therefore \quad \Phi_{D1} \approx 88\,\% \;;\; \Phi_{C1} \triangleq 83\%.$$

Correction for thickness for $\Phi_{C1} = \frac{88 - 83}{2} \times 1 = 2.5\,\%\,(+)$

Correction for interference of pile 2 on $\Phi_{C1} = 19\sqrt{\frac{D}{b'}}\,\frac{d + D}{b}$

Here $b = b' = 60$ m, $D = 7$ m, $d = 2$ m.

$$\therefore \quad \text{Correction} = 19\sqrt{\frac{7}{60}}\left(\frac{9}{60}\right) = 0.97\,\%\,(+)$$

$\therefore$ Corrected $\Phi_{C1} = 83 + 2.5 + 0.97 \triangleq 86.5\,\%$

$\therefore$ Pressure head at $C_1 = H_S \times \Phi_{C1} = 6.5 \times 0.865 \triangleq 5.63$ m ...(ii)

Pressure at point E_2 and D_2 of the second pile

$$\frac{d_2}{b} = \frac{1}{\alpha} = \frac{7}{60} = 0.117$$

$$\therefore \quad \Phi_{E2} = 31\,\% \text{ and } \Phi_{D2} = 21\,\%$$

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Correction for thickness of floor to Φ_{E2} ;

Floor thickness provided near the pile = 1.7 m (Bligh's)

$$\therefore \quad \text{Correction} = \frac{31-21}{7} \times 1.7 = 2.43\ \%\ (-\text{ve})$$

Correction for interference of first pile on $\Phi_{E2} = 19\sqrt{\frac{D}{b'}}\ \frac{d+D}{b}$

Here $D = 2$ m ; $d = 7$ m ; $b = b' = 60$ m.

$$\therefore \quad \text{Correction} = 19\sqrt{\frac{2}{60}}\left(\frac{7+2}{60}\right) = 0.53\ \%\ (-\text{Ve})$$

$\therefore$ Corrected $\Phi_{E2} = 31 - 2.43 - 3.53 = 28.04\ \%$

$\therefore$ Pressure head at $E_2 = 6.5 \times 0.2804 = 1.83$ m.

Assuming the linear variation of pressure for intermediate points, the pressures and floor thickness for points A, B and C are as under :

Point A, just to the d/s of weir wall :

$$P_A = 5.63 - \frac{5.63 - 1.83}{60}(33 + 8) = 3.03 \text{ m}$$

$$\therefore \quad t_A = \frac{3.03}{1.24} = 2.45 \text{ m.}$$

Thickness provided by Bligh's theory = 2.8 m. Hence safe.

Thus provide thickness of 2.8 m for a length of 6 m from A to B.

Point B, 6 m from d/s end of the weir wall :

$$P_B = 5.63 - \frac{5.63 - 1.83}{60}(33 + 8 + 6) = 2.66 \text{ m}$$

$$\therefore \quad t_B = \frac{2.66}{1.24} = 2.14 \text{ m.}$$

Thickness provided by Bligh's theory = 2.30 m. Hence, safe.

Thus provide a thickness of 2.3 m for a length of 6 m from B to C

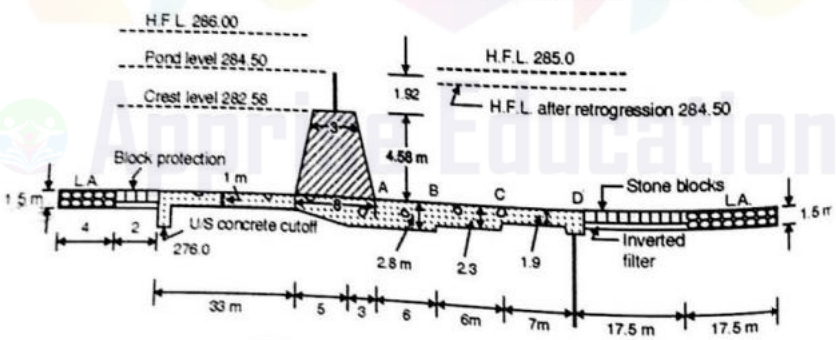

FIG. 12.24. VERTICAL DROP WEIR.

Point C, 12 m from d/s and of the weir wall :

$$P_C = 5.63 - \frac{5.63 - 1.83}{60}(33 + 8 + 12) = 2.27 \text{ m.}$$

$$\therefore \quad t_C = \frac{2.27}{1.24} = 1.83 \text{ m.}$$

Thickness provided by Bligh's theory = 1.7 m. Hence, unsafe.

Increase the thickness to 1.9 m from C to D for a length of last 7 m of the impervious floor. The final section of the weir is shown in Fig. 12.24.

12.11. DESIGN OF SLOPING GLACIS WEIR

Sloping glacis weir is a weir of later origin consisting of (*i*) upstream glacis of slope varying from 1:1 to 2:1 (*ii*) flat crest of 2 to 3 m width with falling shutters, (*iii*) down stream glacis of slope varying from 3:1 to 5:1 and (*iv*) horizontal apron or cistern of adequate length, along with cutoff, block protection, inverted filter and launching aprons, as shown in Figs. 12.4 (*a*). The main advantage of a sloping glacis weir is that hydraulic jump is formed on the d/s sloping glacis to dissipate the energy of overflowing water. Suction pressure is created in the vicinity of formation of hydraulic jump. Hence we shall first study various elements of hydraulic jump, along with computation of suction pressure.

1. Elements of hydraulic jump :

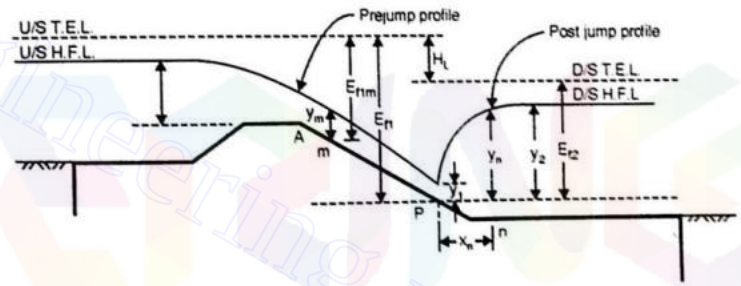

FIG. 12.25. ELEMENTS OF HYDRAULIC JUMP

It can be found from any standard book on hydraulics/fluid mechanics that the conjugate depths y_1 and y_2, before and after the hydraulic jump are related by the equation:

$$y_1\ y_2\ (y_1 + y_2) = \frac{2q^2}{g} \qquad \text{...(12.30)}$$

where q is the discharge intensity.

The loss of energy (H_L) due to the formation of hydraulic jump is given by

$$H_L = \frac{(y_2 - y_1)^3}{4\ y_1\ y_2} \qquad \text{...(12.31)}$$

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Also, the velocities V_1 and V_2, before and after the jump are given by

$$V_1 = q/y_1 \quad ...(12.32)$$

and

$$V_2 = q/y_2 \quad ...(12.33)$$

Thus, there are six elements of hydraulic jump, i.e. q, H_L, y_1, V_1, y_2 and V_2 which are related by the above four equations. If out of the above six elements, any two elements (such as q and H_L) are known, the other four elements can be computed with the help of the above four equations. However, the simultaneous solution of the above equations is quite tedious. However, the solution can be easily obtained with the help of Blench curves (Fig. 12.26) which relate q and H_L to the specific energy E_{f2} after the jump, which in turn is related to y_2 and V_2 as under :

$$E_{f2} = y_2 + V_2^2/2g \quad ...(12.34)$$

Thus, knowing q and H_L, E_{f_2} can be found from Blench curves (Fig. 12.26) and hence specific energy E_{f_1} before the jump is found from the relation :

$$E_{f1} = E_{f2} + H_L \quad ...(12.35)$$

Thus, specific energies E_{f_1} and E_{f_2} before and after the jump are known. The pre-jump and post jump depths y_1 and y_2 are then computed from the following basic relations :

$$E_{f1} = y_1 + \frac{V_1^2}{2g} = y_1 + \left(\frac{q}{y_1}\right)^2 \frac{1}{2g} \quad ...(12.36)$$

and

$$E_{f2} = y_2 + \frac{V_2^2}{2g} = y_2 + \left(\frac{q}{y_2}\right)^2 \frac{1}{2g} \quad ...(12.37)$$

The solution of above (cubic) equations is very tedious, since it involves trial and error steps. However, the solution of the above equations can be simplified by using Montague's curves (Fig. 12.27 *a*) which directly give the values of y_1 and y_2 for different values of E_{f_1} and E_{f_2}. Fig. 12.27 (*b*) gives Montague's curves on enlarged scale for y upto 1.2 m. In the Montague's curves the dividing line gives the locus of critical depth (y_c), while the depth y_1 is the depth in the super critical stage ($y < y_c$) and the depth y_2 is in sub-critical stage ($y > y_c$) of the curves. However, the determination from Montagues' curves gives only rough values of y_1 and y_2. Fine values of y_1 and y_2 can finally be obtained from the above two equations (Eqs. 12.36 and 12.37) through trial and error process.

The level of downstream horizontal apron upto which sloping glacis is provided is found by deducting E_{f_2} from the downstream T.E.L. If this is provided, the hydraulic jump will be formed just at the end of the sloping glacis. A slightly lower level of the horizontal floor will ensure that the jump will be formed at the point P on the glacis. The length of the jump is usually taken equal to $5(y_2 - y_1)$. Hence in order obtain turbulence free flow on the downstream side, the length of the horizontal apron should be atleast equal to $5(y_2 - y_1)$, though this length can be reduced by provision of chute blocks, basin blocks and dentated sill, as discussed in Chapter 11.

FIG. 12.26 BLENCH CURVES

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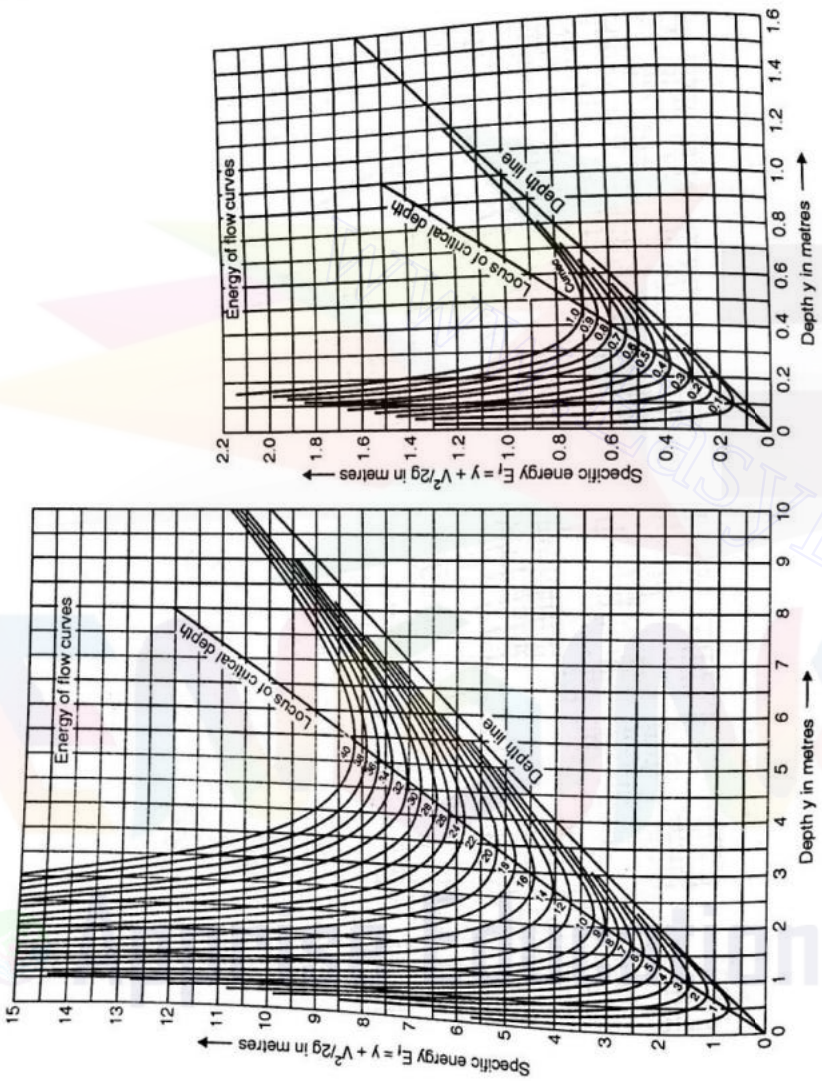

FIG. 12.27. MONTAGUE'S CURVES

2. Determination of water surface profile before the jump:

In order to determine the water surface profile before the hydraulic jump, let us choose any point m on the glacis (Fig. 12.25). It is required to find the depth y_m of the water surface measured above point m.

Procedure

1. Find the elevation of the point m on the d/s sloping glacis, from the geometry.
2. Find specific energy E_{f1m} at point m by subtracting the elevation of m from the u/s T.E.L.
3. Knowing E_{f1m}, find the values of y_m at m either from Montague curve or analytically from Eq. 12.36.
4. Determine the water surface elevation at m by the relation :

   Water surface elevation at point m = R.L. of $m + y_m$
5. Vary the position of point m along the glacis from point A to point P and repeat the above procedure.
6. Join the points corresponding to the water surface elevations so found by a smooth curve, thus getting the prejump profile.

3. Determination of water surface profile after the jump

Procedure :

(*i*) Compute the Froude number F_1 of the incoming flow, using the following relation

$$F_1 = \frac{V_1}{\sqrt{g\,y_1}} = \frac{q}{\sqrt{g\,y_1^3}} \qquad ...(12.38)$$

(*ii*) Select a point n at a horizontal distance x_n from the point P where the jump is located.

(*iii*) Find the value of x/y_1 for point n where $x = x_n$ and y_1 is the prejump depth.

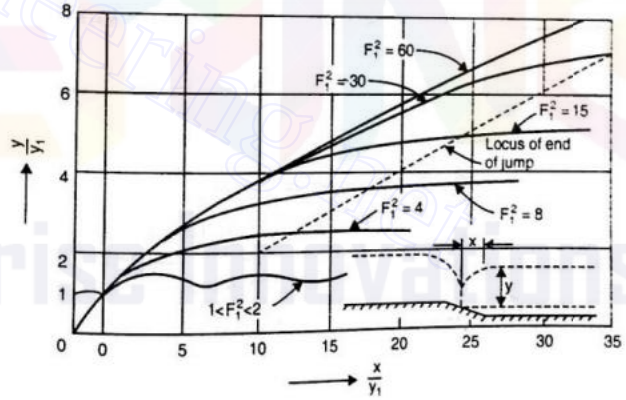

FIG. 12.28. CURVE FOR DETERMINATION OF WATER SURFACE PROFILE AFTER THE JUMP

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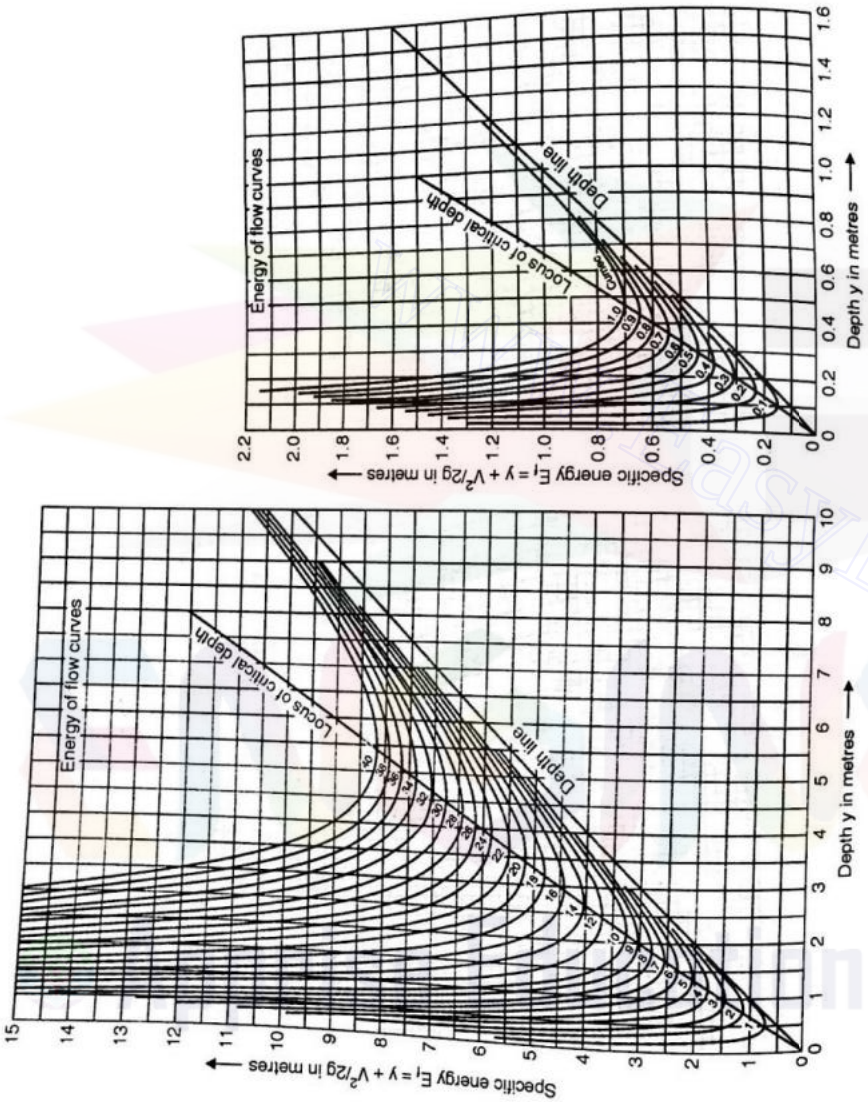

FIG. 12.27. MONTAGUE'S CURVES

2. Determination of water surface profile before the jump:

In order to determine the water surface profile before the hydraulic jump, let us choose any point m on the glacis (Fig. 12.25). It is required to find the depth y_m of the water surface measured above point m.

Procedure

1. Find the elevation of the point m on the d/s sloping glacis, from the geometry.
2. Find specific energy E_{f1m} at point m by subtracting the elevation of m from the u/s T.E.L.
3. Knowing E_{f1m}, find the values of y_m at m either from Montague curve or analytically from Eq. 12.36.
4. Determine the water surface elevation at m by the relation :
   Water surface elevation at point m = R.L. of $m + y_m$
5. Vary the position of point m along the glacis from point A to point P and repeat the above procedure.
6. Join the points corresponding to the water surface elevations so found by a smooth curve, thus getting the prejump profile.

3. Determination of water surface profile after the jump

Procedure :

(i) Compute the Froude number F_1 of the incoming flow, using the following relation

$$F_1 = \frac{V_1}{\sqrt{g\,y_1}} = \frac{q}{\sqrt{g\,y_1^3}} \qquad ...(12.38)$$

(ii) Select a point n at a horizontal distance x_n from the point P where the jump is located.

(iii) Find the value of x/y_1 for point n where $x = x_n$ and y_1 is the prejump depth.

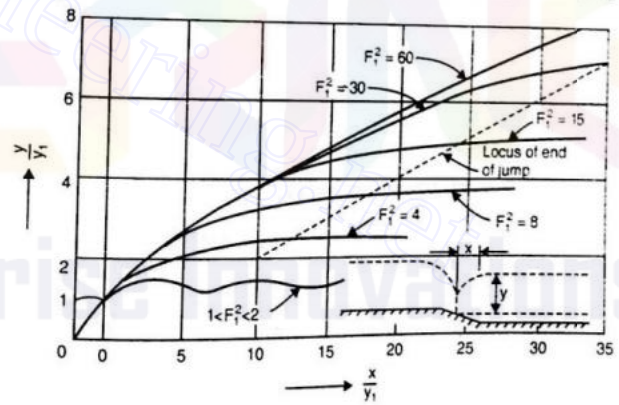

FIG. 12.28. CURVE FOR DETERMINATION OF WATER SURFACE PROFILE AFTER THE JUMP

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(*iv*) Find the value of y/y_1 for this point, corresponding to the Froude number F_1 using curves of Fig. 12.28, for the above value of x/y_1.

Thus $y_n = y$ ordinate at $n = y_1 \times$ value so obtained.

(*v*) Water surface elevation = R.L. of $P + y_n =$ U/S TEL $- E_{f_1} + y_n$

(*vi*) Vary the position of n along the horizontal apron and find the water surface elevation for each position by repeating the above steps. Join these points by a smooth curve to get the water surface profile after the jump.

4. Determination of suction pressure in hydraulic jump trough

The uplift pressure which occurs in the *trough* of the hydraulic jump is called the *suction pressure* or the *dynamic pressure*. The suction pressure head (h_{us}) at any point in the trough is equal to the vertical ordinate between the subsoil hydraulic gradient line and the water surface profile before or just at the point of formation of hydraulic jump, as shown in Fig. 12.29 (*b*). Because of the trough formation, this ordinate is quite large at and near the point *P*.

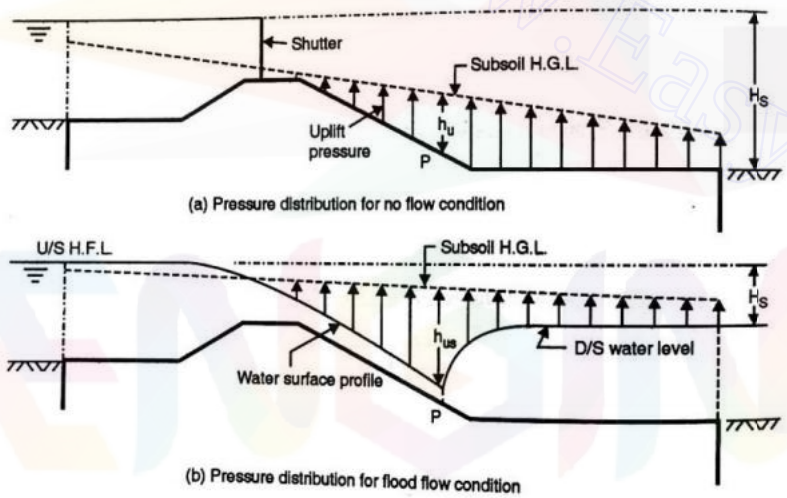

FIG. 12.29. UPLIFT PRESSURE DISTRIBUTION

Fig. 12.29 (*a*) shows the subsoil hydraulic gradient line (H.G.L.) for no flow condition. The uplift pressure (h_u) at any point *P* is equal to the vertical ordinate between the floor level and subsoil H.G.L. Fig. 12.29 (*b*) shows the subsoil H.G.L. along with water surface profile before and after the jump, for flood flow condition. The suction pressure h_{us} at any point is equal to the vertical ordinate between the water surface profile and the subsoil H.G.L. This pressure is maximum at the point *P* of the formation of hydraulic jump. Comparison of both diagrams shows that uplift pressure is greater for flood flow condition, specially at and near the vicinity of point *P* of the jump formation. The floor and glacis should be sufficiently thick to resist this suction pressure. However,

for design purposes, the actual suction pressure (or uplift pressure) is usually taken equal to two-thirds of the theoretical value (h_{us}) so obtained. *This is because of the following reasons :*

(*i*) Reduction of actual suction pressure due to back ward rolling of flow.

(*ii*) Reduction of suction (or uplift) pressure due to the vertical component of momentum, and

(*iii*) Monolithic casting of the concrete floor/glacis leading to beam bending action.

The floor thickness at any point on the d/s glascis and horizontal apron is found from the relation.

$$t = \frac{h}{\rho - 1}$$

where $h = \frac{2}{3} h_{us}$ for flood flow condition and $h = h_u$ for no flow condition.

5. D/S and U/S Protection against scour

The following provisions are made against scour at the upstream and downstream sides of impervious apron :

(*i*) Min. Depth of d/s cutoff : 1.5 *R* to 2.0 *R* below d/s H.F.L. after retrogression.

(*ii*) Min. depth of u/s cutoff : 1.25 *R* to 1.5 *R* below u/s H.F.L.

(*iii*) Length of d/s filter unit : 1.5 d_2 where d_2 is the depth of d/s cutoff below d/s floor.

(*iv*) Length of d/s launching apron : 1.5 d_2, with minimum thickness of 1 m on launched slope of 2:1.

(*v*) Length of u/s block protection : 1.0 d_1 where d_1 is the depth of U/s cutoff below u/s floor.

(*vi*) Length of u/s launching apron : 1.5 d_1 with min. thickness of 1 m on launched slope of 2:1.

DESIGN EXAMPLE OF A SLOPING GLACIS WEIR

Example 12.4. *Design a sloping glacis weir for the following site conditions :*

1. *Maximum discharge intensity on weir crest = 10 cumecs/m length*
2. *H.F.L. before construction of weir = 255.0 m*
3. *R.L. of river bed = 249.5 m*
4. *Pond level = 254.0 m*
5. *Height of crest shutters = 1 m*
6. *Anticipated downstream water level in the river when the weir is discharging with pond level upstream = 251.5 m.*
7. *Bed retrogression = 0.5 m*
8. *Lacey's silt factor = 0.9*
9. *Permissible exit gradient = 1/7*
10. *Permissible afflux = 1 m.*

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Solution

***Step* 1.** Pond level = 254.0 m

Height of shutters = 1 m

∴ Crest level = 254.0 – 1 = 253.0 m

***Step* 2.** $q = 1.70\,(K)^{3/2}$

$$\therefore \quad K = \left(\frac{q}{1.70}\right)^{3/2} = \left(\frac{10}{1.70}\right)^{2/3} = 3.25 \text{ m}$$

∴ Elevation of u/s T.E.L. = R.L. of crest + K = 253.0 + 3.25 = 256.25 m.

***Step* 3.** Regime scour depth $R = 1.35\left(\frac{q^2}{f}\right)^{1/3} = 1.35\left(\frac{10^2}{0.9}\right)^{1/3} \simeq 6.5$ m

$$\therefore \quad \text{Regime velocity} = \frac{q}{R} = \frac{10}{6.5} = 1.54 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V^2}{2g} = \frac{(1.54)^2}{2 \times 9.81} = 0.12 \text{ m.}$$

***Step* 4.** U/S H.F.L. = u/s T.E.L. – 0.12 = 256.25 – 0.12 = 256.13 m.

Level of d/s T.E.L. = H.F.L. before construction $+ \frac{V^2}{2g}$

= 255.0 + 0.12 = 255.12 m

∴ Afflux = u/s H.F.L. – d/s H.F.L. = 256.13 – 255.0 = 1.13 m.

which is very near to the permissible afflux of 1 m.

The d/s H.F.L. after retrogression = H.F.L. before construction – 0.5

= 255.0 – 0.5 = 254.5 m.

D/S T.E.L. after retrogression = 255.12 – 0.5 = 254.62 m.

∴ Loss of head at high flood is

H_L = u/s T.E.L. – d/s T.E.L. after retrogression

= 256.25 – 254.62 = 1.63 m.

***Step* 5.** When the weir is discharging with pond level upstream,

Head over the crest = 254.0 – 253.0. = 1 m

∴ Discharge $q = 1.70\,(K)^{3/2} = 170(1)^{3/2} = 1.70$ cumecs/metre.

Loss of head at that time will be given by

H_L = pond level – Anticipated d/s water level at that time

= 254.0 – 251.5 = 2.5 m.

Step* 6. *Hydraulic jump calculations

Hydraulic jump will be formed on the d/s sloping glacis. The hydraulic jump calculations are done in the tabular form (Table 12.12).

TABLE 12.12. HYDRAULIC JUMP CALCULATIONS

S.No.	Items	High flood condition	Pond level condition
1.	Discharge intensity q	10 cumecs	1.70 cumecs
2.	Head loss H_L	1.63 m	2.50 m
3.	Downstream Specific Energy (E_{f2}) from Blench curves (See Fig. 12.26)	4.30	1.70
4.	Level at which jump will form : (d/s T.E.L. after retrogression – E_{f2})	254.62 – 4.30 = 250.32 m	251.5 – 1.7 = 249.8 m
5.	Upstream Specific Energy $E_{f1} = E_{f2} + H_L$	4.30 + 1.63 = 5.93 m	1.70 + 2.50 = 4.20
6.	Prejump depth D_1 corresponding to E_{f1} from specific energy curve (See Fig. 12.27) or calculated from the Eq. $E_{f1} = D_1 + \frac{(q/D_1)^2}{2g}$	1.03 m	0.15 m
7.	Post jump depth D_2 corresponding to E_{f2} from specific energy curves or calculated from the Eq. $E_{f2} = D_2 + \frac{(q/D_2)^2}{2g}$	3.96	1.68 m
8.	Height of jump = $D_2 - D_1$	3.96 – 1.03 = 2.96 m	1.68 – 0.15 = 1.53 m
9.	Length of concrete floor required beyond the jump = 5 ($D_2 - D_1$)	14.65 m Adopt 15 m length	7.65 m

Step* 7. *Design of glacis

Let the crest width = 2 m Upstream slope 2 : 1

Downstream glacis slope 3 : 1

Adopt downstream floor R.L. = 249.6 m (*i.e.* 0.2 m below the lowest level at which jump will form).

Horizontal length of floor beyond the toe of the glacis = 15 m.

Step* 8. *Depth of sheet piles

R = 6.5 m.

∴ R.L. of bottom of scour hole at u/s = 256.13 – 1.5(6.5) = 246.38 m.

∴ Keep R.L. of bottom of u/s pile at 246.30 m.

R.L. bottom of scour hole at d/s = 254.5 – 2 × 6.5 = 241.5 m.

Keep R.L. of bottom d/s pile at 241.5 m.

Provide the intermediate pile at d/s toe of the glacis as the second line of defence, with its bottom at the same R.L. of 241.5 m.

Step* 9. *Length of the impervious floor

Maximum percolation head = Pond level – d/s floor level

or H_S = 254.0 – 249.6 = 4.4 m.

Depth of d/s cut off = d = 249.6 – 241.5 = 8.1 m.

Permissible $G_E = \frac{1}{7}$ where $G_E = \frac{H_S}{d}\frac{1}{\pi\sqrt{\lambda}}$

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$$\therefore \quad \frac{1}{\pi\sqrt{\lambda}} = G_E \frac{d}{H_S} = \frac{1}{7} \times \frac{8.1}{4.4} = 0.263$$

Hence, from Khosla exit gradient curve

$$\text{For} \quad \frac{1}{\pi\sqrt{\lambda}} = 0.263, \ \alpha = 1.5 = \frac{b}{d}$$

$\therefore$ b = length of impervious floor $= \alpha \cdot d = 1.5 \times 8.1 = 12.2$ m

Floor length already provided from other considerations

= (horizontal length of u/s glacis) + (length of crest)

+ (horizontal length of d/s glacis)

+ (length of floor beyond the toe of a glacis)

$= [2(253.0 - 249.5)] + [2] + [3(253.0 - 249.6)] + [15] = 7 + 2 + 10.2 + 15 = 34.2$ m.

Thus the floor length already provided is much more than that required from permissible exit gradient considerations and no upstream floor is required. However, provide a minimum of 1.8 m length of u/s floor so that the total length of impervious floor becomes 36 m.

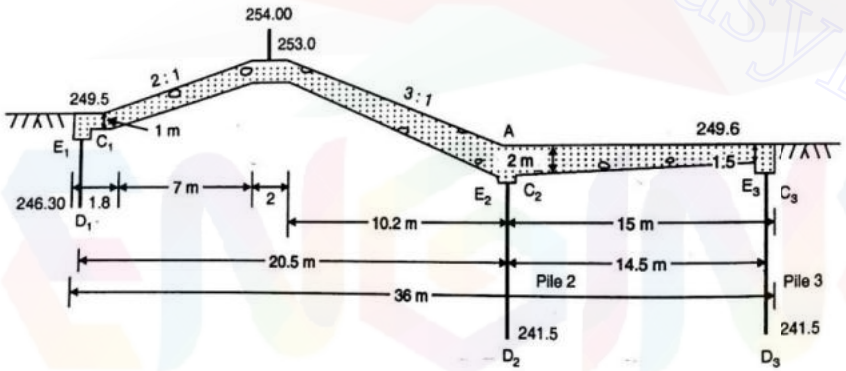

FIG. 12.30.

***Step* 10.** Having known the profile of the glacis and the floor (Fig. 12.30), let us now calculate the pressure at 'key points' by assuming the preliminary thickness of the floor at various points as under.

Floor thickness at u/s of crest = 1.0 m

Floor thickness at d/s end of glacis = 2.0 m (at intermediate pile)

Floor thicknes at d/s end of floor = 1.5 m

The calculation of pressures at various key points have been done in Table 12.13. Refer Fig. 12.30.

TABLE 12.13. CALCULATIONS FOR SUBSOIL UPLIFT PRESSURE (LENGTH OF FLOOR b = 36 m)

Pile line and the pressure point	*Depth of pile d*	$\frac{1}{\alpha} = \frac{d}{b}$	b_1	$\frac{b_1}{b}$	$\alpha = \frac{b}{d}$	*Percentage pressure*				
						Before correction	*Correction for thickness*	*Correction for slope*	*Correction for interference*	*After correction*
Upstream pile	3.2	0.089	...	...	...	...	...	...	...	...
Point C_1	...	...	...	...	...	72	+ 3.1	...	+ 3.7	78.8
Point D_1	...	...	...	...	...	82	...	...	...	82
Intermediate pile	8.1	...	21	0.583	4.44	...	...	...	...	...
Point C_2	...	...	...	...	...	31.5	+ 3.5	...	+ 6.4	41.4
Point D_2	...	...	...	...	...	45.5	...	...	...	45.5
Point E_2	...	...	...	...	...	58.5	− 3.2	+ 2.3	− 2.4	55.2
Downstream pile	8.1	0.225	...	...	...	...	...	...	...	...
Point D_3	...	...	...	...	...	29	...	...	...	29
Point E_3	...	...	...	...	...	44	− 3.6	...	− 6.4	34

Step* 11. *Elevation of subsoil H.G. Lines for various conditions. After having known the % pressures at various key points, the elevation of subsoil H.G. line for (*i*) maximum static head, (*ii*) high flood condition and (*iii*) flow at pond level condition can be determined as illustrated in Table 12.14.

TABLE 12.14. ELEVATION OF SUBSOIL H.G. LINES FOR VARIOUS FLOW CONDITIONS

Condition	*Datum (d/s floor level or d/s water level) (m)*	*u/s water level (m)*		*Ht. / Ele. of subsoil H.G. line above datum (metres)*								
				u/s pile line			*Intermediate pile line*			*d/s pile line*		
				Φ_{E_1} = 100%	Φ_{D_1} = 82%	Φ_{C_1} = 78.8%	Φ_{E_2} = 55.2%	Φ_{D_2} = 45.5%	Φ_{C_2} = 41.4%	Φ_{E_3} = 34%	Φ_{D_3} = 29%	Φ_{C_3} = 0%
No flow condition $H_s = 254.0 - 249.6 = 4.4$ m	249.60	254.00	Ht.	4.40	3.61	3.46	2.43	2.00	1.82	1.50	1.28	0
			Ele.	254.00	253.21	253.06	252.03	251.60	251.42	251.10	250.88	249.60
High flood condition $H_s = 256.13 - 254.5 = 1.63$ m	254.50	256.13	Ht.	1.63	1.34	1.28	0.90	0.74	0.67	0.55	0.47	0
			Ele.	256.13	255.84	255.78	255.40	255.24	255.17	255.05	254.97	254.50
Flow at pond level $H_s = 254.0 - 251.5 = 2.5$ m	251.50	254.00	Ht	2.50	2.05	1.97	1.38	1.14	1.04	0.85	0.72	0
			Ele.	254.00	253.55	253.47	252.88	252.64	252.54	252.35	252.22	251.50

Step 12. Plotting the prejump profile for the two flow conditions.

The uplift pressure at any point will be equal to the difference in elevation between the subsoil H.G. line and the elevation of water surface corresponding the jump profiles

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for the two flow conditions (or for the d/s floor for no flow conditions). Hence, it is essential to plot the water surface profile of the jump.

Plotting of the water surface on the d/s glacis, before the formation of the jump can be done by finding depths D_1 at various points with the help of Montague curves (Fig. 12.27). The calculations are arranged in Table 12.15.

TABLE 12.15. PRE-JUMP PROFILE (R.L. OF CREST = 253.0 m)

Distance from d/s end of crest (m)	*R.L. of glacis (m)*	*High flood condition q = 10 u/s T.E.L. = 256.25*		*Pond level flow q = 1.70 u/s H.F.L. = 254.0*	
		E_{f1} = 256.25 – glacis level	D_1	E_{f1} = 254.0 – glacis level	D_1
3.0	252.0	4.25	1.3	2.00	0.31
6.0	251.0	5.25	1.15	3.00	0.23
8.04 (jump location for high flood)	250.32	5.93	1.03	—	—
9.0	250.00			4.00	0.16
9.60 (jump location for pond level flow)	249.9			4.25	0.15

Step* 13. *Calculations for floor thickness. Fig. 12.31(*a*) shows the subsoil H.G. line for the two conditions. The floor is to be designed for the maximum unbalanced head found by substracting elevation of water surface profile (or d/s bed level as the case may be) from the elevation of the corresponding H.G. line.

(*i*) *At the point of formation of hydraulic jump for high flood condition :*

Unbalanced head for high flood condition (from Fig. 12.31 *a*)

$$= 4 \text{ m}$$

Unbalanced static head for no flow condition

$$= 2.56 \text{ m}$$

$\therefore$ Design head = 4 m

Since the design head is greater for high flood condition, the floor is designed for 2/3rd of this head

$$= \frac{2}{3} \times 4 = 2.67 \text{ m.}$$

$$\therefore \quad \text{Thickness} = \frac{2.67}{2.24 - 1} = 2.15 \text{ m.} \quad (\rho = 2.24)$$

(*ii*) *At the point of formation of hydraulic jump for flow at pond level condition:*

Unbalanced head for pond level condition = 2.9 m

2/3 of this head = 1.93 m

Unbalanced static head for no flow condition = 2.2 m

$\therefore$ Design head = 2.2 m

$$\text{Thickness} = \frac{2.2}{1.24} = 1.8 \text{ m}$$

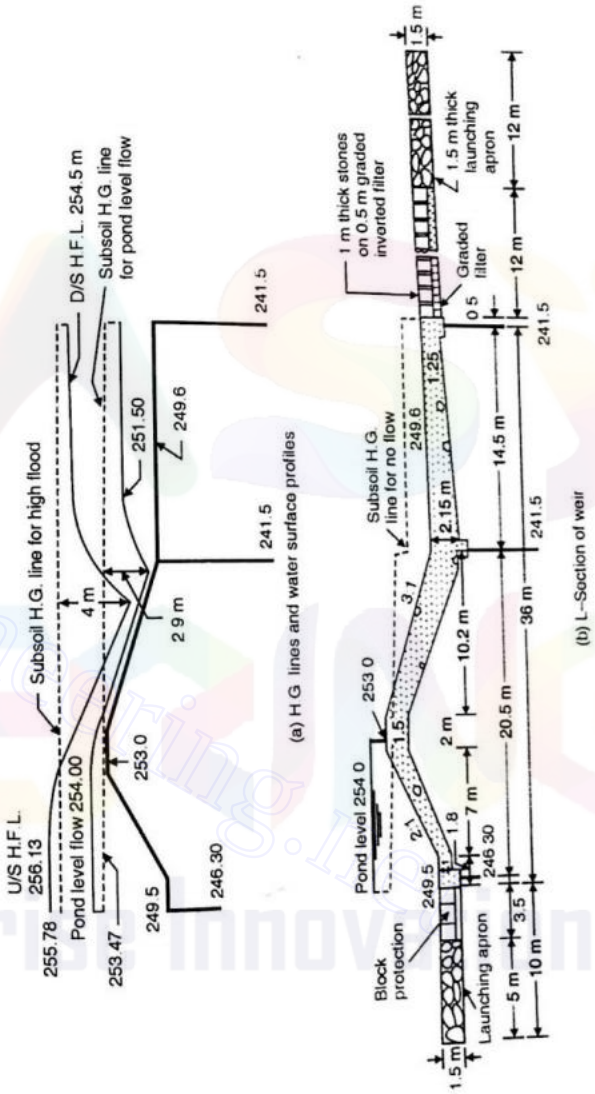

FIG. 12.31 SLOPING GLASCIS WEIR

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This is less than the thickness of 2.15 m found for high flood condition determined above.

Hence, provide a thickness of 2.15 m at the d/s end of the glacis decreasing it to 1.5 m at the end of the crest. A nominal thickness of 1 m is to be provided for the floor to the u/s of the glacis, as shown in Fig. 12.31(*b*).

(*iii*) *Floor thickness at down stream side of sloping glacis*

It will be seen from Fig. 12.31(*a*), (*b*) that the design head for the floor to the d/s of the sloping glacis will be governed by the unbalanced static head which is greater than 2/3 the unbalanced head under the two flow conditions.

Pressure head at point E_3 at d/s end of the floor = 1.5 m.

Thickness required $\frac{1.5}{1.24} = 1.2$ m.

Thus the d/s floor has a thickness of 2.15 m at the d/s end of glacis, and a thickness of 1.25 m at the d/s pile. This reduction in thickness may be done linearly as shown in Fig. 12.31(*b*).

Step 14. Upstream protection works :

D = depth of u/s scour holes below bed level

$= 249.5 - 246.38 = 3.12$ m.

Minimum length of u/s block protection $= D = 3.12$ m.

Provide 1 m thick stone blocks laid over 0.5 m thick graded stones for a length of 3.5 m.

Minimum length of u/s launching apron $= 1.5\,D = 1.5 \times 3.12 \simeq 4.7$ m with a thickness of 1 m on a launched slope of 2 : 1.

However, provide 1.5 m thick launching apron for length of 5 m at the u/s end.

Step 15. Downstream protection works :

D = depth of d/s scour holes below bed level $= 249.6 - 241.5 = 8.1$ m,

Length of inverted filter $= 1.5\,D \simeq 12$ m.

Provide 12 m length of inverted filter consisting of 1 m thick stones laid on 0.5 m thick graded inverted filter.

Length of d/s launching apron $= 1.5\,D = 1.5 \times 8.1 \simeq 12$ m. Its minimum thickness on a launched slopes should be 1 m.

Hence, provide 12 m length of launching apron 1.5 m thick.

The longitudinal section of the weir is shown in Fig. 12.31(*b*).

12.12. DIVIDE WALL

A divide wall or 'groyne' is an embankment, protected on all sides by stones or concrete blocks. It is built at right angles to the axis of the weir separating the weir and the under sluices. The divide wall extends upstream to a little beyond the beginning of the head regulator and downstream to the 'talus' or 'launching apron' of under sluice portion.

The divide wall is normally constructed of solid masonry. It is necessary to provide well foundation for at least 30 m length from the extreme and taking the well below the deepest possible scour. The divide wall should be designed for the following conditions:

(1) Silt pressure upto the full tank level on the pocket side when the river level is low. At this time there is no water on the weir side.

(2) During floods or high river stage, when the under sluices are discharging, a difference of water level of about 1 m, with higher water level on the weir side be taken for design purposes.

(3) The top width of the wall is kept 1.5 to 2.5 m, while the bottom width is worked out so as to contain the resultant within the middle third of the base.

Functions of the divide wall

(1) The floor level of the under sluices or the pocket portion is generally kept lower than the floor level of the main weir. Hence, a divide wall is essential to separate the two floors. This prevents the turbulent action.

(2) If divide wall is not provided, currents approach the scouring sluices from all directions and their effectiveness is reduced. The divide wall helps in concentrating scouring action of the under sluices for washing out the silt deposited in the pocket by ensuring a straight approach through the pocket.

(3) The divide wall prevents cross-current and flow parallel to the weir. The cross currents and the flow parallel to weir cause formation of vortices and result in deep scour. To prevent this, sometimes, divide walls are also provided at intervals along the main portion of a long weir or at points of change of floor level at both main portion of a long weir or at points of change of floor level both on u/s and d/s side.

(4) The divide wall provides a comparatively 'still pocket' in front of canal head regulator. This helps in more silt deposit in the pocket and entry of comparatively clear water into the canal.

(5) Divide wall incidently serves as one of the side walls of the fish ladders.

12.13. THE FISH LADDER

In big rivers, migrating fishes move from upstream to down stream in search of relatively warm water in the beginning of winter season. Before the monsoons (in May and June) they travel upstream in search of clear water. It is, therefore, essential to make the provision of some space in the construction of the weir for the uninterrupted movements of such migratory fish.

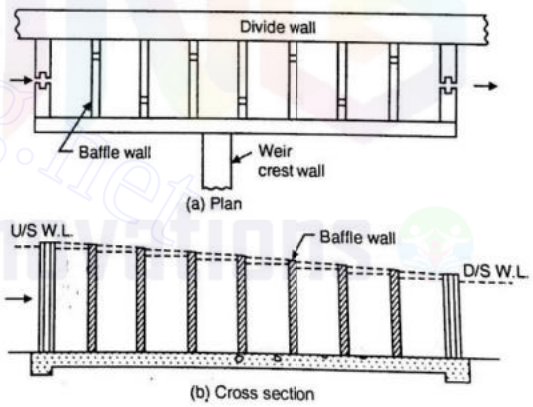

FIG. 12.32. FISH LADDER.

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Fig. 12.32 shows the plan and cross-section of a fish ladder. The difference of water levels on the upstream and downstream sides on the weir is split up into water steps by means of the baffle walls constructed across the inclined chute of fish ladder. Most fish can travel upstream in water flowing with a velocity of 3 m/second. The design of fish ladder should be such that velocity of current in the chute is not more than this value. To have effective control, a grooved gate is provided at the extreme u/s and d/s end walls of the fish ladder. The fish ladder is situated near the under sluices where there is always some water in the river section below them.

12.14. THE UNDER SLUICES

The under sluices or the scouring sluices maintain a deep channel in front of the head regulator and dispose off heavy silt and a part of flood discharge on the d/s side of the barrage.

Functions of scouring sluices

(1) They preserve a clear and defined river channel approaching the regulator.

(2) They control the silt entry into the canal.

(3) They scour the silt deposited in the river bed above the approach channel.

(4) They pass the low floods without dropping the shutter of the main weir, the raising of which entails goods deal of labour and time.

(5) They provide greater waterway for floods, thus lowering the flood levels.

(6) They help in impounding fair amount of receding flood so as to secure full storage.

Capacity of under sluices

The discharging capacity of the under sluices should be fixed from the following considerations :

(1) To ensure proper scouring, its capacity should be at least double the canal discharge.

(2) The under sluices should have sufficient capacity to discharge winter freshlet (with canal inflow) without the necessity of dropping the weir shutters.

(3) During high floods, they should be capable of passing about 10 to 15 % of the maximum flood discharge.

Design considerations

(1) The crest of the under sluices should be lower than the crest of the head regulator by a least 1 to 1.2 metres if special silt exclusion device is not provided. If a silt excluder is provided this difference in level should be at least 1.8 to 2 m.

(2) The thickness and the length of the impervious floor should be designed on the same lines as the floor of the weir portion.

(3) Since the discharge intensity through the under sluices is higher, heavier protection should be provided to the downstream. The length of the talus below the under sluices may be determined from the Bligh's recommendations as under :

$$l_3 = 27C\sqrt{\frac{H_s}{13} \times \frac{q}{75}} \text{ metres.} \qquad ...(12.39)$$

where q = discharge intensity in cumecs/metre.

(4) The length of the impervious apron upstream of the sluice gate may be determined from the Bligh's recommendations as under :

$$l_2 = 3.9\,C\sqrt{\frac{H_s}{13}} \text{ metres.} \qquad ...(12.40)$$

(5) From the consideration of retrogression, the downstream floor should be depressed upto the likely retrogression bed level downstream.

(6) A bridge downstream of the under sluices should always be provided so that the cranes could move on it to lift the gates if the Stoney's arrangement fail.

12.15. THE CANAL HEAD REGULATOR

A head regulator is a structure constructed at the head of a canal taking off from a reservoir behind a weir or a dam. A head regulator may consist of a number of spans separated by piers and operated by gates similar to that provided in a barrage. The modern tendency is to use bigger spans of 6 to 18 m controlled by Stoney gates or sector gates.

Functions of head regulator

(1) To make the regulation of supply in the canal easy.

(2) To control the silt entry in the canal.

(3) To shut out river floods.

Fig. 12.33 shows the section of canal head regulator.

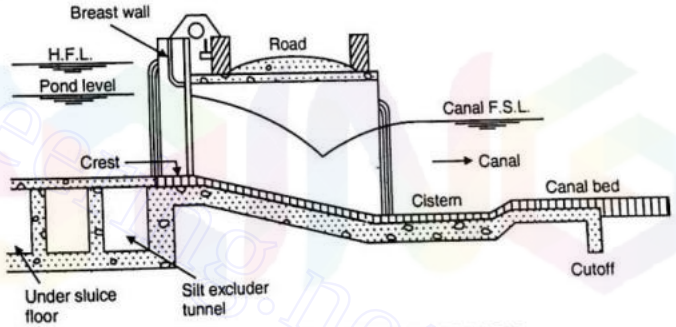

FIG. 12.33. CANAL HEAD REGULATOR.

Design considerations

(1) The water way of the head regulator should be sufficient of pass the full supply discharge of the canal, with ample factor to safety to allow for any silting up of canal. For a head regulator with broad crest and sloping glacis at the downstream, the following discharge equation is applicable :

$$Q = 1.7(L - knH)\,H^{3/2} \qquad ...(12.41)$$

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where Q = total discharge in cumecs
L = length of water way in metres
H = head causing flow
n = number of end contractions
k = constant, depending upon the shape of the nose of the pier ; varies from 0.01 to 0.03.

(2) The regulators are normally aligned at 90° to the weir, but greater angles, upto 110° are considered preferable for providing smooth entry.

(3) The crest level of the head regulator should be higher than the crest of the under sluices by a minimum of 1 to 1.2 metres if silt excluder is not provided and greater than 1.8 to 2 m if silt excluding device is provided.

(4) The hydraulic jump calculations over the sloping glacis are done in the same way as for sloping glacis weir. The length of cistern below the end of the glacis should at least be equal to $5(D_2 - D_1)$. The level of the cistern bed should be well below the minimum level of the formation of hydraulic jump under various discharge conditions.

(5) The design of the impervious floor should be done on the basis of uplift pressure in the manner similar to that of sloping glacis weir. The worst condition will be during the high floods when the canal is shut and maximum static head acts. In case the floor thickness becomes excessive, a reinforced concrete mat should be provided to resist the uplift pressure by bending action.

(6) A concrete cut-off at the end of the impervious floor should always be provided to keep the exit gradient well within the limits.

(7) The piers separating the various spans of the regulator should be extended beyond the sloping glacis so that they may provide support to the upward bending reinforced concrete mat of the cistern floor.

(8) In order to prevent spilling of water towards the canal during high floods, a reinforced concrete breast wall should be provided from pond level to well above the high flood level. The breast wall is supported between the piers, and is designed for its self weight as well as the water pressure from upstream. They may also be designed for additional weight of gate lifting arrangements, if supported on them.

(9) For the proper operations of the regulator, a bridge is provided, spanning over the piers.

(10) The stability of the piers of the head regulator should be tested to withstand the overturning moment caused by the high pressure head during floods.

(11) There is no empirical formula to determine the length of the *talus* of downstream of head regulator. Generally a *talus* of 4 to 5 times the depth of canal and 0.8 to 1 m thick in concrete blocks or stones are considered to be sufficient.

Types of Regulation

There are two methods of regulation adopted at a head regulator to control the entry of silt into the canal :

(*i*) Still pond regulation.

(*ii*) Open flow regulation.

Still pond regulation. In this method, the pocket sluices are entirely closed and the canal draws water from the still pond in the pocket. The water in excess of the canal requirements is thus not allowed to escape under the sluice gates. The velocity of water in the pocket is very much reduced on account of excessive water-way since only the supply required for the canal enters the pocket. The silt is thus deposited in the pocket and clear water enters the canal. When the silt deposited has a level about $\frac{1}{2}$ to 1 metre below the crest level of the regulator, the supply in the canal is shut off for about 24 hours and the sluice gates are opened to scour the deposited silt and discharge it downstream. The process is repeated.

Open flow regulation. In this system, the under sluices may be kept open so that the river supply in excess of the canal requirements is escaped. Top water passes into the canal while the bottom water maintains a certain velocity in the pocket to keep the silt to remain in suspension. The advantage of this system is that the canal is not to be closed for scouring the silt. However, the method is very treacherous on account of the uncertain approach channel conditions in the river.

12.16. SILT CONTROL AT HEADWORKS

The entry of silt into the canal can be controlled by :

(1) Providing a divide wall in the river at the canal side so as
  (*a*) to create a trap or pocket.
  (*b*) to create the scouring capacity of under sluices by concentrating the current towards them.
(2) Paving the bottom of the approach channel to reduce disturbance.
(3) Installing a silt excluder.
(4) Making entry of clear top water in the canal by
  (*a*) providing raised sill in the canal.
  (*b*) lowering sill level of scouring sluices.
(5) Reducing the velocity of water at the intake by providing wider head regulator.
(6) Avoiding unsteady flow by making the entry smooth.
(7) Handling carefully the regulation of weir.

There are two types of special works constructed to control the silt entering into the canal :

1. Silt excluder. 2. Silt extractor.

1. Silt excluder. Silt excluder is a device by which silt is excluded from water entering the canal. It is constructed in the bed in front of head regulator.

The fundamental principle on which a silt control device acts lies in the fact that in a flowing stream carrying silt in suspension, the concentration of silt charge in the lower layer is greater than in the upper ones. Hence, the device is so designed that the top and bottom layers are separated without any disturbance. The top water is then led towards the canal while the bottom water containing high silt charge is wasted.

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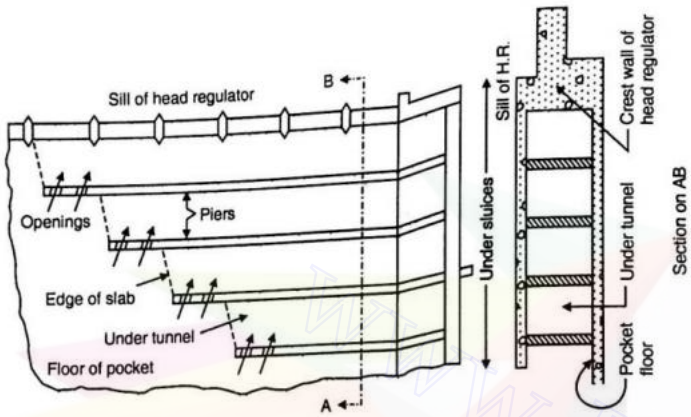

FIG. 12.34. SILT EXCLUDER.

Fig. 12.34 shows a silt excluder — a type used at Khanki weir. The excluder consists of a number of under tunnels resting on the floor of the pocket. The top level of the R.C. roof of the tunnels is kept the same as the sill level of the head regulator. The various tunnels are made of different lenths — the one near the head regulator being of same length as that of the width of the head regulator and the successive tunnels being of decreasing lengths as shown. This arrangement separates the water into two clear layers. The top layer (above the roof of the under tunnels) enters the head regulator, while the bottom layer, containing relatively heavier silt charge goes to the under tunnels and discharges to the d/s of the river through under sluices. The capacity of these tunnels is kept about 10 % of the canal discharge, and the tunnels are so designed that a minimum velocity of 2 to

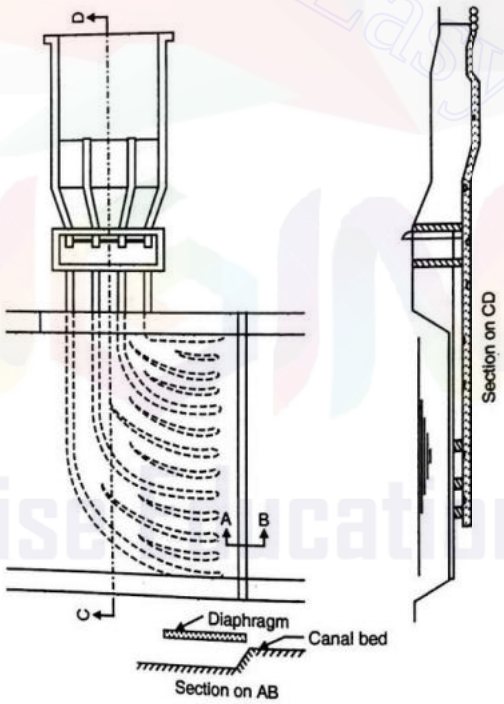

FIG. 12.35. SILT EXTRACTOR.

3 m/sec is maintained. Knowing the discharge and the scouring velocity the total waterway required for the under tunnels can be determined.

2. **Silt Extractor or Silt Ejector.** A silt extractor or silt ejector is a contrivance by which the silt after it has entered the canal is extracted or thrown out. While a silt excluder is a preventive measure, the silt extractor is a curative measure and is constructed on the canal at some distance away from the head regulator.

Fig. 12.35 shows a silt extractor. It consists of a horizontal diaphragm placed slightly above the canal bed. The canal bed is depressed slightly below the diaphragm so that the height of the silt tunnels below the diaphragm is about 0.5 to 0.8 metres. The diaphragm thus separates the top water level from the bottom layer containing heavier silt charge. The disturbance at the entry of the tunnel is reduced by proper design. However, to prevent the tunnel from being clogged, the velocity is accelerated quickly by steadily reducing sectional area of the tunnels by stream-lined vanes. The discharge through the tunnels are governed by the gate opening of the escape chamber. The escape channel is given a steeper slope so that the silt is discharged back to the river through the shortest route.

12.17. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 12.5. *An impervious floor of a weir on permeable soil is 16 m long and has sheet piles at both the ends. The upstream pile is 4 m deep and the down stream pile is 5 m deep. The weir creates a net head of 2.5 m. Neglecting the thickness of the weir floor, calculate the uplift pressures at the junction of the inner faces of the pile with the weir floor, by using Khosla's theory.*

(Civil Services Exam. 1990)

Solution

Fig. 12.36 shows the floor profile. Here, $b = 16$ m.

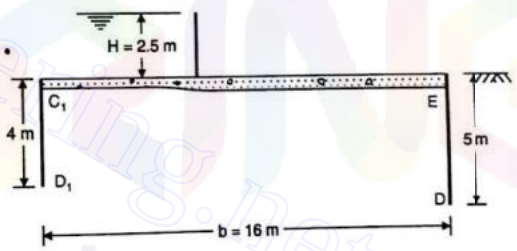

FIG. 12.36

(i) Pressure at point E

$$\Phi_E = \frac{100}{\pi} \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

$$\alpha = b/d = 16/5 = 3.2$$

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$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+(3.2)^2}}{2} = 2.176$$

$$\therefore \quad \Phi_E = \frac{100}{\pi}\cos^{-1}\left(\frac{2.176-2}{2.176}\right) = \frac{100}{\pi}(85°.35) \times \frac{\pi}{180} = 47.42\%$$

Let us now apply correction for interference of u/s pile, given by

$$C = -19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

where $d = 5$ m ; $D = 4$ m; $b' = 16$ m

$$\therefore \quad C = -19\sqrt{\frac{4}{16}}\left(\frac{5+4}{16}\right) = -5.34\%$$

$\therefore$ Corrected $\Phi_E = 47.42 - 5.34 = 42.08\%$

or $P_E = 0.4208 \times 2.5 = \mathbf{1.052\ m}$

(ii) Pressure at point C_1

For finding Φ_{C1}, imagine the pile to be situation at the downstream end. Then

$$\Phi_{C_1} = 100 - \Phi_E$$

where $\Phi_E = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$ where $\alpha = b/d = 16/4 = 4$

$$\therefore \quad \lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+(4)^2}}{2} = 2.562$$

and $\Phi_E = \frac{100}{\pi}\cos^{-1}\left(\frac{2.562-2}{2.562}\right) = \frac{100}{\pi}(77.34°) \times \frac{\pi}{180} = 42.96\%$

$$\therefore \quad \Phi_{C_1} = 100 - \Phi_E = 100 - 42.96 = 57.04\%$$

Let us now apply correction for interference of d/s pile, given by

$$C = +19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

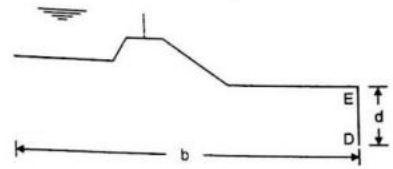
where $d = 4$ m ; $D = 5$ m; $b' = b = 16$ m

$$\therefore \quad C = 19\sqrt{\frac{5}{16}}\left(\frac{4+5}{16}\right) = +5.97\%$$

$$\therefore \quad \Phi_{C_1} = 57.04 + 5.97 = 63.01\%$$

$$\therefore \quad P_C = 0.6301 \times 2.5 = \mathbf{1.575\ m}$$

Example 12.6. *The concrete floor of a head regulator is level with the channel bed (except for the short crest hump) and is 13 m long. The floor is provided with cut off usually at its upstream and downstream ends. The depth of upstream cutoff is 1.5 m (below the floor level) and that the downstream wall is 2 m. Using Khosla's*



$$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) \; ; \quad \lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \; ; \quad \alpha = b/d$$

FIG. 12.37

theory (See Fig. for definition sketch and formula) determine the thickness of the floor at its mid length and also at its junction with upstream and downstream cutoff walls. The floor thickness may not be less than 30 cm anywhere. The upstream FSL is 1.5 m above the floor level. If the permissible exit gradient is 0.18, is the floor safe against failure by piping ? ***(Civil Services Exam. 1982)***

Solution: Take $\rho = 2.24$ for concrete.

(1) Pressure and thickness at the junction E of d/s cutoff with floor

$d = 2$ m ; $b = 13$ m ; $\alpha = b/d = 13/2 = 6.5$; $H_S = 1.5$ m

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+(6.5)^2}}{2} = 3.7882$$

$$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{3.7882-2}{3.7882}\right) = 0.3435 = 34.35\%$$

Correction for interference of u/s sheet pile, given by

$$C = (-)\,19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = -19\sqrt{\frac{1.5}{13}}\left(\frac{2+1.5}{13}\right) = -1.74\%$$

Neglect the correction due to thickness of the floor.

$\therefore$ Corrected $\Phi_E = 34.35 - 1.74 = 32.61\%$

and $P_E = 0.3261 \times 1.5 = 0.489$ m

$$\text{Floor thickness}, t_E = \frac{P_E}{\rho - 1} = \frac{0.489}{2.24-1} = 0.394 \simeq \mathbf{0.4\ m}$$

(2) *Pressure and thickness at junction C_1 of u/s cutoff with floor*

$\Phi_{C_1} = 100 - \Phi_E$, by principle of reversibility of flow.

where $\alpha = 13/1.5 = 8.667$; $\lambda = [1+\sqrt{1+(8.667)^2}]/2 = 4.862$

$$\therefore \quad \Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{4.862-2}{4.862}\right) = 0.3 = 30\%$$

$$\therefore \quad \Phi_{C_1} = 100 - \Phi_E = 100 - 30 = 70\%$$

Correction for interference of d/s sheet pile, given by

$$C = (+)\,19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = +19\sqrt{\frac{2}{13}}\left(\frac{1.5+2}{13}\right) = +2\%$$

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Neglect the correction for thickness of floor.

$\therefore$ Corrected $\Phi_{C_1} = 70 + 2 = 72\%$

$\therefore$ $P_{C_1} = 0.72 \times 1.5 = 1.08$ m

However, this uplift pressure will be counter-balanced by the downward weight of impounded water. Hence provide a nominal (minimum) thickness of 0.3 m at the u/s end.

(3) ***Pressure and thickness at mid-length***

Assuming linear variation, uplift pressure at mid length is given by

$$P_M = (1.08 + 0.489)/2 \simeq 0.785 \text{ m}$$

$\therefore$ Floor thickness, $t_M = \dfrac{0.785}{2.24 - 1} = 0.633$ m $\simeq$ **0.64 m**

(4) ***Exit gradient***

$$G_E = \frac{H_S}{d} \cdot \frac{1}{\pi\sqrt{\pi}} \quad \text{where } d = 2 \text{ m} \; ; \; \lambda = 3.7882 \text{ and } H_S = 1.5 \text{ m}$$

$$\therefore \quad G_E = \frac{1.5}{2} \times \frac{1}{\pi\sqrt{3.7882}} = 0.123 < 0.18 \text{ (permissible)}$$

Hence the floor is *safe* against failure by piping.

Example 12.7. *A stream has a width of 30 m, depth of 3 m and a mean velocity of 1.25 m/sec. Find the height of weir to be built on the stream floor to raise the water level by 1 m. Assume value of discharge coefficient as 0.95.* ***(Engg. Services Exam. 1995)***

Solution

The regular depth of water in the stream, equal to the d/s depth (D) of water = 3 m. Since water level is raised by 1 m due to construction of weir, the u/s water depth, $D_u = 3 + 1 = 4$ m.

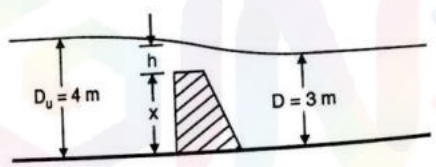

FIG. 12.38

Let x be the height of weir. Then the head h over the weir will be equal to $(4 - x)$ m.

Now Q = discharge in stream

$= B \times D \times V = 30 \times 3 \times 1.25 = 112.5$ cumecs

But $Q = C \,.\, B \,.\, h^{3/2}$ (weir formula)

where $C = \dfrac{2}{3} C_d \sqrt{2g} = \dfrac{2}{3} \times 0.95 \sqrt{2 \times 9.81} = 2.805$

$\therefore$ $112.5 = 2.805 \times 30\,(4 - x)^{3/2}$

From which $(4 - x) = (1.337)^{2/3} = 1.213$ m

Hence $x = 4 - 1.213 = \mathbf{2.787\ m}$

Example 12.8. *A weir has a solid horizontal floor length of 50 m with two lines of cutoff of 8 m depth, below the river bed at its two ends. The floor thickness is 1 m at the upstream end and 2 m at the downstream end, with its upper level being in flush with the river bed. For an effective head of 5 m over the weir, calculate the uplift pressure at the two inside corner points (junction of bottom of floor with the cutoff) and also exit gradient.* ***(Engg. Services Exam. 1984)***

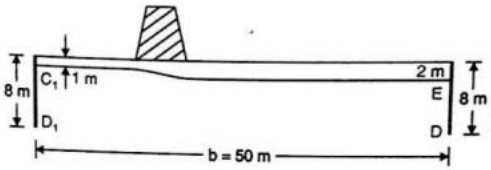

FIG. 12.39

Solution

Give : $H_S = 5$ m; $b = 50$ m

$d_1 = d_2 = 8$ m

Downstream cutoff

$$\alpha = b/d = 50/8 = 6.25$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + (6.25)^2}}{2} = 3.665$$

$$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{3.665 - 2}{3.665}\right) = 0.3499 \quad (i.e.\ 34.99\%)$$

$$\Phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{3.665 - 1}{3.665}\right) = 0.2409 \quad (i.e.\ 24.09\%)$$

Correction due to thickness $= (-)\dfrac{\Phi_E - \Phi_D}{d} \times t = \dfrac{34.99 - 24.09}{8} \times 2 \simeq 2.73\%$

Correction due to interference of u/s cutoff, given by

$$C = (-)\,19\sqrt{\frac{D}{b'}}\left(\frac{d + D}{b}\right) = -19\sqrt{\frac{8}{50}}\left(\frac{8 + 8}{50}\right) = 2.43\%$$

Hence correct $\Phi_E = 34.99 - 2.73 - 2.43 = 29.83\%$

or $P_E = 0.2983 \times 5 \simeq \mathbf{1.40\ m}$

Upstream cutoff

Since $d_1 = d_2 = d$, α and λ are the same as computed above.

Hence $\Phi_{C_1} = 100 - \Phi_E = 100 - 34.99 = 65.01\%$

and $\Phi_{D_1} = 100 - \Phi_D = 100 - 24.09 = 75.91\%$

Correction for thickness $= +\dfrac{75.91 - 65.01}{8} \times 1 \simeq +1.36\%$

Correction due to interference of d/s cutoff, given by

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$$C = +19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) = +2.43\%, \text{ computed above.}$$

$\therefore$ Corrected $\Phi_{C_1} = 65.01 + 1.36 + 2.43 = 68.8\%$

or $P_{C_1} = 0.688 \times 5 = \mathbf{3.44\ m}$

Exit gradient

$$G_E = \frac{H_S}{d}\cdot\frac{1}{\pi\sqrt{\lambda}} = \frac{5}{8}\times\frac{1}{\pi\sqrt{3.665}} = \mathbf{\frac{1}{9.62}}$$

Example 12.9. *A river discharges 1000 cum/sec of water at high flood level of RL 103 m. A weir is constructed for flow diversion with a crest length of 255 m and total length of concrete floor as 40 m. The weir has to sustain the under seepage at a maximum static head of 2.4 m. The silt factor and the safe exit gradient for the river bed material are 1.1 and 1/6 respectively. Determine the depth of cutoff required at the downstream end of the concrete floor. Take the level of downstream concrete floor as R.L. 100 m. Check for exit gradient.* ***(Engg. Services Exam. 1999)***

Solution : Given $Q = 1000$ cumecs ; $L = 255$ m; $f = 1.1$

$\therefore$ $q = Q/L = 1000/255 = 3.922\ m^3/s/m$

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3} = 1.35\left\{\frac{(3.922)^2}{1.1}\right\}^{1/3} \simeq 3.25\ m$$

D/s H.F.L. = 103 m

R.L. of d/s cutoff = D/s H.F.L. $- 1.5R = 103 - 1.5 \times 3.25 \simeq 98.13$ m

R.L. of d/s concrete floor = 100.0 m.

$\therefore$ Depth of d/s cutoff = $100 - 98.13 = \mathbf{1.87\ m} = d$.

Exit gradient, $G_E = \frac{H_s}{d}\cdot\frac{1}{\pi\sqrt{\lambda}}$ (where $H_s = 2.4$ m)

Here, $\alpha = b/d = 40/1.87 = 21.39$

$$\therefore \quad \lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+(21.39)^2}}{2} = 11.2$$

$$\therefore \quad G_E = \frac{2.4}{1.87}\times\frac{1}{\pi\sqrt{11.2}} = \frac{1}{8.19} < \frac{1}{6}$$

Hence the weir floor is safe against piping.

Example 12.10. *A weir across an alluvial river has a horizontal floor of length 60 m and retains 6 m of water under full pond condition. If the downstream sheet pile is driven to a depth of 6 m below the average bed level, calculate the exit gradient. Further, assuming a porosity of 30% and the relative density of soil particles as 2.70, estimate the critical exit gradient and the factor of safety of the system with respect to the exit gradient.* ***(Civil Services Exam. 1989)***

Solution :

Given : $b = 60$ m; $H_S = 6$ m ; $d = 6$ m

$\therefore$ $\alpha = b/d = 60/6 = 10$

$\therefore$ $\lambda = (1+\sqrt{1+\alpha^2})/2 = (1+\sqrt{1+10^2})/2 = 5.5249$

$$G_E = \frac{H_S}{d}\times\frac{1}{\pi\sqrt{\lambda}}$$

$$= \frac{6}{6}\times\frac{1}{\pi\sqrt{5.5249}} = \frac{1}{7.38} = 0.1354$$

$$e = \frac{n}{1-n} = \frac{0.3}{1-0.3} = 0.429$$

Critical hydraulic gradient $= \frac{G-1}{1+e} = \frac{2.7-1}{1+0.429} = 1.19$

$\therefore$ Factor of safety with respect to exit gradient $= \frac{1.19}{0.1354} = \mathbf{8.79}$

Example 12.11. *For the following data, make necessary hydraulic calculations for a vertical drop weir using the Bligh's theory*

Maximum flood discharge	*= 2800 cumecs*
H.F.L. before construction	*= 285.0 m*
Minimum water level	*= 278.0 m*
Downstream bed level	*= 278.0 m*
F.S.L. of canal	*= 284.0 m*
Allowable afflux	*= 1 m*
Coefficient of creep	*= 12*
Permissible exit gradient	*= 1/6*

(Civil Services Exam. 1987)

Solution

This question was set from Author's book (Example 12.3). For solution see example 12.3.

Example 12.12. *Following particulars were recorded from a barrage :*

(i)	*Maximum reservoir level*	*= 212 m*
(ii)	*Pond level*	*= 211 m*
(iii)	*Downstream high flood level in the river*	*= 210 m*
(iv)	*Maximum design flood discharge*	*= 3500 m^3/s*
(v)	*Crest level of the barrage*	*= 207 m*
(vi)	*Crest level the head regulator*	*= 208 m*
(vii)	*Coefficient of discharge*	*= 2.1 $m^{1/2}$/sec for barrage*
(viii)	*Coefficient of discharge*	*= 1.5 $m^{1/2}$/sec for head regulator*
(ix)	*River bed level*	*= 205 m*
(x)	*design discharge of main canal*	*= 500 m^3/sec*

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Determine the number of gates required for the barrage and the head regulator if each gate has 10 m clear span. Neglect (1) end contractions due to piers and abutments and (2) velocity of approach.

If the stilling basin is provided downstream of the barrage for the energy dissipation, find the length and R.L. of the basin floor. Assume that the length of the basin is 5 times the conjugate depth required for hydraulic jump. Neglect losses due to friction.

(Engg. Services Exam. 2001)

Solution

1. Design of water way for barrage during flood

High flood discharge = 3500 m^3/s

H.F.L. = maximum reservoir level = 212 m

Crest level of barrage = 207 m

∴ Head causing flow, $H = 212 - 207 = 5$ m.

Coefficient of discharge for barrage, $C = 2.1$

Hence the length of waterway, L is given by the weir formula :

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

Neglecting the velocity of approach and ignoring end contractions,

$$L \triangleq L_e = \frac{Q}{CH^{3/2}} = \frac{3500}{2.1\,(5)^{3/2}} = 149.07 \text{ m} \triangleq \mathbf{150\ m} \text{ (say).}$$

Provide 15 bays each of 10 m clear span.

Hence Number of gates for the barrage = **15**

2. Design of waterway for canal head regulator

Design discharge of canal = 500 cumecs.

Pond level = 211 m

R.L. of crest of head regulator = 208 m

Head causing flow = 211 − 208 = 3 m.

Coefficient of discharge, C = 1.5 for head regulator.

Hence length of water way, L is given by the weir formula :

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

Neglecting the velocity of approach and ignoring end contractions

$$L \triangleq L_e = \frac{Q}{CH^{3/2}} = \frac{500}{1.5\,(3)^{3/2}} = 64.2 \text{ m}$$

However, provide L = 70 m, in the form of 7 bays each of 10 m clear span.

Hence number of gates for the head regulator = **7**

3. Design of stilling basin

U/S T.E.L. ≙ u/s H.F.L. = 212 m (neglecting the velocity of approach)

D/S T.E.L. ≙ d/s H.F.L. = 210 m (neglecting the velocity of flow

$H_L = 212 - 210 = 2$ m.

Linear discharge, $q = Q/L = 3500/150 = 23.33$ cumecs/m

Hence we have to compute the value of E_{f_2} for $H_L = 2$ m and $q = 23.33$

Since Blench curves are not given, we have to compute E_{f_2} analytically as under.

$$y_c = (q^2/g)^{1/3} = [(23.33)^2/9.81]^{1/3} \triangleq 3.81 \text{ m}$$

Now let $Y = y_2/y_c$ and $Z = H_L/y_c = 2/3.81 = 0.5249$

Since $Z < 1$, the value of Y is given by

$$Y = 1 + 0.93556\,Z^{0.368} = 1 + 0.93556\,(0.5249)^{0.368} = 1.738$$

Hence $y_2 = Y \cdot y_c = 1.738 \times 3.81 = 6.62$ m

∴ Length of cistern $= 5\,y_2 = 5 \times 6.62 = \mathbf{33.1\ m}$

Also, E_{f_2} is given by the following relation :

$$\frac{E_{f_2}}{y_c} = Y + \frac{1}{2\,Y^2} = 1.738 + \frac{1}{2\,(1.738)^2} = 1.9035$$

∴ $E_{f_2} = 1.9035\,y_c = 1.9035 \times 3.81 = 7.25$ m

Hence level at which jump will form = d/s T.E.L. $- E_{f_2}$

$= 210 - 7.25 = 202.75$ m

However, provide cistern at R.L. **202.6 m**

PROBLEMS

1. Explain with help of a diagram, the various component parts, along with their functions, of a diversion headwork.

2. Describe, in brief, various types of weirs. Distinguish clearly between a weir and a barrage.

3. Discuss in brief various causes of failure of weirs and their remedies.

4. Explain Khosla's method of independent variables. How do you apply corrections for (*i*) thickness of floor, (*ii*) inclination of floor and (*iii*) interference of piles ?

5. What is meant by scour ? What precautions do you take against it in weir design?

6. Explain the procedure for the design of a vertical drop weir.

7. What do you understand by launching apron? How is it designed?

8. Write short notes on the following : (*i*) Fish ladder, (*ii*) Divide wall, (*iii*) Under sluices.

9. What are the methods of controlling entry of silt at the headworks of a canal ?

10. Describe, with the help of sketches, the working of a silt excluder.

11. What do you understand by 'silt extractor' and 'silt ejector' ?

12. Write a note on 'location of headworks'.

13. The cross-section of a weir is shown in Fig. 12.40. Calculate (*a*) Uplift pressure at point A and E, (*b*) Thickness of concrete apron at point B, (*c*) Exit gradient.

Use Khosla's curves.

Ans. (*a*) $\Phi_A = 81.5\%$; $\Phi_E = 25.9\%$ (*b*) $t_B = 2.2$ m (*c*) $G_E = \frac{1}{8.9}$

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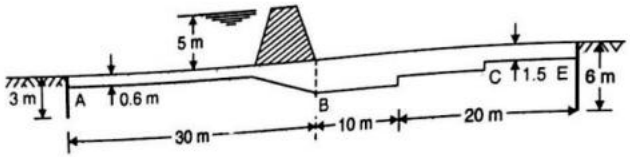

FIG. 12.40.

14. Using Khosla's curves, determine the following for the apron shown in Fig.12.41.
   (*i*) Uplift pressure at points E, D, C, E_1, and D_1.
   (*ii*) Exit gradient.

Neglect the effect of thickness of the floor.

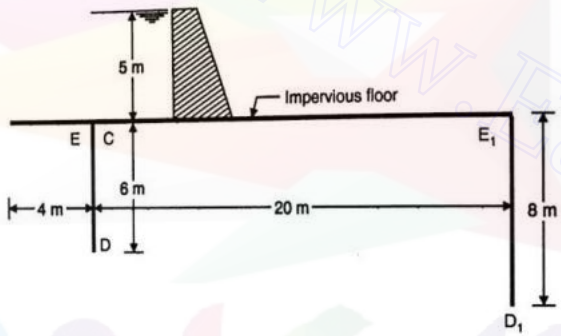

FIG. 12.41.

[**Ans.** (*i*) $\Phi_E = 87\,\%$; $\Phi_D = 65.6\,\%$; $\Phi_C = 59.1\,\%$; $\Phi_{E1} = 42.9\,\%$; $\Phi_{D1} = 32.5\,\%$;

(*ii*) $G_E = \frac{1}{7.1}$

15. Design a vertical drop weir, on Bligh's theory for the following data :
   (*i*) Maximum flood discharge = 1200 cumecs
   (*ii*) H.F.L. before construction of weir = 172.5 m
   (*iii*) River bed level = 168.0 m
   (*iv*) F.S.L. of canal = 171.5 m
   (*v*) Allowable aflux = 1 m
   (*vi*) Coefficient of creep = 11.

Assume any other data not given. The weir wall need not be designed, and its dimensions may be taken as follows : (*i*) top width = 3 m, (*ii*) bottom width = 6 m.

SECTION-III

DISTRIBUTION SYSTEM

Chapters :

13. FLOW IRRIGATION
14. IRRIGATION CHANNELS–I : SILT THEORIES
15. IRRIGATION CHANNELS-II : DESIGN PROCEDURE
16. WATERLOGGING AND CANAL LINING
17. CANAL OUTLETS
18. CANAL REGULATION WORKS
19. CROSS DRAINAGE WORKS

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13

Flow Irrigation

13.1. CANALS : CLASSIFICATION

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

(a) Classification based on the nature of source of supply

(1) Permanent canal. (2) Inundation canal.

A canal is said to be *permanent* when it is fed by a permanent source of supply. The canal is a well made up regular graded channel. It has also permanent masonry works for regulation and distribution of supplies. A permanent canal is also sometimes known as *perennial canal* when the sources from which canal takes is an ice fed perennial river.

Inundation Canals usually draw their supplies from rivers whenever there is a high stage in the river. They are not provided with any headworks for diversion of river water to the canal. They are, however, provided with a canal head regulator. The head of the canal has to be changed sometimes to suit the changing pattern of river course.

(b) Classification based on financial output

(1) Productive canal. (2) Protective canal.

Productive Canals are those which yield a net revenue to the nation after full development of irrigation in the area. *Protective canal* is a sort of relief work constructed with the idea of protecting a particular area from famine.

(c) Classification based on the function of the canal

(1) Irrigation canal. (2) Carrier canal.
(3) Feeder canal. (4) Navigation canal.
(5) Power canal.

An *irrigation canal* carries water to the agricultural fields. A *carrier canal*, besides doing irrigation, carries water for another canal. Upper Chenab canal in West Punjab (Pakistan) is the example of one such canal. A *feeder canal* is constructed with the idea of feeding two or more canals. Examples of such canals are : Rajasthan feeder canal and Sirhind feeder.

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(d) Classification based on boundary surface of the canal

Based on the type of *boundary surface,* canals may be of the following types :

1. Alluvial canals
2. Non-alluvial canals
3. Rigid boundary canals.

and

An *alluvial canal* is the one which is excavated in alluvial soils, such as silt. A *non-alluvial canal* is the one which is excavated in non-alluvial soils, such as loam, clay, hard soil (murram), rock etc. *Rigid boundary canals* are those which have rigid sides and rigid base, such as *lined canals.*

(e) Classification based on the discharge and its relative importance in a given network of canals

(1) Main canal. (2) Branch canal.
(3) Major distributary. (4) Minor distributary.
(5) Water course.

Main canal generally carries water directely from the river or reservior. Such a canal carries heavy supplies and is not used for direct irrigation except in exceptional circumstances. Main canals act as water carriers to feed supplies to branch canals and major distributaries.

Branch canals are the branches of the main canal in either direction taking off at regular intervals. In general, branch canals also do not carry out any direct irrigation, but at times direct outlets may be provided. Branch canals are usually feeder channels for major and minor distributaries. They usually carry a discharge of over 5 cumecs.

Major distributaries usually called *Rajbha,* take off from a branch canal. They may also sometimes take off from the main canal, but their discharge is generally lesser than branch canals. They are real irrigation channels in the sense that they supply water for irrigation to the field through outlets provided along them. Their discharge varies from 1/4 to 5 cumecs.

Minor Distributaries or *minors* take off from branch canals or from distributaries. Their discharge is usually less than 1/4 cumecs. They supply water to the water courses through outlets provided along them.

A water course or *field channel* is a small channel which ultimately feeds the water to irrigation fields. Depending upon the size and extent of the irrigation scheme, a field channel may take off from a distributary or minor. Sometimes, it may even take off from the branch canal for the field situated very near to the branch canal.

***(f)* Classification based on canal alignment**

According to the alignment, a canal may be classified as under (Fig. 13.1):

(1) Contour canal.
(2) Watershed canal.
(3) Side slope canal.

The characteristic features of these canals are discussed in the next article.

13.2. CANAL ALIGNMENT

A canal has to be aligned in such a way that it covers the entire area proposed to be irrigated, with shortest possible length and at the same time its cost including the cost of cross drainage works is a minimum. A shorter length of canal ensures less loss of head due to friction and smaller loss of discharge due to seepage and evaporation, so that additional areas can be brought under cultivation.

According to alignment, the canals may be of the following types (Fig. 13.1)

(1) Ridge canal. (2) Contour canal.
(3) Side slope canal.

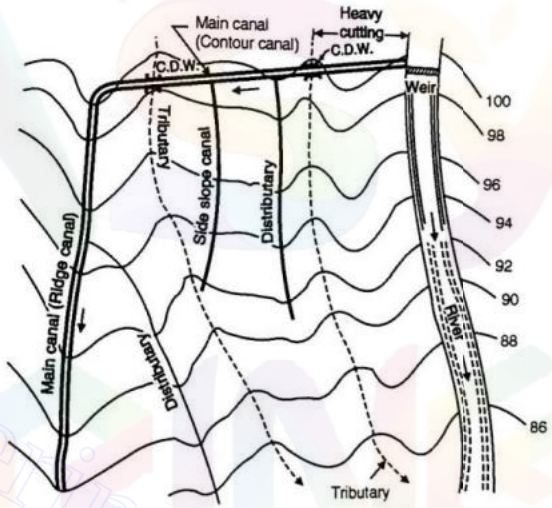

FIG. 13.1. CANAL ALIGNMENT.

1. Ridge Canal

A ridge canal or a watershed canal is aligned along a watershed and runs for most of its length on a watershed. When a channel is on the watershed, it can command areas on both banks and hence a large area can be brought under cultivation. Also, no drainage can intersect a watershed and, hence, the necessity of constructing cross drainage works are obviated.

When the watershed takes a very sharp loop, the canal should be aligned straight to save considerable idle length, as shown in Fig. 13.2. The area of the loop between the watershed and the canal cannot be irrigated by this canal as there will be higher ground on this side and some other arrangement has to be made if the irrigation is important in that area. The canal checks the drainage of this part and a cross-drainage work has, therefore, to be provided. A cost analysis should, therefore, be made before

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aligning the canal straight to reduce its length. The canal has also to leave watershed to by-pass towns and villages situated at the watershed.

2. **Contour Canal**

A channel aligned nearly parallel to the contours of the area is called a contour canal.

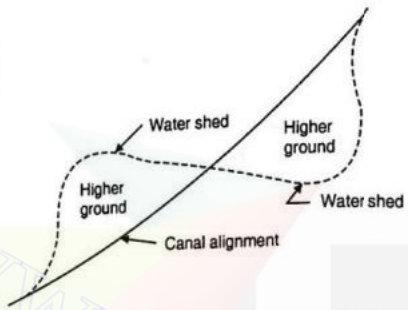

FIG. 13.2. CANAL ALIGNMENT FOR SHARP TURNS IN WATERSHED.

When the canal takes off from a river in a hilly area, it is not possible to align the canal on the watershed as the watershed on the top of the hill may be very high and the areas which need irrigation are concentrated in the valley. The canal is then aligned roughly parallel to the contours of the area. The contour chosen for the alignment should be so placed as to include all culturable area of the valley on one side of the canal.

The contour canal can irrigate only on one side. As the ground level on the other side is quite high, there is no necessity of a bank on this side. Hence, a contour canal is sometimes constructed with one bank only, and is known as a *single bank canal*. However, when both the banks are provided, it is known as a *double bank canal*.

The contour canal does not follow the same contour all along. To enable the water to flow by gravity, some surface slope is given. The rate at which the canal alignment leaves one contour and takes up another depends upon its surface slope. It is usual, in highly undulated tracks, to carry the channel in deep cutting across the ridges or spurs and in high embankments across the valleys in order to reduce the unnecessary length of the channel in long detours and sharp curves in the alignment, as shown in Fig. 13.1.

3. **Side Slope Canal**

It is a channel aligned roughly at right angles to the contours of the country and is neither on the watershed nor in the valley. Such a channel would be roughly parallel to the natural drainage of the country and, hence, it does not intercept any cross-drainage. However, it has very steep bed slope, since the direction of the steepest slope of the ground is at right angles to the contours of the country.

General considerations for alignment

1. The alignment of the canal should be such as to ensure (*i*) the most economical way of distributing the water to the land, (*ii*) as high a command as possible, and (*iii*) minimum number of cross drainage works.

2. The alignment of a canal on a watershed, being the most economical, is preferred. As a general rule, all the watershed lying in a command should be occupied by distributaries.

3. The length of the main canal from the point where it takes off from a river to a point where it mounts on a watershed should be minimum.

4. The contour alignment should be changed this way or that way in order to reduce the number of cross-drainage works to a minimum.
5. The alignment should avoid villages, roads, cart tracks, cremation places, places of worship and other valuable properties.
6. The alignment should pass through the balanced depth of cutting. If not, it should involve minimum depth of cutting or minimum height of filling.
7. The number of kinks and acute curves should be minimum.
8. Idle length of canal should be minimum and branches etc. should be economically planned.
9. The alignment should not be made in a rocky, brakish or cracked strata.

Alignment of a field channel or water course :

Though the maintenance of a field channel is the responsibility of the farmers, its alignment should have the following features :

1. They should be laid along field boundaries.
2. They should be capable of supplying sufficient water to the tail end.
3. Separate field channels should be provided for high and low lands.
4. The field channels should not pass through rocky, brakish, or cracked strata.

13.3. CURVES

Curves should not be provided along the length of a canal except where necessary. Introduction of a curve in a channel disturbs the regime of channel. The concave side is always under erosion and the convex side has a tendency to silt.

If the provision of curve is a must, minimum radius should be provided as shown in Table 13.1.

TABLE 13.1

Capacity of channel (cumecs)	*Minimum radius of the curve (metres)*
Less than 0.3	100
0.3 to 3	159
3 to 15	300
15 to 30	600
30 to 85	900
over 85	1500

13.4. INUNDATION CANALS

Introduction of inundation canal in a particualr area is resorted under the following situations :

1. When the flood in the river is of such a duration that the water level in the canal remains high and fairly constant for quite a long time.
2. When there is an early flood in the river so that Kharif crops can be irrigated.
3. When the river has a late last flood at the end of rainy season so that Rabi crops can be irrigated.

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4. When the area to be irrigated is in close proximity of the canal system so that a large area is not submerged in case there is a worst flood in the river.

5. When the soil has a sufficient self-stabilising power so that bed and banks of the canal remain stable.

In general an inundation canal is similar to a permanent canal. However, the major difference between them is that while in the case of a permanent canal a headwork (such as a weir, barrage or a dam) is constructed, no such works are needed for an inundation canal. But instead, an open cut in the river bank is provided for the inundation canal to take off from the river. The bed level of the cut is kept much above the bed level of the river.

Site selection for the offtake of inundation canals

The selection for the offtake point is very important because the success of inundation canal mostly depends upon it. Following are the main points to be considered for the site selection :

1. At the side of offtake, river course should be straight, and the banks should be stable and high. The river should also flow with an average velocity and should have normal width. The variation of water level should also be low. In case the offtake has to be located on a curve it should be located on the outer side of the curve (*i.e.* the concave side) so that there is no problem of silting in head reaches.

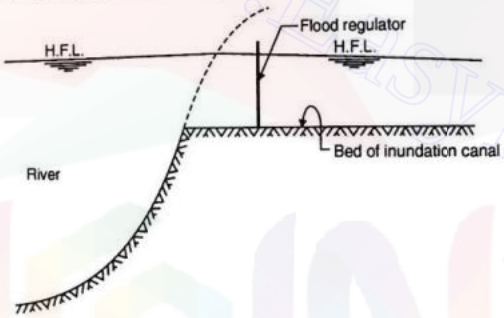

FIG. 13.3. INUNDATION CANAL.

2. The offtake should be located as near the area to be irrigated as possible.

3. If a bypass of a river is available the offtake should be located at the downstream end of the bypass. There may be less of silt trouble in this case since the velocity in the bypass will be almost equal to that required in the inundation canal.

Presence of a shoal creates a still pond in the pocket and thus the silt entry is restricted and reduced.

Design considerations for inundation canals.

The trouble in the inundation canal arises due to lack of control of water levels, causing deposits of sand and formation of bar at mouth with a sudden fall in the river and sanding the canal. Many a times, the flood water finds its way through the canal and submerges vast stretch of land.

Design considerations for inundation canal are as follows :

1. A head regulator, generally known as *flood regulator*, may be provided a few miles downstream of the offtake point. If the flood regulator is provided at the offtake point, there is a danger of the same being washed away during floods.

2. An escape channel, with an off-take at d/s of the flood regulator and joined to the river may be provided to discharge the flood water back into the river when the flood regulator is closed or to divert the surplus water.

3. Vertical lift gates in tiers may be provided at the flood regulator which may be helpful in excluding silt.

4. Full supply level of inundation canal should be kept equal to the fair irrigating level. It is kept at a level at which water is steady in the river for 40 to 50 days during an inundation season.

5. Canal bed level is kept as low as practicable to take advantages of the depth of water for more period even when the river is in low stage.

6. Full supply discharge of the canal is fixed, based on the requirements of Rabi and Kharif crops.

7. Bed slope may be fixed depending upon the slope of the country, keeping in view that non-scouring velocities are not generated. The slope of the canal bed is generally in the range of 0.10 to 0.25 m/km.

8. Bed width may be calculated thereafter. The cross-section may be kept liberal. B/D ratio is kept small. Inundation canal is made deeper and narrower than other irrigation channels.

Advantages of inundation canal

The advantages are as follows :

1. The cost of construction for inundation canal system is very low because of absence of a headwork.

2. Since the silt and fine sand find its way into the canal, the water carried by inundation canal system has a good manurial qualities.

3. Since the water supply in the canal is intermittent and, hence, irrigation in the particular area is also intermittent. Therefore, there is less trouble of waterlogging and other bad effects of over irrigation.

Disadvantage of inundation canal

The disadvantages are as follows :

1. As there is no headwork the head of the canal is liable to be washed away during floods.

2. The water supply available from canal is not reliable. As water is available only during floods, there is no water for irrigation during other seasons.

3. Duty achieved by inundation canal water is low.

4. The cultivator lacks interest in his work due to unreliable supply.

5. The inundation canal is constantly under silt trouble.

6. The size of the canals will have to be of larger dimensions to admit the full requirement of water within a limited time factor.

7. The cross-section of the canal is not well defined. Thus there is very rough handling of water.

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8. The channel alignment is also not very precise. Many a time there are sharp bends at which there may be silting or scouring.

Maintenance of the inundation canal system

There may be a lot of silting trouble in an inundation canal in the head reaches. Following provisions are made in order to maintain the inundation canal system.

1. Construction of *subsidiary heads* so as to supply additional water when the main heads get silted up.

2. Construction of *feeder canal which links up* a number of inundation canals taking off from the same river. Thus the necessity of constructing many subsidiary heads are avoided and the advantages of a good head can be shaped with other canals.

3. Building a head regulator with an escape a few km downstream of the take-off. This will permit only required amount of water to get into the canal. The surplus water may enter into escape and is discharged back into river.

13.5. BANDHARA IRRIGATION

Bandhara is a special type of irrigation scheme, somewhere in between inundation type and permanent type of irrigation. It is essentially a minor irrigation scheme wherein small streams which otherwise allow their flow to be wasted are dammed at places by Bandharas (small weirs), and canals are taken off from them for irrigation of small areas.

A series of such dams or bandharas are constructed and the water available in the monsoon is thus made use of. The Bandhara irrigation scheme is very economical. The irrigation capacity of each Bandhara may vary from a few hectares to about 400 hectares.

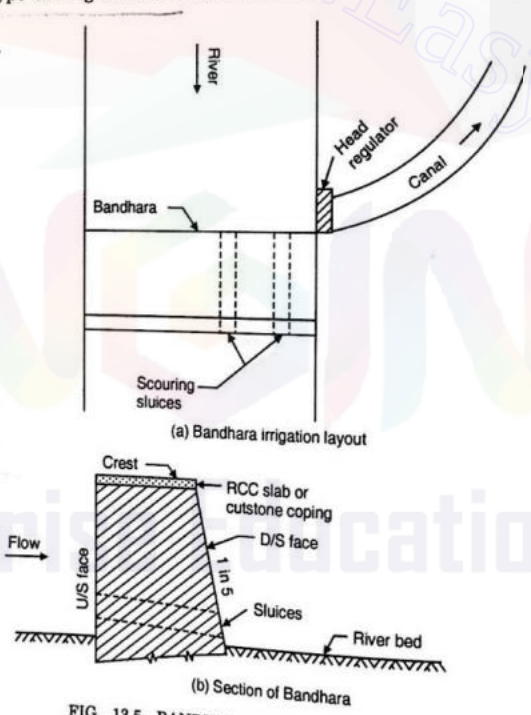

FIG. 13.5. BANDHARA IRRIGATION SYSTEM.

Location of Bandharas

1. The river should preferably be a perenial one so that water is available from the river all through the year.

2. At the site, good foundation should be available for the construction of Bandhara.

3. The section at the site should be straight, narrow and well defined.

4. The offtaking canal from such a site should have a good command.

5. Culturable commanded area so available should be large and fertile.

6. The natural banks on both sides of the site should be high so that no land is submerged on the upstream side.

7. The canal taking off should not interfere with important drainage of the country.

8. The cost of construction of the works should be low.

Working of Bandhara System

The Bandharas are always constructed in series so that water spilling over from one Bandhara is checked by the other Bandhara. Each Bandhara and canal system taking off from the Bandhara forms one complete unit. The canal usually takes off from one side and the water in the canal is controlled by a head regulator. The length of the canal system should not be more than 8 km. The canal terminates in a low lying area commanded by Bandhara. This low lying area is termed as *Thal*. Once this type of system is introduced and developed, the farmers can maintain it themselves.

Design of Bandhara

1. Bandhara should be designed as a low dam. It is usually trapezoidal in shape with upstream face vertical and downstream face having a slope of 1 in 2 to 1 in 5.

2. *Crest width of Bandhara should be kept equal to* $\sqrt{H}$, *where* H is the height of Bandhara, with a minimum of 1.2 m.

3. *Discharge* flowing over Bandhara is given by $Q = 1.7\,Lh^{3/2}$, as the section is considered as a broad crested weir. (L is the length of Bandhara, h is the height of water above the crest of Bandhara in metres and Q is in cumecs.)

4. *Scouring sluices* of 1 m × 1.2 m should be provided at the bottom at suitable intervals to clear silt.

5. The section of the Bandhara shown in Fig. 13.5 is checked for the following conditions :

(*a*) When there is *flow in the river*, there will be afflux on the upstream and, hence, the section should be checked for the differential head.

(*b*) When there is *no flow in the river*, water will be upto the crest level of Bandhara on upstream and there will be no water on downstream.

Phad System of Irrigation

The irrigable area under Bandhara is called *Thal*. A thal may be divided into phads or small blocks. One phad may contain a number of fields owned by a number of cultivators but the crops grown upon all the fields of a phad will be the same in a particular year. Thus if there are four phads in a thal, there will be four types of crops grown in a thal. The crop grown in a phad may be changed from time to time to give rotation to the crops.

Advantages of Bandhara System

1. The system of irrigation has a low initial cost.

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8. The channel alignment is also not very precise. Many a time there are sharp bends at which there may be silting or scouring.

Maintenance of the inundation canal system

There may be a lot of silting trouble in an inundation canal in the head reaches. Following provisions are made in order to maintain the inundation canal system.

1. Construction of *subsidiary heads* so as to supply additional water when the main heads get silted up.

2. Construction of *feeder canal which links up* a number of inundation canals taking off from the same river. Thus the necessity of constructing many subsidiary heads are avoided and the advantages of a good head can be shaped with other canals.

3. Building a head regulator with an escape a few km downstream of the take-off. This will permit only required amount of water to get into the canal. The surplus water may enter into escape and is discharged back into river.

13.5. BANDHARA IRRIGATION

Bandhara is a special type of irrigation scheme, somewhere in between inundation type and permanent type of irrigation. It is essentially a minor irrigation scheme wherein small streams which otherwise allow their flow to be wasted are dammed at places by Bandharas (small weirs), and canals are taken off from them for irrigation of small areas.

A series of such dams or bandharas are constructed and the water available in the monsoon is thus made use of. The Bandhara irrigation scheme is very economical. The irrigation capacity of each Bandhara may vary from a few hectares to about 400 hectares.

Location of Bandharas

1. The river should preferably be a perenial one so that water is available from the river all through the year.

2. At the site, good foundation should be available for the construction of Bandhara.

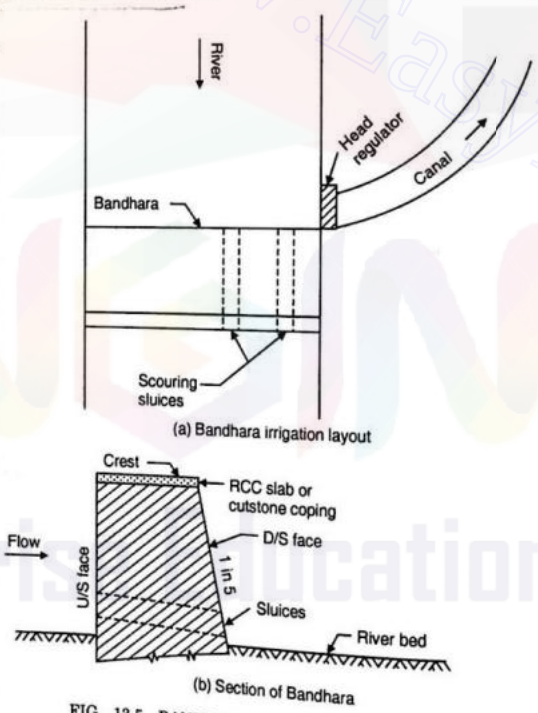

FIG. 13.5. BANDHARA IRRIGATION SYSTEM.

3. The section at the site should be straight, narrow and well defined.

4. The offtaking canal from such a site should have a good command.

5. Culturable commanded area so available should be large and fertile.

6. The natural banks on both sides of the site should be high so that no land is submerged on the upstream side.

7. The canal taking off should not interfere with important drainage of the country.

8. The cost of construction of the works should be low.

Working of Bandhara System

The Bandharas are always constructed in series so that water spilling over from one Bandhara is checked by the other Bandhara. Each Bandhara and canal system taking off from the Bandhara forms one complete unit. The canal usually takes off from one side and the water in the canal is controlled by a head regulator. The length of the canal system should not be more than 8 km. The canal terminates in a low lying area commanded by Bandhara. This low lying area is termed as *Thal*. Once this type of system is introduced and developed, the farmers can maintain it themselves.

Design of Bandhara

1. Bandhara should be designed as a low dam. It is usually trapezoidal in shape with upstream face vertical and downstream face having a slope of 1 in 2 to 1 in 5.

2. *Crest width of Bandhara should be kept equal to* $\sqrt{H}$, *where* H is the height of Bandhara, with a minimum of 1.2 m.

3. *Discharge* flowing over Bandhara is given by $Q = 1.7\,Lh^{3/2}$, as the section is considered as a broad crested weir. (L is the length of Bandhara, h is the height of water above the crest of Bandhara in metres and Q is in cumecs.)

4. *Scouring sluices* of 1 m × 1.2 m should be provided at the bottom at suitable intervals to clear silt.

5. The section of the Bandhara shown in Fig. 13.5 is checked for the following conditions :

(*a*) When there is *flow in the river*, there will be afflux on the upstream and, hence, the section should be checked for the differential head.

(*b*) When there is *no flow in the river*, water will be upto the crest level of Bandhara on upstream and there will be no water on downstream.

Phad System of Irrigation

The irrigable area under Bandhara is called *Thal*. A thal may be divided into phads or small blocks. One phad may contain a number of fields owned by a number of cultivators but the crops grown upon all the fields of a phad will be the same in a particular year. Thus if there are four phads in a thal, there will be four types of crops grown in a thal. The crop grown in a phad may be changed from time to time to give rotation to the crops.

Advantages of Bandhara System

1. The system of irrigation has a low initial cost.

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The maintenance cost is less

2. Small quantities of water which would have otherwise gone waste is utilised to a maximum in this system.

3. Length of canal and distribution system being small, seepage and evaporation losses are very less.

4. The area to be irrigated being close to the source, it yields a high duty and intensive irrigation.

Disadvantages of Bandhara System

1. The irrigable area for one Bandhara is more or less fixed and, hence, even if greater quantity of water is available for irrigation it goes waste.

2. If the river is of non-perennial type, the supply of water becomes seasonal and unreliable in summer.

3. Small sources of water are dammed up for irrigation, hence the population living on the downstream of Bandhara which depends upon the river or stream for water supply for their domestic needs goes without water during dry periods.

Suitability of Bandhara System

Bandhara system is suitable for isolated catchments. It is also very suitable in tracts where there are number of small rivers and streams.

PROBLEMS

1. Explain various types of canals, according to various classification systems.
2. Explain various consideration for alignment of a canal.
3. Distinguish between a contour canal and a ridge canal.
4. Write a note on inundation canal.
5. Describe the Bandhara irrigation system.

14

Irrigation Channels – I : Silt Theories

14.1. TYPE OF CHANNELS

The hydraulic design of a channel or irrigation canal is governed mainly by two factors : (*i*) quantity of silt in the water carried by the channel and (*ii*) type of boundary surface of the channel. Based on these, we may have three types of channels/canals :

1. Alluvial channels
2. Non-alluvial channels

and 3. Rigid boundary channels or lined channels.

Alluvial channels are those which are excavated in alluvial soils, such as silt, and which carry a lot of silt along with water. The boundary or the perimeter of such a channel is therefore made of silt, commonly known as 'alluvium'. The silt content of water flowing in such a channel may vary along its length. The silt content may increase if the velocity of flow is such that it scours the bed and sides of the canal. Similarly the silt content of water may decrease if the velocities are such that 'silting' (or settlement of silt) takes place. The aim of the designer would be to design the channel in such a way that a non-silting non-scouring velocity is obtained for the given design discharge.

Non-alluvial channels are those which are excavated in non-alluvial soils (such as loam, clay, hard soil, rock etc). Such channels generally do not have silting problems as water can flow with *non-silting velocities* (i.e. higher velocities) without fear of scouring of sides and bed of the canal.

Rigid boundary channels are those which have sides and bottom made of rigid material. All lined canals fall under this category.

This Chapter deals with the design of alluvial and non-alluvial channels, while the design of rigid boundary channel (i.e. lined canal) has been given in Chapter 16.

14.2. DESIGN OF ALLUVIAL CHANNELS

The channel or canal which takes off from a river has to draw a fair share of silt flowing in the river. This silt is carried either in suspension or along the bed of the canal. The silt load carried by the channel imposes a difficult problem in a channel design in alluvial soils. The velocity to be allowed in a channel design should

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be such that the silt flowing in the channel is not dropped on the bed. In case a channel silts up, its capacity reduces and so it will irrigate less area. Also the velocity should not be large enough to erode away the bed and sides of the channel. If the sides and bed of a channel are eroded away, the cross-section increases and besides other damages because of scour, its full supply depth decreases ; it can, therefore, command much less area. A velocity which will just keep the silt in suspension, without scouring the channel is known as *non-silting and non-scouring velocity*.

Many investigators have worked on various existing channels towards the design of non-silting-non-scouring channel section. Notable amongst them are works done by Mr. R.G.Kennedy of Punjab Irrigation Department and Mr. Gerald Lacey, Chief Engineer Irrigation Department U.P. The ideas propagated by them have come to be known as Kennedy's theory and Lacey's theory respectively.

For the design of an irrigation channel, the design discharge Q, and the surface and soil properties such as the rugosity coefficient N and silt factor f are known. The problem consists in determination of the *four unknowns* :

(*i*) Area of cross-section (A),

(*ii*) Hydraulic mean depth (R),

(*iii*) Velocity of flow (V),

and (*iv*) Bed slope (S). To start with, following two equations are available :

$$Q = A \times V \quad \text{... (Continuity equation)} \qquad ...(1)$$

and

$$V = f(N, R, S) \quad \text{... (Flow equation)} \qquad ...(2)$$

The flow Eq. (2) may be Manning's equation, or Kutter's equation or any other similar flow equation. However, since there are four unknowns, two more equations must be available for the complete and unique solution. The additional two eqations may be obtained from the following criteria :

(*i*) Providing channel of best discharging section thus getting another equation between A and R.

(*ii*) Limiting equation of velocity from considerations of scouring and silting.

(*iii*) Governing the slope by the available ground slope.

(*iv*) Fixing a suitable B/D ratio on experience.

Criterion (*i*) is not good for alluvial soil where a non-silting non-scouring velocity is a must. Criteria (*ii*) and (*iii*) are utilised by Kennedy's method of design of channels. Wood's Table gives a table of suitable B/D ratio for various discharges. However, Lacey's theory furnishes four equations for the complete determination of the four unknowns, without depending upon the earlier flow equation by Manning or Kutter.

14.3. KENNEDY'S THEORY

Kennedy selected a number of sites on Upper Bari Doab Canal System, one of the oldest in Punjab (Pakistan) for carrying out investigations about velocity and depth of the channel. The sites selected by him did not require any silt clearance for more than thirty years and were thus supposed to be flowing with non-silting non-scouring velocity.

Kennedy's study revealed the following :

1. The flowing water has to counteract some amount of friction against the bed of the canal. This gives rise to vertical eddies rising up gently to the surface. These eddies are responsible for keeping most of the silt in suspension. Some eddies may start from sides but these are for most of its part horizontal and so do not have any silt supporting power. The *silt supporting power* is, therefore, proportional to the bed width of the stream and not to its wetted perimeter.

2. He also defined critical velocity as non-silting non-scouring velocity and gave a relation between critical velocity to the depth of flowing water. The relation is

$$V_0 = 0.55\, D^{0.64} \qquad ...(14.1)$$

In general $V_0 = CD^n$ ...(14.1 *b*)

where V_0 = Critical velocity,

D = Depth of water over bed portion of a channel.

n = Any index number.

Since the equation has been derived on the basis of observations on one canal system only, it is applicable to only those channels which are flowing in sandy silt of the same quality or grade as that of Upper Bari Doab Canal System.

Kennedy later realised the importance of silt grade on critical velocity and introduced a factor m known as *critical velocity ratio* (C.V.R) in his equation. The equation is then written as

$$V_0 = 0.55\, m\, D^{0.64} \qquad ...(14.2)$$

where $m = \text{C.V.R.} = \dfrac{\text{critical velocity for the area}}{\text{critical velocity for Upper Bari Doab canal system}}$ ...[14.2(*a*)]

Sand coarser than the standard were assigned value of m from 1.1 to 1.2 and those finer than the standard from 0.9 to 0.8. Generally, in a system of canal, higher C.V.R. is assumed in head reaches and lower value of C.V.R. is assumed towards its tail end.

The value of constant C in Eq. 14.1(*b*) for various grades of material may be assumed as follows :

TABLE 14.1. VALUE OF *C*

Types of material	*Value of C*
Light Sandy silt	0.53
Coarser light silt	0.59
Sandy loam	0.65
Coarse silt	0.70

The value of m in Eq. 14.2 for different types of silts are given in Table 14.2.

TABLE 14.2. VALUE OF *m*

Type of silt	*Value of m*
1. Light sandy silt in the rivers of Northern India	1.00
2. Somewhat coarser light sandy silt	1.10
3. Sandy, loamy silt	1.20
4. Rather coarser silt or debris of hard soil	1.30
5. Silt of river Indus in Sind	0.70

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Kennedy made use of Kutter's equation for finding the mean velocity of flow in the channel :

$$V = \frac{23 + \frac{1}{N} + \frac{0.00155}{S}}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{N}{\sqrt{R}}} \sqrt{RS} = C\sqrt{RS} \quad \text{(in metric units)} \qquad ...(14.3)$$

Thus the limitations of Kutter's equation became incorporated in Kennedy's design procedure.

Fixation of longitudinal slope (S) of channel : Kennedy did not give any equation for the slope of the canal. The slope is decided according to the slope of the ground available. From the plotted longitudinal slope of the alignment, there is always a certain range of slope, that can be given to a channel at the site. For every such slope, a separate channel can be designed to suit a given discharge. For example, when $Q = 2$ cumecs, $N = 0.0225$ and $m = 1$, we get the following channel dimensions, for various slopes :

Slope	B (m)	D(m)	B/D
1 in 5000	7	0.68	10.8
1 in 4000	3.2	0.85	3.8
1 in 2000	1.5	1.4	1.07

Unfortunately, Kennedy's theory provides no clue as to which of channels will be best suited for a particular discharge and type of soil available. Hence some guidance is necessary to find out as to which channel would suit the need best. To provide this guidance, suitable B/D ratios were fixed in various State Departments on the basis of experience.

Fixation of B/D ratio

(a) Woods Table : Wood's normal table is one such criterion which was prepared in Punjab and is very much in use. An extract of Wood's table, alongwith its metric conversion, is given in Table 14.3.

TABLE 14.3. WOOD'S NORMAL DESIGN TABLE : N = 0.0225

Discharge		B/D ratio	B		D		Slope 1 in...	V/V₀	Mean V	
cusecs	cumecs		ft	m	ft	m			ft. / sec.	m / sec.
10	0.283	2.9	4.75	1.45	1.60	0.49	3333	0.92	1.13	0.344
25	0.708	3.4	7.25	2.21	2.15	0.66	3636	1.01	1.39	0.424
50	1.416	3.7	10.25	3.13	2.75	0.84	4000	1.00	1.56	0.476
100	2.832	4.2	14.50	4.42	3.40	1.04	4444	1.00	1.82	0.555
250	7.079	4.5	22.00	6.70	4.70	1.43	4444	1.01	2.30	0.700
500	14.158	5.7	32.00	9.75	5.65	1.72	5000	1.00	2.54	0.775
1000	28.315	7.6	50.00	15.28	6.50	1.98	5000	1.03	2.89	0.880
2000	56.630	11.3	83.50	25.41	7.40	2.26	5714	1.03	3.10	0.945
5000	141.575	22.5	185.00	56.50	8.20	2.50	6666	0.98	3.20	0.975
10000	283.150	41.0	345.00 360.09	105.00 110.00	8.50 8.80	2.59 2.68	6666 8000	1.02 0.93	3.38 3.13	1.030 0.955
20000	566.300	78.0	696.00	212.00	8.90	2.72	8000	0.94	3.20	0.975

Thus, if B/D ratio is selected from the recommended values, one more equation will be available. Hence unique design of the channel can be done, including the bed slope using Kennedy's theory. This has been illustrated in Case 2 of § 14.4 and in Example 14.2.

(b) Empirical Formula and Table for B/D ratio

For channels having $Q < 15$ cumecs :

$$D = 0.5\sqrt{B} \qquad ...(14.4)$$

However, for channels having $Q \geq 15$ cumecs, the water depth may be selected from the following table (Table 14.4) :

TABLE 14.4. EMPIRICAL RECOMMENDATIONS OF WATER DEPTH (D)

Discharge (cumecs)	Depth D (m)
15	1.7
30	1.8
75	2.3
150	2.6
300	3.0

(c) Recommendations of Central Water Power Commission (C.W.P.C) for B/D ratio

C.W.P.C. has given a graph for B/D ratios for canals carrying discharges upto 300 cumecs, as shows in Fig. 14.1.

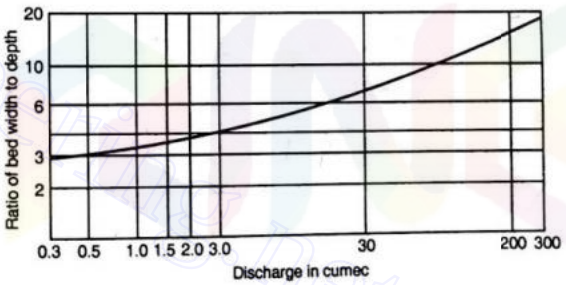

FIG. 14.1. C.W.P.C. RECOMMENDATIONS FOR B/D RATIO.

The above curve can be represented by the following equation :

$$\frac{B}{D} = r = [15 + 6.44\,Q]^{0.382}] \qquad ...(14.5)$$

For example, when $Q = 14$ cumecs, we get

$$\frac{B}{D} = [15 + 6.44 \times 14]^{0.382} \simeq 5.9$$

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and when $Q = 28$ cumec, we get

$$\frac{B}{D} = [15 + 6.44 \times 28]^{0.382} = 7.5 \text{ m}$$

These values are quite close to the one given in Wood's Table (Table 14.3)

14.4. KENNEDY'S METHOD OF CHANNEL DESIGN

For the design of channel, the following equations are used :

Continuity Equ. : $Q = Area \times Velocity$ ...(1)

Kutter's Equ. : $$V = \frac{23 + \frac{1}{N} + \frac{0.00155}{S}}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{N}{\sqrt{R}}}\sqrt{RS} = C\sqrt{RS}$$ ...(2)

Kennedy's Equ. : $V_0 = 0.55\, m\, D^{0.64}$ ...(3)

Buckley's recommendation for *N*

The value of rugosity coefficient N for earthen channels of North India, as suggested by Buckley are given in Table 14.11.

CWPC Recommendations for N

Table 14.5 gives the Central Water and Power Commission (CWPC) practice of selecting Kutter's coefficient N for the canals taking off from reservoirs carrying silt free water.

TABLE 14.5. CWPC PRACTICE FOR *N*

Type of Soil	*Canal Discharge (cumecs)*	*Value of N*
1. Soil other than rock	upto 0.14	0.03
	0.14 to 1.4	0.025
	1.4 to 14	0.0225
	Above 14	0.020
2. Rocky cuts	(i) When rock portion at least 15 cm above the excavated bed level is left out in working out cross-sectional area.	0.035 – 0.05
	(ii) When no portion above bed level is left out	0.05 – 0.080

Recommendations of U.P. Irrigation Department for *N*

The Irrigation Department of Uttar Pradesh adopts the following value of N :

Discharge (cumecs)	*N*
upto 14	0.0225
above 14	0.020

Value of *N* based on nature or condition of channel surface :

In should be noted that the value of N also depends upon the nature or condition of the channel surface, as is clear from Table 14.6.

TABLE 14.6. VALUE OF *N*

Nature or condition of channel surface	*Value of N*			
	Perfect	*Good*	*Fair*	*bad*
1. Alluvial Regime	0.017	0.020	0.0225	0.025
2. Loose gravel	0.0275	0.030	0.033	0.035
3. Rock cut, smooth and regular	0.025	0.030	0.033	0.035
4. Rock cut, irregular	0.035	0.040	0.045	0.050
5. With cobble bed and earthen banks	0.025	0.030	0.035	0.040
6. With earthen bed and rubble sides	0.025	0.030	0.033	0.035

Indian Standard Recommendations for rugosity coefficient N

Table 14.7 gives the values of rugosity coefficient N, as recommended by Indian Standard IS : 7112-1973.

TABLE 14.7. VALUES OF N FOR UNLINED CHANNELS (IS : 7112-1973)

Description of channel	*Value of N*		
	Minimum	*Normal*	*Maximum*
(1) Earth straight and uniform			
(a) Clean, recently complete	0.016	0.018	0.020
(b) Clean, after weathering	0.018	0.022	0.025
(c) Gravel, uniform section clean	0.022	0.025	0.030
(d) With short grass few, weeds	0.022	0.027	0.033
(2) Earth winding and sluggish			
(a) No vegetation	0.023	0.025	0.030
(b) Grass, some weeds	0.025	0.030	0.033
(c) Dense weeds or aquatic plants in deep channels	0.030	0.035	0.035
(d) Earth bottom and rubble sides	0.030	0.035	0.040
(e) Stone bottom and weedy banks	0.025	0.035	0.040
(f) Cobble bottom and clean sides	0.030	0.040	0.050
3. Dragline excavated or dredged			
(a) No vegetation	0.025	0.028	0.033
(b) Light brush on banks	0.035	0.050	0.060
4. Channels not maintained (weed and brush uncut)			
(a) Dense weeds, high as flow depth	0.050	0.080	0.120
(b) Clean bottom, brush on sides	0.040	0.050	0.080
(c) Same, highest stage of flow	0.045	0.070	0.110
(d) Dense brush, high stage	0.080	0.100	0.140

Note : (1) For normal alluvial soils, it is usual in India to assume a value of $N = 0.02$ for bigger channels ($Q > 15$ cumecs) and $N = 0.0225$ for smaller channels ($Q < 15$ cumecs)

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(2) Suitable value of N should be adopted keeping in view the local condition and the above values as a guide.

DESIGN PROCEDURE

Case 1. Given Q, N, m and S (from L-section)

1. Assume a trial value of D in metres.
2. Calculate the velocity V_0 from the equation

$$V_0 = 0.55\, m\, D^{0.64}$$

3. Get area of section A from the continuity Equation :

$$A = \frac{Q}{V_0}$$

4. Knowing D and A, calculate the bed width B from geometry of canal section. The side slope of the canal in alluvial soil is assumed to be $\frac{1}{2}:1$ when the canal has run for some time.

$$\therefore \quad A = BD + \frac{D^2}{2}$$

From which B can calculated.

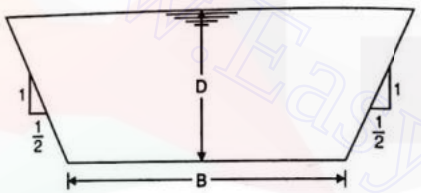

FIG. 14.2. CHANNEL SECTION.

5. Calculate the perimeter and the hydraulic mean depth from the following relations:

$$P = B + D\sqrt{5}$$

$$\therefore \quad R = \frac{A}{P} = \frac{BD + D^2/2}{B + D\sqrt{5}}$$

6. Calculate the actual mean velocity of flow (V) from Kutter's equation. If this value of velocity (V) is the same as V_0 found in step 2, the assumed depth is correct. If not, repeat the calculations with a changed value of D till the two velocities are the same.

Thus we get the solution for a *given value of S*. Case 1 is the most common one and its solution is normally done with the help of Garret's diagram. Garret's diagrams (Fig. 15.3) provide solution of Kennedy's regime equation and Kutter's flow equation for various values of slopes as explained in Chapter 15.

Case 2. Given Q, N, m and B/D ratio from Wood's table :

(1) Calculate A in terms of D.

Let $\frac{B}{D} = x$ or $B = D\,x$

$$\therefore \quad A = BD + \frac{D^2}{2} = xD^2 + \frac{D^2}{2} = D^2(x + 0.5).$$

(2) The value of velocity V_0 is known in terms of D by Kennedy's equation.

$$V_0 = 0.55\, m\, D^{0.64}$$

Substitute the values of V_0 and A in the continuity equation and solve for D. Thus

$$Q = A \times V_0 = D^2(x + 0.5) \times 0.55\, m\, D^{0.64}$$

or

$$Q = 0.55\, m\, (x + 0.5)\, D^{2.64}$$

Hence

$$D = \left[\frac{Q}{0.55m(x + 0.5)}\right]^{\frac{1}{2.64}}$$

In the above relation Q, m and x are known. Hence D is determined.

3. Knowing D, calculate B and R from the following relations :

$$B = xD$$

and

$$R = \frac{BD + D^2/2}{B + D\sqrt{5}}$$

4. Calculate the velocity V_0 from Kennedy's equation

$$V_0 = 0.55\, m\, D^{0.64}$$

5. Knowing V_0 and R, determine the slope S from Kutter's flow equation. The equation can be solved by trial and error.

14.5. SILT SUPPORTING CAPACITY ACCORDING TO KENNEDY'S THEORY

According to Kennedy's theory, the amount of silt held in suspension is proportional to the upward force of vertical eddies, and varies as the bed width (B) and some power of the velocity of flow in the channel.

Let V_0 = critical velocity in the channel

Q_t = quantity of silt transported by the channel

Q = discharge in the channel.

$$\therefore \quad Q_t \propto BV_0^n$$

or

$$Q_t = aB\,V_0^n \qquad ...(i)$$

where a = some constant.

Let p = % of silt in water = Q_t/Q

$$\therefore \quad Q_t = pQ.$$

Assuming a wide channel with vertical slopes,

$$Q = B.D.V_0 \qquad \text{(approx.)}$$

$$\therefore \quad Q_t = p.B.D.V_0 \qquad ...(ii)$$

Equating (i) and (ii), we get

$$aBV_0^n = pB.D.V_0$$

$$\therefore \quad V_0^{\,n-1} = \frac{1}{a}\,p.D.$$

or

$$V_0 = \left(\frac{p}{a}\right)^{\frac{1}{n-1}} D^{\frac{1}{n-1}}$$

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or $$V_0 = CD^{\frac{1}{n-1}} \quad ...(iii)$$

This equation is similar to Kennedy's equation

$$V_0 = CD^y = CD^{0.64} \quad ...(iv)$$

Equating the indices of both these equations, we get

$$\frac{1}{n-1} = 0.64$$

$$\therefore \quad n = 2.56 \triangleq 5/2.$$

Hence, the quantity of silt transported is given by

$$Q_t = a\,B\,V_0^{5/2} \quad ...(14.6)$$

Kennedy did not determine the value of the constant a.

Example 14.1. *Design an irrigation channel on Kennedy's theory, to carry a discharge of 45 cumecs. Take $N = 0.0225$ and $m = 1.05$. The channel has a bed slope of 1 in 5000.*

Solution

Step 1: Let us assume a trial depth $D = 2$ m

Step 2: $V_0 = 0.55\ m\ D^{0.64} = 0.55 \times 1.05\,(2)^{0.64} = 0.9$ m/sec.

Step 3: $A = Q/V_0 = 45/0.9 = 50\ \text{m}^2$

Step 4 : For a trapezoidal section having $\frac{1}{2}:1$ side slopes,

$$A = BD + 0.5\,D^2$$

$$\therefore \quad 50 = B\,(2) + 0.5\,(2)^2, \text{ from which } B = 24 \text{ m}$$

Step 5 : Perimeter $P = B + D\sqrt{5} = 24 + 2\sqrt{5} = 28.47$ m

$$\therefore \quad R = A/P = 50/28.47 = 1.756 \text{ m}$$

Step 6 : Compute velocity of flow from Kutter's equation $V = C\sqrt{RS}$

where $S = 1/5000$

and
$$C = \frac{23 + \dfrac{1}{0.0225} + \dfrac{0.00155}{1} \times 5000}{1 + \left[23 + \dfrac{0.00155}{1} \times 5000\right]\dfrac{0.0225}{\sqrt{1.756}}} = 49.4$$

$$\therefore \quad V = 49.4\sqrt{\frac{1.756 \times 1}{5000}} = 0.926 \text{ m/sec.}$$

Step 7 : Thus $V_0 < V$. In order to increase V_0, increase the value of D. Let $D = 2.2$ m in the next trial.

$$\therefore \quad V_0 = 0.55 \times 1.05\,(2.2)^{0.64} = 0.957 \text{ m/sec.}$$

$$\therefore \quad A = 45/0.957 = 47.04\ \text{m}^2$$

$$\therefore \quad 47.04 = B\,(2.2) + 0.5\,(2.2)^2 \text{ from which } B = 20.28 \text{ m}$$

$$P = 20.28 + 2.2\sqrt{5} = 25.2 \text{ m}$$

$$R = 47.04/25.2 = 1.867 \text{ m}$$

$$C = \frac{23 + \dfrac{1}{0.0225} + \dfrac{0.00155}{1} \times 5000}{1 + \left[23 + \dfrac{0.00155}{1} \times 5000\right]\dfrac{0.0225}{\sqrt{1.867}}} = 49.918$$

$$V = 49.918\sqrt{\frac{1.867 \times 1}{5000}} = 0.965 \text{ m/sec.}$$

Ratio of the two velocities $= 0.965/0.957 = 1.008 \triangleq 1$.

Hence no further trial is necessary.

Thus, $D = \mathbf{2.2\ m}$ and $B = 20.28$ m $\triangleq$ **20.3 m**

Note : If further trials are made, the final values will be $D = 2.23$ m and $B = 19.8$ m.

Example 14.2. *Design an irrigation canal to carry a discharge of 14 cumecs. Assume $N = 0.0225$, $m = 1$ and $B/D = 5.7$.*

Solution :

1. $$\frac{B}{D} = 5.7$$

$$\therefore \quad A = BD + \frac{D^2}{2} = D^2\left[\frac{B}{D} + 0.5\right] = D^2(5.7 + 0.5) = 6.2\,D^2$$

2. $$V_0 = 0.55\ m\ D^{0.64} = 0.55 \times 1 \times D^{0.64}$$

$$Q = A \times V_0 = (6.2\,D^2)(0.55\,D^{0.64})$$

$$\therefore \quad D = \left[\frac{Q}{0.55 \times 6.2}\right]^{\frac{1}{2.64}} = \left(\frac{14}{0.55 \times 6.2}\right)^{\frac{1}{2.64}} = 1.707 \triangleq 1.71 \text{ m.}$$

3. $$\therefore \quad B = 5.7\,D = 5.7 \times 1.71 = 9.73 \text{ m}$$

and
$$R = \frac{BD + D^2/2}{B + D\sqrt{5}} = \frac{(9.73 \times 1.71) + \frac{1}{2}(1.71)^2}{9.73 + 1.71\sqrt{5}} = 1.37 \text{ m.}$$

4. $$V_0 = 0.55\ m\ D^{0.64} = 0.55 \times 1 \times (1.71)^{0.64} = 0.775 \text{ m/sec.}$$

5. But, $$V = \frac{23 + \dfrac{1}{N} + \dfrac{0.00155}{S}}{1 + \left(23 + \dfrac{0.00155}{S}\right)\dfrac{N}{\sqrt{R}}}\sqrt{RS}$$. Keeping $V = V_0$, we get

or
$$0.775 = \frac{23 + \dfrac{1}{0.0225} + \dfrac{0.00155}{S}}{1 + \left(23 + \dfrac{0.00155}{S}\right)\dfrac{0.0225}{\sqrt{1.37}}}\sqrt{1.37 \times S}$$

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or $$0.775 = \frac{23 + 44.5 + \frac{0.00155}{S}}{1 + 0.44 + \frac{2.98}{S} \times 10^{-5}} \times 1.17 \times S^{1/2}$$

which reduces to $$\frac{0.775}{1.17}\left(1.44 + \frac{2.98}{S} \times 10^{-5}\right) = \left(67.5 + \frac{1.55}{S} \times 10^{-3}\right) S^{1/2}$$

or $$67.5\, S^{3/2} + 1.55 \times 10^{-3}\, S^{1/2} - 0.954\, S = 1.98 \times 10^{-5}$$

or $$S^{3/2} + 2.29 \times 10^{-5}\, S^{1/2} - 1.41 \times 10^{-2}\, S = 2.93 \times 10^{-7}$$

Solving this by trial and error, we get $S = \frac{1}{5100}$.

14.6. DRAWBACKS IN KENNEDY'S THEORY

(1) Kennedy did not notice the importance of B/D ratio.

(2) He aimed to find out only the average regime conditions for the design of a channel.

(3) No account was taken of silt concentration and bed load, and the complex silt carrying phenomenon was incorporated in a single factor m.

(4) Silt grade and silt charge were not defined.

(5) Kennedy did not give any slope equation.

(6) Kennedy used Kutter's equation for the determination of the mean velocity and, therefore, the limitations of Kutter's equation got incorporated in Kennedy's theory of channel design.

14.7. FURTHER WORK ON KENNEDY'S THEORY

1. Empirical formulae for other areas

Kennedy's equation could not be made universally applicable even after the introduction of critical velocity ratio. Regional formulae were thereafter developed to suit the requirements of different canal system by different investigators. In these formulae both the coefficient C as well as power of D were varied. A general feature of these formulae indicated a lower value of C for finer soils. The value of C and n in Eq. 14.1 (b) for different canal systems are as follows :

TABLE 14.8. VALUES OF C AND n

Serial No.	Canal system	C	n
1	Godavari Delta	0.391	0.55
2	Krishna Western Delta	0.530	0.52
3	Lower Chenab Canal	0.567	0.57
4	Egyptian Canals	0.283	0.73

2. Lindley's Formulae

It has already been emphasised that depending upon an available slope , many channels can be designed which have different cross-section. As to which of these channels will suit the requirement can not be found out from Kennedy's theory. In order to

make channel design more definite, Lindley of Punjab irrigation proposed his formulae in 1919. He proposed the following two formulae one relating V and D, similar to Kennedy and the other relating V and B.

$$V = 0.567\, D^{0.57} \qquad ...[14.7(a)]$$

and $$V = 0.274\, B^{0.35} \qquad ...[14.7(b)]$$

By eliminating V from Eqs. [14.7 (a),(b)], another relation as originally proposed by Lindley can be obtained. This is given by

$$B = 7.80\, D^{1.61} \qquad ...(14.7)$$

(3) Ranga Raju and Misri's method of canal design on Kennedy's theory

Ranga Raju and Misri (1979) gave a simplified method of a trapezoidal channel design on Kennedy's theory, if the slope is given. They used Manning's formula as flow equation. Assuming a trapezoidal channel section with $\frac{1}{2}:1$ side slopes finally attained by channels in alluvial soils, we have.

$$A = BD + 0.5\, D^2 = D^2\,(B/D + 0.5) = D^2\,(r + 0.5) \quad \text{where } r = B/D \text{ ratio.}$$

and $$P = B + D\sqrt{5} = D\,(B/D + \sqrt{5}) = D\,(r + 2.236)$$

Also, $$R = \frac{A}{P} = \frac{D\,(r + 0.5)}{r + 2.236}$$

$$\therefore \quad V = \frac{Q}{A} = \frac{Q}{D^2\,(r + 0.5)} \qquad ...(i)$$

But from Manning's formula, $V = \frac{Q}{A} = \frac{1}{N} R^{2/3} S^{1/2}$

$$\therefore \quad \left(\frac{Q}{A}\right)^2 = \frac{1}{N^2} R^{4/3} S$$

From which $$S = \frac{Q^2}{A^2} \cdot \frac{N^2}{R^{4/3}} = \frac{Q^2 N^2\,[r + 2.236]^{4/3}}{[D^2\,(r + 0.5)]^2\,[D\,(r + 0.5)]^{4/3}}$$

or $$S = \frac{Q^2 N^2\,(r + 2.236)^{4/3}}{D^{16/3}\,(r + 0.5)^{10/3}} \qquad ...(ii)$$

From Kennedy's basic equation, $V_0 = 0.55\, m\, D^{0.64}$

$$\therefore \quad Q = V_0 \times A = 0.55\, m\, D^{0.64}\,[D^2\,(r + 0.5)]$$

or $$D = \left[\frac{Q}{0.55\, m\,(r + 0.5)}\right]^{1/2.64} = \left[\frac{1.818\, Q}{m\,(r + 0.5)}\right]^{0.3788} \qquad ...(14.8)$$

Substituting this value of D in Equ. (ii), we get

$$S = \frac{Q^2 N^2\,(r + 2.236)^{4/3}}{\left[\frac{1.818\, Q}{m\,(r + 0.5)}\right]^{2.02} [r + 0.5]^{10/3}}$$

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or
$$\frac{S\,Q^{0.02}}{N^2\,m^{2.02}} \simeq \frac{0.299\,(r+2.236)^{4/3}}{(r+0.5)^{1.313}} \qquad ...(14.9)$$

This is the basic relationship between unknown r and known parameters S, Q, N, m. Knowing the left hand side (for given values of S, Q, N and m), the value of r is found by trial and error.

Alternatively, a plot can be made between $SQ^{0.02}/N^2\,m^{2.02}$ and r (Fig. 14.3). The value of r can be read from this plot, for known value of the factor $SQ^{0.02}/N^2\,m^{2.02}$. Knowing, r, we get $B = rD$. Hence from Eq (*i*),

$$V = \frac{Q}{D^2\,(r+0.5)}. \text{ Thus, } B, D \text{ and } V \text{ are known.}$$

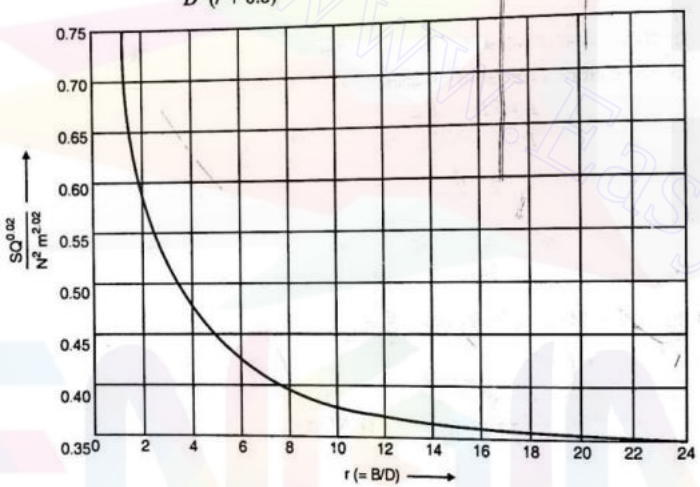

FIG. 14.3.

(4) Semi-empirical curve fitting method of canal design based on Kennedy's theory and CWPC recommendations for *B/D* ratio

For a trapezoidal section having side slopes $\frac{1}{2}:1$, we have

$$A = BD + 0.5\,D^2 = D^2\,(r+0.5) \text{ where } r = B/D \text{ ratio.}$$

Also, from Kennedy's basic equation for velocity,

$$V_0 = 0.55\ m\ D^{0.64}$$

Hence
$$Q = A \times V_0 = D^2\,(r+0.5) \times 0.55\,mD^{0.64} = 0.55\,m\,(r+0.5)\,D^{2.64}$$

Hence
$$D = \left[\frac{Q}{0.55\,m\,(r+0.5)}\right]^{1/2.64} = \left[\frac{1.818\,Q}{(r+0.5)\,m}\right]^{0.3788} \qquad ...(14.8)$$

From the above equation, one can compute both B and D, if suitable explicit equation in the following form is available :

$$r = f\,(Q, N, m, s)$$

One such correlation was obtained by Ranga Raju and Misra (in the form of Eq. 14.9), by using Manning's formula as flow equation; however, limitations of Mannings formula got involved in this equation. A large number of canal designs can be obtained by varying the parameters Q, N, m and S in their practical ranges and a suitable correlation can be established between r, Q, N, m and S. This was done by varying the parameters, r, Q, N, m and S in their practical ranges, thus obtaining 1296 number of canal designs which satisfy both Kennedys as well as Kutter's equations. The resulting solutions so obtained were analysed and the following *unique empirical relationship* was obtained by a method of *curve fitting* :

$$r = \left[\frac{1.607\,S^{1.63}\,Q^{0.033}}{N^{3.26}\,m^{3.293}} - 0.258\right]^{-0.915} \qquad ...(14.10)$$

Thus, for a given set of values of Q, N, m and S, the value of r (i.e. B/d ratio) can be computed from Eq. 14.10. The value of B/D ratio so obtained can then be compared with the curve of Fig. 14.1 recommended by CWPC. If the matching differs by a wide range, a modification in the selected value of S becomes necessary, and this modified value of S can be found as under :

As stated earlier, the curve of Fig. 14.1 can be represented by the following empirical equation :

$$r = [15 + 6.44\,Q]^{0.382} \qquad ...(14.5)$$

Taking the above equation as the optimal condition for the value of r and hence substituting the value of r in Eq. 14.10, we get the following relationship for S :

$$S = \frac{N^2\,m^{2.02}}{1.338\,Q^{0.02}}\left[0.258 + (15 + 6.44\,Q)^{-0.417}\right]^{0.6135} \qquad ...(14.11)$$

From the above equation, the modified slope S is obtained which satisfies both Eqs. 14.11 and 14.5.

Design procedure

(a) Given : Q, N, m and S

Step (i) : Compute, B/D ratio (i.e. r) from Eq. 14.10 which satisfies both Kennedy's equation as well as Kutter's equation.

Step (ii) : Compute D from Eq. 14.8.

Step (iii) : Find $B = rD$

See Example 14.4 for illustration.

(b) Given Q, N, m and CWPC curve for r

Step (i) : Find the required value of r either from the curve of Fig. 14.1 or from Eq. 14.5.

Step (ii) : Compute the optimal value of S from Eq. 14.11.

Step (iii) : Compute D from Eq. 14.8.

Step (iv) : Find $B = rD$

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Example 14.3. *Design a channel on Kennedy's theory, for the following data : $Q = 45$ cumecs, $N = 0.0225$, $m = 1.05$ and $S = 1/5000$. Use the method given by Ranga Raju and Misri.*

Solution : Given $Q = 45$ cumecs, $N = 0.0225$, $m = 1.05$ and $S = 1/5000$.

Substituting the values in Eq. 14.9, we have

$$\text{L.H.S.} = \frac{S\,Q^{0.02}}{N^2\,m^{2.02}} = \frac{1}{5000} \times \frac{(45)^{0.02}}{(0.0225)^2\,(1.05)^{2.02}} = 0.3863$$

Hence $$\frac{0.299\,(r + 2.236)^{4/3}}{(r + 0.5)^{1.313}} = 0.3863$$

From curve of Fig. 14.3, we get $r \simeq 10$

Again solving the above equation by trial and error, starting with $r = 10$, we get $r = 9.7$

Now, from Eq. 14 $$D = \left[\frac{1.818\,Q}{m\,(r + 0.5)}\right]^{1/2.64} = \left[\frac{1.818 \times 45}{1.05\,(9.7 + 0.5)}\right]^{0.3788} = \mathbf{2.16\ m}$$

and $$B = rD = 9.7 \times 2.16 \simeq \mathbf{21\ m}$$

$$V = \frac{Q}{D^2\,(r + 0.5)} = \frac{45}{(2.16)^2\,(9.7 + 0.5)} = 0.946 \text{ m/s}$$

Check $V_0 = 0.55\,m\,D^{0.64} = 0.55\,(1.05)\,(2.16)^{0.64} = 0.945$ m/s. Hence OK.

Example 14.4. *Using the method of curve fitting based on Kennedy's theory, design a channel for the following data :*

$$Q = 45 \text{ cumecs},\ N = 0.0225,\ m = 1.05 \text{ and } S = 1/5000$$

Solution

The ratio $r\,(= B/D)$ is given by Eq. 14.10 :

$$r = \left[\frac{1.607\,S^{1.63}\,Q^{0.033}}{N^{3.26}\,m^{3.293}} - 0.258\right]^{-0.915}$$

$$= \left[\frac{1.607\,(1/5000)^{1.63}\,(45)^{0.033}}{(0.0225)^{3.26}\,(1.05)^{3.293}} - 0.258\right]^{-0.915} = \left[\frac{1.7031 \times 10^{-6}}{4.9876 \times 10^{-6}} - 0.258\right]^{-0.915} = 9.7$$

From Eq. 14.8, we get

$$D = \left[\frac{1.818\,Q}{(r + 0.5)\,m}\right]^{0.3788} = \left[\frac{1.818 \times 45}{(9.7 + 0.5)\,1.05}\right]^{0.3788} \simeq \mathbf{2.16\ m}$$

Hence $B = r\,D = 9.7 \times 2.16 = 20.95 \simeq \mathbf{21\ m}$

Example 14.5. Solve example 14.4, using curve of *CWPC for B/D ratio.*

Solution : From the Eq. 14.5 of curve of Fig. 14.1, we have

$$r = (15 + 6.44\,Q)^{0.382} = (15 + 6.44 \times 45)^{0.382} \simeq 8.89$$

(Against the value of $r = 9.7$ obtained in the previous example)

The optimal value of S is computed from Eq. 14.11 :

$$S = \frac{N^2\,m^{2.02}}{1.338\,Q^{0.02}}\left[0.258 + (15 + 6.44\,Q)^{-0.417}\right]^{0.6135}$$

$$= \frac{(0.0225)^2\,(1.05)^{2.02}}{1.338\,(45)^{0.02}}\left[0.258 + (15 + 6.44 \times 45)^{-0.417}\right]^{0.6135} = \mathbf{1/4920}$$

Also, from Eq. 14.8 :

$$D = \left[\frac{1.818\,Q}{(r + 0.5)\,m}\right]^{0.3788} = \left[\frac{1.818 \times 45}{(8.89 + 0.5)\,1.05}\right]^{0.3788} \simeq \mathbf{2.23\ m}$$

Hence $B = r\,D = 8.89 \times 2.23 = 19.82 \simeq \mathbf{20\ m}$

14.8. LACEY'S REGIME THEORY

"Dimensions, width, depth and slope of a regime channel to carry a given discharge loaded with a given silt charge are all fixed by nature." This idea was first put forward by Lacey. Lindley's theory is also based on the same concept. Lacey succeeded in evolving more generally applicable equations based on his own experiments and the experiments of the past investigators.

Regime channel. Lacey defined regime channel as a stable channel transporting a regime silt charge. *A channel will be in regime if it flows in unlimited incoherent alluvium of the same character as that transported and the silt grade and silt charge are all constant.*

Incoherent alluvium. It is a soil composed of loose granular graded material which can be scoured with the same ease with which it is deposited.

Regime silt charge. It is the minimum transported load consistant with fully active bed.

Regime silt grade. This indicates the gradation between the small and the big particles. It should not be taken to mean the average mean diameter of a particle.

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

1. The channel is flowing in unlimited incoherent alluvium of the same character as that transported.
2. Silt grade and silt charge are constant.
3. Discharge is constant.

If the above three conditions are met with fully, then the channel is said to be in *true regime.* However, it is seldom that the above conditions are realized in field. Hence, Lacey gave the idea of initial and final regime for actual channel.

Initial regime. One of the conditions of attaining regime of a channel is that there should be freedom for the channel to form its own section. Initial regime is the state of channel that has formed its section only and yet not secured the longitudinal slope.

Final regime. When a channel is constructed with defective slope, it tries to throw off the incoherent silt on the bed to increase their slopes. To attain the final regime, the channel forms its section first before the final slope. The channel after attaining its section and longitudinal slope will be said to be in final regime.

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Permanent regime. When a channel is protected on the bed and side with some kind of protecting material, the channel section cannot be scoured up and so there is no possibility of change of section or longitudinal slope ; the channel will then be said to be in permanent regime. Regime theory is not applicable to such channels.

There is only one section and only one longitudinal slope at which the channel will carry a particular discharge with a particular silt grade. Natural silt transporting channels have a tendency to assume a semi-elliptical section. The coarser the silt, greater is the waterway of such a channel and narrower the depth. The finer the silt, greater is the depth and the channel closely approximates a semi-circle.

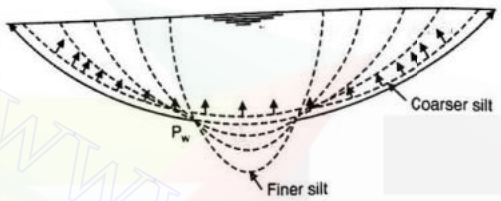

FIG. 14.4. CHANNEL SECTION ACCORDING TO LACEY'S THEORY.

If a channel is constructed with too small a cross-section for a particular discharge and the slope steeper than required, scour will occur till final regime is attained. Similarly silting will occur in a channel till final regime is attained in a channel with a wider cross-section and flatter slope than required.

Lacey also states that the silt is kept in suspension due to the force of vertical eddies. According to him, the eddies are generated from bed and sides, both normal to surface of generation. Hence, vertical component of eddies generated from sides will also support the silt. Lacey, therfore, assumed hydraulic mean depth (R) as variable, unlike Kennedy who assumed depth D as variable. Since Lacey assumed a semi-ellipse as the cross-section of a regime channel, assumption R as a variable seem to more logical.

Lacey's Regime Equations

On the basis of arguments mentioned above, Lacey plotted a large mass of data to obtain a relationship between (*i*) V vs. R and (*ii*) A vs. V. Lacey recognised the importance of silt grade in the problem and introduced a function f known as *silt factor* in the regime relationship. The first two equations, originally suggested by Lacey in F.P.S. units and converted in M.K.S. units, are given below :

$$V = \sqrt{\tfrac{2}{5} f R} \qquad ...(14.12)$$

$$A f^2 = 140.0 V^5 \qquad ...(14.13)$$

where A = area of the channel section and V = velocity of flow in it.

Regime Flow Equation

By plotting a large mass of data from a number of different sources, Lacey obtained the relationship

$$V = 10.8\, R^{2/3} S^{1/3} \qquad ...(14.14)$$

where S = slope of water surface

This equation is very useful for finding velocities during high floods. A beginner has not to worry himself to find out the exact values of N in order to apply Manning's and Kutter's formulae. Equation 14.14 contains no silt factor and rugosity coefficient. Such variables were considered to be implicit in the actual depth and slope adopted by the channel in loose alluvium.

Eqs. 14.12, 14.13 and 14.14 are known as *Lacey's fundamental equations* from which other flow equations can be derived as shown below.

1. Perimeter Discharge (*P-Q*) Relation

By combining Eq. 14.12 and Eq.14.13, a relationship between P and Q can be obtained.

Raising both sides of Eq. 14.12 to the fourth power, we get

$$V^4 = \frac{4}{25} f^2 R^2 .$$

Eliminating f^2 between this equation and equation 14.13, we have

$$A\left[\frac{25V^4}{4R^2}\right] = 140.0\, V^5. \quad \text{or} \quad A\left[\frac{25}{4R^2}\right] = 140\, V$$

Multiplying both sides of this equation by A, we get

$$\frac{25 A^2}{4 R^2} = 140 V A = 140\ Q \quad \text{where} \quad \frac{A^2}{R^2} = P^2$$

Hence
$$P^2 = \frac{4 \times 140}{25} Q$$

or
$$P = 4.733\, Q \simeq 4.75 \sqrt{Q} \qquad ...(14.15)$$

Equation 14.15 is a very important equation in as much as it is universally true. Power of Q does not vary, though the coefficient 4.75 varies between certain range.

2. *V-Q-f* Relation

Multiplying Eq. 14.13 by V on both sides, we get

$$A V f^2 = 140.0\, V^6$$

or
$$Q f^2 = 140.0\, V^6$$

$$\therefore \quad V = \left[\frac{Q f^2}{140.0}\right]^{1/6} \qquad ...(14.16)$$

This relation is utilized in *channel design* by Lacey's theory. The value of Q and f in a channel design are known. The value of V can, therefore, be readily found from Eq.14.16.

3. Regime-Slope Equations : (*S – Q – f*), (*R – S – f*) *and* (*S – f – q*) *relationships.*

From Eq.14.14, cubing both sides of the equation

$$V^3 = 1260\, R^2 S \qquad ...(14.17)$$

From Eq. 14.12 cubing both sides of the equation

$$V^3 = (2/5)^{3/2} f^{3/2} R^{3/2}$$

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Hence, $(2/5)^{3/2} f^{3/2} R^{3/2} = 1260\, R^2 S$

or
$$S = \frac{f^{3/2}}{4980 R^{1/2}} = \frac{0.0002 f^{3/2}}{R^{1/2}} \quad ...(14.18)$$

Equation 14.17 can also be written as
$$S = \left(\frac{V^2}{R}\right)^{5/3} \frac{1}{1260\,(RV)^{1/3}} \quad ...(14.19)$$

From Eq. 14.12, $V^2/R = \frac{2}{5} f$

Also $RV = q$ where q = discharge per unit width.

Hence
$$S = \left(\frac{2}{5}\right)^{5/3} \frac{f^{5/3}}{1260\, q^{1/3}} = 0.000178 \frac{f^{5/3}}{q^{1/3}} \quad ...(14.20)$$

The above equation can also be represented in terms of total discharge.

Rewritting equation 14.19,

$$\therefore \quad S = \left(\frac{V^2}{R}\right)^{5/3} \frac{1}{1260\,(RV)^{1/3}}$$

Putting $R = A/P$, we get

$$\therefore \quad S = \left(\frac{V^2}{R}\right)^{5/3} \frac{1}{1260\left[\frac{A}{P} V\right]^{1/3}} = \left(\frac{V^2}{R}\right)^{5/3} \frac{P^{1/3}}{1260\,(Q)^{1/3}}$$

But $P = 4.75\sqrt{Q}$

$$\therefore \quad S = \frac{(2/5)^{5/3} f^{5/3} (4.75)^{1/3} Q^{1/6}}{1260\, Q^{1/3}}$$

or
$$S = \frac{f^{5/3}}{3340\, Q^{1/6}} = \frac{0.0003 f^{5/3}}{Q^{1/6}} \quad ...(14.21)$$

4. Regime Scour Depth Relation

From Eq. 14.12, as already derived

$$R^2 = \frac{25}{4}\frac{V^4}{f^2} \quad \text{or} \quad R = \frac{5}{2}\frac{V^2}{f}$$

From Eq. 14.16, we have

$$V^2 = \left[\frac{Q f^2}{140}\right]^{1/3}$$

Hence
$$R = \frac{5}{2}\left(\frac{Q f^2}{140}\right)^{1/3} \frac{1}{f} = 0.47\,(Q/f)^{1/3} \quad ...(14.22)$$

Now
$$q = RV = \frac{5}{2}\frac{V^3}{f}$$

Substituting V from Eq. 14.16, we get

$$q = \frac{5}{2}\cdot\frac{1}{f}\left(\frac{Q f^2}{140}\right)^{3/6} = 0.21\, Q^{1/2} \quad ...(14.23)$$

Substituting for $Q = \left(\frac{q}{0.21}\right)^2$ in equation 14.22, we get

$$R = 0.47\frac{1}{f^{1/3}}\left(\frac{q}{0.21}\right)^{2/3} = 1.35\left(\frac{q^2}{f}\right)^{1/3} \quad ...(14.24)$$

5. Lacey's Non-Regime Flow Equation

Eq. 14.14 is applicable only for regime flow. The equation to be derived now will be applicable for both regime and non-regime channels.

From Eq. 14.14,

$$V^3 = 1260\, R^2 S \quad \text{or} \quad V = 35.5\,(R/V)^{1/2}\sqrt{RS} \quad ...(14.25)$$

Equation 14.25 is similar to Chezy's equation, in which Chezy's coefficient C is represented by $35.5\,(R/V)^{1/2}$

Now putting $V = K\sqrt{fR}$ in Eq. 14.12

Chezy's coefficient C may be expressed as

$$C = \frac{35.5\, R^{1/4}}{K^{1/2} f^{1/4}}$$

Again putting
$$\frac{35.5}{K^{1/2} f^{1/4}} = \frac{K'}{N_a} \quad ...(14.25a)$$

where N_a = Lacey's absolute rugosity coefficient, Eq. 14.25 can be written as

$$V = \frac{K'}{N_a} R^{1/4} S^{1/2} \quad ...(14.26)$$

Lacey called N_a as absolute rugosity coefficient to emphasize that this N_a depends only upon grade and density of boundary material and is independent upon channel conditions. This reason could not be supported by observations.

Comparing it to Manning's equation

$$V = \frac{1}{N} R^{2/3} S^{1/2} \quad ...(14.27)$$

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We get $K' = 1$, if $N = N_a$ and $R = 1$ m

Hence Lacey's flow equation will be

$$V = \left(\frac{1}{N_a}\right) R^{3/4} S^{1/2} \quad ...(14.28)$$

N_a–f relation

As already derived in Eq. 14.25 (*a*) :

$$\frac{K'}{N_a} = \frac{35.5}{K^{1/2} f^{1/4}} \quad ...[14.28(a)]$$

This means $N_a \propto f^{1/4}$ or $N_a = c f^{1/4}$, where $c = a$ constant

For standard grade of silt, $f = 1$ and $N_a = 0.0225$. Hence $c = 0.0225$

Hence $N_a = 0.0225 f^{1/4}$ ...(14.29)

The above equation can also be derived alternatively as under :

Since $K' = 1$ and $K = \sqrt{2/5}$ from Eq. 14.12. We get from Eq. 14.25 (*a*) :

$$N_a = \frac{K' K^{1/2} f^{1/4}}{35.5} = \frac{1\,(2/5)^{1/4} f^{1/4}}{35.5} = 0.0225 f^{1/4}$$

6. Silt Factor–Grain Size Relationship

If m_d is the mean particle size or diameter of silt in mm, the silt factor $f = 1.76\sqrt{m_d}$. It is usually taken as D_{50} size. A fairly correct value of f may be arrived at by averaging the values obtained from various equations listed above. Some values of f for various soils are given as under.

TABLE 14.9. VALUES OF f

Type of soil	*Value of f*
Fine silt	0.50 to 0.70
Medium silt	0.85
Standard silt	1.00
Medium sand	1.25
Coarse sand	1.50

The use of a single f value for correlating various regime equations has been objected to by various authors. According to Mr. T. Blench, the use of a single f value for the entire perimeter is inadequate. This amounts to averaging out the relative importance of side and bed. Ningchien on the other hand propagates that the silt factor also depends upon flow conditions on one hand and concentration of silt on the other.

Summary of Lacey's Equations

Table 14.10 gives a summary of Lacey's formulae.

TABLE 14.10. SUMMARY OF LACEY'S FORMULAE

Serial No.	*Designation of formula*	*Expression in M.K.S. units*	*Equation No.*
1	V–f–R	$V = \sqrt{\frac{2}{5} f R}$	14.12
2	A–f–V	$A f^2 = 140.0\, V^5$	14.13
3	V–R–S	$V = 10.8\, R^{2/3}\, S^{1/3}$	14.14
4	P–Q	$P = 4.75\sqrt{Q}$	14.15
5	V – Q – f	$V = \left(\frac{Q f^2}{140}\right)^{1/6}$	14.16
6	S–f–R	$S = \frac{f^{3/2}}{4980\, R^{1/2}}$	14.18
7	S–f–q	$S = 0.000172\, \frac{f^{5/3}}{q^{1/3}}$	14.20
8	S–f–Q	$S = \frac{f^{5/3}}{3340\, Q^{1/6}}$	14.21
9	Regime scour depth relation	$R = 0.47\,(Q/f)^{1/3}$	14.22
10	— Do —	$R = 1.35\,(q^2/f)^{1/3}$	14.24
11	V–N_a–R–S	$V = \frac{1}{N_a} R^{3/4}\, S^{1/2}$	14.28
12	N_a–f	$N_a = 0.0225\, f^{1/4}$	14.29
13	Shock	$V = \frac{1}{N_a} R^{3/4}\, (S-s)^{1/2}$	14.30

The Idea of 'Shock'

Lacey earlier indicated that the value of absolute rugosity coefficient (N_a) is independent of channel conditions. Later, however, he recognised that N_a is affected by channel conditions. This variation was observed in practice. In order to explain the variation of absolute rugosity coefficient with channel condition, Lacey gave a new idea and introduced the concept of *shock losses*.

In case of movable bed channels, ripple formation takes place on the bed. When flow takes place over these ripples, the pressure on the front on these ripples is more than pressure on their rear. The resistance to flow caused by this difference of pressure is known as *form resistance*. The value of *shock losses* due to irregularities roughly coincides with form resistance. The total resistance imparted to the flow of water is due to frictional resistance and due to 'shock losses' caused by irregularities.

Lacey, however, suggested that the value of N_a should remain constant for all channel conditions, but the slope should be divided in two parts — one to over come 'shock losses' and the other to overcome friction. If s denotes the slope required to withstand 'shock losses', Eq. 14.28 can be written in the form :

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$$V = \frac{1}{N_a} R^{3/4} (S - s)^{1/2} \quad ...(14.30)$$

Thus, the value of N_a in the above equation remains independent of channel condition and has a value congruent to the value of N for a channel in 'very good' condition. Buckley reeommends following value of N for earthen channels :

TABLE 14.11. VALUES OF N BY BUCKLEY

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

The value of slope for these channel conditions can be computed from Eq. 14.28 and 14.30 as indicated below. For channel in *very good condition*, $N = 0.0225$ and $s = 0$. For channel in *good condition*, $N = 0.025$. Assuming $N_a = N$, we get the following values of velocities:

From Eq. 14.28, $$V = \frac{1}{N_a} R^{3/4} S^{1/2} = \frac{1}{0.025} R^{3/4} S^{1/2}$$

From Eq 14.30, $$V = \frac{1}{N_a} R^{3/4} (S - s)^{1/2} = \frac{1}{0.0225} R^{3/4} (S - s)^{1/2}$$

Equating the two velocities, we get

$$\frac{1}{0.025} R^{3/4} S^{1/2} = \frac{1}{0.0225} R^{3/4} (S - s)^{1/2}$$

or $$(S - s)^{1/2} = 0.9\, S^{1/2}$$

From which $$s = 0.19\, S \quad ...(14.31)$$

This means that for a channel in good conditions, 19% of slope is accounted for by shock losses and 81% is available for frictional resistance. Similarly the shock losses for indifferent and poor conditions of a channel can be calculated. The value of shock losses for these channels respectively account for 33.1% and 43.7% of the gross slope available.

Thus, the values of s due to shock losses for channel with different conditions are as under :

(*i*) Channel in very good condition: $s = 0$

(*ii*) Channal in good condition : $s = 0.19\, S$

(*iii*) Channel with indifferent condition: $s = 0.331\, s$

and (*iv*) Channel with poor condition : $s = 0.437\, S$

Lacey's idea of 'shock' was unique, as the same brought the idea of *form resistance* coming into play in flowing mass of water. However, shock losses cannot be actually found out and, hence, equation 14.30 cannot be used in practice. Therefore, different values of N_a (= N value recommended by Buckley) has to used for different channel conditions.

14.9. LACEY'S THEORY APPLIED TO CHANNEL DESIGN

The channel section can be designed on Lacey's theory by using various equations developed above. The design procedure involves no trial and error steps. For the channel design, the discharge Q and mean diameter of silt particles m_d or silt factor f should be known.

Channel design procedure

1. Calculate the silt factor $f = 1.76\sqrt{m_d}$
2. Compute velocity $V = \left(\frac{Qf^2}{140}\right)^{1/6}$
3. Determine area $A = Q/V$
4. Compute perimeter $P = 4.75\sqrt{Q}$
5. Find out bed width B and depth D of the channel section since A and P are known. The side slope of an irrigation channel is usually $\frac{1}{2}:1$. Hence :

Area $$A = BD + \frac{D^2}{2}$$

and perimeter $$P = B + D\sqrt{5}$$

Hence $$D = \frac{P - \sqrt{P^2 - 6.944A}}{3.472} \quad ; \quad B = P - 2.236\, D$$

6. Calculate $$R = \frac{5}{2}\frac{V^2}{f}$$

Also calculate $$R = \frac{BD + D^2/2}{B + 2.236\, D}$$

Both the values of R should be the same ; this will provide a *numerical check* from steps 1 to 5.

7. Find the slope $$S = \frac{f^{5/3}}{3340\, Q^{1/6}}$$

Example 14.6. *A channel section has to be designed for the following data :*

Discharge $Q = 30$ *cumecs*

Silt factor $f = 1.00$

Side slope $= \frac{1}{2}:1$

Find also the longitudinal slope.

Solution

1. The value of f is given as 1.00
2. Velocity $$V = \left[\frac{30 \times 1}{140}\right]^{1/6} = 0.773 \text{ m/sec}$$
3. Area $$A = \frac{30}{0.773} = 38.8 \text{ sq.m}$$

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4. $$P = 4.75\sqrt{Q} = 4.75\sqrt{30} = 26 \text{ m}$$

5. $$D = \frac{P - \sqrt{P^2 - 6.944\,A}}{3.472} = \frac{26 - \sqrt{676 - 6.944 \times 38.8}}{3.472} = 1.67 \text{ m.}$$

$$B = P - 2.236\,D = 26 - 2.236 \times 1.67 = 22.26 \text{ m.}$$

6. Hydraulic mean radius.

$$R = \frac{5}{2}\frac{V^2}{f} = \frac{5}{2} \times \frac{1}{1}(0.773)^2 = 1.49 \text{ m.}$$

Also $$R = \frac{B\,D + D^2/2}{B + D\sqrt{5}} = \frac{1.67\,(22.26) + 1.67^2/2}{22.26 + 1.67\sqrt{5}} = 1.49 \text{ m.}$$

Hence, checked.

7. Slope $$S = \frac{f^{5/3}}{3340\,(Q)^{1/6}} = \frac{1}{3340\,(30)^{1/6}} = \frac{1}{3340\,(1.764)} = \frac{1}{5880}.$$

Hence, the channel has a bed width $B = 22.26$ m and a depth of 1.67 m. The longitudinal slope $S = 1/5880$.

14.10. COMPARISON OF KENNEDY'S AND LACEY'S THEORIES

1. Kennedy introduced the term C.V.R. (m) in his equation to make it applicable for channels of different grades of silt, but he did not give any idea to measure the value of m. Lacey introduced the concept of silt factor f in his equations and suggested a method of determining the value of f by relating it with particle size.

2. Kennedy assumed that silt is kept in suspension because of eddies generated from the bed only, and so he proposed a relation between V and D. Lacey assumed that silt is kept in suspension because of the normal components of eddies generated from the entire perimeter and so he proposed a relation between V and R.

3. Kennedy assumed Kutter's formula for finding the value of mean velocity where in the value of N is to be assumed arbitrarily. Lacey gave his own formula for the velocity and thus a designer has not to choose anything arbitrarily.

4. Kennedy gave no formula for determination of longitudinal slope of the canal. The slope to be given to the canal is based on experience or on Wood's table. Lacey gave a formula for the longitudinal slope of a regime channel.

5. Lacey proposed that the shape of a regime channel should be a semi ellipse. Since the channel section is trapezoidal in shape, it can never attain final regime. Kennedy simply gave the idea that a non-silting and non-scouring channel will be a regime channel.

6. Lacey's theory as applied to channel design does not involve any trial and error procedure whereas Kennedy's theory involves a trial and error procedure for design of channel.

7. Lacey made a distinction between two types of resistance in alluvial channels, one determined by grain size and the other due to irregularities of the channel. Kennedy did not make any such distinction.

8. Basic concept of the theories is the same that the silt remains in suspension due to the force of vertical eddies.

14.11. DEFECTS IN LACEY'S THEORY

1. The theory does not give a clear description of physical aspects of the problem.

2. It does not define what actually governs the characteristics of an alluvial channel.

3. The derivation of various formulae depends upon a single factor f and dependence on single factor f is not adequate. There are different phases of flow on bed and sides and, hence, different values of silt factor for bed and side should have been used.

4. Lacey's equations do not include a concentration of silt as variable.

5. Lacey did not take into account the silt left in channel by water that is lost in absorption which is as much as 12 to 15% of the total discharge of channel.

6. The effect of silt attrition was also ignored. The silt size does actually go on decreasing by the process of attrition among the rolling silt particles dragged along the bed.

7. Lacey did not properly define the silt grade and silt charge.

8. Lacey, however, considers that a regime channel is inherently free from external restraint and shock and has, therfore, a constant N_a for a given size of material. In so far as regime channel is a sediment transporting channel and will normally have a changing pattern of bed ripple formation, this statement is unlikely to be correct.

9. Lacey introduced semi-ellipse as ideal shape of a regime channel which is not correct.

10. Strictly speaking an artificial channel is not a regime channel, and regime theory is not applicable to it.

14.12. TRACTIVE FORCE APPROACH FOR CHANNEL DESIGN

Tractive force approach, which is an improvement over various silt theories discussed above, can be used for the design of alluvial channels. *Tractive force* or *fluid drag* is the force exerted by the flowing water on the bed grains to cause its movement. Tractive force is equal to the tangential component of the weight of water flowing in the channel and acts is the direction of flow. For uniform flow, the *mean tractive force* per unit length of the channel is equal to wAS, where w is the unit weight of water, A is the cross-sectional area of the channel and S is the bed slope of the channel. The *tractive force per unit area* (or the boundary shear stress τ) is equal to the tractive force per unit length of the channel divided by its perimeter P.

Thus, $$\tau = \frac{w\,A\,S}{P} = wRS \qquad \text{...(14.32)}$$

where R = hydraulic mean radius.

The bed grains start moving when the tractive force exceeds the frictional resistance between them. The tractive force required to initiate the general movement of bed grains is called the *critical tractive force* and the corresponding stress is known as *critical tractive stress* (τ_c). The critical tractive stress is a function of sediment concentration

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(i.e. its specific gravity) and the average grain size of the bed material. Several investigators have given expressions for τ_c, and some of these are given below.

1. Shield's expression ...(14.33)

$$\tau_c = 0.06\, w\, (G-1)\, d$$

where τ_c = critical shear stress (kN/m^2)

w = specific weight of water (kN/m^3)

d = grain size diameter (m)

Taking $G = 2.66$ and $w = 9.81$ kN/m^3, equation 14.33 reduces to : ...(14.33 *a*)

$$\tau_c = 0.98\, d$$

2. White's equation ...(14.34)

$$\tau_c = 0.786\, d \ \text{(kN/m}^2\text{)}$$

where d is the grain diameter in m.

Note that the above equations are for *fully developed turbulent flow*. The value of τ_c for channels may be reduced by 10% for slightly sinuous channels, by 25% for moderately sinuous channels and by 40% for very sinuous channels.

Table 14.12. gives the average values of critical tractive stress (τ_c) for some common types of sediments flowing in the channels.

TABLE 14.12. AVERAGE VALUES OF τ_c

Type of sediment	*Average value of τ_c (kN/m^2)*
1. Alluvial soils, silty loam and coarse sand	0.0025
2. Medium sand	0.0017
3. Sandy loam	0.0020
4. Fine Gravel	0.0037

14.13. DESIGN OF UNLINED CHANNELS IN ALLUVIAL SOIL AS PER IS : 7112-1973

Indian Standard IS : 7112-1973 has recommended the following three methods for design of unlined channels in alluvial soils.

1. Channel design by Lacey's Equations
2. Channel design by regime type fitted equations
3. Channel design by tractive force approach

1. Method 1. Channel design by Lacey's equations

In this method, the channel may be designed by the following three equations given by Lacey

$$S = \frac{0.0003\, f^{1/3}}{Q^{1/6}} \quad ...(i) \quad ...(14.21)$$

$$P = 4.75\sqrt{Q} \quad ...(ii) \quad ...(14.15)$$

and

$$R = 0.47\,(Q/f)^{1/3} \quad ...(iii) \quad ...(14.22)$$

The silt factor f may be determined from the following equation :

$$f = 1.76\sqrt{m_d}$$

where m_d = mean particle size in mm of the boundary material = D_{50} in mm. Alternatively, silt factor f be determined from the following equation.

$$f = \frac{2.39\, V^2}{R}$$

where V is the mean velocity of flow (m/s) and R is the hydraulic mean depth (m) of an existing stable channel.

For a design problem, Q and f are known; hence S, P and R can be computed from Eqs (*i*), (*ii*) and (*iii*) respectively. IS : 7112-1973 has also given graphical solution for finding direction B and D corresponding to the above computed values of P and R, for a trapezoidal channel having side slopes $\frac{1}{2}:1$. The corresponding curves for the graphical solution are given in Fig. 14.5.

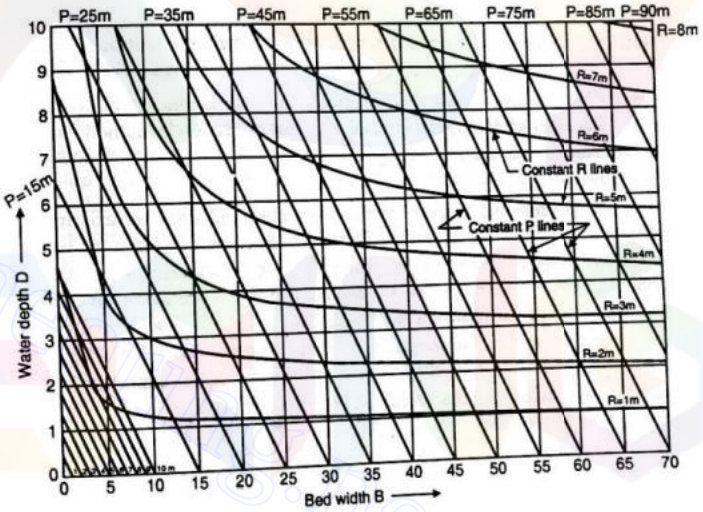

FIG. 14.5. HYDRAULIC CHART OF RELATIONSHIP BETWEEN B, D, P AND R FOR CHANNEL HAVING SIDE SLOPES $\frac{1}{2}:1$.

Alternatively, B, and D can be computed from the simultaneous solution of the following equations :

$$P = B + D\sqrt{5} = B + 2.236\, D$$

and

$$R = \frac{BD + 0.5\, D^2}{B + 2.236\, D}$$

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2. Method 2 : Channel design by regime type fitted equations

Alternatively, the channel can be designed using *regime type fitted equations* given in Table 14.13. These equations have been evolved on the basis of data collected from various States in India.

TABLE 14.13. REGIME TYPE FITTED EQUATIONS (IS : 7112 - 1973)

S.N.	Hydraulic parameter	All India canals	Punjab canals	U.P. Canals	Bengal canals
1.	Bed slope S	$\frac{0.000315}{Q^{0.1651}}$	$\frac{0.000251}{Q^{0.0961}}$	$\frac{0.000360}{Q^{0.1450}}$	$\frac{0.0001346}{Q^{0.0375}}$
2.	Wetted Perimeter P	$4.30\,Q^{0.5231}$	$7.00\,Q^{0.6019}$	$3.98\,Q^{0.5020}$	$5.52\,Q^{0.4190}$
3.	Hydraulic mean Radius R	$0.515\,Q^{0.3406}$	$0.466\,Q^{0.3389}$	$0.448\,Q^{0.3649}$	$0.438\,Q^{0.4454}$

Method 3. Channel design by tractive force approach

IS : 7112-1973 also recommends to design the channel on the tractive force approach, in which the tractive stress (or unit tractive force) exerted on the bed of a channel by the flowing water may be calculated from the following equation :

$$\tau = w \,.\, R \,.\, S. \qquad ...(14.32)$$

The permissible tractive stress is a function of sediment concentration and average particle size (or D_{50} size) of the bed material in case of channels in non-cohesive soils and voids ratio in case of clayey soils.

Design procedure

1. Obtain the permissible tractive stress from observed data on existing channels based on sediment concentration X of flow in the channel and D_{50} size of bed material in the case of non-cohesive soils and voids ratio in the case of cohesive soils. Alternatively, the permissible tractive stress may also be determined by expressions given by various investigators such as Shield, White etc.

2. Select a suitable bed slope of the channel on the basis of average ground slope along the alignment of the channel or on the basis of experience.

3. Taking $\tau = \tau_c$, compute the value of R from the relation :

$$R = \frac{\tau_c}{w \,.\, S} \qquad ...(14.32\ a)$$

4. Assume a suitable value of rugosity coefficient N from Table 14.7 and obtain the mean velocity (V) of flow from Manning formula :

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

5. Compute the area of cross-section from the relation : $A = Q/V$

6. Knowing R and A, determine B and D for a trapezoidal channel having side slopes of $\frac{1}{2} : 1$.

Example 14.7. *Using tractive force approach, design a channel in alluvial soil for the following data :*

(i) *Discharge : 45 cumecs*
(ii) *Bed slope : 1/4800*
(iii) *Manning's $N = 0.0225$*
(iv) *Permissible tractive stress : 0.0035 kN/m^2*
(v) *Side slopes : $\frac{1}{2} : 1$*

Solution :

$$R = \frac{\tau_c}{w \,.\, S} = \frac{0.0035}{9.81 \times 1/4800} = 1.713 \text{ m}$$

$$V = \frac{1}{N} R^{2/3} S^{1/2} = \frac{1}{0.0225} (1.713)^{2/3} (1/4800)^{1/2} = 0.918 \text{ m/s}$$

$$\therefore \quad A = Q/V = 45/0.918 = 49 \text{ m}^2$$

$$P = A/R = 49/1.713 = 28.61 \text{ m}$$

$$\therefore \quad P = 28.61 = B + 2.236\,D \qquad ...(i)$$

and

$$A = 49 = BD + 0.5\,D^2 \qquad ...(ii)$$

Solving these, we get $D \triangleq$ **1.94 m** and $B =$ **24.27 m**

14.14. SEDIMENT TRANSPORT

The quantity of solids entering the channel is an important factor which controls the cross-section and shape of a true regime channel. Regime theory does not consider this important factor in channel design. However, it has now been realised that the channel design will not be successful unless a detailed study of sediment transport has been made.

The sediment moving in a fluid can be broadly divided in two parts :

1. Bed load 2. Suspended load.

Bed load

Bed load may be defined as the load of bed material in the layer where suspension is impossible for fluid dynamic reasons. Sediment grains in the bed layer are not vertically supported by the flow, but *rest on the bed* almost continuously, while sliding, rolling and jumping along. Their weight is supported by the static grain particles in the non-moving bed. They move regularly exchanging places with the similar particles in non-moving bed.

Suspended Load

In case of suspended load, the weight of the particles is suported by the surrounding fluid. Since the particles continuously settle with its settling velocity V_s, in relation to the surrounding fluid, it is kept in suspension soley because the flow provides an upward motion due to turbulent exchange due to which a fluid is continuously exchanged over definite distances between horizontal layers. The rising fluid originates from lower layers of higher concentration and descending fluid originates from higher layer of lower concentration so that a surplus of upward moving sediment particles over downward moving sediment particle occurs. This surplus provides an upward motion of particles which counterbalances the general settling of sediment. Suspended load cause an additional hydrosatic pressure on the bed.

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The concentration of suspended load C at a height y above the bottom can be determined in terms of known concentration C_a at a reference point at a hight a above the bottom by Rouse equation given below :

$$\frac{C}{C_a} = \left[\frac{a}{y} \cdot \frac{D-y}{D-a}\right]^{\frac{V_s}{KV_*}} \quad ...(14.35)$$

where V_s = fall velocity of grain in still water

D = Depth of water

K = Van Karman's constant = 0.4

V_* = Shear friction velocity $= \sqrt{\tau_0/\rho} = \sqrt{\frac{wR'S}{\rho}} = \sqrt{\frac{\rho g R' S}{\rho}} = \sqrt{g R' S}$ ...(14.36)

τ_0 = Intensity of shear stress at bottom.

ρ = mass density of water

R' = modified value of R due to ripple $= R(N'/N)^{3/2}$ ...(see Eq. 14.42)

Note that the effect of ripples is to reduce the hydraulic mean radius to a value R' from R. For a wide channel, $R \triangleq$ bed width B.

Rate of bed load transport

The initiation of bed load movement is caused by the drag force exerted on it by moving water. Many investigators have worked on the correct estimation for the rate of bed transport but none of the ideas extended so far could be widely accepted.

We shall discuss herein the following two most widely used equations :

(1) Meyer-Peter's Equation, and (2) Einsten's Equation

1. Meyer Peter's Equation

Meyer-Peter's equation, given below, is based on experimental work carried out at Federal Institute of Technology, Zurich.

$$\left(\frac{Q_s}{Q}\right)\left(\frac{N'}{N}\right)^{3/2} wSD = 0.047(w_s - w)d + 0.25(w/g)^{1/3}(q_s)^{2/3} \quad ...(14.37)$$

where D = depth of flowing water in the channel (m)

S = bed slope of the channel

d = mean grain diameter (m)

q_s = rate of bed load transport per unit width of channel (kN/m/s)

Q = discharge in the channel, if the sides were frictionless (cumecs)

Q_s = actual discharge in the channel (cumecs)

w_s = specific weight of sediment particles (kN/m^3)

w = specific weight of water (kN/m^3)

N' = Manning's coefficient for plane bed

N = Manning's coefficient for rippled bed

The ratio Q_s/Q takes into account correction for friction of the sides. If the sides are frictionless, $Q_s/Q = 1.0$.

The values of N' and N can be obtained from Strikler's formulae given below:

$$N' = (k_s)^{1/6}/24 \text{ and } N = (k)^{1/6}/24 \quad ...(14.38)$$

where k_s is the *effective grain diameter* or d_{50} size (in m) and k is the representative size (in m) of roughness of rippled bed.

Note that, N is the rugosity coefficient actually observed by experiments on rippled bed of the channel and its value can be taken as 0.020 for discharges over 11 cumeces and as 0.0225 for discharges less than 11 cumecs.

Now wRS = shear stress $\tau = wDS$ for wide channels where $R \triangleq D$.

Hence bed shear, $\tau_b = (Q_s/Q) \cdot wDS$

Also, critical shear stress, $\tau_c = 0.047\, w_s - w)\, d$.

Substituting these values in Eq. 14.37, we get

$$\tau_b (N'/N)^{3/2} = \tau_c + 0.25 (w/g)^{1/3} (q_s)^{2/3}$$

or

$$q_s = 47450\, [\tau_b (N'/N)^{3/2} - \tau_c]^{3/2} \text{ kN/m/hr} \quad ...(14.39)$$

However, if q_s is expressed in N/m/hr and τ_b and τ_c are in N/m^2, we have

$$q_s = 1500\left[\tau_b (N'/N)^{3/2} - \tau_c\right]^{3/2} \text{ N/m/hr} \quad ...(14.39\ a)$$

Lastly if q_s is expressed in kg/m/hr and τ_b and τ_c are in kg/m^2, we have

$$q_s = 4700\, [\tau_b (N'/N)^{3/2} - \tau_c]^{3/2} \text{ kg/m/hr} \quad ....(14.39\ b)$$

2. Einstein's Equations

Einstein's equations based on the principles of law of probability, involves a number of assumptions and experimental coefficients. The equations correlate two dimensionless parameters φ_* and ψ_* (known as *bed load functions*) as under :

$$\varphi_* = \left(\frac{i_B}{i_b}\right)\varphi = \frac{i_B}{i_b}\left[\frac{q_s}{Gw}\left(\frac{1}{G-1}\right)^{1/2}\left(\frac{1}{g\,d^3}\right)^{1/2}\right] \quad ...(14.40)$$

and

$$\psi_* = \left(\frac{i_B}{i_b}\right)\psi = \frac{i_B}{i_b}\left[\frac{(G-1)\,d}{R'\,S}\right] \quad ...(14.41)$$

where q_s = rate of bed load transport per unit width of channel (kN/m/s)

d = diameter of sediment particles (m)

i_B = fraction of q_s which is of diameter d

i_b = proportion of grains of diameter d in the bed.

G = specific gravity of sediment.

w = specific weight of water (kN/m^3)

S = bed slope of the channel

R' = hydraulic mean radius of the channel if the bed were unrippled.

Since R' represents hydraulic mean radius of a channel which has grains roughness only (i.e. unrippled channel), we have from Manning's equation

$$V = \frac{1}{N'}(R')^{2/3} S^{1/2} \quad ...(a)$$

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However, if R is the *actual* hydraulic mean radius for the rippled bed, we have

$$V = \frac{1}{N}(R)^{2/3} S^{1/2} \quad ...(b)$$

Equating the above two equations, we have

$$R' = R\,(N'/N)^{3/2} \quad ...(14.42)$$

Relationship between φ_* and ψ_*

Since the relation between φ_* and ψ_* is quite complex, Einstein gave the relationship in the form of graph shown in Fig. 14.6. In order to save space (i.e. the length of the curve), he subdivided the entire graph into three parts : Graph III (for $0.01\,\varphi_* > 0.0001$), Graph II (for $1.0 > \varphi_* > 0.01$) and Graph I (for $100 > \varphi_* > 1.0$), each part having abscissa marked accordingly, as shown.

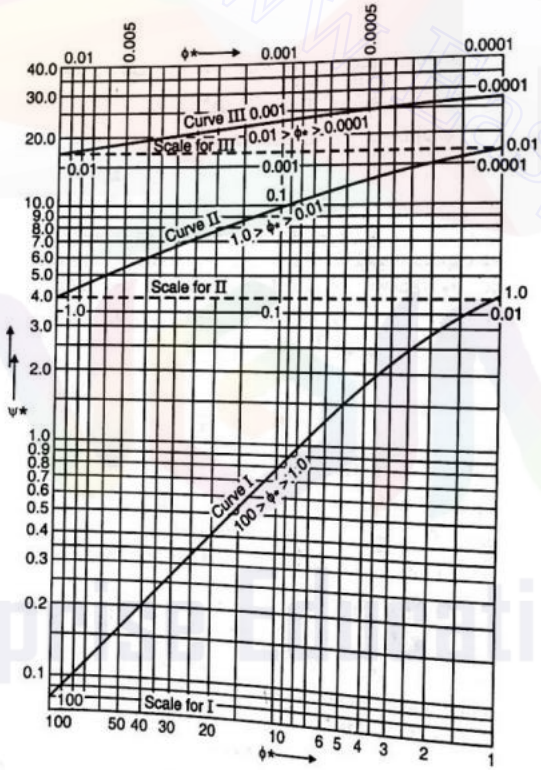

FIG. 14.6. EINSTEIN'S BED LOAD FUNCTIONS

If the value of ψ_* is known (or determined from Eq. 14.41), the corresponding value of φ_* is found from the graph and then the value of q_s is computed from Eq. 14.40.

Note the following values for *uniform soils* :

(*i*) $i_B = i_b$

(*ii*) $$\varphi_* = \varphi = \left[\frac{q_s}{G\,w}\left(\frac{1}{G-1}\right)^{1/2}\left(\frac{1}{g\,d^3}\right)^{1/2}\right] \quad ...(14.40\ a)$$

and (*iii*) $$\psi_* = \psi = \frac{(G-1)\,d}{R'\,S} \quad ...(14.40\ b)$$

Also, for uniform soils, the relation between φ and ψ can be expressed by the following equation :

$$\varphi = \frac{1}{0.465}\,e^{-0.391\psi} \quad ...(14.43)$$

Channel design based on concept of bed load transport

The above equations given by Meyer-Peter and Einstein can be used to design a channel for a given bed load transport. However, solution of these equations yield only the hydraulic mean depth R and the slope S, and not the bed width. Hence bed width B is chosen close to (or slightly less than) Lacey's regime perimeter. Also, the value of Manning's coefficient N for rippled bed has to be *estimated* on the basis of experience. See examples 14.15 and 14.16 for illustration.

ILLUSTRATIVE EXAMPLES

Example 14.8. *Design a channel section by Kennedy's theory given the following data :*

Discharge	$Q = 28$ *cumecs*
Kutter's	$N = 0.0225$
Critical velocity ratio	$m = 1$
Side slope	$= \frac{1}{2} : 1$
	$B/D = 7.6$

Find also the bed slope of the channel.

Solution

Let the depth of the channel be D

Hence width B of the channel $= 7.6\ D$

Velocity in the channel

$$V_0 = 0.55\,m\,D^{0.64} = 0.55\,D^{0.64}, \text{ taking } m = 1$$

Area A of the channel section having a side slopes $\frac{1}{2} : 1$ is

$$A = B\,D + D^2/2 = 7.6\,D^2 + D^2/2 = 8.1\,D^2$$

Continuity Equ. $\quad Q = A\,V_0$

$\therefore \quad 28 = (8.1\,D^2)\,(0.55\,D^{0.64}) = 4.46\,D^{2.64}$

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$$D^{2.64} = \frac{28}{4.46} = 6.28, \text{ from which } D = 2.01 \text{ m},$$

$\therefore$ $B = 2.01 \times 7.6 \simeq 15.3$ m

Hence

Hydraulic mean radius $R = A/P = \dfrac{A}{B + \sqrt{5}\,D} = \dfrac{8.1\,D^2}{7.6\,D + 2.236\,D} = 0.823\,D$

Taking $D = 2.01$ m, we get $R = 0.823 \times 2.01 = 1.66$ m

$V = 0.55\,(2.01)^{0.64} = 0.86$ m/s

$\therefore$ By applying Kutter's formula $V = C\sqrt{RS}$

where $C = \dfrac{23 + \dfrac{1}{N} + \dfrac{0.00155}{S}}{1 + \left(23 + \dfrac{0.00155}{S}\right)\dfrac{N}{\sqrt{R}}}$, we get

$$\frac{23 + \dfrac{1}{N} + \dfrac{0.00155}{S}}{1 + \left(23 + \dfrac{0.00155}{S}\right)\dfrac{N}{\sqrt{R}}}\sqrt{RS} = 0.86$$

or $$\left(23 + \frac{1}{0.0225} + \frac{0.00155}{S}\right)S^{1/2} = \frac{0.86}{\sqrt{1.66}}\left[1 + \left(23 + \frac{0.00155}{S}\right)\frac{0.0225}{\sqrt{1.66}}\right]$$

or $$67.4\,S^{3/2} - 0.885\,S + 0.00155\,S^{1/2} - 1.42 \times 10^{-5} = 0$$

Solving by trial and error, we get $S = \dfrac{1}{6250}$.

Check $$67.4\left(\frac{1}{6250}\right)^{3/2} - 0.885\left(\frac{1}{6250}\right) + 0.00155\left(\frac{1}{6250}\right)^{1/2} - 1.42 \times 10^{-5} = 0$$

$$13.65 \times 10^{-5} - 14.18 \times 10^{-5} + 1.96 \times 10^{-5} - 1.42 \times 10^{-5} = 0$$

$$15.61 - 15.60 \simeq 0$$

Hence, $S = \dfrac{1}{6250}$; **$B \simeq 15.3$ m ; $D = 2.01$ m.**

Example 14.9. *Find the channel section and discharge Q that can be allowed to flow in it, if $B/D = 5.7$, bed slope = 1/5000 and $N = 0.0225$. Use Kennedy's theory.*

Solution

Given $B/D = 5.7$. Hence, $B = 5.7$ D

For a channel with side slope $\frac{1}{2}:1$

$$R = \frac{A}{P} = \frac{B\,D + D^2/2}{B + \sqrt{5}\,D} = \frac{5.7\,D^2 + 0.5\,D^2}{5.7\,D + 2.236\,D} = \frac{6.2\,D^2}{7.936\,D} = 0.781\,D.$$

Assume critical veloctiy ratio $m = 1$.

Then $V = V_0 = 0.55D^{0.64}$ by Kennedy's formula and $V = C\sqrt{RS}$ by Kutter's formula.

Hence, $$0.55\,D^{0.64} = \frac{23 + \dfrac{1}{N} + \dfrac{0.00155}{S}}{1 + \left(23 + \dfrac{0.00155}{S}\right)\dfrac{N}{\sqrt{R}}}\sqrt{RS}$$

Putting $N = 0.0225$

$$\text{R.H.S.} = \frac{23 + \dfrac{1}{0.0225} + 0.00155 \times 5000}{1 + \left[23 + \dfrac{0.00155 \times 5000}{1}\right]\dfrac{0.0225}{\sqrt{0.781\,D}}}\sqrt{0.781\,DS}$$

$$= \frac{75.15}{1 + \dfrac{30.75 \times 0.0225}{0.885\sqrt{D}}}\sqrt{\frac{0.781D}{5000}} = \frac{66.5\,D^{1/2}}{(1 + 0.781\,D^{-1/2})\,70.7}$$

Equating L.H.S. and R.H.S. we get

$$70.7 \times 0.55\,D^{0.64} + 0.55 \times 0.781 \times 70.7\,D^{0.14} = 66.5\,D^{1/2}$$

$$38.88\,D^{0.64} - 66.5\,D^{0.5} + 30.37\,D^{0.14} = 0.$$

Solving the above equation by trial and error, we get $D = 1.7$ m

Hence, $B = 5.7 \times 1.7 = 9.69$ m

$V = 0.55 \times 1 \times (1.7)^{0.64} = 0.773$ m/sec

$A = B\,D + D^2/2 = 6.2\,D^2 = 6.2\,(1.7)^2 = 17.91$ m^2

Discharge $Q = A\,V = 17.91 \times 0.773 =$ **13.85 cumecs.**

Example 14.10. *Design an irrigation channel in alluvial soil according to Lacey's silt theory, given the following data :*

Full supply discharge $= 15$ *m^3/sec*

Lacey's silt factor $= 1.0$

Channel side slopes $= \frac{1}{2} : 1.$

Solution

1. Velocity V in the channel $= \left[\dfrac{Qf^2}{140}\right]^{1/6} = \left[\dfrac{15 \times 1}{140}\right]^{1/6} = 0.689$ m/sec.

2. Area A of the channel section $= \dfrac{Q}{V} = \dfrac{15}{0.689} = 21.765$ m^2.

3. Hydraulic mean radius $R = \dfrac{5}{2}\dfrac{V^2}{f} = \dfrac{5\,(0.689)^2}{2} = 1.187$ m.

4. $$R = 1.187 = \frac{A}{B + D\sqrt{5}} = \frac{21.765}{B + D\sqrt{5}}$$

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or $$B + D\sqrt{5} = \frac{21.765}{1.187} = 18.336 \quad ...(i)$$

and $$A = BD + D^2/2 = 21.765 \quad ...(ii)$$

Solving equations (*i*) and (*ii*), We get

$$B = \triangleq 15.3 \text{ m} \quad \text{and} \quad D = \triangleq 1.37 \text{ m}$$

5. *Check* : $P = 4.75\sqrt{Q} = 4.75\sqrt{15} \triangleq 18.4$ m

Also, $P = B + D\sqrt{5} = 15.3 + 1.37\sqrt{5} = 18.4$ m

Note. In Example 14.26, first A and P were computed from which B and D were determined. A check was then made by finding out the value of R by Lacey's formula as well as from the values of B and D. In this example, first A and R were computed in the order to determine B and D. A numerical check has been made in step (5) by finding values of P from the calculated values of B and D and from that of Lacey's formula. Both the methods are correct and students can follow any procedure.

4. Slope $$S = \frac{1}{3340\,(Q)^{1/6}} = \frac{1}{3340\,(15)^{1/6}} = \frac{1}{5245}.$$

Example 14.11. *The slope of channel in alluvium is $S = 1/5000$; Lacey's silt factor = 0.9 and channel side slope $= \frac{1}{2} : 1$. Find the channel section and maximum discharge which can be allowed to flow in it*

Solution

We have $$S = \frac{f^{5/3}}{3340\,Q^{1/6}}$$

or $$Q = \left[\frac{f^{5/3}}{3340\,S}\right]^6 = \frac{f^{10}}{(3340\,S)^6} = \left[\frac{5000}{3340}\right]^6 \times (0.9)^{10} = \mathbf{3.924\ m^3/sec}$$

Also $$S = \frac{f^{3/2}}{4980\,R^{1/2}}$$

or $$R = \frac{f^3}{(4980\,S)^2} = \frac{(0.9)^3 \times 5000 \times 5000}{4980 \times 4980} = 0.735$$

Now, $P = 4.75\sqrt{Q} = 4.75\sqrt{3.924} = 9.41$ m

Hence $A = P\,R = 9.41 \times 0.735 = 6.916$ sq.m

But $A = B\,D + D^2/2 = 6.916$ m^2 ...(*i*)

$P = B + \sqrt{5}\,D = 9.41$ m ...(*ii*)

Solving equations (*i*) and (*ii*), we get

$$\mathbf{B = 7.45\ m \text{ and } D \triangleq 0.88\ m}$$

Example 14.12. ***Bed load transport by Meyer-Peter's Equation***

A wide channel 3 m deep has a bed slope of 1/4400. The bed material consists of uniform grain size of 0.3 mm, Determine the quantity of bed load moved by the channel by Meyer-Peter's equation. Take representative size of roughness of channel bed, k = 1.5 mm, and specific gravity of sediment particles as 2.65.

Solution

From Eq. 14.39, $q_s = 47450\,[\tau_b\,(N'/N)^{3/2} - \tau_c]^{3/2}$ kN/m/hr.

Here, $$\tau_b = wDS = 9.81 \times 3 \times \frac{1}{4400} = 6.689 \times 10^{-3}\ \text{kN/m}^2$$

$$N' = \frac{(k_s)^{1/6}}{24} \quad \text{where } k_s = d = 0.3 \text{ mm}$$

and $$N = \frac{k^{1/6}}{24} \quad \text{where } k = 1.5 \text{ mm}$$

$$\therefore \quad \left(\frac{N'}{N}\right)^{3/2} = \left(\frac{d}{k}\right)^{1/4} = \left(\frac{0.3}{1.5}\right)^{1/4} = 0.6687$$

$$\tau_c = 0.047\,(w_s - w)\,d$$

where $w = 9.81$ kN/m^3, $w_s = wG = 9.81 \times 2.65 \triangleq 26$ kN/m^3 and $d = 3 \times 10^{-4}$ m

$$\therefore \quad \tau_c = 0.047\,(26 - 9.81) \times 3 \times 10^{-4} = 0.228 \times 10^{-3}\ \text{kN/m}^2$$

Substituting the values in Eq. 14.39, we get

$$q_s = 47450\,[6.689 \times 10^{-3} \times 0.6687 - 0.228 \times 10^{-3}]^{3/2}$$
$$= \mathbf{13.12\ kN/m/hour}$$

Example 14.13. ***Bed load-transport by Einstein's equation***

Determine the bed load transported by the channel of Example 14.12 by Einstein's equation. Neglect side friction.

Solution : For uniform soils

$$\psi_* = \psi = \frac{(G-1)\,d}{R'\,S} \quad \text{and} \quad R' = R\left(\frac{N'}{N}\right)^{3/2}$$

For wide channel, $R = D = 3$ m

As found in example 14.12, $(N'/N)^{3/2} = 0.6687$

$$\therefore \quad R' = 3 \times 0.6687 = 2.006 \text{ m}$$

Hence $$\psi = \frac{(2.65 - 1)\,3 \times 10^{-4}}{2.006 \times 1/4400} = 1.086$$

From Einstein's curve (Fig. 14.6), for $\psi_* = \psi = 1.086$, we get $\varphi_* = \varphi \triangleq 7.0$

From Eq. 14.40 (*a*), $$\varphi_* = \varphi = \frac{q_s}{Gw}\left(\frac{1}{G-1}\right)^{1/2}\left(\frac{1}{gd^3}\right)^{1/2}$$

or $$q_s = \varphi\,Gw\,(G-1)^{1/2}\,(g\,d^3)^{1/2}$$
$$= 7 \times 2.65 \times 9.81\,(2.67 - 1)^{1/2}\,(9.81)^{1/2}\,(3 \times 10^{-4})^{3/2}\ \text{kN/m/s}$$
$$= 4.348 \times 10^{-3}\ \text{kN/m/s} = 3.804 \times 10^{-3} \times 3600\ \text{kN/m/h} = \mathbf{13.7\ kN/m/h}$$

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Example 14.14. ***Determination of concentration of suspended load***

Determine the concentration of suspended load at 1.0 m above the channel bed of Example 14.12 if the concentration at 0.3 m above the bed is 400 ppm. The particles have a fall velocity of 0.03 m/sec.

Solution : Given : $C_a = 400$ ppm at 0.3 above bed and $V_s = 0.03$ m/sec.

Here, $a = 0.3$ m. $D = 3$ m and $y = 1$ m

Now $V_* = \sqrt{g R' S}$, where $R' = 2.006$ m found in the previous example.

$$V_* = \sqrt{\frac{9.81 \times 2.006}{4400}} = 0.0669 \text{ m/sec.}$$

$\therefore$ $k =$ Von Karman's constant $= 0.4$

$$\therefore \quad \frac{V_s}{V_* k} = \frac{0.03}{0.0669 \times 0.4} = 1.121$$

From Eq. 14.35,

$$\frac{C}{C_a} = \left[\frac{a}{y} \times \frac{D-y}{D-a}\right]^{V_s/V_* k}$$

$$\therefore \quad C = 400\left[\frac{0.3}{1} \times \frac{3-1}{3-0.3}\right]^{1.121} = \mathbf{74.1\ ppm}$$

Example 14.15. ***Channel design on Meyer-Peter's equations***

Using Meyer-Peter's equations, design an irrigation channel carrying a full supply discharge of 45 cumecs with a bed load concentration of 55 ppm by weight. The average grain diameter of bed material = 0.3 mm and G = 2.67.

Solution

Quantity of bed load to be transported $= (55 \times 10^{-6}) \times 45 \times 9.81 \times 60 \times 60 = 87.41$ kN/m/h

Lacey's regime perimeter $P = 4.75\sqrt{Q} = 4.75\sqrt{45} = 31.86$ m

Let us fix the channel width $B = 28$ m (i.e. slightly less than P).

$\therefore$ Rate of bed load transported, $q_s = 87.41/28 = 3.12$ kN/m/hr

Let the effective grain diameter, k_s be equal to 0.4 mm (i.e. slightly more than the average grain diameter of 0.3 mm).

Hence $$N' = \frac{(k_s)^{1/6}}{24} = \frac{(0.4 \times 10^{-3})^{1/6}}{24} = 0.0113$$

Lacey's silt factor $= 1.76\sqrt{m_d} = 1.76\sqrt{0.3} = 0.964$

Hence from the Eq. $N_a = 0.0225 f^{1/4}$, we get $N = N_a = 0.0225(0.964)^{1/4} = 0.0223$

$$\therefore \quad \frac{N'}{N} = \frac{0.0113}{0.0223} = 0.507$$

Now, $\tau_b = (Q_s/Q)\, wRS = wRS$ by assuming $Q_s/Q = 1$

$\therefore$ $\tau_b = 9.81\, RS$ kN/m²

Also, $\tau_c = 0.047\,(w_s - w)\, d = 0.047 \times 9.81\,(2.67 - 1) \times 0.3 \times 10^{-3}$

$= 0.231 \times 10^{-3}$ kN/m²

Now $q_s = 47450\,[\tau_b\,(N'/N)^{3/2} - \tau_c]^{3/2}$

Substituting the values of various terms, we get.

$$3.12 = 47450\,[9.81\, RS\,(0.507)^{3/2} - 0.231 \times 10^{-3}]^{3/2}$$

From which, we get $RS = 0.525 \times 10^{-3}$ ...(a)

Also, $A = PR = 31.86\, R$

From Manning's formula $Q = \frac{1}{N} A R^{2/3} S^{1/2}$

$$\therefore \quad 45 = \frac{1}{0.0223} \times 31.86\, R \times R^{2/3} S^{1/2}$$

or $R^{5/3} S^{1/2} = 0.031$

or $R^{10/3} S = 0.000992$ ...(b)

Hence from (a) and (b), $R^{7/3} = 0.000992/0.525 \times 10^{-3} = 1.8897$

From which $R = 1.314$ m

Hence from (a), $S = 0.525 \times 10^{-3}/1.314 = 0.3997 \times 10^{-3} \simeq 1/2500$

For a trapezeridal channel with $\frac{1}{2}:1$ side slopes,

$$R = \frac{BD + 0.5\,D^2}{B + 2.236\,D} \text{ where } B = 28 \text{ m and } R = 1.314$$

$$\therefore \quad 28\,D + 0.5\,D^2 = 1.314\,(28 + 2.236\,D)$$

or $0.5\,D^2 + 25.06\,D - 36.792 = 0$ from which $D \simeq \mathbf{1.43\ m}$

Hence the channel dimension are : $\mathbf{B = 28\ m; D = 1.43\ m}$ and $\mathbf{S = 1/2500}$

Example 14.16. ***Channel design on Eienstein's Equations***

Redesign the channel of example 14.15, using Eienstein's equations.

From Eq. 14.40 (a), $$\varphi_* = \varphi = \frac{q_s}{Gw}\left(\frac{1}{G-1}\right)^{1/2}\left(\frac{1}{g\,d^3}\right)^{1/2}$$

Substituting the given values, we get

$$\varphi = \left(\frac{3.12}{3600}\right)\left(\frac{1}{2.67 \times 9.81}\right)\left(\frac{1}{2.67-1}\right)^{1/2}\left[\frac{1}{9.81\,(0.0003)^3}\right]^{1/2} = 1.573$$

Hence from Fig. 14.6, we get $\psi_* = \psi \simeq 3.4$ for $\varphi = \varphi_* = 1.573$

Now $$\psi_* = \psi = \frac{(G-1)\,d}{R'\,S}$$

$$\therefore \quad R'\,S = \frac{(2.67-1)\,0.0003}{3.4} = 0.1474 \times 10^{-3}$$

But $R' = R\,(N'/N)^{3/2}$ where $N'/N = 0.507$ from previous example.

$\therefore$ $R' = R\,(0.507)^{3/2} = 0.361\,R$

$\therefore$ $0.361\,R\,S = 0.1474 \times 10^{-3}$

or $RS = 0.4083 \times 10^{-3}$ ...(1)

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Also, from Example 14.15, we have ...(2)

$$R^{10/3} S = 0.000992$$

Hence from (1) and (2), we get $R^{7/3} = 2.4296$ or $R = 1.463$ m

and from (1), $S = 0.4083 \times 10^{-3}/1.463 = 1/3580$

For a trapezeridal channel with $\frac{1}{2}:1$ side slopes,

$$R = \frac{BD + 0.5\,D^2}{B + 2.236\,D} \text{ where } B = 28 \text{ m selected in Example 14.15}$$

$$\therefore \quad 1.463 = \frac{28\,D + 0.5\,D^2}{28 + 2.236\,D}$$

or $0.5\,D^2 + 24.73\,D - 40.96 = 0$, from which $D \simeq 1.60$ m

Hence the channel dimensions are :

$$B = 28 \text{ m}; \; D = 1.6 \text{ m and } S = 1/3580$$

14.15. DESIGN OF NON-ALLUVIAL CHANNELS

As defined earlier, non-alluvial channels are those which flow through non-alluvial soils such as loam, clay, mooram and other hard soils, including boulders and rocks. In the non-alluvial soils, water is relatively clear, due to which there is no problem of silting. These channels are, therefore, considered as stable channels. Non-alluvial channels are designed on the basis of *maximum permissible velocity* that could be permitted in the channel without causing any scour either in the bed or in the sides. Table 14.14 gives the values of permissible velocities in various types of bed material.

TABLE 14.14. VALUES OF PERMISSIBLE VELOCITIES

Type of bed material	*Permissible velocity (m/s)*	*Type of bed material*	*Permissible velocity (m/s)*
1. Ordinary soils	0.6 to 0.9	6. Hard soil & Moorum	1.0 to 1.2
2. Loam; lean clay	0.4 to 1.0	7. Gravel	1.2 to 1.5
3. Ordinary clay	0.5 to 1.2	8. Boulders	1.5 to 1.8
4. Heavy clay	0.6 to 1.6	9. Soft rock	1.8 to 2.4
5. Light loose sand	0.3 to 0.6	10. Hard rock	2.5 to 5

Side slopes

The side slopes (H : V) of channels in ordinary soils, including clay, are generally kept as 1:1 in cutting and 1.5 : 1 in filling. However, in grit (or gravel), soft rock and hard rock, the side slopes are kept as 0.5 : 1, 0.25 : 1 and 0.125 : 1 respectively though the sides may be kept even vertical in very hard rocks.

Flow equations

The design of non-alluvial channels is done on the basis of the following two flow equations.

(*i*) Chezy's formula, and (*ii*) Manning's formula

1. Chezy's formula

$$V = C\sqrt{RS} \quad \text{...(14.44)}$$

where C is Chezy's coefficient usually determined from the following equation by Bazin.

$$C = \frac{87}{1 + K/\sqrt{R}} \quad \text{...(14.45)}$$

where the K is Bazin's coefficient the value of which may be taken 1.2 to 1.4 for earth channels in good condition and between 1.7 to 1.8 for earth channels in poor condition.

(i) Manning's formula

$$V = \frac{1}{N} R^{2/3} S^{1/2} \quad \text{...(14.46)}$$

where N is Manning's coefficient (Table 14.15) the values of which depends upon the type of surface of channel.

TABLE 14.15. VALUES OF MANNING'S COEFFICIENT N

Type of surface	*Value of N*
1. Earth channels; clean straight and uniform	0.016 to 0.020
2. Earth channels; clean but weathered	0.018 to 0.025
3. Earth channels; with grass and weeds	0.022 to 0.033
4. Channels in gravel with stones	0.03 to 0.035
5. Channels in rock; smooth and uniform	0.025 to 0.04
6. Channels in rock; rough	0.035 to 0.050

Design procedure

For a channel design, the discharge Q and bed slope S are generally known. The value of maximum permissible velocity and side slopes ($r : 1$) are chosen on the basis of type of soil through which the channel has to flow. Generally, Manning's formula is selected as the flow equation.

Step 1: Determine the area of cross-section from the continuity equation :

$$A = Q/V \quad \text{...(i)}$$

Step 2 : Determine hydraulic mean radius from Manning's formula (Eq. 14.46).

$$R = \left(\frac{VN}{S^{1/2}}\right)^{3/2} \quad \text{...(ii)}$$

Step 3. Determine wetted perimeter (P) from the relation

$$P = A/R \quad \text{...(iii)}$$

Step 4 For a trapezoidal channel with side slopes *r : 1* (Fig. 14.7), the area of cross-section (A) and wetted perimeter (P) are given by the following expressions.

$$A = (B + rD)\,D \quad \text{...(14.47)}$$

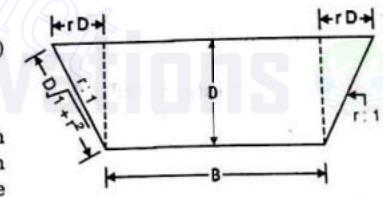

FIG. 14.7

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and $$P = B + 2(D\sqrt{1+r^2}) \quad ...(14.48)$$

In the above two equation, the L.H.S. (i.e. A and P) are known from (*i*) and (*ii*) respectively. Hence B and D can be computed by solution of the above two simultaneous equations.

Most efficient channel of trapezoidal section

A most efficient channel is the one which has minimum wetted perimeter for a given area. Let the side slopes of the trapezoidal section be r (H) : 1 (V). Then if B is the bed width and D is the water depth,

$$A = (B + rD)D \quad ...(i)$$

and $$P = (2\sqrt{1+r^2})D + B \quad ...(ii)$$

Substituting the value of B from (*i*) into (*ii*), we get

$$P = 2D\sqrt{1+r^2} + \frac{A}{D} - rD$$

$$\therefore \quad \frac{dP}{dD} = 0 = 2\sqrt{1+r^2} - AD^{-2} - r$$

or $$A = (2\sqrt{1+r^2} - r)D^2 \quad ...(iii)$$

Hence from (*i*), $$B = \frac{A}{D} - rD = (2\sqrt{1+r^2} - r)D - rD$$

or $$B = 2(\sqrt{1+r^2} - r)D \quad ...(iv) \quad ...(14.49)$$

Subtituting the value of B in (*ii*) and (*i*) we get

$$P = (2\sqrt{1+r^2})D + 2(\sqrt{1+r^2} - r)D$$

or $$P = 2D(2\sqrt{1+r^2} - r] \quad ...(v)$$

and $$A = \{2D(\sqrt{1+r^2} - r) + rD\}D = D^2(2\sqrt{1+r^2} - r)$$

Hence $$R = \frac{A}{P} = \frac{D^2(2\sqrt{1+r^2} - r)}{2D(2\sqrt{1+r^2} - r)} = \frac{D}{2} \quad ...(14.50)$$

Hence for the most efficient channel, $R = D/2$. This is one condition obtained for having minimum value of P. However, P is dependent on two variables : D and r. A second condition can therefore be obtained by putting $dP/dr = 0$, keeping D as constant.

Now $$P = 2D(2\sqrt{1+r^2} - r) = 4D\sqrt{1+r^2} - 2rD$$

$$\therefore \quad \frac{dP}{dr} = 0 = 4D\frac{1}{2}(1+r^2)^{-1/2} \times 2r - 2D = \frac{4rD}{\sqrt{1+r^2}} - 2D$$

or $$\frac{2r}{\sqrt{1+r^2}} = 1 \quad \text{or } 4r^2 = 1 + r^2$$

From which $$r = \frac{1}{\sqrt{3}} \quad ...(14.51)$$

Hence the most efficient trapezoidal channel is the one which has side slopes of $1/\sqrt{3} : 1$ (H : V) and $R = D/2$

Example 14.17. *Design of channel in non-alluvial soil*

Design a channel for $Q = 45$ cumecs in non-alluvial soil having maximum permissible velocity of 0.9 m/sec. The available bed slope is 1 in 4000. Assume Manning's $N = 0.025$.

Solution

Step 1 : $A = Q/V = 45/0.9 = 50 \text{ m}^2$

Step 2 : $$R = \left[\frac{VN}{S^{1/2}}\right]^{3/2} = \left(\frac{0.9 \times 0.025}{(1/4000)^{1/2}}\right)^{3/2} \simeq 1.7 \text{ m}$$

Step 3 $P = A/R = 50/1.7 = 29.45$ m

Step 4 Let us provide a trapezoidal section of the channel with sides slopes of 1:1 (i.e. $r = 1$)

$$\therefore \quad A = (B + D)D = 50 \quad ...(a)$$

and $$P = B + 2D\sqrt{2} = 29.45 \quad ....(b)$$

Substituting the value of B from Eq. (*b*) into Eq. (*a*), we get

$$D^2 + (29.45 - 2.828D)D = 50$$

or $1.828D^2 - 29.45D + 50 = 0$ from which $D = 1.929 \text{ m} \simeq$ **1.93 m**

and $B =$ **24 m**

Example 14.18. *Design of non-alluvial channel*

Using Bazin's formula, design a non-alluvial channel carrying discharge of 15 cumecs with a mean velocity of 0.75 m/s. The channel has bottom width as five times the depth, and has side slopes of 1:1. Assume Bazin's coefficient $K = 1.3$.

Solution

Step 1 : $A = Q/V = 15/0.75 = 20 \text{ m}^2$

Step 2 : $A = (B + D)D$ where $B = 5D$ and $A = 20 \text{ m}^2$

$\therefore$ $20 = (5D + D)D = 6D^2$, from which $D = 1.826 \text{ m} \simeq 1.83$ m

and $B = 5D = 5 \times 1.826 = 9.13$ m

Step 3 : $P = B + 2D\sqrt{2} = 9.13 + 2 \times 1.83\sqrt{2} = 14.31$ m

$\therefore$ $R = A/P = 20/14.31 \simeq 1.4$ m

Step 4 : $$C = \frac{87}{1 + K/\sqrt{R}} = \frac{87}{1 + 1.3/\sqrt{1.4}} = 41.44$$

Step 5 : $V = C\sqrt{RS}$, from which

$$S = \left(\frac{V}{C}\right)^2 \frac{1}{R} = \left(\frac{0.75}{41.44}\right)^2 \frac{1}{1.4} \simeq \frac{1}{4270}$$

Hence the channel dimension are :

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$$B = 9.1 \text{ m}; D = 1.83 \text{ and } S = 1/4270$$

14.16. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 14.19. *A most efficient trapezoidal section is required to give a maximum discharge of 21.5 m^3/s of water. The slope of the channel bottom is 1 in 2500. Taking $C = 70\ m^{1/2}/sec$. in Chezy's equation, determine the dimensions of the channel. Also determine the value of Manning's n taking the value of velocity of flow as obtained for the channel by Chezy's equation.* **(Engg. Services Exam. 1997)**

Solution : We have seen in §14.15 that a most efficient trapezoidal section is the one which has $R = D/2$ and $r = 1/\sqrt{3}$. Corresponding to these, the values of B, A, P and R are as under :

$$\left.\begin{aligned} B &= 2D(\sqrt{1+r^2} - r) \\ A &= D^2(2\sqrt{1+r^2} - r) \\ \text{and} \quad P &= 2D(2\sqrt{1+r^2} - r) \end{aligned}\right] \quad \text{where } r = 1/\sqrt{3} = 0.5774$$

Note: The students are required to prove the above expressions.

Now, $Q = V \times A = C\sqrt{RS} \times A$

Subtituting $Q = 21.5\ (m^3/s)$, $C = 70\ (m^{1/2}/s)$ and $S = 1/2500$, we have

$$21.5 = 70\sqrt{R}\left(\frac{1}{\sqrt{2500}}\right)A = \frac{70}{50}A\sqrt{R} \quad ...(i)$$

Now $A = D^2\left(2\sqrt{1+\frac{1}{3}} - \frac{1}{\sqrt{3}}\right) = 1.732\,D^2$ and $R = 0.5\,D$

$$\therefore \quad 21.5 = \frac{70}{50}(1.732\,D^2)\sqrt{0.5\,D} = 1.7146\,D^{2.5}$$

From which $D = \mathbf{2.75\ m}$

$$B = 2D\left(\sqrt{1+\frac{1}{3}} - \frac{1}{\sqrt{3}}\right) = 1.1547\,D = 1.1547 \times 2.75 \simeq \mathbf{3.18\ m}$$

Hence the channel has the following dimensions :

$B = 3.18$ m; $D = 2.75$ m; side slopes $1/\sqrt{3}\ (H) : 1$

Now from Chezy's formula, $V = C\sqrt{RS} = 70\left(\frac{2.75}{2}\right)^{1/2}\left(\frac{1}{2500}\right)^{1/2} = 1.642$ m/s

From Manning's equation, $V = \frac{1}{n}R^{2/3}S^{1/2}$

$$\therefore \quad 1.642 = \frac{1}{n}\left(\frac{2.75}{2}\right)^{2/3}\left(\frac{1}{2500}\right)^{1/2} = \frac{0.02473}{n}$$

From which, $n = \mathbf{0.0151}$

Example 14.20. *Design a regime channel to carry a discharge of 100 cumecs in soil having a silt factor of 1.1, using Lacy's theory. Assume side slopes of 0.5 horizontal to 1.0 vertical.* **(Engg. Services Exam. 1984)**

Solution : Given : $Q = 100$ cumecs; $f = 1.1$; side slopes 0.5 : 1

$$V = \left(\frac{Qf^2}{140}\right)^{1/6} = \left[\frac{100\,(1.1)^2}{140}\right]^{1/6} = 0.976 \text{ m/sec.}$$

$$A = Q/V = 100/0.976 = 102.46\ m^2$$

$$P = 4.75\sqrt{Q} = 4.75\,(100)^{1/2} = 47.5 \text{ m}$$

$$D = \frac{P - \sqrt{P^2 - 6.944\,A}}{3.472} = \frac{47.5 - \sqrt{(47.5)^2 - 6.944 \times 102.46}}{3.472} \simeq \mathbf{2.36\ m}$$

$$B = P - 2.236\,D = 47.5 - 2.236 \times 2.36 \simeq \mathbf{42.2\ m}$$

Check : $R = \frac{5}{2}\frac{V^2}{f} = \frac{5}{2}\frac{(0.976)^2}{1.1} \simeq 2.16$ m

Also, $R = \frac{BD + D^2/2}{B + 2.236\,D} = \frac{42.2 \times 2.36 + 0.5\,(2.36)^2}{42.2 + 2.236 \times 2.36} \simeq 2.16$ m

Hence OK.

Lastly, $S = \frac{f^{5/3}}{3340\,Q^{1/6}} = \frac{(1.1)^{5/3}}{3340\,(100)^{1/6}} \simeq \mathbf{\frac{1}{6140}}$

Example 14.21. *Design a channel by Lacey's theory for 40 cumecs capacity. The side slopes may be assumed to be 1:1. The average size of the bed material may be taken as 0.8 mm.* **(Civil Services Exam. 1986)**

Solution

Silt factor, $f = 1.76\sqrt{m_d} = 1.76\sqrt{0.8} \simeq 1.57$

$$V = \left(\frac{Qf^2}{140}\right)^{1/6} = \left(\frac{40\,(1.57)^2}{140}\right)^{1/6} = 0.943 \text{ m/sec}$$

$$A = Q/V = 40/0.943 = 42.41\ m^2$$

$$P = 4.75\sqrt{Q} = 4.75\sqrt{40} = 30.04 \text{ m}$$

For a trapezoidal canal with 1:1 side slopes, we have

$$A = D \times \frac{B + (B + 2D)}{2} = D\,(B + D)$$

$$P = B + 2\,(D\sqrt{2}) = B + 2.828\,D$$

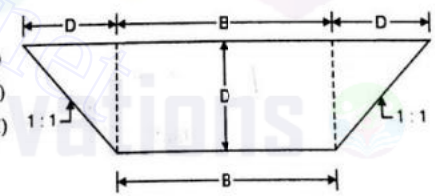

FIG. 14.8.

Hence $D\,(B + D) = 42.41$ ...(i)

and $B + 2.828\,D = 30.04$ ...(ii)

Substituting the value of B from (ii) into (i), we get

$$D^2 + D\,(30.04 - 2.828\,D) = 42.41$$

or $1.828\,D^2 - 30.04D + 42.41 = 0$

from which $D = \mathbf{1.56\ m}$

Hence from (ii), $B = 30.04 - 2.828 \times 1.56 = 25.63\ m \simeq \mathbf{25.7\ m}$

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Check : $R = \frac{5}{2}\frac{V^2}{f} = \frac{5}{2}\frac{(0.943)^2}{1.57} = 1.416$ m

$A = 1.56\,(25.63 + 1.56) = 42.416\ m^2$

$P = 25.63 + 2.828 \times 1.56 = 30.04$

$R = 42.416/30.04 \triangleq 1.412$ m. Hence OK.

$\therefore$ Finally, $S = \frac{f^{5/3}}{3340\,Q^{1/6}} = \frac{(1.57)^{5/3}}{3340\,(40)^{1/6}} = \mathbf{\frac{1}{2912}}$

Example 14.22. *A certain stretch of a lined trapezoidal channel has one side vertical wall and the other 45° sloping wall. It is is to deliver water at 30 m^3/sec with a velocity of 1 m/sec. Compute bed width and flow depth for minimum lining area.* ***(Engg. Services Exam. 1988)***

Solution :

Given : $Q = 30$ cumecs; $V = 1$ m/sec.

$A = Q/V = 30/1 = 30\ m^2$

The canal section is shown in Fig. 14.9.

$A = D\,[B + (B + D)]/2 = 0.5\,D\,(2\,B + D)$

Hence $0.5\,D\,(2\,B + D) = 30\ m^2$ ...(1)

For minimum lining area, wetted perimeter will be minimum.

$P = D + B + D\sqrt{2} = B + 2.414\,D$ ...(2)

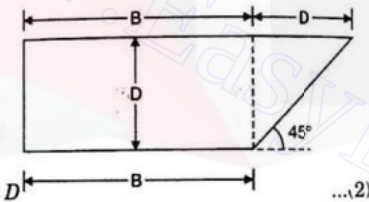

FIG. 14.9.

But from (1), $B = \frac{1}{2}\left(\frac{30}{0.5\,D} - D\right) = \frac{30}{D} - \frac{D}{2}$ ...(3)

Hence from (2), $P = \frac{30}{D} - \frac{D}{2} + 2.414\,D = \frac{30}{D} + 1.914D$ ...(4)

$\therefore$ $\frac{dP}{dD} = 0 = -30\,D^{-2} + 1.914$, from which $D \triangleq$ **3.96 m**

Hence from (3), $B = \frac{30}{3.96} - \frac{3.96}{2} \triangleq$ **5.6 m**

Check : $A = 0.5\,D\,(2\,B + D) = 0.5 \times 3.96\,(2 \times 5.6 + 3.96) \triangleq 30\ m^2$

Hence OK.

Example 14.23. *A trapezoidal channel with a base of 6 m and side slopes of 2 horizontal to 1 vertical conveys water at 17 m^3/sec with a depth of 1.5 m. Is the flow situation sub or super critical ?* ***(Engg. Services Exam. 1988)***

Solution : Given : $B = 6$ m; $D = 1.5$ m; $Q = 17\ m^3$/sec.

The canal section is shown in Fig 14.10, from which

$A = 1.5\left(\frac{6 + 12}{2}\right) = 13.5\ m^2.$

$\therefore$ $V = Q/A = 17/13.5 = 1.259$ m/sec.

Wetted perimeter $P = 6 + 2\sqrt{(3)^2 + (1.5)^2} = 12.71$ m

$\therefore$ $R = A/P = 13.5/12.71 = 1.062$

Froude's number $= \frac{V}{\sqrt{gR}} = \frac{1.259}{\sqrt{9.81 \times 1.062}} = 0.39 < 1$

Hence the ***flow is sub-critical.***

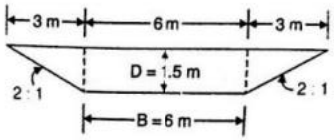

FIG. 14.10.

Example 14.24. *A trapezoidal channel with bottom width 3.5 m and side slopes 1 H : 1 V on the left and 1.5 H : 1 V on the right with n = 0.016 and a bed slope of 2.6 in 10000, carries a discharge of 8 m^3/sec. Determine the normal depth and the average sheer stress on the channel bed.* ***(Engg. services Exam, 2003)***

Solution :

Given $B = 3.5$ m ; $Q = 8\ m^3$/sec.

Left side slope = 1 : 1

Right side slope = 1.5 : 1

The channel section is shown in Fig. 14.11.

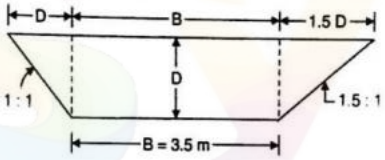

FIG. 14.11.

$$A = D\left[\frac{B + (B + 2.5\,D)}{2}\right] = D\,(B + 1.25\,D) = D\,(3.5 + 1.25\,D) \quad ...(1)$$

$$P = B + D\sqrt{2} + \sqrt{D^2 + (1.5\,D)^2} = B + 3.217\,D = 3.5 + 3.217\,D \quad ...(2)$$

$$\therefore \quad R = \frac{A}{P} = \frac{D\,(3.5 + 1.25\,D)}{3.5 + 3.217\,D} \quad ...(3)$$

Also, $V = Q/A = 8\,/\,D\,(3.5 + 1.25\,D)$ ...(4)

From Mannings formula, we have

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

$$\therefore \quad \frac{8}{D\,(3.5 + 1.25\,D)} = \frac{1}{0.016}R^{2/3}\left(\frac{2.6}{10000}\right)^{1/2}$$

or $$R = \left[\frac{7.9382}{D\,(3.5 + 1.25\,D)}\right]^{1.5} \quad ...(5)$$

Equating (3) and (5), we get

$$\frac{D\,(3.5 + 1.25\,D)}{3.5 + 3.217\,D} = \frac{22.366}{D^{1.5}\,(3.5 + 1.25\,D)^{1.5}}$$

or $$[D\,(3.5 + 1.25\,D)]^{2.5} = 78.281 + 71.951\,D \quad ...(6)$$

Solving Eq. (6) by *trial and error*, we get $D \triangleq$ **1.5 m**

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Hence $$R = \frac{1.5\,(3.5 + 1.25 \times 1.5)}{3.5 + 3.217 \times 1.5} \simeq 0.97 \text{ m}$$

Check : From (5), $$R = \left[\frac{7.9382}{1.5\,(3.5 + 1.25 \times 1.5)}\right]^{1.5} = 0.98 \text{ m}$$

Both the values are nearly equal. Hence OK.

Average shear stress on channel bed $= wRS$

$$= 9.81 \times 0.97 \times \frac{2.6}{10000} = 2.47 \times 10^{-3} \text{ kN/m}^2$$

$$= \mathbf{2.47\ N/m^2}$$

Example 14.25. *A channel is 40 m wide 2.6 m deep, with a slope 1 in 5000. The representative bed material size D_{65} is 0.64 mm, and mean diameter of the bed material is 0.38 mm. Manning's $n = 0.021$. Neglecting the side friction, find the bed load transported by the channel in tonnes per day by Meyer-Peter equation, considering B/D as large and $\tau_c = 0.075\,d$.* **(Engg. Services Exam 2003)**

Solution : Given $B = 40$ m; $D = 2.6$ m; $S = 1/5000$

$D_{65} = 0.64$ mm; $m_d = 0.38$ mm; $n = 0.021$; $\tau_c = 0.075\,d$

From Eq. 14.39 (*b*), bed load transported is given by

$$q_s = 4700\,[\tau_b\,(N'/N)^{3/2} - \tau_c]^{3/2} \text{ kg/m/hr.}$$

where $$\tau_b = w\,DS = 1000 \times 2.6 \times \frac{1}{5000} = 0.52 \text{ kg/m}^2$$

$$N' = \frac{(k_s)^{1/6}}{24} = \frac{(D_{65})^{1/6}}{24} = \frac{(0.64 \times 10^{-3})^{1/6}}{24} = 0.0122$$

N = actual Manning's coefficient $= 0.021$ (given)

$\tau_c = 0.075\,d$ where d = mean diameter $= 0.38$ mm

$= 0.075 \times 0.38 = 0.0285$ kg/m²

$$\therefore \quad q_s = 4700\left[0.52\left(\frac{0.0122}{0.021}\right)^{3/2} - 0.0285\right]^{3/2} \simeq 426 \text{ kg/m/hr}$$

$$= 426 \times 10^{-3} \times 24 \text{ t/m/day} = 10.22 \text{ t/m/day}$$

$\therefore$ Bed load transported by 40 m wide channel

$$= 10.22 \times 40 \text{ t/day} \simeq \mathbf{409\ t/day}$$

Example 14.26. *Calculate the bed width for an irrigation channel to carry a discharge of 5 cumecs. Side slopes of the channel are 1/2 horizontal to 1 vertical. Assume critical velocity ratio as 0.8. Take first trial depth of flow as 1 m. The channel as a bed slope of 0.2 m per km. Using Kutter's equation $V = C\sqrt{RS}$, check the depth. The value of C is 30. V is mean velocity of flow in m/sec and R is hydraulic mean radium in m.* **(Engg. Services Exam. 1995)**

Solution : Given $Q = 5$ m³/sec ; $S = 0.2/1000$; $m = 0.8$

Assuming $D = 1$ m is the first trial,

$$V_0 = 0.55\,m\,D^{0.64} = 0.55 \times 0.8\,(1)^{0.64} = 0.44 \text{ m/s}$$

$$A = Q/V_0 = 5/0.44 = 11.36 \text{ m}^2$$

For a trapezoidal section having 1/2 :1 side slopes,

$$A = BD + 0.5\,D^2 = D\,(B + 0.5\,D)$$

$\therefore$ $11.36 = 1\,(B + 0.5 \times 1)$ from which $B = 10.86$ m

$$P = B + D\sqrt{5} = 10.86 + 1 \times 2.236 \simeq 13.1 \text{ m}$$

and $$R = A/P = 11.36/13.1 = 0.867 \text{ m}$$

Actual $$V = C\sqrt{RS} = 30\left(0.867 \times \frac{0.2}{1000}\right)^{1/2} = 0.395 \text{ m/s.}$$

Thus, $V_0 > V$

In order to reduce V_0 given by Kennedy's equation, reduce D in the second trial.

Second trial : Let $D = 0.8$ m.

$$\therefore \quad V_0 = 0.55 \times 0.8\,(0.8)^{0.64} = 0.381 \text{ m/s}$$

$$A = 5/0.381 = 13.1 \text{ m}^2$$

$$B = \frac{A}{D} - 0.5\,D = \frac{13.1}{0.8} - 0.5 \times 0.8 = 15.99 \text{ m}$$

$$P = B + 2.236\,D = 15.99 + 2.236 \times 0.8 = 17.77 \text{ m}$$

$$R = 13.1/17.77 = 0.737 \text{ m}$$

Actual $$V = 30\left(0.737 \times \frac{0.2}{1000}\right)^{1/2} = 0.364 \text{ m/s}$$

This, $V_0 > V$. Hence reduce D further in the next trial.

Third trial : Let $D = 0.7$ m

$$\therefore \quad V_0 = 0.55 \times 0.8\,(0.7)^{0.64} = 0.35 \text{ m/s.}$$

$$A = 5/0.35 = 14.28 \text{ m}^2$$

$$B = \frac{14.28}{0.7} + 0.5 \times 0.7 = 20.75 \text{ m}$$

$$P = 20.75 + 2.236 \times 0.7 = 22.31 \text{ m}$$

$$R = 14.28/22.31 = 0.64 \text{ m}$$

Actual $$V = 30\left(0.64 \times \frac{0.2}{1000}\right)^{1/2} \simeq 0.34 \text{ m/s}$$

Since $V \simeq V_0$, no further trials are required.

Hence $\mathbf{D = 0.7\ m}$ **and** $\mathbf{B = 20.75\ m}$

Example 14.27. *Design an irrigation channel by Kennedy method to carry 50 cubic meters per second of discharge with base width to depth ratio as 2.5. the critical velocity ratio is 1.1. Assume Kutter's rugosity coefficient as 0.025 and side slope of the channel as 0.5 horizontal to 1.0 vertical.* **(Engg. Services Exam. 1984)**

Solution : Given $B/D = 2.5$; $Q = 50$ cumecs; $m = 1.1$; $N = 0.025$

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For a trapezoidal channel with $\frac{1}{2}:1$ side slopes,

$$A = BD + 0.5\,D^2 = D^2\left(\frac{B}{D} + 0.5\right) = D^2\,(2.5 + 0.5) = 3\,D^2$$

$$V_0 = 0.55\,m\,D^{0.64} = 0.55 \times 1.1\,D^{0.64} = 0.605\,D^{0.64}$$

$$Q = A \times V_0 = (3\,D^2)\,(0.605\,D^{0.64}) = 1.815\,D^{2.64}$$

$$\therefore \quad D = \left(\frac{Q}{1.815}\right)^{1/2.64} = \left(\frac{50}{1.815}\right)^{1/2.64} = \mathbf{3.51\ m}$$

$$B = 2.5\,D = 2.5 \times 3.51 = \mathbf{8.78\ m}$$

and

$$R = \frac{BD + 0.5\,D^2}{B + 2.236\,D} = \frac{8.78 \times 3.51 + 0.5\,(3.51)^2}{8.78 + 2.236 \times 3.51} = 2.224 \text{ m}$$

$$V_0 = 0.605\,(3.51)^{0.64} = 1.351 \text{ m/sec.}$$

But

$$V = \frac{23 + \frac{1}{N} + \frac{0.00155}{S}}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{N}{\sqrt{R}}}\sqrt{RS}$$

$$\therefore \quad 1.351 = \frac{23 + \frac{1}{0.025} + \frac{0.00155}{S}}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{0.025}{\sqrt{2.224}}}\sqrt{2.224 \times S}$$

or

$$1.351 = \frac{23 + 40 + 0.00155/S}{1.3856 + \frac{2.598}{S}10^{-5}} \times 1.491 S^{1/2}$$

or

$$\frac{1.351}{1.491}\left(1.3856 + \frac{2.598}{S} \times 10^{-5}\right) = \left[63 + \frac{1.55}{S} \times 10^{-3}\right] S^{1/2}$$

or

$$63\,S^{3/2} + 1.55 \times 10^{-3}\,S^{1/2} - 1.255\,S = 2.354 \times 10^{-5}$$

or

$$S^{3/2} + 2.46 \times 10^{-5}\,S^{1/2} - 1.99 \times 10^{-2}\,S = 3.737 \times 10^{-7}$$

Solving this by trial and error, we get $S \simeq \mathbf{\frac{1}{2600}}$

Example 14.28. *Design a regime channel for a discharge of 35 m^3/s with silt factor of 0.9 by Lacey's theory, taking side slopes as 1 H : 2V.*

(Engg. Services Exam. 2002)

Solution : *The above Example is practically the same as Example 14.6 except that Q has been increased from 30 to 35 cumecs.*

Given : $Q = 35$ cumecs ; $f = 0.9$

Side slopes $= 1\,(H) : 2\,(V) = \frac{1}{2}\,(H) : 1\,(V)$

1. $V = \left(\frac{Qf^2}{140}\right)^{1/6} = \left(\frac{35 \times 0.9^2}{140}\right)^{1/6} = 0.766$ m/sec.
2. $A = Q/V = 35/0.766 \simeq 45.67\ m^2$
3. $P = 4.75\sqrt{Q} = 4.75\sqrt{35} = 28.1$ m
4. $D = \frac{P - \sqrt{P^2 - 6.944\,A}}{3.472} = 1.833 \text{ m} \simeq \mathbf{1.84}$
5. $B = P - 2.236\,D = 28.1 - 2.236 \times 1.833 \simeq \mathbf{24\ m}$
6. $S = \frac{f^{5/3}}{3340\,Q^{1/6}} = \frac{0.9^{5/3}}{3340\,(35)^{1/6}} \simeq \mathbf{\frac{1}{7200}}$

Example 14.29. *Design an irrigation canal for a discharge of 15 cumecs, $N = 0.0225$, $m = 1$ and $B/D = 5.7$. Use Kennedy's theory.*

(Civil Services Exams 1987)

Solution : ***This question is exactly the same as example 14.2 except that discharge Q has been increased from 14 cumecs to 15 cumecs. For solution, refer Example 14.2.***

Example 14.30. *Design the section of an unlined canal in a loamy soil to carry a discharge of 50 cum/sec with permissible velocity of 1 m/sec. Assume side slopes of 2:1 and B/D ratio as 6.0. Using Manning's formula, calculate the bed slope of the canal, N = 0.0225.* ***(Civil Services Exam. 1988).***

Solution : Given : $Q = 50$ cumecs ; $V = 1$ m/sec; $N = 0.0225$; $B/D = 6.0$

Side slopes $= 2\,(H) : 1\,(V)$

$A = Q/V = 50/1 = 50\ m^2$

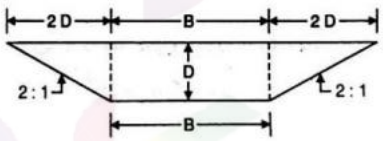

FIG. 14.12.

For a canal of side slopes 2:1

$$A = D\left[\frac{B + (B + 4\,D)}{2}\right] = D\,(B + 2\,D)$$

or $\quad A = D^2\left(\frac{B}{D} + 2\right)$ where $A = 50\ m^2$ and $B/D = 6$

Hence $\quad 50 = D^2\,(6 + 2)$ from which $D = 2.5$ m

$\therefore \quad B = 6\,D = 2.5 \times 6 = 15$ m

$$P = B + 2\sqrt{D^2 + 4D^2} = B + 4.472\,D = 15 + 4.472 \times 2.5 = 26.18 \text{ m}$$

$$R = A/P = 50/26.18 \simeq 1.91 \text{ m}$$

From Manning's formula, $S = \left[\frac{VN}{R^{2/3}}\right]^2 = \left[\frac{1 \times 0.0225}{(1.91)^{2/3}}\right]^2 \simeq \mathbf{\frac{1}{4680}}$

Example 14.31. *In a wide stream, a suspended load sample taken at a height of 0.3 m from the bed indicated a concentration of 1000 ppm of sediment by weight. The stream is 5.0 m deep and has a bed slope of 1/4000. The bed material can be assumed to be uniform size with a fall velocity of 2.0 cm/s. Estimate the concentration of the suspended load at mid-depth.* ***(Civil Services Exam. 1991)***

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Solution : Given $C_a = 1000$ ppm at 0.3 m above bed ; $V_s = 2.0$ cm/sec $= 0.02$ m/sec

$a = 0.3$ m; $D = 5.0$ m and $y = 2.5$ m

$\therefore$ $R = D = 5$ m for wide channel.

Shear friction velocity $V_* = \sqrt{gRS} = \sqrt{9.81 \times 5 \times \frac{1}{4000}} = 0.1108$

$$\therefore \quad \frac{V_s}{V_* k} = \frac{0.02}{0.1108 \times 0.4} = 0.452$$

From Eq. 14.35,

$$\frac{C}{C_a} = \left[\frac{a}{y} \times \frac{D-y}{D-a}\right]^{V_s/V_* k}$$

$$\therefore \quad C = 1000\left[\frac{0.3}{2.5} \times \frac{5-2.5}{5-0.3}\right]^{0.452} = \mathbf{288\ ppm}$$

Example 14.32. *A stable channel is to be designed for a discharge of 40 m^3/sec and silt factor of unity. Calculate the dimensions of the channel using Lacey's regime equation. Also calculate the dimensions of the channel if it were to be designed on the basis of Kennedy's method with critical velocity ratio equal to unity, and the ratio of bed width to depth of flow the same as obtained from Lacey's method.*

(Civil Services Exam. 1993)

Solution : Given : $Q = 40$ cumecs ; $f = 1$

(*a*) ***Canal design by Lacey's theory***

1. $V = \left(\frac{Qf^2}{140}\right)^{1/6} = \left(\frac{40 \times 1^2}{140}\right)^{1/6} = 0.812$ m/sec.

2. $A = Q/V = 40/0.812 = 49.29\ m^2$

3. $P = 4.75\sqrt{Q} = 4.75\sqrt{40} = 30.04$ m

4. For a channel with $\frac{1}{2}:1$ side slope,

$$D = \frac{P - \sqrt{P^2 - 6.944A}}{3.472} = \frac{30.04 - \sqrt{(30.04)^2 - 6.944 \times 49.29}}{3.472}$$

$= 1.833$ m $\simeq 1.84$ m

$B = P - 2.236D = 30.04 - 2.236 \times 1.833 = 25.94 \simeq 26$ m

5. $S = \frac{f^{5/3}}{3340\,Q^{1/6}} = \frac{1^{5/3}}{3340\,(40)^{1/6}} = \mathbf{\frac{1}{6177}}$

(*b*) ***Canal design by Kennedy's theory***

$\frac{B}{D} = \frac{25.94}{1.833} = 14.15$; $m = 1$

Let $N = 0.0225$

1. $A = BD + 0.5D^2 = D^2\left(\frac{B}{D} + 0.5\right) = D^2(14.15 + 0.5) = 14.65\,D^2$

2. $V_0 = 0.55\,m\,D^{0.64} = 0.55 \times 1 \times D^{0.64}$

$$Q = A\,V_0 = (14.65\,D^2)(0.55 \times D^{0.64}) = 8.058\,D^{2.64}$$

$$\therefore \quad D = \left(\frac{Q}{8.058}\right)^{1/2.64} = \left(\frac{40}{8.058}\right)^{1/2.64} = 1.835 \simeq \mathbf{1.84\ m}$$

$$\therefore \quad B = 14.15\,D = 14.15 \times 1.835 = 25.96 \simeq \mathbf{26\ m}$$

Thus, we get the same values of B and D, as obtained by Lacey's theory.

Example 14.33. *Design a Lacey's regime channel flowing through material of average diameter 0.328 mm, for the following data :*

Culturable commended area = 100000 hectares

Intensity of irrigation

Rabi = 40%

Kharif = 30%

Outlet discharge factor

Rabi = 1800 hec/cumec

Kharif = 800 hec/cumec

Assume conveyance losses to be 10% ***(Civil Services Exam. 1994)***

Solution

Area under Rabi $= 100000 \times 0.4 = 40000$ hec

Area under Kharif $= 100000 \times 0.3 = 30000$ hec.

$\therefore$ Outlet discharge for Rabi $= \frac{40000}{1800} = 22.22\ m^3/s$

and outlet discharge for Kharif $= \frac{30000}{800} = 37.5\ m^3$

$\therefore$ Design discharge for canal $= 1.1 \times 37.5 = 41.25$ cumecs

Silt factor $f = 1.76\sqrt{m_d} = 1.76\sqrt{0.328} \simeq 1$

$$V = \left(\frac{Qf^2}{144}\right)^{1/6} = \left(\frac{41.25 \times 1^2}{144}\right)^{1/6} = 0.812 \text{ m/sec}$$

$$A = \frac{Q}{A} = \frac{41.25}{0.812} = 50.81\ m^2$$

$$P = 4.75\sqrt{Q} = 4.75\sqrt{41.25} = 30.51$$

$$D = \frac{P - \sqrt{P^2 - 6.944A}}{3.472} = \frac{30.51 - \sqrt{(30.51)^2 - 6.944\,(50.81)}}{3.472}$$

$= 1.863 \simeq \mathbf{1.87\ m}$

$B = P - 2.236D = 30.51 - 2.236 \times 1.863 = 26.34$ m $\simeq$ **26.4 m**

$$S = \frac{f^{5/3}}{3340\,(Q)^{1/6}} = \frac{1^{5/3}}{3340\,(41.25)^{1/6}} \simeq \mathbf{\frac{1}{6210}}$$

Example 14.34. *Write the Rouse's equation for distribution of suspended sediment in the vertical of a channel and lebel all the terms.*

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In a wide alluvial channel having suspended sediment load, the depth of flow is 2.8 m. If the suspended load sample indicated a concentration of 700 ppm at a point 30 cm below water surface, estimate the concentration at a point 10 cm above the bed in the same vertical. The exponent in Rouse's equation can be assumed as 0.4. ***(Civil Services Exam. 2002)***

Solution

Rouse equation for distribution of suspended load is given by

$$\frac{C}{C_a} = \left[\frac{a\,(D-y)}{y\,(D-a)}\right]^{V_s/k\,V_*} \qquad ...(14.35)$$

Here, the exponent $V_s/k\,V_*$ has been given as equal to 0.4.

Also, $D = 2.8$ m; $a = 2.8 - 0.3 = 2.5$ m; $C_a = 700$ ppm; $y = 0.1$ m

$$\therefore \quad C = 700\left[\frac{2.5\,(2.8-0.1)}{0.1\,(2.8-2.5)}\right]^{0.4} = \mathbf{6109\ ppm}$$

Example 14.35. *A distributory is to be designed to irrigate 3600 ha in Rabi and 1400 ha in Kharif. Kor depth (critical stage watering depth) and Kor period (critical stage water period) for Rabi and Kharif are 13.5 cm and 4 week, 19.00 cm and $2\frac{1}{2}$ weeks respectely. Design the distributory by Lacey's theory and silt factor 0.85.* ***(Civil Services Exam. 2005)***

Solution :

Area for Rabi = 3600 ha

Kor depth for Rabi = 13.5 cm

Kor period for Rabi = 4 weeks

$$\therefore \quad \text{Duty for Rabi crop} = \frac{8.64\,b}{\delta} = \frac{8.64 \times 4 \times 7}{0.135} = 1792 \text{ ha/cumes}$$

$\therefore$ Discharge for Rabi = 3600/1792 ≏ 2.01 cumecs

Area for Kharif = 1400 ha

Kor depth for Kharif = 19 cm

Kor period for Kharif = $2\frac{1}{2}$ weeks

$$\therefore \quad \text{Duty for Kharif} = \frac{8.64 \times 2.5 \times 7}{0.19} = 795.8 \text{ ha/cumes.}$$

$\therefore$ Discharge for Kharif = 1400/795.8 = 1.76 cumecs

Hence design discharge for distributory = 2.01 cumes.

Channel design by Lacey's theory

1. $$V = \left(\frac{Qf^2}{140}\right)^{1/6} = \left[\frac{2.01\,(0.85)^2}{140}\right]^{1/6} = 0.467 \text{ m/sec.}$$

2. $A = 2.01/0.467 = 4.304 \text{ m}^2$

3. $P = 4.75\sqrt{Q} = 4.75\sqrt{2.01} = \mathbf{6.73\ m}$

4. $$D = \frac{P - \sqrt{P^2 - 6.944\,A}}{3.472} = \frac{6.73 - \sqrt{(6.73)^2 - 6.944 \times 4.304}}{3.472}$$

$= 0.806$ m ≏ **0.81 m**

5. $$S = \frac{f^{5/3}}{3340\,(Q)^{1/6}} = \frac{(0.85)^{5/3}}{3340\,(2.01)^{1/6}} = \frac{1}{4919}$$

PROBLEMS

1. Describe Kennedy's silt theory. A channel is silting badly in the head reach. How would you proceed to determine its cause and what remedies would you suggest.

2. Explain the procedure of designing a channel with Kennedy's theory.

3. For a channel, the discharge (Q), rugosity (N), critical velocity ratio (m) and the bed width-depth ratio (B/D) are given. Explain how would you design the channel using Kennedy's theory ?

4. Derive an expression for the silt supporting capacity of a channel according to Kennedy's theory.

5. Explain Lacey's silt theory.

6. Describe the method of designing a canal based on Lacey's theory.

7. Compare Kennedy's and Lacey's silt theories. Why is Lacey's conception superior to that of Kennedy's ?

8. What do you understand by (*a*) regime channel, (*b*) initial and permanent regime of channels ?

9. Using Lacey's theory, design an irrigation channel for the following data :
   Discharge $Q = 50$ cumecs
   Silt factor $f = 1$
   Side slopes $= \frac{1}{2} : 1$ [**Ans** $B = 29.6$ m ; $D = 1.98$, $S = 1/6400$]

10 Using Lacey's basic regime equations, derive an expression for Lacey's scour depth.

11. With the help of basic regime equations given by Lacey, derive the perimeter-discharge relationship.

12. The slope of a channel in alluvial soil is 1/5900. Find the channel section and the maximum discharge which can be allowed to flow in it. Take Lacey's silt factor $f = 1$. The channel is of trapezoidal section, having side slopes $\frac{1}{2} : 1$.
[**Ans.** $b = 22.3$ m, $d = 1.97$ m, $Q = 30$ cumecs]

13. Using Kennedy's theory, desing a channel section for the following data :
   Discharge $Q = 14$ cumecs
   Kutter's $N = 0.0225$
   Critical velocity ratio $m = 1$
   Side slopes $\frac{1}{2} : 1$
   Bed slope $= \frac{1}{5000}$ [**Ans.** $b = 9.7$ m ; $d = 1.71$ m]

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15

Irrigation Channels - II : Design Procedure

15.1. LONGITUDINAL SECTION OF A CANAL

After having fixed the alignment of canal on shajra sheet as indicated in Chapter 13, the longitudinal section of the alignment is taken on the field. The longitudinal section should thereafter be plotted to a horizontal scale of 1 cm = 160 m and to vertical scale of 2 cm = 1 m. Vertical scale should be changed if desired according to the magnitude of fall available.

For drawing longitudinal section, following steps are generally followed :

(1) Plot the ground level along the alignment of the channel with the reference to convenient datum.

(2) Mark the full supply level (F.S.L.) and the bed level of the parent channel just on the upstream of head regulator of the offtaking channel for reference purposes.

(3) Draw the F.S.L. of the off-taking channel keeping following points in view.

(*a*) Keep the F.S.L. of the off-taking channel 15 cm below the parent channel. This is generally done (*i*) to provide for the loss of head at the head regulator, (*ii*) to meet the demand for extra supplies in the canal at a future date, (*iii*) to maintain the flow even if the channels get silted upto some extent in its head reaches. For main canals taking off from river, F.S.L is kept about 1 m lower than the pond level of the reservoir.

(*b*) F.S.L. should be above the ground level for most of its length ; F.S.L. may not be above isolated high patches of ground. Such areas can be irrigated by lift irrigation. The F.S.L. should, however, be only some centimetres above the ground level . Since the canal is running on watershed, the ground level on both sides of the canal will be lower and cross slopes available will be ample to provide irrigation to the entire command.

4. Choose bed slope close to Lacey's slope or that given by Wood's Table. If the slope available on the ground is lesser than that suggested by Lacey or Wood's normal Table then maximum slope as available on the ground may be provided. Efforts should, however, be made to reduce silt factor as far as possible by preventing silt entry at head in such a case.

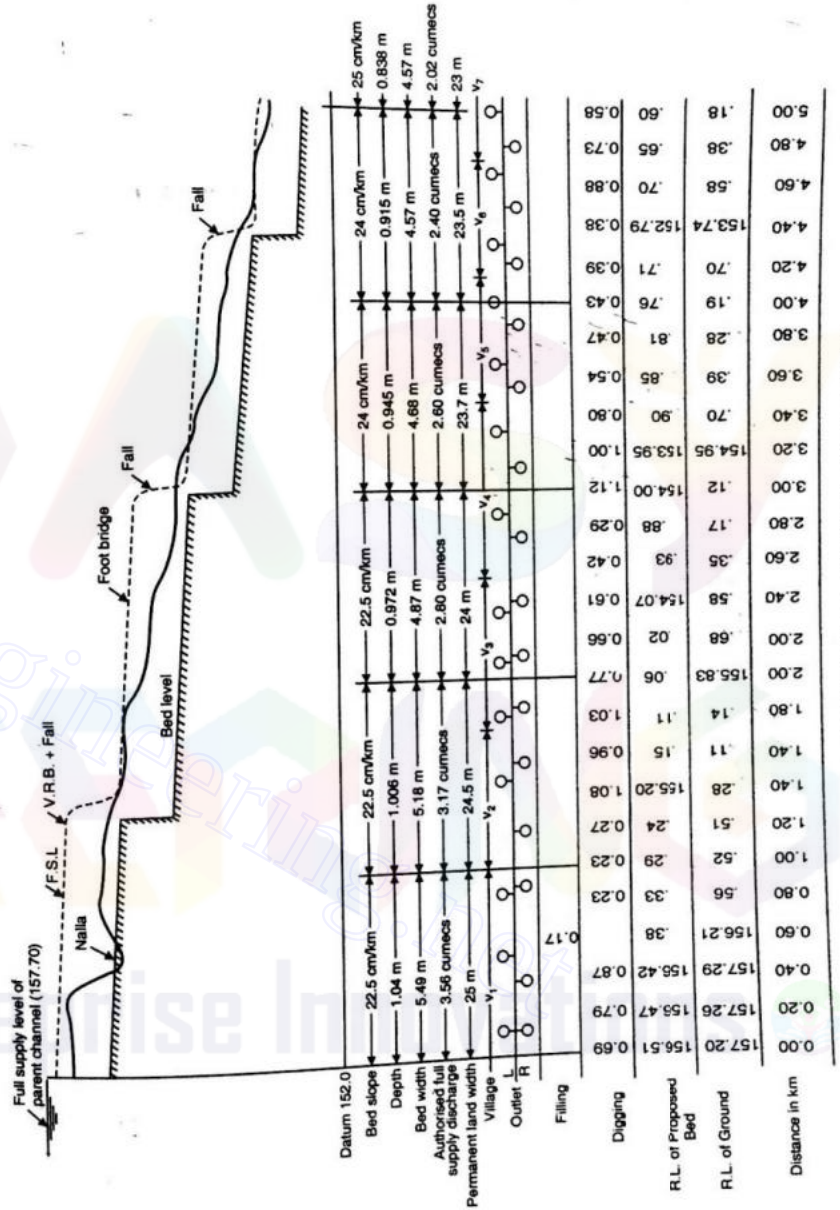

FIG. 15.1. LONGITUDINAL SECTION OF A CANAL.

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5. Provide falls if the slope available in the ground is steeper than the one to be given to the canal bed. The magnitude of the drop should be such that F.S.L. of the canal downstream of fall should remain below the ground level for about half a km before emerging out of the ground level. Unbalanced earth work on both sides of falls should be kept minimum. The location of falls has been discussed in detail in Chapter 18.

(6) Keep the channel in balanced depth of cutting or filling as far as possible.

Fig 15.1 shows the logitudinal section of a canal.

15.2. BALANCING DEPTH

A canal section will be economical when the earth work involved at a particular section has an equal amount of cut and fill. Usually a canal section has a part in cutting and part in filling as shown in Fig. 15.2. If the amount of cut is equal to the amount of fill, it has to be paid for once only. More so, the necessity of a borrow pit or soil bank is entirely avoided. For a given cross-section there is always only one depth of cutting for which the cutting and filling will be equal. The depth is known as *balancing depth*. The balancing depth is worked out as under :

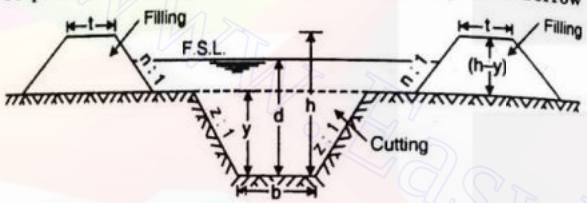

FIG. 15.2. BALANCING DEPTH.

If h = vertical height of top of bank from bed of canal.

b = bed width of the channel.

t = top width of the canal bank.

$n:1$ = side slope of bank in filling.

$z:1$ = side slope of canal in cutting.

d = full supply depth of the channel.

y = depth of cutting

$$\text{Area of the cut} = by + zy^2 = y(b + zy)$$

$$\text{Area of fill} = 2[(h-y)t + n(h-y)^2].$$

Equating the area of cut and fill

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

or

$$y^2(2n - z) - (b + 4nh + 2t)y + h(2t + 2nh) = 0 \quad ...(15.1)$$

A canal is usually constructed with a side slope of 1 : 1 in cutting and a slope 1.5 : 1 in filling . Therefore, putting $n = 1.5$ and $z = 1$ in Eq. 15.1 we get.

$$y^2 - (b/2 + 3h + t)y + h\left(t + \frac{3}{2}h\right) = 0 \quad ...(15.2)$$

From Eq. 15.1 the depth y from the ground line and the bed level is calculated. A line roughly parallel to the hydraulic gradinet line is drawn on the L-section which envelops maximum number of ground points. Bed line is then drawn parallel to it and y metre below this line.

15.3. LOSSES IN CANALS

When water continuously flows through a canal, losses take place due to seepage, deep percolation and evaporation. These losses are some times known as *transmission losses*. These losses should be properly accounted for, otherwise lesser quantity of water will be available for cultivation at the tail end. Water losses in canals can be broadly classified under three heads :

(*i*) Evaporation losses, (*ii*) Transpiration losses., (*iii*) Seepage losses.

(i) Evaporation losses. The loss due to evaporation is generally a small percentage of the total loss in unlined canal. It hardly exceeds 1 to 2 percent of the total water entering into the canal. The evaporation losses depend upon *(i) Climatic factors* such as temperature, humidity and wind velocity, and *(ii) Canal factors* such as water surface area, water depth and velocity of flow. Maximum loss is there in summer months when temperatures are high and wind velocities are also high. Similarly, losses are maximum in unlined canals due to wider water surface area, shallower water depth and low velocity. The average evaporation loss per day may vary between 4 mm to 10 mm.

(ii) Transpiration losses. The transpiration loss takes place through lot of vegetation and weeds growth along the bank of canal. However, this forms a extremely small part of total loss.

(iii) Seepage losses. Seepage losses constitute major portion of loss in an unlined canal. The seepage losses are due to (*i*) absorption of water in the upper layers of soil below the canal bed, and due to (*ii*) *percolation* of water into the water table, thus raising the water table. If, however water table is much lower, seepage losses are only due to absorption. Percolation losses are always much more than the absorption losses.

Rate of water loss

Canal losses may be expressed in any one of the following methods :

(*i*) as cumecs per million square metre of the wetted perimeter.

(*ii*) as depth of water lost per day over the wetted perimeter.

(*iii*) as percentage of the canal discharge.

(*iv*) as percentage per kilometer length of the canal.

Out of these four methods, the first method is the simplest, and is quite popular. In absence of any other data, the transmission losses may be taken as 2.5 cumecs per million square meter of wetted perimeter for unlined channels and as 0.60 cumecs per million sq. metre of wetted perimeter for lined canals.

In U.P., the loss Q_L in cumeces per km length of unlined canal is given by

$$Q_L = \frac{1}{200}(B + D)^{2/3} \quad ...(15.3)$$

where B and D are in metres.

In Punjab and Haryana, the losses in unlined canals in cumecs per million square metre of wetted perimeter is given by the expression

$$Q_L = 1.9\, Q^{0.0625} \quad ...(15.4)$$

The Central Water Power Commission (CWPC) recommends the following values of losses in unlined canals (Table 15.1).

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TABLE 15.1. CWPC RECOMMENDATION FOR LOSSES IN UNLINED CANALS

Soil Type	Transmission loss (cumec/million sq meter of wetted perimeter)
1. Rock	0.91
2. Black cotton soil	1.83
3. Alluvial soil	2.74
4. Decayed rock or gravel	3.0

Losses are sometimes expressed as percentage of total discharge, as under.

Main canal and branches : 15% to 20 %

Distributaries and minors : 6% to 7%

Water courses : 17% to 22%

Losses in lined canals : In Punjab, the following formula is used for the determination of total losses in cumces per Mm^2 (million square meters) of wetted perimeter

$$K = 0.349\, Q^{0.056} \qquad ...(15.5)$$

where Q is the discharge in cumecs.

However as per recommendations of C.W.P.C. and IS: 1745-1968, the transmission losses in lined canals can be taken as 0.6 cumecs/Mm^2 of the wetted perimeter.

15.4. SCHEDULE OF AREA STATISTICS AND CHANNEL DIMENSIONS

The design of channel cross-section from km to km is carried out in a Tabular form called the *Schedule of Area Statistics and Channel Dimensions.*

The schedule of area statistics and channel dimensions is shown in Tables 15.2 and 15.7. Working of the Table is explained below and has later been illustrated by Example 15.3.

TABLE 15.2. SCHEDULE OF AREA STATISTICS AND CHANNEL DIMENSIONS

Below km	Gross commended area (G.C.A.)	Culturable commended area (C.C.A.)	Area to be irrigated			Outlet discharge factor	Outlet discharge	Losses in reach	Total loss	Total discharge	Channel Dimensions						Velocity (V) of flow	V/V_0 Ratio or silt factor f
			Rabi (%)	Perennial (%)	Kharif (%)						Bed slope	Bed width	Water depth	Design discharge	Height of Bank	Width of Bank		
	ha	ha	ha	ha	ha	ha/cumec	cumec	cumec	cumec	cumec	cu/km	m	m	cumec	m	m	m/s	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
0																		
1																		
2																		
⋮																		
n																		

Col. 1. The actual design of channel is carried from km to km. Sometimes an off-taking channel may take off in between the kilometre interval. The channel dimensions are then also found out at the downstream of the cross regulator.

Col. 2. Column 2 indicates the gross commanded area, i.e. the entire area under the command of the channel below the particular kilometre at which it is being designed.

Col. 3. In this column, the area actually under cultivation below the particular km under the command of the channel is indicated.

Col. 4 to 6 indicate the percentage of area under Rabi, perennial and Kharif crops. Mostly Rabi crops are controlling crops for finding the discharge of the channel. Channels either designed for Rabi or Kharif crops suffice the purpose for sugar cane irrigation. However, under some local conditions the discharge needed for sugar-cane irrigation is found out.

Col. 7. Indicates the outlet discharge factor for the controlling crop, calculated by the methods indicated in Chapter 3.

Col. 8. Indicates the outlet discharge required and can be found out by multiplying area to be irrigated to the outlet discharge factor.

Col. 9. Indicates the losses in the reach from kilometre to kilometre. Generally losses per million square metre of the wetted surface area is known and so the channel losses in different reaches have to be calculated on the basis of tentative design for channels.

Col. 10. Indicates the total losses in the channel below any particular kilometre of the channel.

Col. 11. Indicates the total discharge for which the channel has to be designed. This discharge includes outlet discharge and losses.

Col. 12 to 18. These columns pertain to channel dimensions and are filled up after designing the channel in each reach based on Kennedy theory using Garret's diagrams or on Lacey's theory using Lacey's diagrams.

15.5. USE OF GARRET'S DIAGRAMS FOR CHANNEL DESIGN ON KENNEDY'S THEORY

Garret's diagrams give the graphical method of designing the channel dimensions based on Kennedy's theory. The original diagrams were prepared in F.P.S.units. Fig. 15.3 (*a*) and (*b*) show two such diagrams converted into metric units. The diagram has the discharge plotted on the abscissa. The ordinates on left indicate the slope and that on right the water depth in the channel and critical velocity V_0. The discharge lines are curved and bed width lines are shown dotted.

The proccdure in designing the channel consists of the following steps.

1. Find out the discharge for which the channel is to be designed. This can be found from Schedule of Area Statistics.

2. Find out the slope of the channel from its longitudinal section.

3. In Fig. 15.3, follow the discharge line and find out its intersection with the horizontal lines from slope. Interpolation may be done if needed to locate the discharge line. Mark the intersection point P.

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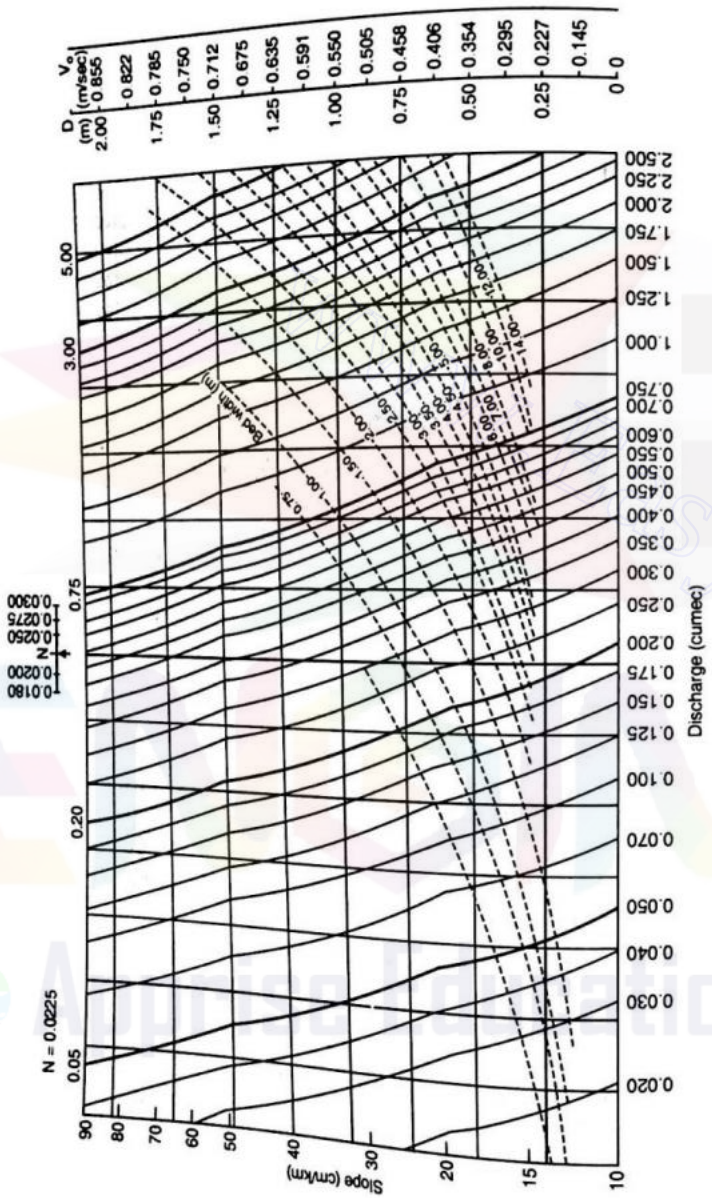

FIG. 15.3 (a) GARRETS DIAGRAMS FOR Q BETWEEN 0.02 CUMEC TO 5.0 CUMECS

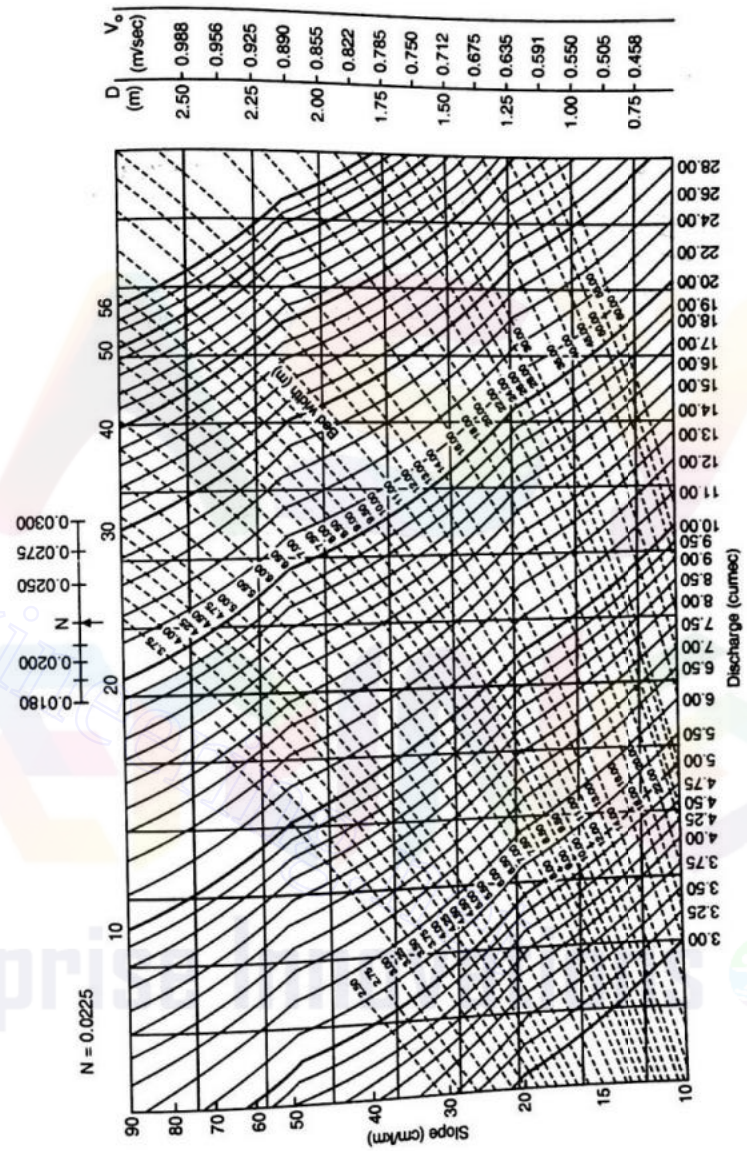

FIG. 15.3. (b) GARRETS DIAGRAMS FOR Q BETWEEN 3 CUMECS TO 56 CUMECS

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4. Draw a vertical line through the point of intersection. This will intersect several bed width curves. Each point of intersection of vertical line and bed width curve gives a depth and critical velocity V_0 corresponding to this depth on right hand side ordinate.

5. Choose a pair of bed width, depth and V_0 corresponding to point of intersection obtained in step 4.

6. Calculate the area (A) of channel section corresponding to bed width-depth obtained in step 5.

7. Calculate the velocity in the channel corresponding to this area A.

8. Calculate the ratio V/V_0. This should be equal to unity or equal to the value of m given for channel design.

9. Repeat the procedure with other values of bed width and depth till the value of V/V_0 obtained is the same as the value of m given for channel design.

To simplify the procedure, the result should be arranged in a tabular form as illustrated in Example 15.1. In practice, the design of channel is carried out by Garret's diagram. The diagram is applicable only for channel sections having a side slope of $\frac{1}{2}:1$.

The diagrams have been drawn for $N = 0.0225$. However, a nomogram at the top of each diagram helps in making the use of these diagrams for any other value of N. For any other value of N, the vertical line drawn at the intersection of the discharge and slope lines has to be shifted either to the left or the right side to the extent given on the nomogram.

15.6. USE OF LACEY'S REGIME DIAGRAMS

These diagrams have been prepared by Lacey to provide a graphical solution for channel design.There are two separate diagrams — one is used for determining bed width (Fig. 15.4 *a* and *b*) and depth and the other to determine the regime slope (Fig. 15.5). The diagrams have been plotted for channel section having a side slopes of $\frac{1}{2}:1$.

Procedure of channel design with the help of these diagrams is as under :

1. To find the *bed width and depth,* use Fig. 15.4 (*a*) and (*b*). Find the intersection of discharge Q with silt factor f.

2. Read the value of bed width from abscissa and the value of depth from the ordinate.

3. To determine *regime slope* S, use Fig. 15.5. Find again the intersection and discharge Q with silt factor f.

4. Read the value of ordinate corresponding to this point of intersection and determine slope S.

15.7. CROSS-SECTION OF AN IRRIGATION CHANNEL

A canal is generally taken in such a way that its section is partly in cutting and partly in filling in order to approach close to the balancing depth. Many times,

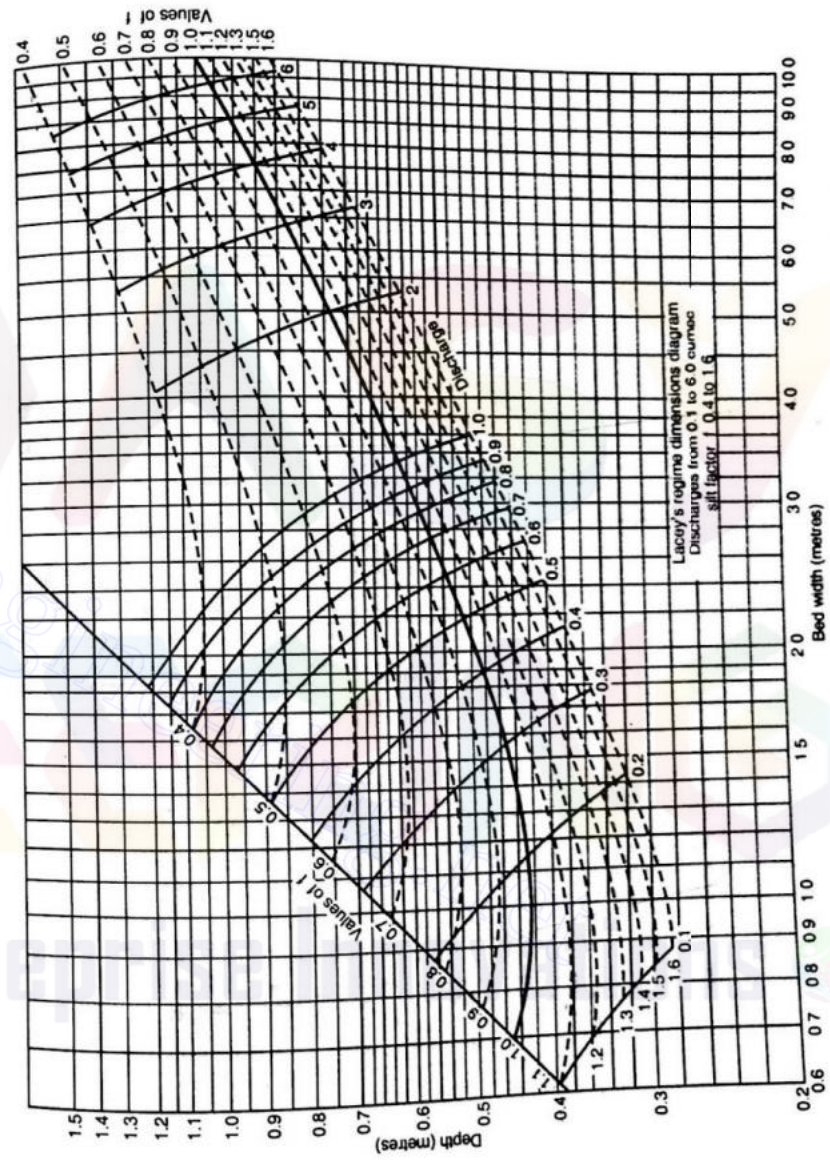

FIG. 15.4. (*a*) LACEY'S REGIME DIAGRAMS FOR B AND D (Q BETWEEN 0.1 TO 6 CUMECS)

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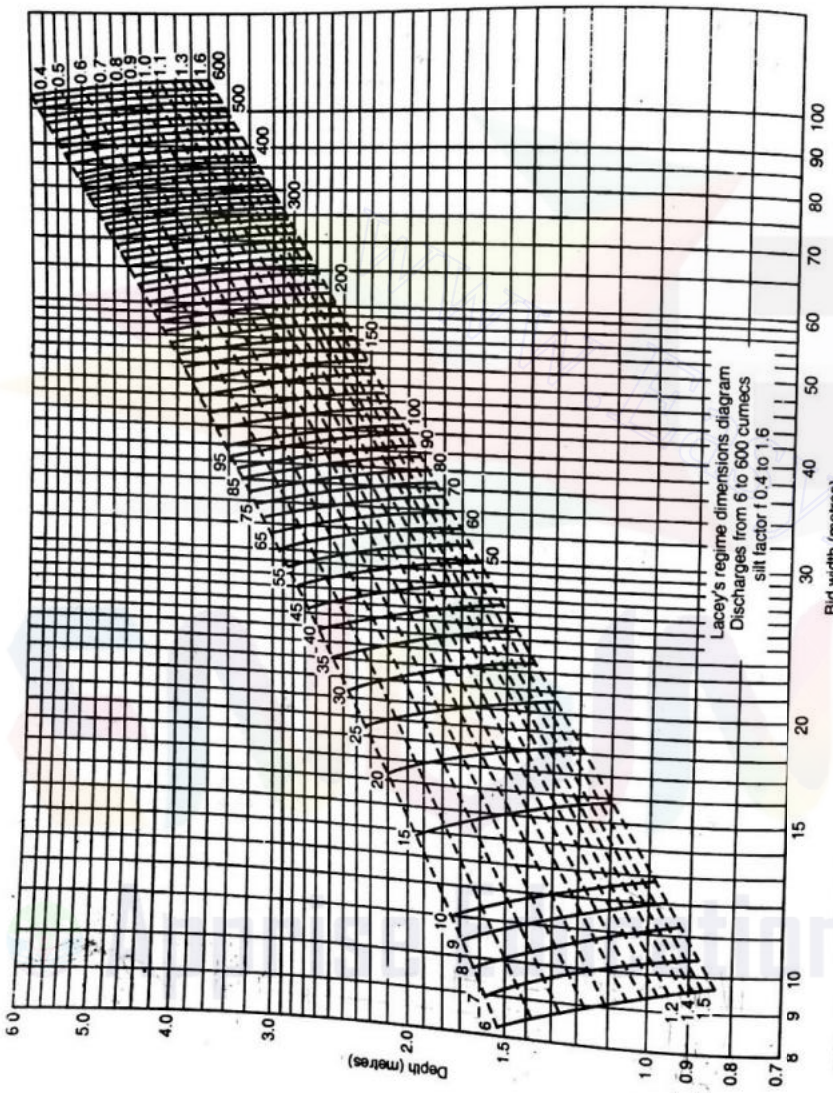

FIG. 15.4. (b) LACEY'S REGIME DIAGRAMS FOR *B* AND *D* (*Q* BETWEEN 6 TO 600 CUMECS)

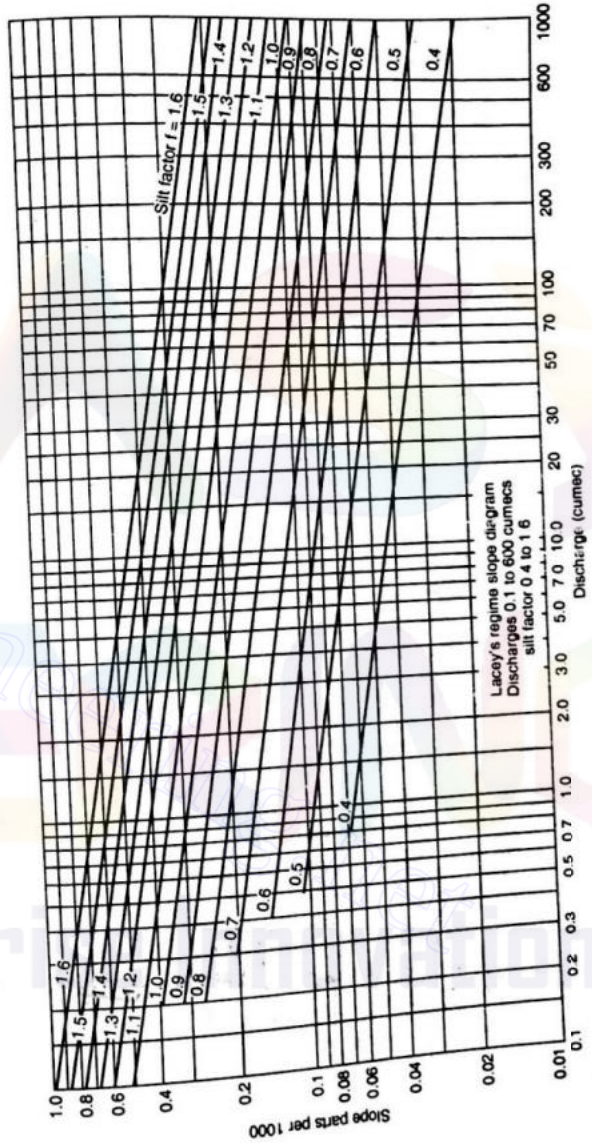

FIG. 15.5. LACEY'S REGIME DIAGRAMS FOR SLOPE

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however, the canal has to be carried through deep cutting or filling. A channel section may, therefor, be either :

1. In cutting 2. In filling, or 3. In partial cutting and filling.

The channel section in these three conditions are shown in Fig. 15.6. When the ground level is above the top of the bank, the canal is said to be in cutting. Similarly, when the ground level is below the bed level of canal, it is said to be in filling. A canal is in partial cutting or filling when the ground level is in between bed level and top of bank.

A canal can have a dowla section on one side or a bank section on both sides. The section may also be designed to have a bank section on one side and dowla section on the other side. Usually the left bank of canal has a dowla section and service road and right bank of the canal has a bank section.

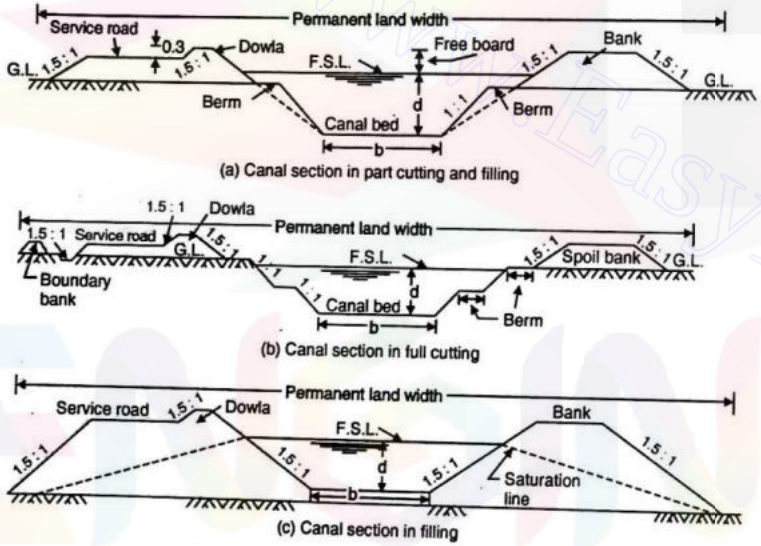

FIG. 15.6. TYPICAL CANAL SECTIONS.

Side Slopes

For the computation of the values of hydraulic mean depth R, the area and wetted perimeter are worked out with $\frac{1}{2}:1$ side slopes for design purpose even though in execution actually flatter slopes depending upon stability and type of soils are adopted. $\frac{1}{2}:1$ side slope is assumed with the presumption that the sides of the channel get silted up to $\frac{1}{2}:1$ slope and channel capacities would be reduced accordingly in due course. However, in case a channel takes off from such storage reservoirs where silt

free water is available, the actual slope to be provided during construction is assumed for design purposes. The side slopes depend upon the nature of the soil and the depth of cutting or filling. The minimum slope usually allowed is given in Table 15.3.

TABLE 15.3. SIDE SLOPES OF CHANNEL

S.No.	Type of soil	Cuttings	Fillings
1	Hard Rock	$\frac{1}{8}:1$ to $\frac{1}{4}:1$	
2	Soft Rock	$\frac{1}{4}:1$ to $\frac{1}{2}:1$	
3	Hard clay or gravel	$\frac{3}{4}:1$ to $1:1$	$1\frac{1}{2}:1$
4	Soft clay & alluvial soils	$1:1$	$1\frac{1}{2}:1$ to $2:1$
5	Sandy loam	$1\frac{1}{2}:1$	$2:1$
6	Light sand	$2:1$	$2:1$ to $3:1$

Berm : Berms are narrow strip of land left at the ground level between the inner toe of the bank and top edge of cutting. The width of the berm is variable but it is kept such that the bed line and bank line remain parallel.

Let $r_1:1$ be the slope in cutting and $r_2:1$ be the slope of the bank in filling. Then from Fig. 15.7, we have

$$AB = r_1 y + \text{initial beam width} + r_2(d-y)$$

But $AB = r_2 d$, for bed line and bank line to be parallel.

Hence from the above two relations, we get

$$\text{Initial berm width} = (r_2 - r_1)y$$

This shows that the initial berm width depends only on the depth of cutting. Since the depth cutting varies with the variation of ground level, the initial berm width also varies.

After the channel runs for some time, the silt is deposited on the sides of the section and gradually the sides get silted to a slope of $\frac{1}{2}:1$. The position of the berm therefore, shifts from ground level to the F.S.L. and its width becomes equal to $(r_2 - \frac{1}{2})D$ where D is the full supply depth. Taking $r_2 = 1\frac{1}{2}$, the final berm width becomes equal to the depth D of the canal when the filling of bank has been done in $1\frac{1}{2}:1$ slope. The berms when fully formed serve the following purposes :

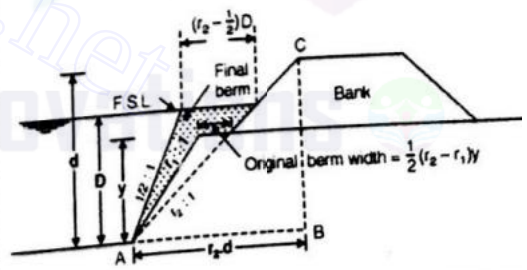

FIG. 15.7. INITIAL AND FINAL POSITION OF BERM

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1. The silt deposited on the sides is very compact and impervious. It serves therefore, as fairly impervious lining which reduce the absorption losses and prevents leaks and consequent breaches.
2. The possibility of leaks and breaches are reduced as they bring the saturation line more inside the body of the banks.
3. The berms provide a bigger water way ; its capacity, therefore, increases and it can thereby absorb positive fluctuations.
4. They protect the bank from erosion because of wave action.
5. Open borrow pits can be taken on the berms.
6. They sometimes provide an additional inspection path.
7. They porvide additional strength to banks and thus make it safe against branches.

Minimum berm width may be adopted as under (Table 15.4) :

TABLE 15.4

Discharge	*Berm width (metres)*
Upto 4.25 cumecs	$0.6 + \frac{1}{2}$. F.S. depth
4.25 cumecs to 28 cumecs	$1.25 + \frac{1}{2}$ F.S. depth
28 cumecs and above	$1.25 + \frac{\text{F.S. discharge}}{28} + \frac{1}{2}$ F.S. depth

Minimum berm width $= 0.5 + \frac{1}{4}$ (width of combined side slopes of cutting and filling)

Maximum berm width $= 0.5 + \frac{1}{2}$ (width of combined side slopes of cutting and filling)

Recommendations of IS : 7112-1973 for berm width

Fig. 15.8 shows three typical sections of canals : (*a*) partly in cutting and partly in filling, (*b*) wholly in filling and (*c*) wholly in cutting as per IS : 7112-1973. Accordingly, the recommendations for berm widths are as under :

(*a*) *When the canal is partly in cutting and partly in filling* (Fig. 15.8 *a*), the berm may be kept at the ground level, with its width $= 2D$ (i.e. twice the fully supply depth).

(*b*) *When the canal is wholly in filling* (Fig. 15.8 *a*) the berm may be kept at the full supply level with its width equal to $3D$.

(*c*) *When the canal is wholly in cutting* (Fig. 15.8 *c*), the berm may be kept at the full supply level with its width $= 2D$.

Additional berm : Additional berm is provided when the channel is in deep cutting. Such a berm may be provided at the ground level, as shown in Fig. 15.8 (*c*). In such a circumstance, the width of the original berm, provided at full supply level may be reduced.

Free Board :

Free board is the gap or the margin of height between F.S.L. and top of the bank. It is possible to ensure that water does not overtop the bank. Free board in

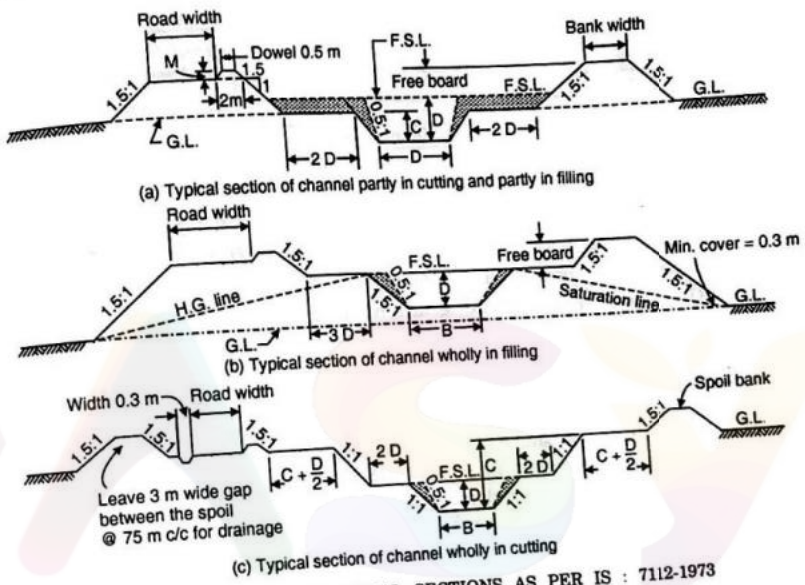

FIG. 15.8. TYPICAL CANAL SECTIONS AS PER IS : 7112-1973

a channel is governed by the consideration of size of canal, its location, water surface fluctuations, rain water inflow etc. The usual value of free board provided is given in Table 15.6.

U.S.B.R. recommendations for free board

(*a*) For $Q \le 0.6$ cumecs, F.B. $= \sqrt{0.46\,D} \triangleq 0.68\sqrt{D}$ ...(15.6 *a*)

(*b*) For $Q \ge 85$ cumecs, F.B. $= \sqrt{0.76\,D} \triangleq 0.87\sqrt{D}$ ...(15.6 *b*)

(*c*) For $85 > Q > 0.6$ cumecs, the free board can be interpolated

Lacey's recommendations for free board

$$\text{F.B.} = 0.20 + 0.15\,Q^{1/3} \quad \text{...(15.7)}$$

where Q is the discharge in cumecs.

Top width of Bank :

The purpose of the bank is to retain water so long as the berm is not formed. They thus have is to withstand full pressure of water above the ground level . They should, therefore, be sufficiently strong to withstand this initial strain. No special design is, however, done and it is customary to provide width of bank as per Table 15.5. If the canal is completely in filling the adequacy of the bank section and stability of its slope should be tested as per the methods suggested in Chapter 10.

Recommendations of IS : 7112-1973 for minimum top width of banks

Table 15.5 gives the Indian Standard recommendations for minimum top width of banks of canals.

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TABLE 15.5. MINIMUM TOP WIDTH OF BANKS (IS : 7112 - 1973)

S.No.	Discharge (cumec)	Minimum top width (m)	
		Inspection Bank	Non-inspection Bank
1	0.15 to 7.5	5.0	1.5
2	7.5 to 10.0	5.0	2.5
3	10.0 to 15.0	6.0	2.5
4	15.0 to 30.0	7.0	3.5
5	30.0 and above	8.0	5.0

Sometimes, the following empirical formula is used for top width :

$$\text{Top width} = D + 0.3\, r^{1/2} \quad \text{where } r = B/D \text{ ratio.} \qquad ...(15.8)$$

It is customary to provide a minimum top width of 1.5 m.

Saturation line or hydraulic gradient line

When water flows through an unlined earth channel, seepage takes place through the banks. Due to this seepage, the banks get saturated in the lower portion while the upper portion remains either dry or moist. The line demarcating these two portions is called the *saturation line* or the *hydraulic gradient line* or *phreatic line*. The saturation line is actually parabolic in shape, but for banks of lesser height (less than 5 m), it is usually taken as a *straight line* (Fig. 15.8 *b*) starting from the point where the full supply line meets the inner sloping face of the bank. The slope of the saturation line is known as *saturation gradient* or *hydraulic gradient*, and its value depends upon the type of the soil of the banks. Indian Standard (IS : 7112-1973) recommends the following empirical values of the hydraulic gradient for banks less than 5 m in height.

Type of soil	Hydraulic gradient (H:V)
1. Silty soils	4:1
2. Silty sand	5:1
3. Sandy soils	6:1

However, for banks more than 5 m in height, the true position of the saturation line (or phreatic line) shall be determined and the stability of its slope should be checked. In all the cases, however, the saturation line shall have a minimum cover of 0.3 m.

If, however, the saturation line cuts the outer face of the bank, or if sufficient cover is not available, back berm or counter berm should be provided (as indicated in § 15.11) or the top width of the banks should be increased.

Dowel or Dowla :

A dowla is provided by the side of inspection road as shown in Fig. 15.9. Top of Dowla is kept above the F.S.L. by a margin of free board. They are provided as

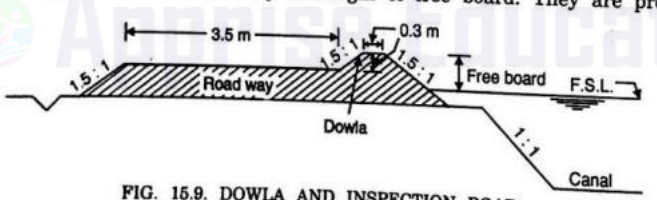

FIG. 15.9. DOWLA AND INSPECTION ROAD.

a measure of safety for automobiles driven on the service road. They act as kerbs on the side of roadway towards the canal. Top width of dowla is kept from 0.3 m to 0.6 m and is about 0.3 m above the service road.

Inspection Roadway :

It is very necessary to have access to all parts of canal system so that proper inspection may be done. It will also increase the efficiency of the maintenance. Main canals and branch canals have roadway on both sides. In case the road can be provided only on one side, it is customary to provide it on the left bank of canal. The banks of the canal are classified as left and right by facing the canal along the direction of flow. Usually the road on right serves the purpose of transport of the material. On smaller branches and distributaries the roadway is provided on left side only and it is meant for inspection. Road is not provided along minors. The canal road is usually unmetalled since they are used for limited purpose only. Since the canal divides the area in two parts, sometimes a cart road is provided on both sides. The cart road may be kept 3.0 to 5.0 m wide. Many a times the canal road provides the only access to the interiors. It is also sometimes required to let the inspection road be a thorough fare. The state P.W.D.(B. and R.) may also use it. In such a case the road surface must be metalled before leaving it open to public.

In case of dowla section, the service road is adjoining to the dowla. The road way is provided 0.3 to 0.8 m above F.S.L. depending upon the size of the channel. In case of bank section, the road way is provided just a few cm above the ground level. It becomes very difficult to inspect a canal when the roadways is in this position because the bank section is on the right and one has to get down at many a places in order to inspect the canal.

Standards of canal cross-section

Table 15.6 gives the commonly followed standards of canal cross-section.

TABLE 15.6. STANDARDS OF CANAL CROSS-SECTION

S.No.	Item	Main and Branch canal		Major distributaries		Minor distributaries	
		150 to 30 cumecs	30 to 10 cumecs	10 to 5 cumecs	5 to 1 cumecs	1 to 0.3 cumecs	Below 0.3 cumecs
1.	Maximum crest width of bank	3 m	2.5 m	2.25 m	2 m	1.5 m	1.0 m
2.	Free board	0.9 m	0.75 m	0.6 m	0.5 m	0.4 m	0.3 m
3.	Width of berms	Minimum berm width = 0.6m + 1/4th of the width of combined side slopes of cutting and embankment Maximum berm width = 0.6m + 1/2th width of combined slopes					
4.	Width of road way	6 m	5 m	5 m	3.5 m	3.5 m	Nil
5.	Depth of earth cover over saturation gradient	1 m	0.8 m	0.5 m	0.5 m	0.5 m	0.5 m
6.	Width of land to be acquired : (Clear of banks) when canal cutting is deeper than balancing depth	5 m	Half the height of the bank above ground subject to a minimum of 1.5 m				
7.	Width of land to be acquired : Clear of banks when canal cutting depth is less than balancing depth	Full height of bank plus 5 m	Full height of bank plus 5 m	Full height of bank above ground level plus 1.5 m			

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15.8. BORROW PIT

When the earth work in filling exceeds excavated quantity at a particular section, borrow pits are required to make good the requirements of filling. *Outside borrow pits* are generally not preferred due to mosquito nuisance. *Inside borrow pits* are usually advocated as they get silted up during the course of time. The outside borrow pit if used should not be dug deeper than 0.3 m, so that the land may be reclaimed after the completion of the job. The outside borrow pit should start from a distance not less than 5 m from the toe of embankment in case of small channels and 10 m from the toe in case of large channels. Inside borrow pit should have a width not more than half the width of the canal and should be dug in the centre. Their depth should not be more than 1 m. Inside borrow pit should not be continuous and a portion not less than half the length of the borrow pit should be left between successive borrow pits. The arrangement of internal borrow pit is shown in Fig. 15.10.

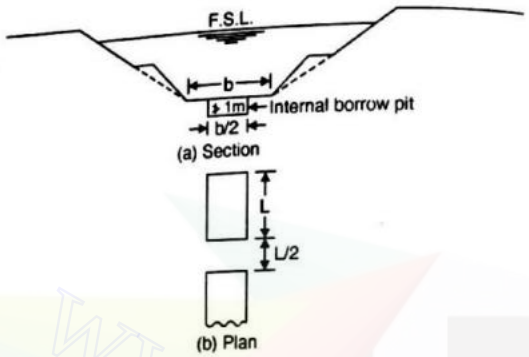

FIG. 15.10. ARRANGEMENT OF INTERNAL BORROW PITS.

15.9. SPOIL BANK

When the quantity of excavated earth is much in excess of the quantity required for filling it has to be deposited in the form of spoil banks. If the quantity of the extra earth available is small, it is used to widen the sevice road, but the service road can only be widened to a certain limit beyond which its maintenance may be difficult. The height of the spoil bank is not kept large, but on costly land the height of spoil bank is increased to involve minimum width of land. A longitudinal drain is always made between the spoil bank and service road. Cross drains should also be provided at about 100 m interval and the spoil banks should be discontinuous at these points.

15.10. PERMANENT LAND WIDTH

Width of land required for construction of canal cross-section is known as land width. *Permanent land width* is the distance between the outer toes of the canal bank plus a few metres on both sides for the construction of a drain or line of trees etc. It is the land width which is permanently required before starting the construction of canal.

During construction stage of the canal, some extra land width is required for staking of materials and machinery and for borrow pits. This is known as *temporary land width* and is acquired for temporary use. The land is returned to its owner after completion of the job. A nominal compensation has to be paid to the owner for temporary acquisition.

15.11. BACK BERM, COUNTER BERM

Even after providing the usual embankment section of a bank, the saturation gradient may cut the downstream side of the bank. In such a case the saturation line should always be covered by 0.3 to 0.6 m of earth. This is best done by the help of counter berm as shown in Fig. 15.11.

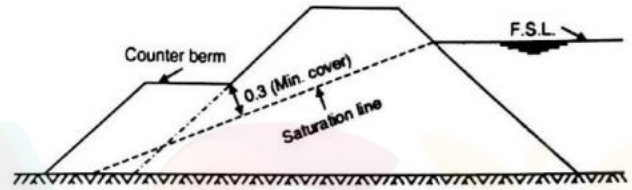

FIG. 15.11. COUNTER BERM.

15.12. MAINTENANCE OF IRRIGATION CHANNELS

After the construction of irrigation system in an area is completed, it becomes essential to maintain it for its proper and efficent functioning. There are various reasons due to which a canal may cease to function effciently. These are :

(*i*) Silting of canal (*ii*) Breaching of canal due to weak banks
(*iii*) Weed growth (*iv*) Overflow of canal banks

1. Silt Removal :

When the silt is deposited on the bed and sides, the capacity of the canal reduces. It is better to exclude silt by providing silt excluder and ejector as explained in Chapter 12. Curved wing-cum silt vanes and silt tunnels-cum-curved wings can be fitted near the head of an off-taking canal for exclusion of silt. However, none of these methods can put a complete check over the entry of silt in the channel and hence silt is deposited in the channel bed even after the best care.

The following measures are adopted to remove the silt :

(*a*) ***Flushing***. Flushing of the canal with clear water will lift up deposited silt. Absolutely clear water should be used for flushing but if this is not available then the water which contains minimum quantity of silt should be used for flushing. Flushing should generally be over done to cause some scour. This will create room for further silting and will thus reduce the frequency of flushing.

(*b*) ***Silt scouring fleet***. The method consists of having three lower barges connected to the upper barge by a cable operated by winch. The lower barges have movable shutters. The silt is kept agitated by manoeuvering the barge up and down. The method was used in Punjab but was unsuccessful.

(*c*) ***Bundle of thorny bushes*** tied together and pressed down by weight of stones are pulled inside the channel by animals. They are quite useful in dislodging the fine muddy silt.

(*d*) ***Iron rakes*** are also dragged in the channel to dislodge silt.

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(*e*) ***Reduction of area of flow***. Loaded boats are put across the section to reduce the area of flow and increase the velocity of flow.

(*f*) ***Stirring of silt by water jet.*** A pump fitted with a pipe and nozzle is placed on a barge. The high velocity jet is directed to the bed to stir the silt and prevent silting.

(*g*) ***Dredging.*** A dredger is very rarely employed for removing silt from canal as it is very costly method.

(*h*) ***Excavation***. The silt deposited in a channel is cleared off by manual labour. The method is quite costly as it requires recurring expenditure. This method is generally adopted for silt clearance in distributaries and minors. The silt must be deposited clear off the channel so that it does not find its way back to the channel.

2. Strengthening of Canal Banks

To prevent breaching of the canal bank, it should be strengthened properly so that valuable loss of irrigation and property is prevented due to breaching a canal section. There are four methods of strengthening a canal bank :

(*a*) External silting system. (*b*) Internal silting system.

(*c*) Formation of berm by internal silting.; (*d*) Formation of back berm.

(*a*) ***External silting system.*** In this method subsidiary banks are constructed which run parallel to the main banks. Cross bunds are constructed within the subsidiary bank and main bank at a distance of 150 to 1500 m to form compartment of silting or *silting tanks*. Water is allowed to get into the compartment from upstream side and is held there for some time before discharging it back to the canal from outlet end. When the cross bunds are separated at a distance of 150 to 300 m, the capacity of the bank is small and only a portion of full supply discharge is taken into the compartment. It is then known as *into out system*. But when the length of the compartment is large say 1200 to 1500 m, full discharge of the canal can be taken inside the compartment and it is then known as *long reach system*. The method is practised only when no water is required for irrigation downstream of the reach.

(*b*) ***Internal silting system.*** In this system the canal banks are set back away from their original positions. The section of the canal provided is large than required and, therefore, its velocity is low. The section, therfore, gets silted up very quickly. To induce silting and accelerate the process, low submersible spurs are constructed. The silted berm is shown in Fig. 15.12.

Hanging groyne or suspended groyne is also very useful for inducing silting.

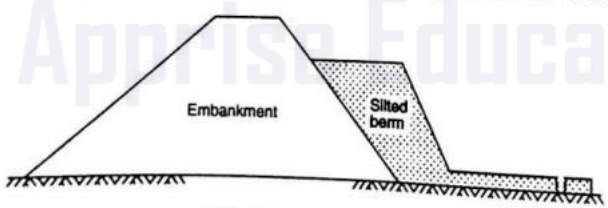

FIG. 15.12. SILTED BERM

(*c*) ***Formation of berms by internal silting.*** Banks can also be strengthened by formation of berm by internal silting. The silting of the berm can be accelerated by constructing permeable spurs from the side of the channel section.

(*d*) ***Formation of back berm.*** A back berm may be formed if the saturation line crosses the downstream slope of the bank as explained in § 15.11.

3. Weed control

Water weeds are unwanted plants that grow profusely in water under certain favourable conditions. They tend to reduce the discharging capacity of channel by reducing the area of the channel section and velocity of flow. The problem of weed growth is more marked in Deccan where the heavy weed growth may reduce the channel discharge to even less than 15%. The nuisance has, therefore, to be checked to permit the channel to function efficiently.

There are a variety of weeds growing on canal bed, water surface and water marks. They tend to thrive better in a range of 20° to 30° C. Weed growth is not possible in channels having high velocity of flow but when the channel has velocity less than 0.6 m per second weed growth is generally possible. The deposition of silt has no direct effect on weed growth, yet profuse weed growth is known to take place where silt is deposited. Light has a considerable effect on weed growth. Weed growth is accelerated in presence of light.

The *weed growth* can be checked by passing higher velocity than regime velocity in the channel. This will keep the silt in suspension and will make water turbid. Thus the light rays are cut off and silt is not deposited on channel bed.

Yet another way of weed control is *rush rotation*. In the process of rush rotation, the channel is run with full supply discharge for some time and then it is left completely dry for some time. This helps in excluding more light when higher depth is flowing, thus reducing the weed growth. During closure weed is unable to resist scorching rays of sun. Long duration closure has killing effect on the weed growth.

Weed removal may done by plucking them by hand and burning them when canal is dry.

15.13. MAINTENANCE OF SERVICE ROADS

The canal service road is unmetalled and, therefore, in monsoon season grass and small bushes grow on the road surface. The surface of road also wears and tears off due to some traffic over it. Sometimes unauthorised traffic also pass over canal roads which make the conditions still bad. The maintenance of service roads, therefore, consists of :

(*i*) Removal of grass and small bushes. (*ii*) Levelling of road surface.

(*iii*) Ramming and watering of top wearing surface.

Maintenance is usually carried out after the monsoon season. To check unauthorised traffic of cart, sometimes a check barrier is constructed at every crossing. The check barrier consists of a small earth mound with a slope of 1 : 4 on upstream and downstream side with some top width. Jeeps can easily be crossed over these mounds but for trucks and bullock-carts it becomes a real barrier.

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15.14. CANAL BREACHES

Canal breach or the tearing of the canal bank takes place when canal is in filling. During a canal breach, a big gap forms on the side and the large canal discharge rushes out doing unlimited damage.

Reasons for canal breaching.

1. ***Breaching due to defective design and construction of the bank.*** The canal bank is not strong enough to resist water pressure and a breach occurs.

2. ***Breaching due to exposure of saturation gradient.*** The soil particles on the outer slope are dislodged and flown away along with the seeping water. A cavity is thus formed in the bank which ultimately develops into a breach.

3. ***Breaching due to rush of water through rat holes or "shories".***

4. ***Piping*** near downstream toe may ultimately cause subsidence of canal banks.

5. ***A willful cut*** in the canal bank either for unauthorised irrigation or for allowing the flood water to get into canal to save an area from submergence.

On Lower Ganga Canal near Nadrai Aqueduct some *owlposts* are provided. It is believed that owls eat away rats and hence prevent the development of rat holes.

Repair of Breaches

A breach can be repaired by the procedure described below :

1. Immediately after a breach occurs, the canal discharge should be diverted to a nearby escape channel on the upstream side or the canal should be closed from the head to stop the outflow completely. Many a times it is not possible to close the canal from head as the irrigation of the area in between may suffer.

2. Provide steps in the sides of the gap so that the earth to be deposited for closing the gap forms a good bond with old surface.

3. Derive two lines of ballapiles in the gap in the inner side of the bank and fill the space in between by planks and bushes and with sand bages on top.

4. Step 3 should cut down the outflow considerably and velocities will reduce. If a second line of defence is needed, the same may be provided by two rows of sand bags with earth in between behind the pile line.

5. Collect a sufficent amount of earth on both sides of the gap. The earth collected should either be from spoil bank nearby or from borrow pits.

This earth material is then dumped up instantaneously from both sides of the gap to form a ring bund on outer side.

6. The opening in between the pile line and the gap is then filled with earth in layers, which should be properly compacted at optimum moisture content and the new bank is prepared.

The bank so formed is inspected carefully for few days to watch its proper functioning.

15.15. REGULATION OF CANAL SYSTEM

The process of distribution of water is called *regulation*. The principle underlaying the distribution of water is apportioning available canal supplies on the basis of irrigable

area. If the water is needed only occasionally then it may be supplied according to the general agreement between cultivators and canal administration.

A canal has to be run with full supplies if there is a keen demand in the commanded area, with part supplies if a part of commanded area only needs water and with no supplies if the demand falls below a certain minimum. The closure of a main canal is limited, as the area covered is large and there is always some demand in some parts of its commanded area though the channel has to be run with low supplies frequently. When the demand is only on a part of the system the distributaries which need water are kept open and the rest are closed. The difficulty in the distribution arises when there is a keen demand in the entire system but the available supplies from the river is unable to meet this demand. A straight forward but less effective solution should be to run all the distributaries with reduced supplies. Another alternative, a more effective one is, to run the distributaries with full supplies in rotation. This system is known as *Osrabandi* in U.P. and *Warabandi* in Punjab. Amongst the various disadvantages of the former method, the important ones are (*i*) silting of the channels, (*ii*) increased weed growth, (*iii*) increased seepage and water logging, (*iv*) low head at outlets. The advantage of the latter system is that the full closure of smaller channels may facilitate silt clearance, inspection and repair of masonry works.

A roaster is generally prepared which details the supplies allotted to different channels and names of different channels which will be closed down from week to week. The roaster is usually adhered to during season. Sometimes the demand in the area is very fluctuating, when a flexible regulation from week to week is adopted in accordance with the anticipated demand in the ensuing week. The anticipated demand is furnished to the engineer incharge through telegram sent by the revenue staff for their respective areas.

In order to strictly adhere to the regulation, it is imperative to know the discharge flowing in channel by plotting stage discharge curves as explained in 15.22. The procedure of measurement of discharge is therefore, described in subsequent articles.

15.16. MANAGEMENT OF IRRIGATION WATER

With increased need of water for irrigation it has become absolutely necessary to avoid wastage of irrigation water and use it as efficiently as possible. Government of India has now created 32 Command Area Developmet Authorities (CADA) in 12 different states so that irrigation potential created can be properly utilized. Various steps to check the wastage of water are discussed below :

(*i*) ***Land development*** . This includes land consideration, proper layout, shaping and grading of land and proper cultivation of land at the appropriate times.

(*ii*) ***Field application of water.*** An appropriate method of application of water has to be selected. CADA is now helping farmers to manage the field ditches with regulating structure beyond the outlets for proper supply of water to farm land.

(*iii*) ***Irrigation by rotation.*** This method should be practiced when available supplies are lower than the demand.

(*iv*) ***Method of charging.*** Volumetric basis of charging is extremely helpful in preventing wastage. However, if its implimentation pose a problem, charging on

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a crop season-area basis is a proper alternative. It requires careful planning and formation of committees for different outlets so that farmers at head and tail can get together and decide about allocation.

(*v*) ***Improved agronomic practice.*** Scientific principles in finding NIR and FIR, proper choice of high yielding varieties of crops and crop rotation help in more efficient use of water.

(*vi*) ***Educating farmers*** to develop a scientific and technical temperament and seeking their active participation in construction, maintenance and operation of field channels helps in better use of irrigation water.

15.17. MEASUREMENT OF DISCHARGE OF A CANAL

It is necessary to know the discharge flowing in a channel section. The efficient use of irrigation water largely depends upon the measurement of water. The measurement of discharge may give an idea of the amount of silting and canal losses.

Principal methods commonly used for masurement of discharge are mentioned below :

1. Area velocity method.
2. Chemical method.
3. Weir method.
4. Meter flume method.
5. Stage discharge curve method.

15.18. AREA VELOCITY METHOD.

Area-velocity method is a more general method and is applicable for rivers and canals both. This method consists of measuring area of the channel section and the mean velocity of flow through it. The product of the area with mean velocity yields the discharge.

Measurement of velocity : The main problem in measuring the discharge is the accurate determination of velocity. The velocity distribution in an open channel flow is shown in Fig. 15.13. The velocity is minimum at centre and decreases towards sides. For measuring the velocity in the channel section, a straight and well defined reach of the channel section should be selected. The site should also be free from the disturbing influence of hydraulic structures. A structure will be affected by back water curve whereas the discharge site located on the down-stream of the structure will be affected by the falling curve. The cross-sectional area of the channel should be fairly constant.

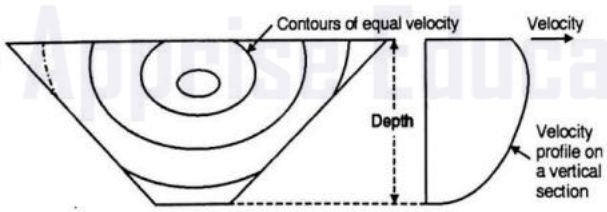

FIG. 15.13. VELOCITY DISTRIBUTION IN CHANNEL SECTION.

The velocity is measured by the following methods :

1. Surface floats.
2. Subsurface floats.
3. Twin floats.
4. Velocity rod or floats.
5. Current meter.
6. Pitot tube.

Surface float method : Surface floats are made of light material such as cork or hollow wooden blocks so that they may float on the surface. They are painted on top and sometimes attached with a flag to easily distinguish it from any other floating object.

Two wire ropes are stretched across the channel section one at the beginning and the other at the end of selected reach for conducting the test. This distance is called *test reach*. A third rope is stretched upstream of the test reach by 15 to 20 m. All the three ropes are marked at various points so as to divide the channel width in various compartments (Fig. 15.16) the centre line of which defines the part or track for floats. The floats are dropped in water along various paths near the third rope. They acquire uniform velocity by the time they enter the test reach. The time of travel of the test reach of only those floats which follow their correct track is recorded.

$$\text{Then the velocity of float} = \frac{\text{Distance travelled by float}}{\text{Time taken}}$$

For large channels the length of test reach should be 50 m to 75 m. For lined and small channels this length should be from 15 to 20 m. Wind affects the velocity of float. Observation, therfore, should not be taken where there is strong breeze.

The method of observing the velocity is the same with other types of floats also.

Sub-Surface floats or double floats: The surface float only gives the surface velocity. It is usually required to find the mean velocity of channel section. Sub-surface float finds the mean velocity across a vertical section.

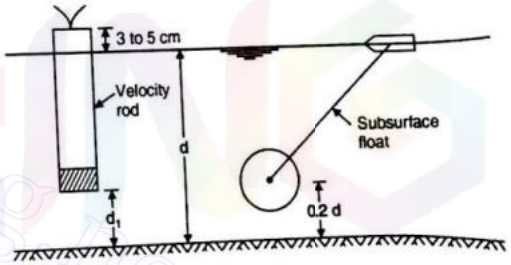

FIG. 15.14. SUB-SURFACE FLOAT OR ROD.

The sub-surface float (Fig. 15.14) consists of hollow metal sphere or cylinder attached to the surface float by means of a chord. The length of chord is adjustable so as to reduce or increase the depth of the sub-surface float. Mean velocity is obtained by adjusting the length of the chord such that the centre of sphere is about 0.2 *d* above the bed level.

Twin float

Twin float consists of two hollow metal spheres attached with a chord. Upper sphere floats on the surface and the lower sphere is so weighed as to keep the float vertical. Mean velocity is obtained if the lower float is just clear off the bottom.

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Velocity rod or rod floats: They are made of hollow tube or wooden rod. They are usually made telescopic with one tube sliding into the other. They are weighted with lead shots in such a way that when the rods are let afloat, they stand vertical. Their length is adjusted in such a way that the bottom of float may be clear off the bottom by a few centimetres. The mean velocity is calculated by the formula

$$V_m = V\left\{1.02 - 0.116\sqrt{\frac{d_1}{d}}\right\} \quad ...(15.9)$$

where V = the observed velocity

d = depth of flow in the channel

d_1 = clearance between lower-end of rod and bed.

Current meter : A current meter is a device which measures the velocity in the channel directly. The instrument consists of a wheel which is caused to revolve by the force of current when immersed in water and is provided with a gear to record the number of revolutions of the wheel. According to the revolving parts used the current meter may be classified into :

(1) Conical cup type current meters such as Anemometer and Price current meter.

(2) Hellicoidal vanes type current-meter such as Haskell current meter and Amsler current-meter.

Current meters with conical cups are more popular.

Rating of current. The current meter is placed in an artificial channel of known velocity. The instrument is suspended in the channel to a suitable depth. The number of revolutions recorded by the meter corresponding to the velocities is determined. A graph is plotted with velocity per second as ordinate and number of revolutions per second as abscissa. This is known as *rating curve*. Velocity in the channel is directly proportional to the number of revolutions and hence rating curve is a straight line.

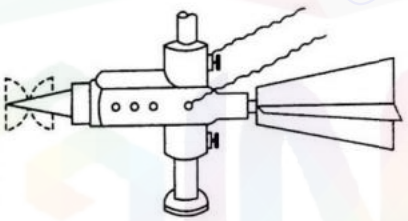

FIG. 15.15. CURRENT METER (AMSLER TYPE)

Measurement of Velocity: On channels of not too large a depth, the instrument is usually mounted on a rod on which it can be fixed in any position. The rod is made to rest on the bed of the channel, where the velocity is to be measured. Whereas on large channels, the current meter is suspended from a boat by a cable and is weighed down by a heavy stream-lined anchor.

Current meter is used in the following ways :

(i) Single point method. The current meter is immersed to 0.6 of the depth of the stream. At this depth velocity recorded is the mean velocity. The channel section is divided into various compartments and the velocity and the centre of each compartment is noted. Single point method is suitable for streams having a larger depth where

the meter cannot be suspended at 0.8 d. The method gives error of 5% to the mean value.

(ii) Double point method. In this method the meter is introduced at 0.2 d and 0.6 d of the stream. Average of the two velocities gives the mean velocity in the section. Maximum error in this method is about 2.5%. This is suitable for ordinary streams.

(iii) Integration method. The meter is introduced in the channel at various depths such as 0.1 d, 0.2 d, 0.3 d, 0.4 d and so on. The velocities recorded at these depths are then averaged to find the mean velocity. Integration method gives accurate results.

Sometimes when the velocity in the channel section is high, it may not be possible to introduce the meter at 0.8 d or 0.6 d. In that case it is introduced at 0.2 d and the surface velocity is found out, The surface velocity is multiplied by an appropriate coefficient to find the mean velocity.

Pitot tube. The method is very suitable for laboratory though not so suitable for the field as the entrance of the tube may get clogged by fine silt. The pitot tube is suspended in the stream with its nozzle pointing upstream. Due to dynamic pressure exerted on the mouth of the tube by flowing water, the level of water in the vertical limb of the tube rises above the water surface of the channel by an amount h. The velocity in the channel is then equal to. $\sqrt{2gh}$.

Measurement of area of flow. The cross-sectional area is generally divided into a number of small vetical strips. For measuring the width of the strip, a steel cable is suspended from two towers erected on either side of the bank. From the cables are hung ropes or chains to touch surface of water at regular intervals and divide the surface of water into suitable strips of known width. Depth of water at each point is measured by a sounding rod. The width and depth of each strip being known, cross-sectional area of various strips can be calculated.

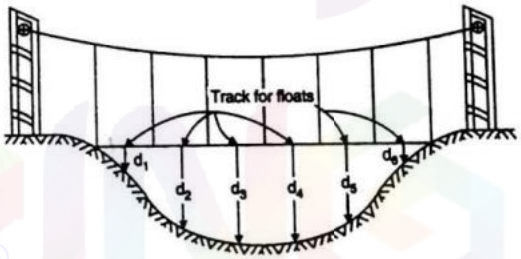

FIG. 15.16. MEASUREMENT OF AREA.

Thus if A_1, A_2, A_3, etc. are the areas of the strips and V_1, V_2, V_3, etc the corresponding mean velocities, then the channel discharge is given by

$$Q = A_1 V_1 + A_2 V_2 + A_3 V_3 ..... = \Sigma AV \quad ...(15.10)$$

15.19. CHEMICAL METHOD

A solution of known strength and weight of some chemical, preferably salt, is injected uniformally over the cross-section. It is then allowed to mix thoroughly as it travels downstream with water such that stream has uniform strength. From some

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where on the downstream side a sample of water is collected. The concentration of salt of the sample is determined by titrating the sample with silver nitrate and potassium chromate. Since the water may contain salt originally, the concentration of salt in undosed water is also determined. By knowing the concentration of salt in the sample, the volume of water Q flowing per second can be calculated by :

$$Q = \frac{w_1 - w_0}{w_2 - w_0} q \qquad ...(15.11)$$

where w_0 is the weight of salt/m^3 in undosed water

w_1 is the weight of salt/m^3 of injected salt soultion.

w_2 is the weight of salt/m^3 of dosed water obtained from the sample

q = quantity of solution injected in m^3/sec.

15.20. WEIR METHOD

The water is allowed to flow over a broad crested or a sharp crested weir and the head over the weir is measured to find the discharge. A rectangular or triangular notch can also be fitted and the head over the notch can be related to the discharge flowing over it.

General equation for the flow over a broad or sharp crested weir is :

$$Q = \frac{2}{3} C_d \sqrt{2g}\,(L - 0.2\,H_1)\,(H_1^{3/2} - h_a^{3/2}) \qquad ...(15.12)$$

where $H_1 = H + h_a$

h_a = head due to velocity of approach = $V_a^2/2g$

H = Head of water over the crest.

Value of C_d varies in accordance with the shape of the crest.

For Broad Crested Weirs $C_d = 0.557$.

Narrow Crested Weirs $C_d = 0.623$.

For clear overfalls $C_d = 0.666$.

When the discharge is measured over a V-notch

$$Q = \frac{8}{15} C_d \sqrt{2g}\, H^{5/2} \tan \theta/2 \; . \qquad ...(15.13)$$

where θ is the angle of Notch.

15.21. METER FLUME

Discharge measurement by a meter flume in open channel is similar to the discharge measurement by a Venturimeter in a pipe flow. The channel width is restricted to form a throat with smooth entry and exit transitions on both sides.

The depth of water at the throat and at the upstream normal section is recorded by means of two guage wells provided as shown in Fig.15.17. The throat has a higher discharge intensity and a higher velocity compared to normal section. The depth at the throat, therefore, is lesser than that at normal section. The difference in water level indicates the pressure loss. Knowing the depth of water, the area, a_1 at the

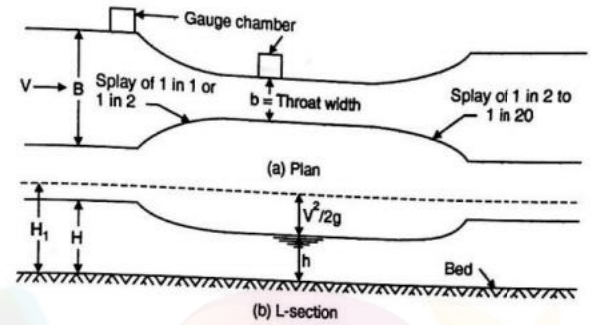

FIG. 15.17. METER FLUME.

normal section and a_2 at the throat section can be calculated. If H is the depth at the normal section and h is depth at throat the discharge Q is given by the formula:

$$Q = \frac{C\, a_1\, a_2 \sqrt{2g\,(H - h)}}{\sqrt{a_1^2 - a_2^2}} \qquad ...(15.14)$$

where C is the coefficient of flume to be determined experimentally.

15.22. STAGE DISCHARGE CURVE

Stage discharge curve is a curve between discharge and depth of water in the channel. Stage or depth is plotted as ordinate and discharge is plotted as abscissa. The curve may also be plotted on logarithmic scale ; then the curve may be in the form of a straight line. In order to plot stage discharge curve statistical data is collected initially for discharge at various stages of flow. This range of discharge data collected should cover the usual range between minimum and maximum flow in a river. A stage discharge curve is shown in Fig. 15.18.

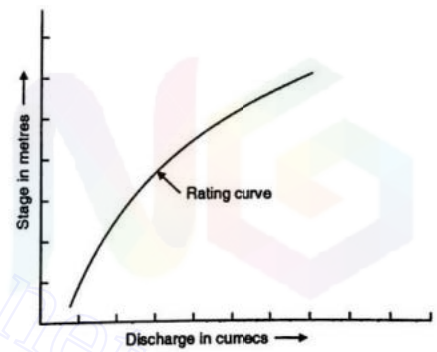

FIG. 15.18. STAGE DISCHARGE CURVE.

Once a stage discharge curve is plotted, it can be used for finding out the discharge at any subsequent period by measuring the stage. The stage discharge curve must be checked periodically as the regime conditions of the channel are always changing.

Example 15.1. *Design a channel by Kennedy's theory, using Garret's diagram for the following data :*

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Full supply discharge = 7 *cumecs*
Rugosity coefficient = 0.0225
Bed slope = 1/4444
Critical velocity ratio = 1.
Side slope of channel = $\frac{1}{2}$: 1.

Solution

$$S = \frac{1}{4444} = 22.5 \text{ cm/km.}$$

From the Garret's diagram [Fig. 15.3 (*b*)] find the intersection of the slope line (S = 22.5 cm/km) with discharge line (Q = 7 cumecs) and draw a vertical line. This line cuts many bed width lines ; various values of B and D so obtained are tabulated. The values of B and D which give critical velocity ratio equal to 1 will be adopted.

$Q = 7$ cumecs; $N = 0.0225$; $S = 22.5$ cm/km.

Bed width (B) from diagram	Depth (D) from diagram	Area A $= BD + \frac{D^2}{2}$	V_0 from diagram	$V = \frac{Q}{A}$	$\frac{V}{V_0}$	Remarks
6	1.5	10.13	0.72	0.693	0.956	small
7	1.35	10.26	0.673	0.682	1.015	more
6.75	1.38	10.15	0.685	0.685	1.005	satisfactory

Hence adopt $B = 6.75$ m and $D = 1.38$ m.

Example 15.2. *Design an irrigation channel in alluvial soil according to Lacey's theory, using Lacey's regime diagrams, given the following data :*

Fully supply discharge = 15 *cumecs*
Lacey's silt factor = 1.0
Channel side slopes = $\frac{1}{2}$: 1.

Solution.

From Lacey's regime diagram [Fig. 15.4 (*b*)] read the values of B and D corresponding to the intersection of silt factor line f with discharge line.

Hence the value are : $B = 15.1$ m; $D = 1.38$ m.

Similarly from Fig. 15.5 the value of slope

$$S = 0.19 \text{ part per } 1000 = \frac{0.19}{1000} = \frac{1}{5250}$$

Example 15.3. *Design and prepare the longitudinal section, schedule of area statistics and channel dimensions of an irrigation channel from the following data.*

Longitudinal levelling of channel alignment gives the following results :

Below km	Ground level (m)	Below km	Ground level (m)
0.0	157.20	2.6	155.35
0.2	157.26	2.8	155.17
0.4	157.29	3.0	155.12
0.6	156.21	3.2	154.95
0.8	156.56	3.4	154.70
1.0	156.52	3.6	154.39
1.2	156.51	3.8	154.28
1.4	156.28	4.0	154.19
1.6	156.11	4.2	154.10
1.8	156.14	4.4	153.74
2.0	155.83	4.6	153.58
2.2	155.68	4.8	153.38
2.4	155.58	5.0	153.18

Assume datum at level 150.00 m.
Full supply level of parent channel 157.70 m
Bed level of parent channel 156.00 m.
Commanded area are given as follows :

Below km	Gross commanded area (Hectares)	Culturable commanded area (Hectares)
0	24,000	16,000
1	22,000	14,000
2	18,000	12,000
3	16,000	11,000
4	14,000	10,000
5	12,000	8,000

Total loss below km 5 is 0.5 cumec. Losses are to be calculated at a rate 2 cumecs per million square meter of wetted perimeter.

Intensity of irrigation	*Rabi (Wheat) 30%*	*Kharif (Rice) 12 $\frac{1}{2}$ %*
Kor Period	*4 weeks*	*2 $\frac{1}{2}$ weeks*
Kor-depth	*13.4 cm*	*19 cm*
Side slope of the channel is	$\frac{1}{2}$:1	

$m = 1.$; *Kutter's* $N = 0.0225$.

Design Procedure

1. The longitudinal section of the ground is plotted in Fig. 15.1.
2. The slope of the channel is assumed with the help of Lacey's diagrams and is tabulated in schedule of area statistics.
3. A trial slope line for full supply level is marked keeping in view the guidelines of § 15.1. The position and depth of falls are tentatively marked.
4. The actual channel design is thereafter carried out by filling schedule of area statistics.

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5. The outlet discharge factor for controlling crops is determined as follows :

(a) Outlet discharge factor for Rabi $= \frac{8.64b}{\Delta}$

where $b = 4 \times 7$ days ; $\Delta = 0.134$ m.

Hence outlet discharge factor $= \frac{8.64 \times 4 \times 7}{0.134} = 1800$ hectares/cumec

(b) Outlet discharge factor for Kharif $= \frac{8.64\,b}{\Delta}$

where $b = 2.5 \times 7$ days ; $\Delta = 0.19$ m.

Hence, outlet discharge factor $= \frac{8.64 \times 2.5 \times 7}{0.19} = 760$ hectares/cumec.

The outlet disharge factor for the controlling crops is one which gives maximum discharge.

Let us assume the culturable commanded area as 16,000 hectares.

Area under Rabi crop with intensity of irrigation as 30%

$$= 16{,}000 \times \frac{30}{100} = 4800 \text{ hectares}$$

Area under Kharif crops with intensity of irrigation as $12\frac{1}{2}$ %

$$= 16{,}000 \times \frac{12.5}{100} = 2000 \text{ hectares}$$

$\therefore$ Discharge needed for Rabi crops $= \frac{4800}{1800} = 2.67$ cumecs

and Discharge needed for Kharif crops $= \frac{2000}{760} = 2.63$ cumecs

Hence outlet discharge factor to be adopted = 1800 hectares per cumec.

Design of channel at various sections

The design is started from the tail end where the losses are known.

1. At km 5.00

Culturable commanded area below 5 km = 8000 hectares

Intensity of Rabi irrigation = 30 %

Area to be irrigated for Rabi crops

$$= 0.30 \times 8000 = 2400 \text{ hectares}$$

Outlet discharge factor = 1800 hectare/cumec

$\therefore$ Discharge needed for Rabi crop $= \frac{2400}{1800} = 1.34$ cumecs

Total losses after 5 km = 0.5 cumec (given)

Total discharge at km 5.00 = 1.34 + 0.5 = 1.84 cumecs

Design discharge (Q) will be 10% more than total discharge.

Therefore, design discharge = 1.1 × 1.84 = 2.02 cumecs

$V/V_0 = 1.0$ and $N = 0.0225$

Assume slope $S = 0.25$ m/km = 1/4000

From Garret's Diagram Fig. 15.3 (a) :

Discharge Q (cumecs)	Slope S	Bed width B (m)	Water Depth D (m)	Area (A)= $BD + D^2/2$ (m^2)	Velocity V = Q/A (m/sec)	Critical Velcoity (V_0) m/sec	C.V.R.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
2.02	$\frac{1}{4000}$	5.50	0.73	4.27	0.472	0.448	1.05
		4.90	0.79	4.16	0.485	0.472	1.03
		4.55	0.84	4.17	0.484	0.488	0.992

Hence adopt values : $B = 4.55$ m ; $D = 0.84$ m.

2. At km 4.00

Gross commanded area below km 4.00 = 1400 hectares

Culturable commanded area = 10,000 hectares

Intensity of Rabi irrigation = 30 %

Area to be irrigated for Rabi crops $= \frac{30}{100} \times 10{,}000 = 3000$ hectares

Outlet discharge required $= \frac{3000}{1800} = 1.67$ cumecs.

Losses below km 5.00 = 0.5 cumec.

Losses in channel reach between km 5.00 to km 4.00.

For calculation of losses, the perimeter of the section at km 5.00 shall be taken into consideration as the section at km 4.00 at this stage cannot be calculated.

Wetted perimeter of the channel section at km 5.00

$$= B + \sqrt{5}D = 4.55 + \sqrt{5} \times (0.840) = 6.445 \text{ m.}$$

Thus loss in reach of km 5.00 to km 4.00

$$= \frac{6.445 \times 1000 \times 2}{10^6} = 0.013 \text{ cumec.}$$

$\therefore$ Total losses below km 4.00 = Losses below km 5.00 + Losses in the reach of km 5.00 to km 4.00

$$= 0.50 + +0.013 = 0.513 \text{ cumec.}$$

$\therefore$ Total discharge required at km 4.00

$$= 1.67 + 0.513 = 2.183 \text{ cumecs}$$

Adopt design discharge (Q) 10% more than total discharge

$$= 1.1 \times 2.183 = 2.40 \text{ cumecs.}$$

Similar method is followed for the determination of discharges at km 3.00, km 2.00 etc.

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The channel design and other details are tabulated in Schedule of Area Statistics (Table 15.7).

TABLE 15.7. SCHEDULE OF AREA STATISTICS AND CHANNEL DIMENSIONS

Below km	Gross commended area	Culturable commended area	Area to be irrigated			Outlet discharge factor	Outlet discharge	Losses in reach	Total losses	Total discharge	Channel Dimensions						Velocity (V) of flow	V/V_0
			Rabi (wheat) 30%	Sugar	Rice (12.5 %)						Bed slope	Bed width	Water depth	Design discharge	Height of Bank	Width of Bank		
	ha	ha	ha	ha	ha	ha/cumec	cumec	cumec	cumec	cumec	cm/km	m	m	cumec	m	m	m/s	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
0	24000	16000	4800	...	2000	1800	2.66	0.0150	0.570	3.23	22.5	5.50	1.04	3.56	...	2.0	0.570	1.015
1	22000	14000	4200	...	1750	1800	2.33	0.0140	0.555	2.885	22.5	5.20	1.007	3.17	...	2.0	0.555	1.00
2	18000	12000	3600	...	1500	1800	2.00	0.0136	0.541	2.541	22.5	4.85	0.975	2.80	...	2.0	0.538	1.00
3	16000	11000	3300	...	1375	1800	1.833	0.0136	0.527	2.360	24.0	4.70	0.945	2.60	...	2.0	0.530	1.00
4	14000	10000	3000	...	1250	1800	1.67	0.0130	0.513	2.183	24.0	4.55	0.915	2.40	...	2.0	0.521	1.00
5	12000	8000	2400	...	1000	1800	1.34	...	0.500	1.84	25.0	4.55	0.840	2.02	...	2.0	0.484	0.992

Example 15.4. *Calculate the economical depth of cutting for the cross-section of the canal shown in Fig. 15.19. The bed width of the channel is 5 m and top width of banks are 2 m. Side slope of excavation is 1 : 1 and of bank $1\frac{1}{2}$: 1. Height of banks from bed is 2.92 m throughout.*

Solution

For calculating the economical depth of digging we use the relation :

Quantity of digging = Quantity of banking

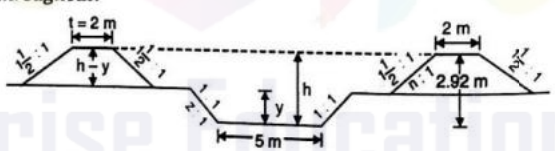

FIG. 15.19. CROSS-SECTION.

That is, sectional area of digging = sectional area of two banks

$$By + zy^2 = 2(h-y)\,t + 2n\,(h-y)^2$$

$$B = 5 \text{ m},\ z = 1,\ h = 2.92,\ t = 2 \text{ m},\ n = 1.5$$

Substituting the value in the above equation, we get

$$5y + y^2 = 2\,(2.92 - y)\,2 + 2 \times 1.5(2.92 - y)^2$$

or $$5y + y^2 = 11.68 - 4y + 25.5 - 17.52y = 3\,y^2$$

or $$2y^2 - 26.52\,y + 37.18 = 0 \quad \text{or} \quad y^2 - 13.26\,y + 18.59 = 0$$

From which $y = 1.6$ m.

Therefore, the economical depth of cutting = **1.60 m.**

PROBLEMS

1. What do you understand by the balancing depth ? Derive an expression for the same.
2. What do understand by schedule of area statistics ? How is it prepared ?
3. Explain, step by step, the method of designing an irrigation channel if the culturable commanded area and the details of crops along the cannal are given.
4. Draw sketches to show the section of canal, (*a*) partly in cutting and partly in filling, (*b*) wholly in cutting, (*c*) wholly in filling.
5. What is berm ? Why is it provided in the canal ?
6. Write short notes on the following :

(*i*) Free board in canals, (*ii*) Permanent land width, (*iii*) inspection road, (*iv*) berm, (*v*) counterberm, (*vi*) dowla.

7. Explain the measures adopted to remove silt from the canal.
8. How do you control weed growth in a canal ?
9. Explain various methods of discharge measurement is a canal.
10. Using Lacey's diagrams, design in irrigation channel on Lacey's theory for the following data :

Discharge $Q = 50$ cumecs
Silt factor $f = 1$
Side slopes $= \frac{1}{2} : 1$ $\left(\text{Ans} = 29.5 \text{ m};\ D = 1.95\ ;\ S = \frac{1}{6400}\right)$

11. Using Garret's diagrams, design channel section on Kennedy's theory for the following data :

Discharge $Q = 14$ cumecs
Kutters $N = 0.0225$
Critical velocity ratio $m = 1$
Side slopes $= \frac{1}{2} : 1$
Bed slope $= \frac{1}{5000}$.

12. What do you understand by balancing depth ? Derive an expression for the same. Hence, find the value of the balancing depth for the canal shown in Fig. 15.20. [Ans. 1.54 m]

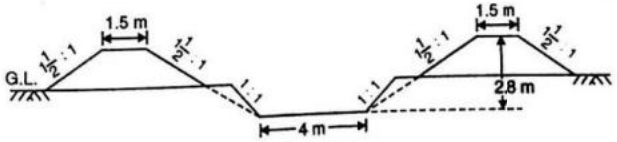

FIG. 15.20.

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When the agricultural land productivity affected by higher water table due to complete saturation of soil is called waterlogging

16

Waterlogging and Canal Lining

16.1. WATERLOGGING

An agricultural land is said to be waterlogged when its productivity or fertility is affected by high water-table. The depth of water-table at which it tends to make the soil water-logged and harmful to the growth and subsistence of plant life depends upon the height of capillary fringe, which is the height to which water will rise due to capillary action. The height of capillary fringe is more for fine-grained soil and less for coarse-grained ones. The normal height of the capillary fringe met within agricultural soils varies from 0.50 to 1.60 m. The crop yield is adversely affected when the capillary meniscus surface rises to within 0.6 m of ground surface. The land will, therefore, be waterlogged when the water-table is within 1.50 m [i.e. 0.6 + 0.9] to 2.1 m [i.e. 0.6 + 1.5] below the ground surface.

The adverse effects of high water-table upon the yield of crops also depend upon the nature of crop grown. Plants with the longer roots going deeper are more susceptible to high water-table.

The depth of water table which adversely affects the growth of different crops is given below :

Crops	*Depth of water table*
1. Wheat	0.9 to 1.2 metres
2. Cottom	1.5 to 1.8 m
3. Rice	0.6 m
4. Sugarcane	0.3 m
5. Fooder crop	1.2 m
6. Luceine	2.1 to 2.4 m

The problem of waterlogging in our country is a matter of deep concern. The total water logged land is estimated at 26.3 lakh hectares in area. West Bengal and Punjab have the water logged area as 10.33 lakh and 10.12 lakh hectares. Uttar Pradesh comes third in the list with an area of 2.03 lakh hectares. The waterlogging is spreading. The magnitude of the problem will be evident from the fact that during 40 years period from 1918 to 1958, the water-table in Kalinadi-Ganga and Hindan-Kalinadi Doab (U.P.) has risen by 2 to 2.3 m and in Arid Pandu, Arind Sengar and Sengar Yamuna Doab by about 3 m. The percolation and adsorption losses from canals is a dual loss

in as much as water that could beneficially be utilized for irrigation is wasted and in addition the same causes damage to agricultural land due to waterlogging.

16.2. EFFECTS OF WATERLOGGING

The infertility of the soil when an area becomes waterlogged is usually due to the following reasons :

1. *Inhibiting activity of soil bacteria*

The liberation of plant food is dependent upon the activity of soil bacteria, which requires adequate amount of oxygen in the air for proper functioning. When the soil pores within the root zones of the crops normally grown are so saturated as to effectively cut off the normal circulation of air, the land is said to be waterlogged.

2. *Decrease in available capillary water*

Plant life draws its substance from the soil-solution round the soil particles which is drawn into the plants by capillary action and osmosis. If the water-table is high, the roots of the plants are confined to the top layers of the soil above the water table while if the water table is lower, the roots of plants have more room for growth.

3. *Fall in soil temperature*

A waterlogged soil warms up slowly and due to lower temperature, action of soil bacteria is sluggish and plant food available is less.

4. *Defective air circulation*

When the water-table is high, the drainage becomes impossible and the carbon dioxide liberated by the plant roots cannot be dissolved and taken away. Consequently fresh air containing oxygen is not drawn and activity of soil bacteria and plant growth suffers.

5. *Rise of salt*

The rise of water-table also causes accumulation of alkali salts in the surface soil by the upward flow of water which is established in waterlogged lands. If the underlying layers contain alkali salts in solution they are brought up with water which evaporates leaving the salt on the surface.

The alkaline deposit changes the pH value of soil. Soils with pH value 7.0 to 8.5 gives normal yields, with pH value 8.0 to 9.0 the yield decreases ; when pH value rises to 11.0, the soil becomes infertile.

6. *Delay in cultivation operations*

In waterlogged areas, cultivation operations such as ploughing and mulching are either impossible or difficult and in any case they are delayed. Sowing of crops and their growth are also delayed. Crop yield is poor and it arrives late in market causing loss to cultivators.

7. *Growth of wild flora*

In waterlogged soils, natural flora such as *water hyacinth* grows profusely. This reduces the crop yield. A cultivator has to waste money and time both for clearing it out.

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8. *Adverse effect on community health.*

The climate of a waterlogged area becomes damp. Formation of stagnant pools may become breeding places for mosquitoes. The climate thus becomes extremely detrimental to the health of community.

16.3. CAUSES OF WATERLOGGING

Waterlogging in any particular area is normally the result of several contributory factors. The main factors causing waterlogging are given below :

1. *Inadequate surface drainage*

When the surface drainage is not adequate, the heavy precipitation in the area is not drained off quickly and the rain water remains stagnant over the area for considerable time. This gives rise to heavy percolation and water-table rises in the area.

2. *Seepage from canal system*

In nature, the water-table is in state of equilibrium. The amount of inflow is practically equal to the amount to outflow. Thus equilibrium is upset by the construction of a new canal system as a new constant source of inflow due to seepage is introduced. The water-table in the area, therefore, rises. The rise takes place up to a level where the increased inflow is balanced by the increased outflow. The increased outflow is mainly because of increase in soil evaporation due to nearness of water-table, increased transpiration from plants and increased discharge in seepage drains. Seepage losses from canal have been discussed in §16.5.

3. *Over-irrigation of fields*

When the irrigation water applied to the field is in excess of the requirement of the crop, deep percolation takes place which is retained in the intermediate zone augmenting the ground water storage.

4. *Obstruction of natural drainage*

If a natural drainage is obstructed by irrigation channel, rail or road embankments, it will not be able to pass the rain water of catchment. There will thus be flooding of land and consequent water logging.

5. *Obliteration of natural drainage*

Sometimes the cultivators plough up and obliterate an existing natural drainage. This results in stoppage of storm water flow, consequent flooding and waterlogging.

6. *Inadequate capacity for arterial drainage*

The arterial drainage or Nadi may not have adequate capacity to pass the heaviest floods in the entire catchment. As such the functions of all the drains connected to the arterial drain is seriously hampered. The flood water of local drains thus spreads over the countryside for days and heavy percolation into the subsoil causes alarming rise in water table.

7. *Construction of a water reservoir*

Similar to the seepage from a canal, the seepage from the reservoir augments the water-table and may cause water-logging.

8. *Natural obstruction to the flow of ground water*

Sometimes subsoil does not permit free flow of subsoil water due to some natural obstruction. This may accentuate the process of raising the water-table.

The creation of a high false water-table or perched water-table also leads to waterlogging.

16.4. REMEDIAL MEASURES

In devising anti-waterlogging measures, the nature and magnitude of various factors, enumerated in previous article, should be correctly assessed and allowed for. Various remedial measures adopted for prevention of waterlogging are discussed below :

1. *Efficient Surface drainage.* An efficient drainage system which permits a quick flow of rain water in short period helps to reduce the waterlogging. They have a low initial cost of construction.

2. *Under-drainage by tile drains.* The drainage of agricultural land is done more satisfactorily by the drains (Refer § 16.6), A suitable tile drain can hold the water table at a predetermined level which will be most beneficial to the crops. It has a large initial cost.

3. *Reducing percolation from canals.* Measures for substantial reduction of percolation losses from canal have to be undertaken. This can be achieved in the following way :

(a) *Lining of irrigation channels* so as to make bed and sides of canal impervious.

(b) *Lowering of full supply level of irrigation channels.* If the full supply level of irrigation channel is reduced, there will be lesser seepage loss from embakment. The effective head between full supply level and field will also reduce and, therefore, chances of a wasteful use of water is avoided.

(c) *Construction of intercepting drains.* These drains are constructed along the canals particularly at high embankment to arrest seepage flow. The distance of seepage drains from the canal should be such that while it intercepts seepage and percolation, it is not so near that it may draw water directly from the canal. This distance will depend upon the difference of level between the water surface in the canal and the natural ground water-table.

4. *Restriction of Irrigation*

(*a*) The cultivators should be educated for economic use of water and induced to divide his field into "Kiaries" to avoid wastage. He should also be encouraged to supplement his water requirement from open wells and tube wells.

(*b*) Area with high water table may be allowed only for Kharif irrigation and during Rabi the cultivators may irrigate from open wells and tube wells.

5. *Lining of water courses.* The losses by percolation from cultivator's watercourse are of the order of 20% and above. Their lining, therefore, further checks the inflow of canal water to subsoil through water courses.

6. *Removing obstruction in natural drainage.* Drainage crossing with road, railways and canals should be remodelled to make it more efficient.

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7. *Prevention of seepage from water reservoir.* Adequate and suitably designed toe filters are provided so that seepage ultimately finds its way into the natural stream.

8. *Depletion of ground water storage by pumping.* The surplus groundwater which causes undesirable rise in the water-table can be pumped out by :

(*a*) *Shallow well pumping.* Water is pumped from top aquifers to depress the water-table. This water may be utilized for irrigation in some other area.

(*b*) *Deep well pumping.* The water is pumped out from several water bearing strata by a series of wells scattered over large areas and discharge is used for further irrigation.

In area where the danger of waterlogging has become imminent, further canal irrigation should not be introduced. Instead, tube wells should be sunk and the area should be irrigated by tube wells.

Irrigation from masonary wells also reduces waterlogging.

9. *Changes in crop pattern.* A change in crop pattern may minimise the damage to plant line.

10. *Adoption of sprinkler method for irrigation.* This reduces the percolation losses from watercourses as only predetermined amount of water is applied to the land.

16.5. LOSSES IN CANAL

The losses in canal comprise evaporation from the surface and seepage through the bed and sides of the drains.

Loss due to evaporation from a canal system depends upon the climatic conditions of the region and hence it can never be prevented. However, losses by evaporation forms a minor part, hardly 1 to 5% of seepage loss and hence, in most of the cases evaporation loss is not significant.

Loss due to seepage is the most significant as this forms the major portion of the loss of the canal water. The seepage loss depends mainly upon the following factors :

1. Position of subsoil water-table.
2. Porosity of scil and subsoil.
3. Extent of absorbing medium.
4. Design of canal cross-section.
   (*a*) Depth of water in canal : greater the depth, greater is the loss of water.
   (*b*) Velocity of water in the canal : the loss decreases with increase in velocity.
5. Physical properties of canal water :
   (*a*) Temperature of water. The loss increases with increase in temperature of water.
   (*b*) Amount of silt carried in suspension. The loss decreases with an increase in amount of silt carried on suspension.
6. Conditions of canal system. The loss decreases with the age of canal and increases with the extent of absorbing medium.

The losses in canal are usually *measured* by a simple method known as *inflow and outflow method.* In the method a long reach of the canal is selected. Discharge

observations are taken at the beginning and end of this reach for several days continuously. A fairly high level with constant gauge should be maintained in the reach. The outlets or any offtaking channel should be completely closed during observation period. The difference between the discharge entering the reach and that leaving the reach is the loss occurring in the reach. The losses are expressed in the following different ways :

(*a*) *As cumecs per million square metres of wetted perimeter*:

(*i*) *In north India*

Ordinary clay loam	1.77 to 2.65
Loose sandy soil	5.39 to 6.19
Gravelly soil	10.61 to 21.21

(*ii*) *In South India*

Decayed rock or gravel	3.05
Alluvial or red soil	2.44
Black cotton soil	1.53
Rock	0.91

In U.P. the following formulae are in use to determine the canal losses.

$$dq = 4.8603 \times 10^{-3}\, C(B+D)^{2/3}$$

where, dq = Loss in cumecs per km
B = Bed width in m
D = Depth in m
C = Constant, the value of which is 0.7 for main and branch canals and 1.0 for other canals.

(*b*) *As a depth of water lost in 24 hours over the area of wetted perimeter.*

(*c*) *As a percentage of the discharge of the channel.*

(*d*) *As a percentage per km length of the canal.*

The magnitude of seepage losses from canal will be evident from fact that on Upper Ganga Canal the losses for main canal and branches are 15%, for distributaries and minors 20% and for water courses 22%. The total evaporation losses are less than 1%. On Upper Bari Doab Canal system, the losses estimated for main canal and branches are 12.2%, for distributaries and minors 9.2% and for water courses 12.5%.

Losses in canal irrigation U.P. are assessed as below :

Sandy soil	25 to 50%
Sandy loams	15 to 25%
Fine sandy loams	10 to 20%
Heavy clay loams	5 to 15%

Depending upon the position of water-table the seepage loss from the canal occurs in two ways, namely :

(*i*) Absorption (*ii*) Percolation.

1. *Absorption.* When the water-table is considerably below the ground level, the water seeping through pores is unable to join the water-table and wets the subsoil locally forming a saturated bulb as shown in Fig.16.1. The zone in between this saturated zone and the zone of capillary moisture remains unsaturated.

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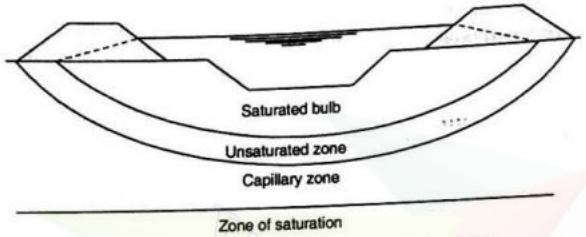

FIG. 16.1. SEEPAGE LOSS BY ABSORPTION.

2. *Percolation.* When the water-table is close to the ground level, the seepage water may establish a direct and continuous flow in between the canal section and water table. Thus the space between the water-table and canal bed enclosed by the two flow lines, marked in Fig. 16.2 remains completely saturated with water. When once a direct flow is established, the seepage loss depends upon the head diffrernce between the full supply level of canal and water-table.

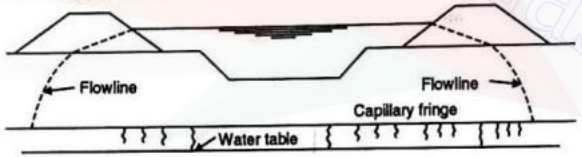

FIG. 16.2. LOSS OF CANAL WATER DUE TO PERCOLATION.

16.6. LAND DRAINAGE

Benefits of Drainage

The introduction of drainage system in an area has the following advantages:

1. Adequate drainage improves soil structure and increases and perpetuates the productivity of soil.
2. Waterlogged saline and alkaline lands can be reclaimed.
3. Adequate drainage leads to early ploughing and planting.
4. Drainage of the area lengthens crop growing seasons.
5. Drainage reduces water-table in an area and thereby :
   (*a*) provides more available plant food by increasing depth of root zone soil.
   (*b*) increases soil ventilation.
   (*c*) favours growth of soil bacteria.
   (*d*) assures high soil temperature.
6. Drainage decreases soil erosion and gullying by increasing water infiltration into soil.
7. Excess salt from soil can be leeched out.

Types of Drains

The drains can be divided into two main types, namely :

(*i*) Open drains, (*ii*) Closed drains.

Open drains can be further sub-divided into :

1. ***Shallow surface drains.*** It reduces percolation to ground water but is of no use if water has already permeated to ground water reservoir. This drains excess irrigation water applied to the field and accelerates removel of storm water.

2. ***Deep open drains.*** These can be used to reduce water logging without the provision of the tile drains. These will then have to be provided at a distance of 0.75 km or even less. They are also commonly used as outlet drains for closed drain system.

Under-drains or Tile-drains : Tile drains are located at suitable depths below the ground surface above the impervious clay stratum impeding the natural percolation of water. They are preferably placed in a medium or high permeability.

16.7. DESIGN AND MAINTENANCE OF OPEN DRAINS

Open drains are very useful in removing the local accumulation of water expeditiously. These drains are not needed if the country has a good slope with sufficient number of well defined drainage lines nor they are needed in arid areas with very little rainfall. They are mostly needed in deltaic tracts where there are some under-ground obstructions of impervious material across natural drainage lines.

The drainage system in an area is generally introduced many years after the introduction of a canal system especially when the rise of water-table is very slow.

Layout. The alignment of drains should follow natural drainage lines, keeping in view the amount of falls required for the surface slope generally called *drainage head.*

Slope. A slope of 1/300 to 2/300 is generally satisfactory in deltaic tract.

Capacity of drain. The drain is designed for a capacity to carry subsoil water in addition to the natural drainage of the country and the quantity of water expected from rains. Capacity may be kept such as to remove surface water in 3 to 5 days, since no harm to crop occurs during their submergence upto 3 to 5 days.

It is considered uneconomical to design the earthen section of surface drain to cope with maximum anticipated discharge of the catchment. In Uttar Pradesh, the design discharge for the earthen section is kept only 50% of the maximum anticipated discharge but the masonry works on the drains are designed for maximum discharge. For the plains of Ganga Yamuna Doab, the maximum anticipated discharge is calculated at 0.054 cumecs per hectare of the catchment area while the design discharge for masonry is calculated at 0.108 cumecs per hectare of catchment area. Drainage system in Punjab is designed at 0.0343 cumecs per hectare.

Section. The design of a drain is done as per the design of irrigation canal, but critical velocity is not given any consideration. However, if the drain has to carry the rain water, then $V/V_0 \geq 1$. It is desirable to provide a cunnette (a small drain in the bed of larger drain) in the centre of the drain with bed gently slopping towards it to carry the small seepage discharge in case a drain is receiving both storm and seepage water. The *depth* of deep drain generally varies from 2 to 4 m. The *full supply level* is kept about 0.6 m below the ground level.

Maintenance. The main problem of maintenance is the removal of acquatic weeds. A reference may be made to § 15.12 for the same.

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16.8. UNDER-DRAINS OR TILE DRAINS

The drainage of waterlogged agricultural land by surface drains is not very satisfactory for the following reasons :

1. Valuable agricultural land is wasted in the construction of surface drains.
2. Bridges across the drains are essential. Even after the bridge construction, the natural communication for agricultural operation is restricted.
3. Valuable plant foods are washed down in the open drains.

Thus the drainage of agricultural land is done more satisfactorily by covered tile drains. A tile drain is located at suitable depth below the ground surface and is usually covered with filter of coarse sand and bajri. The gravitational water at the level of tile drain is under pressure due to the head of water above it. It enters the tile drains through the open joints generally up to 10 cm wide at the end of tiles. The drains have their outlets in natural or artificial channels which are so situated that they are not usually submerged in normal rains.

16.9. LAYOUT OF A TILE DRAIN SYSTEM

The layout of tile drain is governed by the topography of the area. Various possible alternative plans of layout for tile drainage system is shown in Fig. 16.3 and are discussed below :

1. **Natural System.** The natural system is preferred in rolling topography, where drainage of isolated area is required. In this system the mains and connected laterals are provided in natural course as shown in Fig. 16.3(*a*).

2. **Grid iron layout.** The drainage system consists of laterals and submains. Laterals are provided only on one side of submain. The system is used when the land is practically level or where the land slopes away from the submain on one side and when the entire area has to be drained.

3. **Herring bone pattern.** In this system laterals join the submains from each side alternately [Fig. 16.3(*c*)]. It is adopted when the submain is laid in depression. The land along the submain is double drained but since it is in a depression, it probably requires more drainage then the land on the adjacent slopes.

4. **Double main system.** The system has two mains with separate laterals for each as shown in Fig. 16.3(*d*). This system is adopted when the bottom of depression is wide. This arrangement reduces the length of the laterals and eliminates the break in slope of the lateral at the edge of depression.

5. **Intercepting tile drains.** In this system there is no lateral drain. A submain is provided at the toe of slope as shown in Fig. 16.3(*e*). This arrangement is adopted when the main source of drainage is from a hilly land.

Mains and submains should be kept away from trees as the roots of trees can easily enter the open joints of the under drains.

16.10. FLOW OF GROUND WATER TO DRAINS

The tile drains should be fixed in such a fashion that the water table can be maintained at a pre-determined level. This can be achieved by keeping it about 0.30 m below the level at which water table is desired to be maintained.

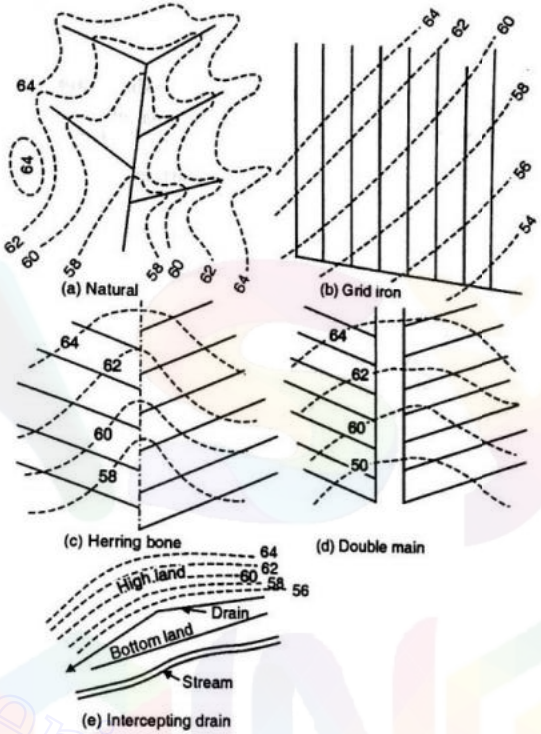

FIG. 16.3. LAYOUT OF TILE DRAINS.

With reference to Fig. 16.4, let L be the spacing of the drains and a its height above the impervious stratum. Maximum height of drained water-table above the stratum

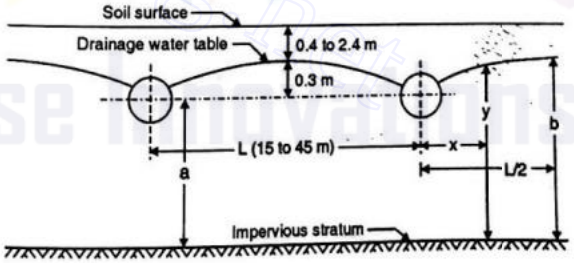

FIG. 16.4. GROUND WATER IN A DRAINED FIELD.

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is b. Let y be the height of the drained water-table above the stratum at distance x from the centre of the drain.

Assumptions

1. The hydraulic gradient at a distance x from the drain is dy/dx.
2. Flow lines are parallel and cross-sectional area of flow at a distance x is $y \times 1 (= y)$.
3. Flow q towards the drain is inversely proportional to the distance from the drains.

If Q_D is the design flow per meter length of drain,

then $$q = \frac{1}{2} Q_D \quad \text{when } x = 0$$

and $$q = 0 \quad \text{when } x = L/2.$$

Therefore $$\frac{q}{Q_D/2} = \frac{L/2 - x}{L/2} \qquad ...(16.1)$$

As per Darcy's Law

$$q = k\,i\,A = ky\frac{dy}{dx} \qquad ...(16.2)$$

From Eq. 16.1 and Eq. 16.2

$$\frac{Q_D}{L}(L/2 - x) = ky\frac{dy}{dx} \qquad ...(16.3)$$

or $$\frac{Q_D}{2kL}(L - 2x) = y\,dy$$

Integrating $$\frac{Q_D}{2kL}(Lx - x^2) = \frac{y^2}{2} + C$$

When $x = 0$, $y = a$: So $C = -a^2/2$

$$\therefore \quad k = \frac{Q_D(Lx - x^2)}{L(y^2 - a^2)} \qquad ...(16.4)$$

When $x = L/2$ $y = b$, hence

$$L = \frac{4k(b^2 - a^2)}{Q_D} \qquad ...(16.5)$$

Eq. 16.5 shows that spacing of the drains is independent of drain size. Actually an increase in drain diameter produces slight increase in yield. Q_D will depend upon infiltration discharge into the ground which should be removed by the drain. Its value is usually *assumed at 1% of average annual rainfall at the place, to be removed by the drain in 24 hours.*

Let P_{av} (metres) be the average annual rainfall at the place. Then

$$Q_D = \frac{0.01\,P_{av}(L \times 1)}{24 \times 3600} = \frac{P_{av}\,.\,L}{8.64 \times 10^6} \text{ cumecs/m length of drain.}$$

Substituting the value of Q_D in Eq. 16.5, we get

$$L = \frac{4k(b^2 - a^2)}{P_{av}L/8.64 \times 10^6}$$

or $$L = \left[\frac{(8.64 \times 10^6)\,4k(b^2 - a^2)}{P_{av}}\right]^{1/2}$$

See Example 16.4 for illustration.

Drainage coefficient

Drainage coefficient is the rate at which water is removed by the drain. Its value depends upon the rainfall, but also varies with type of soil, type of crop, degree of surface drainage etc. It is expressed as the depth of water in cm (or m) to be removed in 24 hours from the drainage area. The common values of the drainage coefficient, suggested for humid regions by the U.S. Soil Conservation Service are : 1 to 2.5 cm/day for mineral soils and 1.25 to 10 cm per day for organic soils for different crops. Its value is so fixed that 1% of the average annual rainfall in that area is removed per day.

16.11. LINING OF IRRIGATION CHANNELS

The process of reducing seepage losses and saving by providing an impermeable layer on the surface of water carrying channel.

1. Necessity

Lining of canal is necessary :

(*i*) to minimise the seepage losses in canal.
(*ii*) to increase the discharge in canal section by increasing the velocity.
(*iii*) to prevent erosion of bed and side due to high velocities.
(*iv*) to retard the growth of weeds, and
(*v*) to reduce maintenance of canal.

2. Advantages of Lining.

The main advantages derived by canal lining are mentioned below :

1. The lining of canals prevents seepage loss and thus more area can be irrigated by the water so saved. The cost of irrigation is, therefore, reduced.
2. The lining of canal is an important anti-water logging measure as it reduces seepage to the adjoining land.
3. The lining provides a smooth surface. The rugosity coefficient, therefore, decreases. The resistance to flow also decreases and hence the velocity of flow in the lined canal increases.
4. The increased velocity minimises the losses due to evaporation.
5. The increased velocity helps to provide a narrow cross-section for lined channels.
6. Higher velocity helps in providing a flatter hydraulic gradient or bed slope. Thus better command can be obtained.
7. Higher velocity prevents silting of channel.
8. Lining makes the banks more stable in light textured soil.
9. Lining reduces maintenance costs and possibility of breaching due to increased stability of section.
10. Lining of a canal prevents or reduces weed growth.

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11. Lining of a canal increases available head for power generation as a flatter gradient can be provided.
12. Canal lining assures economical water distribution.
13. Canal lining prevents water to come in contact with harmful salts during transit.

3. **Disadvantages of Canal Lining**

The canal lining has certain disadvantages although the advantages far out-weigh the meagre disadvantages. The disadvantages of canal lining are mentioned below :

1. Canal lining requires a heavy initial investment.
2. Lining being permanent, it is difficult to shift the outlets very often.
3. It is very difficult to repair the damaged lining.
4. A lined channel section is without a berm. The additional safety provided by the berm for vehicular and pedestrian traffic is, therefore, absent in a lined channel.

4. **Suitability of Canal Lining Material**

A canal lining material, to be suitable, should have the following properties :

1. The material used for lining should provide complete *water tightness.*
2. The material used should have low coefficient of rugosity so as to make the section *hydraulically more efficient.*
3. The material chosen for canal lining should be *strong and durable.*
4. The lining should not have a very *high initial cost.* Subsequent maintenance cost of canal lining should be *very low.*
5. The material used should be able to *resist growth of weeds and attack of burrowing animals.*
6. The material used should be unaffected by *tramping of cattles.*
7. The material should withstand high velocity.
8. The material should permit construction of required slope easily.

16.12. TYPES OF LINING

The following are the important types of concrete lining used in India :

(*a*) ***Hard surface type lining*** :

1. Cement concrete lining.
2. Shotcrete lining.
3. Precast concrete lining.
4. Cement mortar lining.
5. Brick lining.
6. Stone blocks, or undressed stone lining.
7. Asphaltic lining.

(*b*) ***Earth type lining*** :

8. Soil cement lining.
9. Clay puddle lining.
10. Sodium carbonate lining.

(*c*) ***Buried and protected membrane type lining*** :

11. Prefabricated light membrane lining.
12. Bentonite soil and clay membrane lining.
13. Road oil lining.

1. **Concrete lining** : Concrete lining has found only a limited use in India because of the high initial cost. Concrete as a lining material has excellent hydraulic properties.

Thickness of concrete lining : The thickness of lining is governed by the requirement of imperviousness and structural strength. Usual thickness provided is from 5 to 10 cm for M 15 concrete and is 7.5 cm to 15 cm for M 10 concrete.

Bank slope : Earthen banks of canals upon which lining has to rest are made at a self-supporting slope so that no earth pressure is exerted upon lining.

Reinforcement : Reinforcement is not provided except where it may be particularly required. Reinforcement interferes with the working of mechanical equipment but if provided it prevents formation of wide gap and possible faulting in case of failure of such grade. It also reduces width of temperature crack.

Joints : Construction joints localise volume changes due to shrinkage of concrete and avoid cracking. Lining is usually provided in panels not more than 3 m. Different types of joints provided are shown in Fig. 16.5.

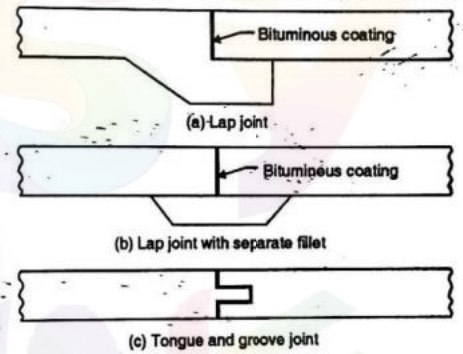

FIG. 16.5. DIFFERENT TYPES OF JOINTS.

Where lining operations are continuous, transverse grooves or longitudinal and transverse grooves both, at a spacing of 3 to 5 m are formed.

Construction of concrete lining: Prepare a subgrade and compact it fully. Saturate subgrade to a depth of 30 cm in sandy soil and 15 cm in other soils and then treated in any of the following manner before laying concrete :

(*a*) Lay a base coat of 1 : 4 cement sand slurry on the subgrade. This is cheapest and best treatment.

or (*b*) Spread oil paper on the subgrade.

or (*c*) Spread crude oil on subgrade.

The concrete used for lining generally has a mix ratio 1 : 4 : 8 or 1 : 3 : 6 or 1 : 4 : 6. The concrete is usually laid in alternate blocks. The form work is removed after seven days and the remaining blocks are laid then. After the concreting is over, the entire lined surface is properly cured.

2. **Shotcrete lining** : In this type of lining, a mixture of cement and sand (1 : 4) is shot at the subgrade through a nozzle. The thickness of this type of lining varies from 2.5 to 6.5 cm. Shotcrete consumes large amount of cement. Shotcrete can be placed on irregular subgrade and hence fine dressing of subgrade is not required. Shotcrete is also used for repair of old but sound concrete lining.

3. **Precast concrete lining** : This type of concrete lining consists of precast slabs usually 90 cm × 30 cm in size. The thickness of each slab is from 5 to 6.5 cm. The blocks are manufactured with an interlocking arrangement. The slabs are laid

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on well prepared and compacted subgrade. The joint is grouted with cement or is sealed with asphalt to prevent any seepage through joint

4. **Cement mortar lining** : Cement mortar lining has never been used as such. It is usually used as sandwich material between brick layers. Thickness for this type of lining is kept from 1 to 4 cm. The method of construction is similar to that of concrete lining. A large amount of cement is consumed in this type of lining and hence it is quite costly.

5. **Brick lining** : This type of lining consists of a single or double layer of brick masonry or a layer of brick masonry followed by a layer of tiles laid is mortar. The first layer is laid on a 12 mm layer of 1 : 6 cement mortar. The mortar is first spread on a properly compacted and wetted subgrade. A 12 mm thick layer of plaster in 1 : 3 cement mortar is laid over the first layer. Then the second layer is laid over it in 1 : 3 cement mortar. The size of brick especially made is restricted to 30 × 15 × 5 cm for convenience of handling.

This type of lining has been used in Punjab on Bhakra and Haveli canals. On Bhakra canal its performance has been found to be extremely satisfactory. This lining is hydraulically as efficient as that of concrete lining. In case of any settlement of subgrade, the mortar joints between bricks provide for numerous cracks which are so fine that they do not permit any seepage through them. In case of any abnormal settlement, the failure in this types of lining will be local and lining can be repaired easily.

6. **Asphaltic lining** : Asphaltic lining is carefully controlled mixture of asphalt and grade aggregate mixed and placed at a high temperature of 200°C and covered with a 30 cm layer of earth material for a protection. The mix is placed either by hand or by an equipment similar to that of concrete.

7. **Soil cement lining** : Soil cement is a mixture of cement and soil. Soil used in soil cement should not contain more than 35% of fine fraction passing 200 sieve. The mixture, in proportion of 90 to 95 % soil and 10 to 5% cement is thoroughly mixed dry and is then added with water to raise the moisture content to optimum. It is, therefore, placed on the subgrade and properly compacted. After laying in a reach is complete, it is covered with wet sand or wet straw for seven days for curing.

In U.S.A. plastic soil cement has been used with a higher consistency to get a faster rate of construction.

8. **Clay puddle lining** : Clay puddle is produced from clay by first exposing clay to weathering. It is then mixed with water to bring it to the saturation and is pugged thoroughly by trampling under man's or cattle's feet. The thickness of lining is 30 cm. It is then protected by a layer of earth material.

9. **Sodium carbonate lining** : The mixture consists of clayey soil and sodium carbonate in a proportion of at least 10% clay and 6% sodium carbonate. Thickness of lining is kept as 10 cm. This type of lining is used on small canal and water coarse and not found to be durable.

10. **Stone block lining** : This consists of undressed stone block set in mortar and laid over a prepared subgrade. The lining is able to check seepage effectively but

has a considerable resistance to flow of water. Dressed stone block lining is effective but costly.

11. **Pre-fabricated light weight membrane** : They are matted fibres of asbestos or jute and is coated with asphalt. It is laid on a smooth and prepared subgrade, and is covered with a layer of earth material.

12. **Bentonite and clay membrane** : This consists of bentonite or clay blanket 4 cm thick laid over a prepared subgrade, and covered with earth.

13. **Road oil lining** : The road oil sprinkled on subgrade in a thickness of about 1.5 mm is sufficient enough to saturate subgrade to a depth of 8 cm. The subgrade is then rolled so that oil enters the soil pores.

16.13. DESIGN OF LINED CANAL

Fig. 16.6 shows two types of lined canal sections : (*i*) triangular shaped or curved channel and (*ii*) trapezoidal shaped or flat bottom shaped channel. The side slopes are so selected that they are nearly equal to the angle of repose of the soil so that no earth pressure is imposed on the lining. The corners are rounded off to improve the hydraulic efficiency. Triangular shaped section is adopted for small discharges, as it is the best discharging section. For higher discharges, trapezoidal section may be adopted. If the limiting velocity is taken as 1.8 m/sec, then circular section can be adopted only if the discharge is less than 85 cumecs. However, Indian Standard recommends only trapezoidal section *for all discharges.*

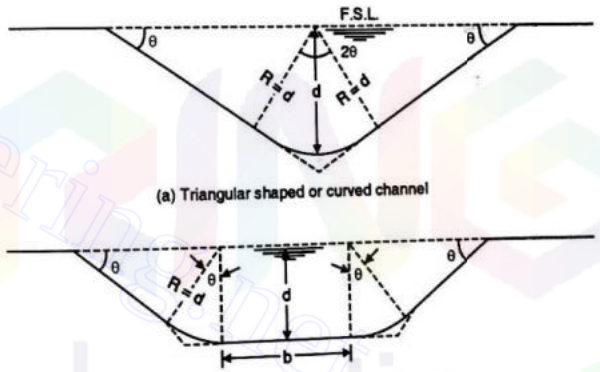

FIG. 16.6. SECTION OF LINED CHANNEL.

In order to design a lined canal section following data is required: (*i*) discharge Q, (*ii*) rugosity coefficient N, (*iii*) longitudinal slope S, (*iv*) side slopes, and (*v*) limiting velocity V.

The procedure used for design of trapezoidal shaped channel is given below

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1. Knowing limiting velocity V, the rugosity coefficient N and longitudinal slope, S, work out the value of hydraulic mean depth R by the use of Manning's equation:

$$V = \frac{1}{N} R^{2/3} S^{1/2} \qquad ...(16.6)$$

2. Find the area of cross-section from the continuity equation :

$$A = \frac{Q}{V}$$

3. Find out the wetted perimeter of the section

$$P = \frac{A}{R}$$

4. Also, for trapezoidal shaped channel (Fig. 16.6 *b*), the area of cross-section

$$A = bd + \pi d^2 \frac{\theta}{\pi} + \frac{2d^2 \cot\theta}{2} = bd + d^2\theta + d^2 \cot\theta$$

or $$A = bd + d^2(\theta + \cot\theta) \qquad ...(16.7)$$

Perimeter $$P = b + 2d\theta + 2d\cot\theta = b + 2d(\theta + \cot\theta) \qquad ...(16.8)$$

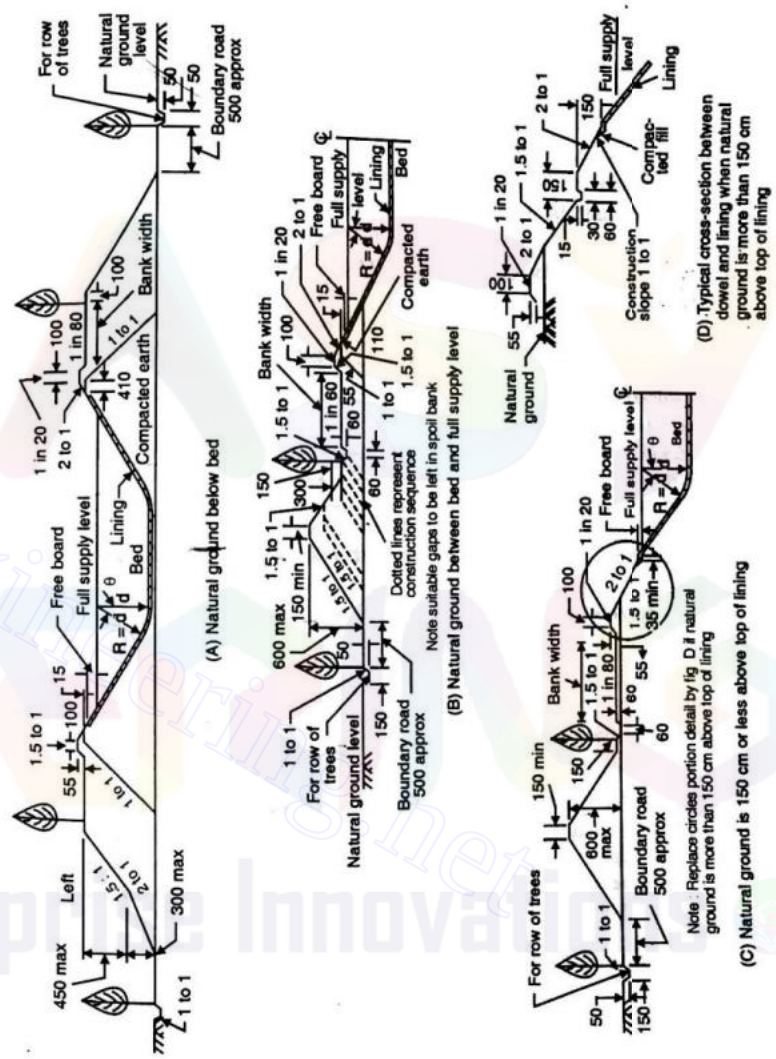
Since A and P are known from steps (3) and (4), the value of bed width and depth d can be worked out from equations 16.7 and 16.8. The values of N may be taken from Table 16.1. See Example 16.1 for illustration.

TABLE 16.1. VALUES OF RUGOSITY COEFFICIENT *N* FOR LINED CHANNELS WITH STRAIGHT ALIGNMENT (IS : 4745-1968)

S. No.	Surface characteristics	Value of N
1.	***Concrete with surface as indicated below*** :	
	(*a*) Formed, no finish	0.013–0.017
	(*b*) Trovel finish	0.012–0.014
	(*c*) Float finish	0.013–0.015
	(*d*) Float finish, some gravel on bottom	0.015–0.017
	(*e*) Gunite, good section	0.016–0.019
	(*f*) Gunite, wavy section	0.018–0.022
2.	***Concrete bottom float finished sides as indicated below*** :	
	(*a*) Dressed stone in mortar	0.015–0.017
	(*b*) Random stone in mortar	0.017–0.020
	(*c*) Cement rubble masonry	0.020–0.025
	(*d*) Cement rubble masonry plastered	0.016–0.020
	(*e*) Dry rubble (riprap)	0.020–0.030
3.	***Gravel bottom sides as indicated below*** :	
	(*a*) Formed concrete	0.017–0.020
	(*b*) Random stone in mortar	0.020–0.023
	(*c*) Dry rubble (riprap)	0.023–0.033
4.	***Brick***	0.014—0.017
5.	***Asphalt***	
	(*a*) Smooth	0.013
	(*b*) Rough	0.016
6.	***Wood planed clean***	0.011–0.013
7.	***Concrete lined excavated rock with.:***	
	(*a*) Good section	0.017–0.020
	(*b*) Irregular section	0.020–0.027

16.14. INDIAN STANDARD ON DESIGN OF LINED CANALS (IS : 4745-1968)

Fig. 16.7 shows the details of cross-section of lined canal according to IS : 4745-1968.



(A) Natural ground below bed

(B) Natural ground between bed and full supply level

(C) Natural ground is 150 cm or less above top of lining

(D) Typical cross-section between dowel and lining when natural ground is more than 150 cm above top of lining

FIG. 16.7. TYPICAL CROSS-SECTION OF LINED CANNAL (IS : 4745-1968) (ALL DIMENSIONS AREA IN CENTIMETRES

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1. *Side slopes.* Lining is usually made to rest on slopes corresponding to the angle of repose of the natural soil so the slopes shall be such that no earth pressure is exerted on the back of the lining.

2. *Free board.* A free board of 0.75 m shall be provided. Free board shall be measured from the full supply level to the top of lining.

3. *Bank widths.* The following bank widths shall be provided:

(*a*) 8 m for cutting and filling reaches for main canals,

(*b*) 6.5 m for cutting reaches for branches canals, and

(*c*) 6.5 m on left side and 5 m on the right side for filling reaches for branch canals

4. *Dowel.* This shall be provided in accordance with Fig. 16.7.

5. *Roadways and banks.* These shall be provided in accordance with Fig. 16.7.

6. *Velocity.* Allowable velocity of about 1.5 to 2.0 m/sec is recommended for lined canal.

Table 16.2 gives the values of *limiting velocities* in different kinds of linings.

TABLE 16.2. LIMITING VELOCITIES IN DIFFERENT TYPES OF LININGS

Type of Lining	*Permissible Velocity (m/sec)*
1. Cement concrete lining (un-reinforced)	2.0 to 2.5
2. Burnt clay tile and brick lining	1.6 to 1.8
3. Asphatic concrete	1.4 to 1.6
4. Flag stone lining	1.5 to 1.7

Critical velocity ratios are not applicable to lined canals but the possibility of silting cannot be neglected and, hence, the critical velocity ratio should be aimed at higher than unity.

The velocity may be computed from the following formula :

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

where V = mean velocity

R = hydraulic mean depth

S = slope of the canal, and

N = Rugosity coefficient as given in Table 16.1

Example 16.1. *Design a trapezoidal shaped concrete lined channel to carry a discharge of 100 cumecs at a slope of 25 cm/km. The side slopes of the channel are 1.5 : 1. The value of N may be taken as 0.016. Assume the limiting velocity as 1.5 m/sec.*

Solution. Use Manning's equation

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

where $S = \frac{25}{1000 \times 100} = \frac{1}{4000}$

$V = 1.5$ m/sec

$N = 0.016$

$$\therefore \quad 1.5 = \frac{1}{0.016} R^{2/3} \frac{1}{\sqrt{4000}}$$

or $R = 1.87$ m.

The channel section is assumed to be trapezoidal as shown in Fig. 16.6(*b*).

For side slope $1\frac{1}{2} : 1$, $\cot\theta = 3/2 = 1.5$ and $\theta = 0.59$ radian.

Area of cross section of the channel

$$A = \frac{Q}{V} = \frac{100}{1.5} = 66.67 \text{ m}^2$$

Perimeter of the section

$$P = \frac{A}{R} = \frac{66.67}{1.87} = 35.65 \text{ m.}$$

From Eq. 16.7 $\quad 66.67 = bd + d^2(\theta + \cot\theta)$

$35.65 = b + 2d\,(\theta + \cot\theta)$

Substituting for θ and $\cot\theta$

$$66.67 = bd + d^2\,(2.09) \quad ...(a)$$

$$35.65 = b + d\,(4.18) \quad ...(b)$$

From Eq.(*b*)

$$b = 35.65 - 4.18\,d$$

Substituting it in equation (*a*)

$$d\,(35.65 - 4.18\,d) + 2.09\,d^2 = 66.67$$

$$d^2 - 17.1\,d + 31.9 = 0$$

$$\therefore \quad d = 2.145 \text{ m}$$

and $b = 35.65 - 4.18\,(2.145) = 26.68$ m.

Thus required bed width = **26.68** m and depth ≙ **2.15** m.

Example 16.2. *Solve example 16.1 when the limiting value of velocity is not given, but b/d = 8.*

Solution. From Eq. 16.7, substituting the values of θ and $\cot\theta$,

$$A = bd + d^2\,(\theta + \cot\theta)$$

or $A = bd + 2.09\,d^2$

and $P = b + 4.18\,d.$

Therefore, velocity in the channel

$$V = Q/A = \frac{100}{bd + 2.09\,d^2} \quad ...(1)$$

Hydraulic mean depth

$$R = \frac{A}{P} = \frac{bd + 2.09\,d^2}{b + 4.18\,d}$$

Using Manning's equation

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$$V = \frac{1}{N} R^{2/3} S^{1/2} = \frac{1}{0.016} R^{2/3} \frac{1}{\sqrt{4000}} = \frac{R^{2/3}}{1.01} \quad ...(2)$$

Hence $R^{2/3} = 1.01\ V$

Substituting the values of R and V, we have

$$\left[\frac{bd + 2.09\,d^2}{b + 4.18d}\right]^{2/3} = 1.01\left[\frac{100}{bd + 2.09d^2}\right]$$

This is a equation between b and d. But $b = 8\,d$

$$\therefore \quad \left[\frac{8\,d^2 + 2.09\,d^2}{8\,d + 4.18\,d}\right]^{2/3} = 1.01\left[\frac{100}{8\,d^2 + 2.09d^2}\right]$$

or $$\left[\frac{10.09}{12.18}d\right]^{2/3} = 1.01\left[\frac{100}{10.09\,d^2}\right]$$

or $$d^{8/3} = \frac{101}{10.09}\left[\frac{12.18}{10.09}\right]^{2/3} = \frac{101 \times 1.134}{10.09} = 11.348$$

$$\therefore \quad d = (11.348)^{0.375} = 2.487 \simeq 2.49 \text{ m}$$

$$\therefore \quad b = 8\,d = 8 \times 2.487 = 19.89 \text{ m}$$

Therefore, required bed width $= 19.89$ m $\simeq$ **20 m**

and depth $= 2.49$ m $\simeq$ **2.5 m**

Example 16.3. *Design a concrete lined channel to carry a discharge of 45 cumecs at a slope of 1 in 10,000. The side of the channel are $1\frac{1}{4}:1$ and Manning's N may be taken as 0.018.*

Solution. The channel is assumed to be of triangular section as shown in Fig. 16.6 (*a*)

Area $$A = \pi d^2 \frac{\theta}{\pi} + 2\frac{d^2}{2}\cot\theta = d^2(\theta + \cot\theta)$$

where $d =$ depth of central portion

Perimeter $$P = 2\pi d\,\frac{\theta}{\pi} + 2d\cot\theta = 2d(\theta + \cot\theta)$$

Hydraulic mean depth,

$$R = \frac{d^2(\theta + \cot\theta)}{2d(\theta + \cot\theta)} = d/2.$$

From Manning's equation

$$V = \frac{1}{N} R^{2/3} S^{1/2} = \frac{1}{0.018}\left(\frac{d}{2}\right)^{2/3} \frac{1}{\sqrt{10000}} = \frac{d^{2/3}}{2.86} \quad ...(a)$$

Also $$V = \frac{Q}{A} = \frac{45}{d^2(\theta + \cot\theta)}$$

where $\cot\theta = 1.25$, since side slope is $1.25:1$

$\therefore \quad \theta = 0.675$ radian

Hence, $$V = \frac{45}{d^2(0.675 + 1.25)} = \frac{45}{d^2(1.925)} \quad ...(b)$$

Equating (*a*) and (*b*) we get

$$\frac{d^{2/3}}{2.86} = \frac{45}{d^2(1.925)}$$

Hence $$d^{8/3} = \frac{45 \times 2.86}{1.925} = 66.8$$

From which $d =$ **4.83 m**

Example 16.4. *Design a concrete lined channel of trapezoidal section to carry a discharge of 250 cumecs at a slope of 1 in 6000. The side slopes of the channel are to be made as $1\frac{1}{2}:1$ and a limiting depth of 3 m is to be maintained. Take N for the lining material as 0.015.*

Solution : For a channel of trapezoidal section, we have

$$A = bd + d^2(\theta + \cot\theta)$$

and $$P = b + 2\,d(\theta + \cot\theta)$$

For $1\frac{1}{2}:1$ side slope, $\cot\theta = 1.5$ and $\theta = 0.59$ radian.

Taking the above values along with $d = 3$ m, we get

$$\therefore \quad A = bd + d^2(0.59 + 1.5) = 3\,b + 9\,(2.09) = 3b + 18.81 \quad ...(1)$$

and $$P = b + 2\,d\,(0.59 + 1.5) = b + 6\,(2.09) = b + 12.54 \quad ...(2)$$

Hence $$R = \frac{A}{P} = \frac{3\,b + 18.81}{b + 12.54} \quad ...(3)$$

Now, $$Q = V \times A = \frac{1}{N} A\,R^{2/3} S^{1/2}$$

or $$250 = \frac{1}{0.015}(3\,b + 18.81) \times \left(\frac{3\,b + 18.81}{b + 12.54}\right)^{2/3} \frac{1}{\sqrt{6000}}$$

or $$\frac{(3\,b + 18.81)^{5/3}}{(b + 12.54)^{2/3}} = 290.47 \quad ...(4)$$

Solving the above equation by trial and error, we get

$b \simeq$ **44.6 m**

Hence $b =$ **44.6 m** and $d =$ **3 m.**

Example 16.5. *The annual rainfall in a certain area is 80 cm. Find the spacing of drains if one percent of average annual rainfall is to be drained in 24 hours.*

Given : depth of impervious stratum from top of soil surface = 10 metres. Position of drains is 1.8 m below top soil surface and the depth of highest position of water table below top soil surface = 1.5 m. Take permeability $k = 1 \times 10^{-4}$ m/sec.

Solution: Let the spacing of the drains be L metres

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One percent of average rainfall in an area equal to $L \times 1$ is

$$= \frac{0.80 \times L \times 1}{100} \text{ cubic metres per metre length}$$

This is to be removed in 24 hours.

Hence $$Q_D = \frac{0.80\,L}{100 \times 24 \times 3600} \text{ cumecs}$$

$$b = 10 - 1.5 = 8.5 \text{ m} \quad \text{and} \quad a = 10 - 1.8 \text{ m} = 8.2 \text{ m}$$

Now $$L = \frac{4\,k\,(b^2 - a^2)}{Q_D} = \frac{4 \times 1 \times 10^{-4}\,[(8.5)^2 - (8.2)^2]}{Q_D}$$

or $$L = \frac{4 \times 10^{-4}\,[(8.5)^2 - (8.2)^2] \times 100 \times 24 \times 3600}{0.80\,L} = \frac{21643.2}{L}$$

From which $L = 147.1$ m.

Hence, *drains should be spaced at* **147 m c/c.**

Example 16.6. Given $L = 30$ m ; $Q_D = 4 \times 10^{-6}$ *cumecs* $a = 8$ m *and* $b = 8.3$ *m. Find k.*

Solution

Substituting these values in Eq. 16.5 :

$$L = 30 = \frac{4\,k\,[(8.3)^2 - (8)^2]}{4 \times 10^{-6}}$$

Hence $$k = \frac{30 \times 4 \times 10^{-6}}{4 \times [(8.3)^2 - (8)^2]} = \mathbf{6.13 \times 10^{-6}\ m/sec}$$

Example 16.7. *Given* $L = 50$ *m,* $a = 10$ *m,* $b = 10.3$ *m and* $k = 1 \times 10^{-5}$ *m/sec. If the drains carry 1% of average annual rainfall in 24 hours, find the average annual rainfall.*

Solution

$$Q_D = \frac{4 \times 1 \times 10^{-5}\,[(10.3)^2 - (10)^2]}{50} = 4.872 \times 10^{-6} \text{ cumecs.} \qquad ...(1)$$

If P_{av} is the average annual rainfall then one percent of average rainfall $= \frac{P_{av}}{100}$

Since discharge Q_D is for 1 m length assumed, area being served by one drain $= L \times 1$

$$Q_D = \frac{P_{av} \times L \times 1}{24 \times 3600 \times 100} = \frac{50\,P_{av}}{24 \times 3600 \times 100} \qquad ...(2)$$

Hence $$4.872 \times 10^{-6} = \frac{50\,P_{av}}{24 \times 3600 \times 100}$$

From which $P_{av} = 0.84$ m = **84 cm**

Example 16.8. *There are two different areas A and B to be drained by means of closed tile drainage system.*

If $\frac{k_A}{k_B} = \frac{2}{1}$; $\frac{L_A}{L_B} = \frac{1}{1.5}$ *and* $\frac{(b^2 - a^2)_A}{(b^2 - a^2)_B} = \frac{5}{6}$

find the ratio $\frac{(Q_D)_A}{(Q_D)_B}$. *If the drains carry 1% of average annual rainfall in 24 hours in both the cases, find the ratio* $\frac{P_A}{P_B}$.

Solution

As per equation 16.5, $$Q_D = \frac{4\,k\,(b^2 - a^2)}{L}$$

$$\frac{(Q_D)_A}{(Q_D)_B} = \frac{4\,k_A\,(b^2 - a^2)_A\,L_B}{4\,k_B\,(b^2 - a^2)_B\,L_A} = \frac{2}{1} \times \frac{5}{6} \times 1.5 = \mathbf{2.5}$$

If 1% of average annual rainfall is carried by the drain in 24 hours then

$$Q_D = \frac{L \times 1 \times P}{24 \times 60 \times 60 \times 100}$$

$$\frac{(Q_D)_A}{(Q_D)_B} = \frac{L_A \times P_A}{L_B \times P_B}$$

$$\frac{P_A}{P_B} = \left(\frac{L_B}{L_A}\right)\left\{\frac{(Q_D)_A}{(Q_D)_B}\right\} = 1.5 \times 2.5 = \mathbf{3.75}$$

16.15. ECONOMICS OF CANAL LINING

At times, the choice of canal lining is to be done on financial considerations besides the technical feasibility. For determining its economic viability, an analysis of benefits accrued from canal lining has to be worked out in terms of money. The benefits accrued from lining should be greater than extra cost to be incurred on lining.

Calculation of benefit

The major benefit then can be readily assessed in terms of money is from the saving of seepage water which would have been lost from unlined channel. This water when supplied to farmers will yield revenue.

Let q cumecs be the water saved in the lined reach and R_1 rupees be the cost of water per cumec. Then the total saving $= q \,.\, R_1$. rupees.

The other benefit is through a considerable reduction in the maintenance cost of a lined channel, compared to that of an unlined channel. Let p be the percentage saving in annual maintenance cost which is rupees R_2 for unlined channel. Then the saving in annual maintenance cost $= p\,R_2$ rupees.

Thus total benefit (B) per year $= (q\,R_1 + p\,R_2)$ rupees. Saving in maintenance cost is usually taken to be around 40% so that $p = 0.4$

Besides the above benefits, there will be a considerable indirect benefits accrued from the prevention of water logging caused due to excess seepage, but as these are difficult to assess in terms of money, the *minimum* annual benefit (B) is considered as $B = (q\,R_1 + p\,R_2)$

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Annual cost of extra expenditure on lining

Let the extra expenditure on canal lining be C rupees. If P_t is the total perimeter of lining, L is length of lining and c is the cost of lining in Rupees per m^2, $C = c\,P_t\,.\,L$. The extra expenditure is to be calculated by subtracting the saving in land cost, cost of drainage works, cost of bridges and earth work etc. from the total cost of lining.

Let the life of lining be N years and i be the percentage rate of interest expressed as fraction per year. Then the capital recovery factor or annual cost of extra expenditure on lining is equal to :

$$= C \times i\,(i+1)^N\,[(1+i)^N - 1]^{-1} \qquad ...(16.9)$$

Therefore *benefit cost ratio* (B.C.R.) is given by

$$\text{B.C.R.} = \frac{[q\,R_1 + p\,R_2]\,[(1+i)^N - 1]}{C \times i\,(1+i)^N} = \frac{B\,[(1+i)^N - 1]}{Ci\,(1+i)^N} \qquad ...(16.10)$$

Note : If rate of interest is 8% per annum, $i = 0.08$.

For the project justification, the benefit cost ratio should be greater than 1. See Example 16.4 for illustration.

16.16. DRAINAGE AND PRESSURE RELEASE ARRANGEMENTS BEHIND CANAL LINING

The thickness of canal lining is usually very small, and hence the lining can not withstand any water pressure from the back of the lining. The water pressure or *hydrostatic pressure* may be caused by seeping rainwater in the backfill, even when the water table in the area remains below the canal bed. Continuous rain water infiltration behind the lining will make the backfill saturated. In the case of drawdown in the canal, the water in the backfill gives rise to build up of hydrostatic pressure behind the lining. Hence it is necessary to relieve this hydrostatic pressure by the provision of drainage and pressure release arrangements. This can be done by the following methods:

(*i*) Provision of pressure release valves embedded in drainage relief pockets

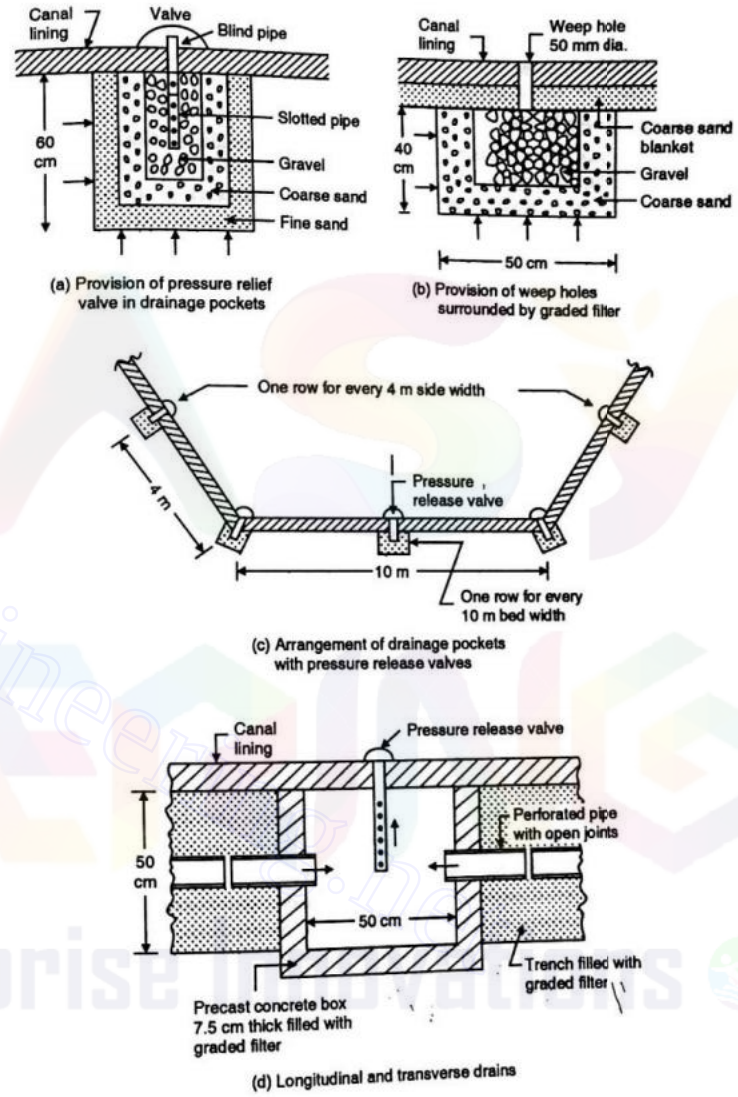
(*ii*) Provision of weep holes in the bed and sides of the lined canal.

(*iii*) Provision of longitudinal and transverse drains below lining.

1. Provision of pressure release valves

A pressure relief/release valve is a device like a check valve which allows the water from the subgrade to flow into the canal but prevents entry of canal water into the subgrade. Thus, it is a *one way valve* or a flap valve opening upwards into the canal. These valves open out as soon as the differential pressure becomes about 10 cm. The diameter of the valve varies from 50 mm (used on side slopes) to 150 mm (used generally is the bed). These valves are made of abrasion resistant material, generally of cast iron or heavy duty plastic. Non-metallic valves give better performance than metallic valves.

Pressure release valves are provided in small *drainage relief pockets* of size 60 cm × 60 cm filled with graded filter beneath the lining, as shown in Fig. 16.8 (*a*). The *graded filter* should be designed in such a way that there is no flow of bed soil through the valve. The drainage relief pockets are cubical pockets provided at isolated



(a) Provision of pressure relief valve in drainage pockets

(b) Provision of weep holes surrounded by graded filter

(c) Arrangement of drainage pockets with pressure release valves

(d) Longitudinal and transverse drains

FIG. 16.8. PRESSURE RELIEF ARRANGEMENTS IN LINED CANAL.

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locations in the bed as well as sides of the lined canal, below the canal lining, at suitable spacing (Fig. 16.8 *c*). These are provided at 15 to 20 m intervals, along rows in such a way that at least one row is provided in the bed for every 10 m bed width, and one row on the sides for every 4 m side width.

2. Provision of weep holes

Weep holes are small holes (about 50 mm dia.) made in the lining in the bed as well as sides. The bottom of the weep hole open out in small pocket etc. containing inverted graded filter consisting of two or three materials (Fig. 16.8 *b*). These holes are suitably spaced in the bed and sides of lined canal.

3. Provision of longitudinal and transverse drains

A longitudinal drain consists of a continuous perforated open jointed pipe encased in a graded filter, having outlets in a masonry or precast concrete box of 50 cm × 50 cm size filled with graded filter, as shown in Fig. 16.8 (*d*). The box has an outlet into the lined canal through a pressure relief valve. Transverse drains, having cross-section similar to that of a longitudinal drain, are provided at a suitable interval in the bed as well as sides.

16.17. SALT PROBLEMS IN IRRIGATED SOILS

The gradual accumulation of salts on the ground or in the root zone of crop leads to saline and alkaline conditions. This results in loss of permeability, toxicity and anaerobic conditions. Plants fail to derive their nutrients at the required rate owing to the high soil moisture tension. This causes unhealthy growth of plants resulting in loss of yield. The irrigated soil may become saline or alkaline because of the following reasons :

1. Rise in water table

A rise in water table with a high evaporation results in build up of dissolved salts. High evaporation and transpiration rates with poor drainage of soil is the major cause of salinity of soil.

2. Climatic conditions

In the region of heavy rainfall, leaching of soil takes place due to heavy infiltration. In arid and semi-arid climates, rainfall is small hence the leaching is limited and the soil is likely to become saline and alkaline.

3. Quality of irrigation water used

If the quantum of dissolved salts in irrigation water exceeds a certain limit, the excess salts accumulate in the root zone and create salt problem. Sodium ions and chlorides are very harmful compared to sulphates and bicarbonates.

4. Nature of soil and under-irrigation

Heavy soils like clays have poor permeability due to which leaching is extremely slow and hence, are more prone to salinity and alkalinity compared to sandy soils. Brackish water can be used for sandy soils (light soils) if enough water can be provided to satisfy irrigation and leaching requirements. Under-irrigation by sprinkler may result in salinity due to non availability of leaching water.

5. Sea water intrusion

In coastal areas, sea water intrudes below the ground water. If ground water is extracted from below the interface, saline water comes out which can make the land saline or alkaline if used as a source of irrigation.

Leaching Requirement

When soil or the irrigation water is saline, the accumulated salt should be leached out by irrigating it with more water. The fraction of irrigation water that must be leached through the root zone to keep the salinity of soil below a specific limit is termed as *Leaching Requirement* L_R. It is usually expressed as a ratio :

$$L_R = \frac{\text{Depth of drainage water } D_d \text{ per unit area}}{\text{Depth of irrigation water } D_i \text{ per unit area}}$$

From the consideration of salt balance

$$\frac{D_d}{D_i} = \frac{C_i}{C_d} = \frac{E_{ci}}{E_{cd}} \text{ and hence } L_R = \frac{D_d}{D_i} = \frac{C_i}{C_d} = \frac{E_{ci}}{E_{cd}} \qquad ...(16.11)$$

where C_i and C_d are the salt concentration in irrigation and drainage water respectively and E_{ci}, E_{cd} are corresponding electrical conductivity.

The depth of irrigation water D_i to be applied consists of (*i*) consumptive requirement D_c (= C_u) and (*ii*) Leaching requirement D_d.

$$D_i = D_c + D_d = D_c + L_R D_i$$

Hence
$$D_i = \frac{D_c}{1 - L_R} = \frac{D_c \cdot E_{cd}}{E_{cd} - E_{ci}} \qquad ...(16.12)$$

The electrical conductivity of drainage water (E_{cd}) may be assumed on the basis of permissible salt tolerance limit of the crop to be grown. Generally, E_{cd} is assumed to be equal to $2\,E_{ce}$, where E_{ce} is the electrical conductivity of *saturated soil extract* (i.e. water solution extracted from the soil at its saturation percentage). Hence Eq. 16.11 reduces to

$$L_R = \frac{E_{ci}}{2\,E_{ce}} \qquad ...(16.11a)$$

Leaching water can be added with irrigation water or can be applied separately. When the intake of soil is low, i.e. for heavy soil, a better practice is to apply small quantities more frequently. Leaching before irrigation season is often more beneficial. After leaching, the deeper layers are at field capacity and, hence, seeding can be done without additional water needed for germination.

Reclamation of salt affected land

The reclamation of salt affected land is costly and should be resorted to only if it is very necessary. Various measures adopted for reclamation depends upon the reasons causing the salinity or alkalinity. Some of these measures are discussed below.

(*i*) ***Anti water logging measures***

Surface drainage, sub-surface drainage, pumping and recycling, controlling of seepage loss etc. are some of the methods that has already been discussed earlier. As water logging conditions reduce, the salt affected land also improves and becomes culturable.

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(ii) *Use of proper quality of water for irrigation*

The quality of water fit for irrigation has been discussed in Chapter 3. If the salinity or alkalinity of land has been caused due to the continued use of poor quality water, its replacement with proper quality of water improves the land condition.

(iii) *Leaching of soil salts*

In order to regain the salt affected land, the salt has to be leached out by supplying additional quantity of water. Leaching water can be added along with irrigation water where there is no limit on supply.

If the intake rate of soil is not sufficient to absorb extra dose, it is advisable to irrigate more frequently with smaller depths in each application rate.

If limited supply is available during the crop seasons, leaching should be practiced during off season periods usually by flooding several times. The drying of beds between consecutive leaching applications stimulates root growth, aids aggregation and permits formation of surface cracks which helps in better infiltration during subsequent flooding.

For reclamation of land covered with encrusted salt layers, alternate flooding and surface drainage prior to leaching can remove appreciable quantities of soluble salts and increase efficiency of leaching carried out subsequently. Leaching is good for saline soils. On alkaline soils, the use of certain chemicals with subsequent leaching is more useful.

(iv) *Choice of crops*

Proper choice of salt tolerant crops, crop rotation etc. increases fertility and reclamation of salt affected land. Various salt tolerant crops are : barley, sugar beet tobacco, turnips, mustard, cotton, sugarcane and some varieties of grass. Research scientists have also evolved salt tolerant varieties of rice, wheat, bajra etc. The acidic nature of cell saps in the roots of these crops neutralize the alkalinity of the soil. The movement of roots through the soil increases its permeability. This results in aerobic conditions and nitrification of soil owing to the enhanced bacterial activity.

(v) *Chemical Methods*

The method is based on the principle that the addition of some salts or acids can neutralize harmful salts. Sulphur, sulphuric acid, gypsum, iron sulphate and lime stone are some of the common chemicals used for leaching operations. Use of Gypsum ($CaSO_4 . 2H_2O$) is popular, because of its low cost and cheap availability. Chemical reaction which takes place in this process is

$$2\,Na\,Y + Ca\,SO_4 = Ca\,Y_2 + Na_2\,SO_4$$

where Y represents the exchangeable constitutent. Owing to the replacements of Na ion by Ca ion, soil regains its normal character making it permeable, fiberous, aerated and fit for agriculture. The quantity depends upon the quantity of exchangeable sodium. It is generally assessed that 4.1 tonnes of gypsum/hectare of land for 33 cm depth of soil is needed for reclamation. Powdered gypsum is thoroughly mixed with soil and the field levelled and flooded with water so that exchange between Ca & Na takes place and the dissolved Na ion can be leached down by percolating water.

If soil condition is good and irrigation water is saline, gypsum is not added to the soil ; instead gypsum solution is added to irrigation water before it is applied to the field. Addition of chemical fertilizers, usually nitrogen and organic matter along with chemical treatment is necessary for early reclamation and better fertility.

Example 16.9. *An unlined channel in alluvial soil has a seepage loss of 2.5 cumecs/million sq.m of wetted perimeter. The channel has a wetted perimeter of 25 m and has maintenance cost of Rs. 1 per sq.m of wetted perimeter. There is a huge scarcity of water in the area and as such the canal is to be lined with cement concrete lining 12 mm thick so as to reduce seepage loss to 0.02 cumecs/million sq.m of wetted perimeter. The lined channel will have a wetted perimeter of 20 m. The extra cost of lining works out to be Rs. 100/-per sq.m. If the average annual revenue per cumec of water is Rs. 25 lakhs and percentage reduction in annual maintenance cost is 40%, decide whether it is economically feasible to provide canal lining. Assume the life of canal lining as 50 years and the interest rate is 6 per cent per annum.*

Solution

For the purpose of calculations assume 1 km length of channel.

(a) Annual benefit

(i) *Seepage.* Area of wetted perimeter for unlined channel in 1 km

$$= 25 \times 1000 = 25000 \text{ sq. m}$$

$$\text{Seepage loss} = 25000 \times 2.5 \times 10^{-6} = 0.0625 \text{ cumec;}$$

Area of wetted perimeter for lined channel in 1 km $= 20 \times 1000 = 20{,}000$ sq. m.

$$\text{Seepage loss} = 20000 \times 0.02 \times 10^{-6} = 0.0004 \text{ cumecs.}$$

So, saving in water loss $= 0.0625 - 0.0004 = 0.0621$ cumec.

$$\text{Annual revenue saved} = \text{Rs. } 0.0621 \times 25 \times 10^5$$

$$= \text{Rs. } 155250.$$

(ii) *Maintenance.* Annual maintenance cost of unlined channel.

$$= 1 \times 25000 = \text{Rs. } 25000.$$

Saving in maintenance cost $= 0.4 \times 25000 = \text{Rs. } 10000$

Total annual benefit $= \text{Rs. } 155250 + 10000 = \text{Rs. } 165250.$

(b) Annual cost

Area of lining per km $= 20.0 \times 1000 = 20000$ sq.m.

Cost of lining $C = 100 \times 20000 = 2000000$

Interest rate $= 6\%$ per annuam. Hence $i = 0.06$. Also, $N = 50$.

Annual cost of lining or capital recovery factor

$$= \left[\frac{C \times i\,(i+1)^N}{(1+i)^N - 1}\right] = \frac{2000000 \times 0.06 \times (1.06)^{50}}{(1.06)^{50} - 1}$$

$$= \text{Rs. } 126888$$

$$\text{Benefit cost ratio} = \frac{165250}{126888} = 1.302$$

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