A distributary canal takes off from a branch canal having CBL at 204.0 m and FSL at 205.8 m. The gross commanded area at the head of the distributary is 30,000 hectares, and after each km it is reduced by 5,000 hectares. Out of this command, the culturable area is only 75%. The intensity of irrigation for the Rabi and Kharif season is 32% and 15% resp. Design suitable channel sections for the first 5 Km of this distributary, assuming the following data:

(i) Total losses below 3 km = 0.44 cumec
(ii) Channel losses occur @ 2 cumecs/ million m^2 of wetted perimeter.
(iii) Kor period for Rabi (wheat) = 4 weeks
(iv) Kor depth for Rabi = 14 cm
(v) Kor Period for Kharif (Rice) = 2.5 weeks
(vi) Kor depth for Kharif = 20 cm
(vii) Manning's n = 0.0225
(viii) Critical velocity ratio = 0.95

The ground levels at every 200 mts, along the line of proposed alignment, have been obtained and are tabulated below.

Solution.

The channel is to be designed from its tail (where the losses are known) towards its head, km by km.

The G.C.As and C.C.As at various km are, first of all, worked out in following table

Table 1

Below km	Gross Commanded area (in hectares)	Culturable Commanded area (in hectares)
0 (i.e. head)	30,000	22,500
1	25,000	18,750
2	20,000	15,000
3	15,000	11,250

Outlet discharges for the two crop seasons are determined as given below:

(i) For Rabi, $D = \frac{8.64 B}{\Delta}$, where $B = 4$ weeks $= 28$ days, $\Delta = 14$ cm $= 0.14$ m

$$D = \frac{8.64 \times 28}{0.14} = 1728 \text{ ha/cumec}$$

(ii) For Kharif, $D = \frac{8.64 B}{\Delta}$, where $B = 2.5$ weeks $= 2.5 \times 7$ days $= 17.5$ days, $\Delta = 20$ cm $= 0.20$ m

$$D = \frac{8.64 \times 17.5}{0.20} = 756 \text{ ha/cumec}$$

Intensity of irrigation for Rabi = 32%

& Intensity of irrigation for Kharif = 15%

If G is the gross culturable area at any point, then 0.32G is the Rabi area and 0.15G is the Kharif area

Discharge required for this Rabi area $= \frac{0.32\,G}{\text{Outlet factor for Rabi}}$

$$= \frac{0.32\,G}{1728} = \frac{G}{5400}$$

Similarly, discharge required for Kharif area $= \frac{0.15\,G}{756} = \frac{G}{5040}$

Since the discharge required for Kharif crop is more than that required for Rabi crop, the outlet factor of Kharif crop becomes the controlling factor.

Discharge needed at various kms for the given command are worked in the following Table

Table 2

Below Km (1)	Culturable commanded area (from previous table) (2)	Discharge required for Kharif crop (in cumecs) $= \frac{\text{Col.}(2)}{5040}$
0	22,500	4.46
1	18,750	3.72
2	15,000	2.98
3	11,250	2.23

(i) Design at km 3

Losses below km 3 = 0.44 cumec (given)

Discharge required for this crop at this point (Table 2) = 2.23 cumecs

Total discharge required = 2.23 + 0.44 = 2.67 cumecs

Design discharge = 10% more than required
= 1.1 × 2.67 = 2.937 cumecs = 2.94 cumecs (say)

$$C.V.R = \frac{V}{V_0} = 0.95$$

$$n = 0.0225$$

Lacey's regime slope for this discharge and silt factor = 1, is approximately 22 cm per km.

Let us keep the slope @ 22.5 cm/km.

$$\therefore \quad S = \frac{0.225}{1000} = \frac{1}{4444}$$

From Garret's diagram [Plate 4.1 (a)], assuming $\frac{1}{2}:1$ side slopes, the channel section is designed as shown in the table below.

Table 3

Discharge (cumecs)	S	B (m)	y (m)	$A = \left(B + \frac{y}{2}\right)y$ (m²)	$V = \frac{Q}{A}$ (ms⁻¹)	V_0 (ms⁻¹)	$\frac{V}{V_0}$	Remarks
2.94	$\frac{1}{4444}$	5.0	0.98	5.38	0.55	0.53	1.04	much larger than 0.95
		4.5	1.05	5.28	0.56	0.58	0.96	OK

Hence, adopt $B = 4.5$ m

$y = 1.05$ m

$S = \frac{1}{4444}$ (i.e., 22.5 cm per km)

These dimensions, quite nearly satisfy the bed width – depth relationship, given by $y = 0.5\sqrt{B}$; and hence, the assumed slope is all right and can be adopted.

(ii) Design at km 2

Outlet discharge required below km 2, from Table 2 $= 2.98$ cumecs.

Losses below km 3 $= 0.44$ cumec.

Losses in channel between km 3 to km 2.

For the calculation of these, the perimeter of the section at km 3 shall be taken, as the section at km 2 is not known so far.

$$\text{Wetted perimeter} = B + \sqrt{5} \cdot y = 4.5 + \sqrt{5} \times 1.08 = 6.92 \text{ m}$$

$$\text{Loss @ 2 cumecs/million sq.m} = 2 \times \left[\frac{6.92 \times 1000}{10^6}\right] = 0.014 \text{ cumec}$$

Total Losses below km 2

$= $ Losses below km 3 + Losses between km 3 and km 2

$= 0.44 + 0.014 = 0.454$ cumec.

Total discharge required at km 2 $= 2.98 + 0.454 = 3.434$ cumecs

Design discharge $= 1.1 \times 3.434 = 3.7874$; say 3.79 cumecs

Use the same slope of 22.5 cm in 1 km i.e., $\frac{1}{4444}$.

Using Garret's diagrams [Plate 4.1 (b)], we design the channel section as shown in table below

Table 4

Q	S	B (m)	y (m)	$A = (B + \frac{y}{2})y$ (m^2)	$\frac{Q}{A} = V$ (ms^{-1})	V_0 (ms^{-1})	$\frac{V}{V_0}$	Remarks
3.79 cumecs	$\frac{1}{4444}$	6.0	1.05	6.85	0.55	0.58	0.95	OK

Hence, adopt $B = 6.0$ m

$y = 1.05$ m

$S = \frac{1}{4444}$

(iii) Design at km 1

Outlet discharge required below km 1, from Table 2 $= 3.72$ cumecs

Losses below km 2 worked out earlier $= 0.454$ cumec

Losses between km 2 and km 1: To work out these losses, the perimeter of the section at km 2 shall be taken, as the section at km 1 is not known so far.

$$\therefore \text{Wetted perimeter} = B + \sqrt{5}\, y = 6.0 + \sqrt{5} \times 1.08 = 6.0 + 2.42$$
$$= 8.42 \text{ m}$$

Losses @ 2 cumecs/million sq.m (i.e; in length of 1 km, i.e., 1000 m)

$$= 2 \times \left[\frac{8.42 \times 1000}{10^6}\right] = 0.017 \text{ cumec}$$

Total Losses below km 1 = 0·454 + 0·017 = 0·471 cumec

Total discharge required at km 1 = 3·72 + 0·471
= 4·191 cumecs

Design discharge = 1·1 × 4·191 = 4·61 cumecs

Let us adopt a slope of 20 cm in 1 km, i.e; $S = \frac{1}{5000}$

Using Garret's diagram [Plate 4·1 (c)], the required channel section is designed as shown in Table below

Table 5

Q	S	B (m)	y (m)	$A = (B + \frac{y}{2})y$ (m²)	$V = \frac{Q}{A}$ (m/s)	V_o (m/s)	$\frac{V}{V_o}$	Remarks
4·61 cumecs	$\frac{1}{5000}$	6·0	1·2	7·92	0·582	0·615	0·947	O.K.

Hence adopt $B = 6·0$ m
$y = 1·2$ m
$S = \frac{1}{5000}$ (20 cm in 1 km)

(iv) Design at 0 km

Outlet discharge required at 0 km, from Table 2 = 4·46 cumecs
Losses below km 1, as worked out earlier = 0·471 cumec
Losses between km 0 to 1 : To work out these losses, the perimeter of the section at km 1 shall be taken, as the section at km 0 is not known so far.

Now, wetted perimeter = $B + \sqrt{5}\,y$
$= 6·0 + \sqrt{5} \times 1·2 = 6 + 2·68 = 8·68$ m

Losses @ 2 cumecs/million sq·m in a length of 1 km
$= 2\left[\frac{8·68 \times 1000}{10^6}\right] = 0·01736 = 0·017$ cumec

Total losses below km 0 = 0·471 + 0·017 = 0·488 cumec

Total discharge required at 0 km = 4·46 + 0·488 = 4·948 cumecs

Design discharge = 1·1 × 4·948 = 5·44 cumecs.

Let us adopt a slope of 20 cm in 1 km, i.e, $S = \frac{1}{5000}$

Using Garret's diagrams [Plate 4·1 (c)], the required channel section is worked out is shown in the Table below,

Table 6

Q	S	B (m)	y (m)	$A = \left[B + \frac{y}{2}\right]y$ (m²)	$V = \frac{Q}{A}$ (ms⁻¹)	V_o (ms⁻¹)	$\frac{V}{V_o}$	Remarks
5·44 Cumecs	$\frac{1}{5000}$	7·2	1·20	9·36	0·58	0·615	0·944	O.K.

Hence, adopt $B = 7.2\,m$

$$y = 1.2\,m$$
$$S = \frac{1}{5000} \text{ (20cm in 1km)}$$

All the data worked out above, has been entered at their proper places in the 'schedule of area statistics and channel dimensions' (Table 7). The table has been completed with the help of Canal Standards.

The L-section of the distributary is drawn, as shown in figures, starting from the head (i.e., 0 Km) by keeping its FSL at head at 0.2 m below the FSL of the branch channel.

→ The cross-sections at various km are drawn, by assuming $1\frac{1}{2}:1$ slopes in filling and 1:1 slopes in cutting.

→ The dowel level is kept 0.15 m higher than the bank level, and road level at 0.15 m below the bank level.

Note. The slopes of 1:1 shall afterwards become $\frac{1}{2}:1$, due to silting, and that is why in design calculations, $\frac{1}{2}H : 1V$ slopes are taken.

1. Following particulars were recorded from a barrage:

(i) Maximum reservoir level = 212 m

(ii) Downstream high flood level in the river = 210 m

(iii) Maximum design flood discharge = 3500 m^3/s

(iv) Pond level = 211 m

(v) Crest level of the barrage = 207 m

(vi) Crest level of the head regulator = 208 m

(vii) Coefficient of discharge = 2.10 $m^{1/2}/sec$ (for barrage)
= 1.50 $m^{1/2}/sec$ (for head regulator)

(viii) River bed level = 205 m

(ix) Design discharge of main canal = 500 m^3/s.

Determine the number of gates required for the barrage and the head regulator if each gate has 10m clear span. Neglect:

(1) end contractions due to piers and abutments;

(2) velocity of approach

If a silting basin is provided d/s of the barrage for the energy dissipation, find the length and RL of the basin floor. Assume that the length of basin is 5 times the conjugate depth required for hydraulic jump. Neglect losses due to friction.

Solution (1) Waterway for Barrage:

High flood discharge of 3500 m^3/s has to pass through the gated openings provided at the barrage site, with the following data.

Maximum water level at high flood U/S of barrage = 212.00 m

Crest level of barrage = 207.00 m

Opening available or Head causing flow above the crest of barrage
= 212.00 − 207.00 = 5.00 mts.
(ignoring vel. head)

Now, $Q = C \cdot L \cdot H^{3/2}$

where C = Co-efficient of discharge through barrage
= 2.10 (given)

L = Effective waterway of barrage (openings)

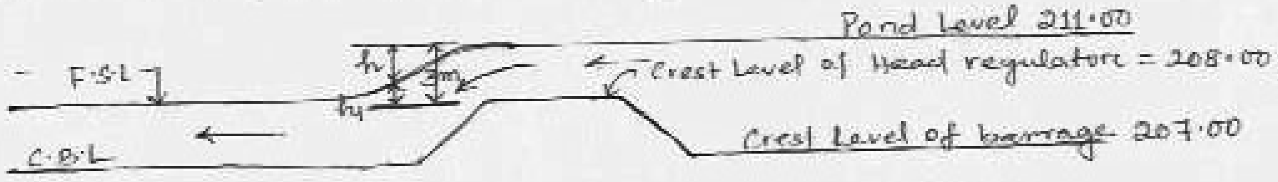
H = 5.00 mts.

$\therefore\ 3500 = 2.10 \times L \times (5)^{3/2}$

or $L = 149.1$ m; 150 m

Effective water way = Total water way (ignoring end contractions)
Hence, provide 15 bays, each of 10 m clear span
∴ No. of gates required for barrage = 15 Ans.

(2) Water way for Canal Head Regulator



The discharge through canal regulator = 500 m^3/s
Co-efficient of discharge for regulator opening = 1·5
Head over the crest of regulator at pond level = 211·00 − 208·00
= 3·00 mts.

$$Q = C_1 L_1 H_1^{3/2}$$
$$\text{or } 500 = 1.5 \times L_1 \times (3)^{3/2}$$
$$\text{or } L_1 = \frac{500}{1.5 \times (3)^{3/2}} = 64 \text{ mts.}$$

Provide 7 bays each of 10 m clear span, thereby providing 70 m water way.
No. of gates required for canal head regulator each of 10 m span = 7. Ans.

(3) Design of Stelling Basin
U/s HFL = 212·00
D/s HFL = 210·00

THEORIES OF SEEPAGE AND
DESIGN OF WEIRS & BARRAGES.

FAILURE OF HYDRAULIC STRUCTURES FOUNDED ON PERVIOUS FOUNDATION

Failure due to (I) Surface flow < By hydraulic Jump / By scouring.

(II) Sub-surface flow

Water seeping below the body of the hydraulic structure (pervious foundation or alluvial soil), endangers the stability of the structure and may cause its failure, either by:

(i) piping; or

(ii) by Direct uplift.

(i) Failure by Piping (or) Undermining.

When the seepage water retains sufficient residual force at the emerging d/s end of the work, it may lift up the soil particles. This leads to increased porosity of the soil by progressive removal of soil beneath the foundation. The structure may ultimately subside into hollow so formed, resulting in the failure of the structure. [As the process of removal of soil particles goes on continuously (i.e.) a depression is formed which extends backwards towards the u/s through the bottom of foundation. A hollow pipe like formation thus develops under the foundation due to which the weir may fail by subsiding.]

(ii) Failure by Direct uplift.

The water seeping below the structure, exerts an uplift pressure on the floor of the structure. If this pressure is not counterbalanced by the weight of the concrete or masonry floor, the structure will fail by a rupture of a part of the floor.

The above concepts of the failure of hydraulic structures due to sub-surface flow were introduced by Bligh on the basis of experiments & research work conducted after the failure of Khanki weir, which was designed on experience & intuition without any rational theory.

Bligh's Theory

According to Bligh's Theory, the percolating water follows the outline of the base of the foundation of the hydraulic structure.

In other words, water creeps along the bottom contour of the structure.

→ The length of the path thus traversed by water is called length of creep.

→ It is assumed in this theory that the loss of head is proportional to the length of the creep.

If H_L is the total head loss b/w the u/s & d/s, and L is the length of creep; then loss of head per unit of creep length is called "hydraulic gradient." (H_L/L)

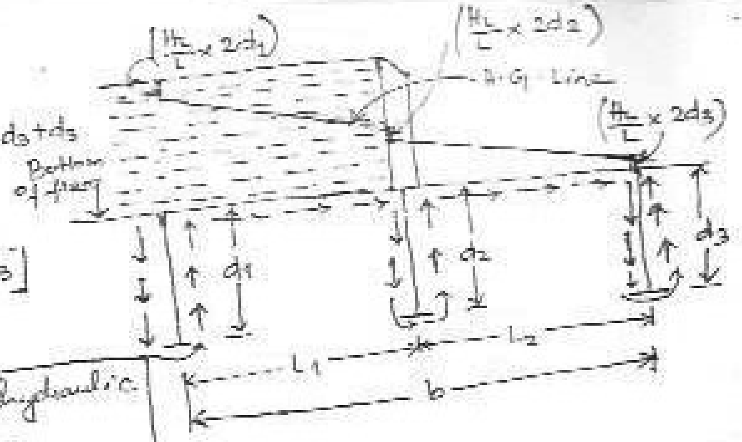
→ Bligh make no distinction between horizontal & vertical creep.

→ Expression for hydraulic gradient (Bligh's Theory)

Consider a section as shown in fig.

Let H_L be the difference of water levels b/w upstream & downstream ends.

Water starts percolating at A and emerges at B.



Total length of creep = L

$= d_1 + d_1 + L_1 + d_2 + d_2 + L_2 + d_3 + d_3$

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

$= (L_1 + L_2) + 2[d_1 + d_2 + d_3]$

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

$$\boxed{\text{Head loss per unit length (or) hydraulic gradient} = \left[\frac{H_L}{b + 2(d_1 + d_2 + d_3)}\right] = \frac{H_L}{L}}$$

Head losses equal to at the places of three vertical cut-offs $= \left(\frac{H_L}{L} \times 2d_1\right), \left(\frac{H_L}{L} \times 2d_2\right), \left(\frac{H_L}{L} \times 2d_3\right)$

(i) Safety Against Piping on Undermining.

According to Bligh, safety against piping can be ensured by providing sufficient creep length, given by $L = CH_L$, where C is Bligh's coefficient for the soil.

$\therefore$ Hydraulic gradient, $\frac{H_L}{L} = \frac{1}{C}$

(must be kept under a safe limit inorder to ensure safety against piping.)

Type of soil	Values of C	Safe H.G.
1. Fine micaceous sand (N.I Rivers)	15	$\frac{1}{15}$
2. Coarse grained sand (C & S.I Rivers)	12	$\frac{1}{12}$
3. Sand mixed with boulder & gravel, & for loam soil	5 to 9	$\frac{1}{5}$ to $\frac{1}{9}$
4. Light sand & mud	8	$\frac{1}{8}$

(ii) Safety against uplift pressure. [can be achieved by providing sufficient impervious floor thickness]

Ordinates of H.G. line above the bottom of the floor represent the residual uplift water head at each point.

If h' meters is this ordinate, then water pressure equal to h' will act at this point, and has to be counterbalanced by the weight of the floor thickness say t.

Uplift pressure $= \gamma_w h'$.

Downward pressure $= (\gamma_w \cdot G) \cdot t = \gamma t$

where G = Specific gravity of floor material.

γ = Unit weight of floor m

t = thickness of flr

For equilibrium, $\gamma_w h' = (\gamma_w G) t$

$h' = G \cdot t$

Subtracting t on both sides, we get

$h' - t = t(G - 1)$

or $\boxed{t = \left(\frac{h' - t}{G - 1}\right) = \left(\frac{h}{G - 1}\right)}$

where $(h' - t) = h$ is the ordinate of H.G line above top of the floor.
$(G-1)$ is the submerged specific gravity of floor material.

Hence, thickness of the floor can be easily determined by using the above equation. This is generally increased by 33%, so as to allow a suitable factor of safety.

- Floor thickness has to be designed according to the above equation only for the d/s floor and for the worst conditions i.e.; when maximum ordinates of H.G line occur.
- The water standing on the u/s floor, more than counterbalances the uplift caused by the same water, and hence, only a nominal floor thickness is required on the u/s side, so as to resist wear, impact of flowing water etc.
- While designing aprons of hydraulic structures on Bligh's theory for sub-surface flow, the floor thickness, is designed in accordance with the above rules, and sufficient length of pucca floor given by $L = C \cdot H_L$ is provided, so as to ensure a safe value of H.G.

LANE'S WEIGHTED CREEP THEORY

- Bligh calculated the length of the creep by simply adding the horizontal creep length & vertical creep length, thereby making no distinction between the two creeps.
- Lane stipulated that horizontal creep is less effective in reducing uplift (or in causing loss of head) than vertical creep.
  He suggested a weighted factor of $1/3$ for the horizontal creep, as against 1.0 for the vertical creep.

  Total Lane's creep length (L_l) is given by

$$L_l = (d_1 + d_1) + \frac{1}{3} L_1 + (d_2 + d_2) + \frac{1}{3} L_2 + (d_3 + d_3)$$

$$\text{or } L_l = \frac{1}{3}(L_1 + L_2) + 2(d_1 + d_2 + d_3)$$

$$\text{or } \boxed{L_l = \frac{1}{3} \times b + 2(d_1 + d_2 + d_3)}$$

To ensure safety against piping, $L \nless C_1 H_L$, where H_L is the head causing flow, and C_1 is Lane's creep coefficient.

- Lane's theory was an improvement over Bligh's theory, but however, was purely empirical without any rational basis, and is practically nowhere used, and is having only a theoretical importance.

KHOSLA'S THEORY AND CONCEPT OF FLOW NET

HISTORY

- Many of the important hydraulic structures, such as weirs & barrages, were designed on the basis of Bligh's theory between 1910 to 1925.
- In 1926–27, the upper Chenab canal syphons, designed on Bligh's theory, started posing undermining troubles.
- Investigation started which ultimately lead to Khosla's theory.
- The main principles of this theory are summarised below:

(1) The seeping water does not creep along the bottom contour of pucca floor as stated by Bligh, but this water moves along a set of stream-lines.

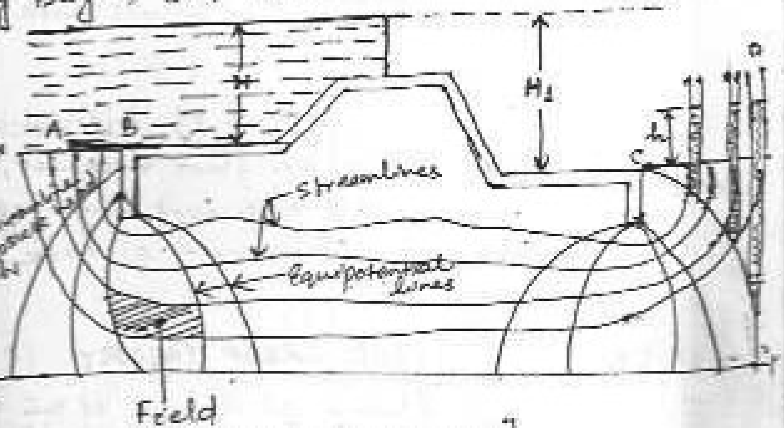
This steady seepage in a vertical plane for a homogeneous soil can be expressed by Laplacian equations

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial z^2} = 0$$

where $\phi = Kh$ = Flow potential

K = coefficient of permeability

h = Residual head at any point within the soil.



[Khosla's Flow Net]

The above equation represents two sets of curves intersecting each other orthogonally. One set of lines is called streamlines, and the other set is called Equipotential lines. The resultant flow diagram showing both the set of curves is called a Flow net.

STREAMLINES

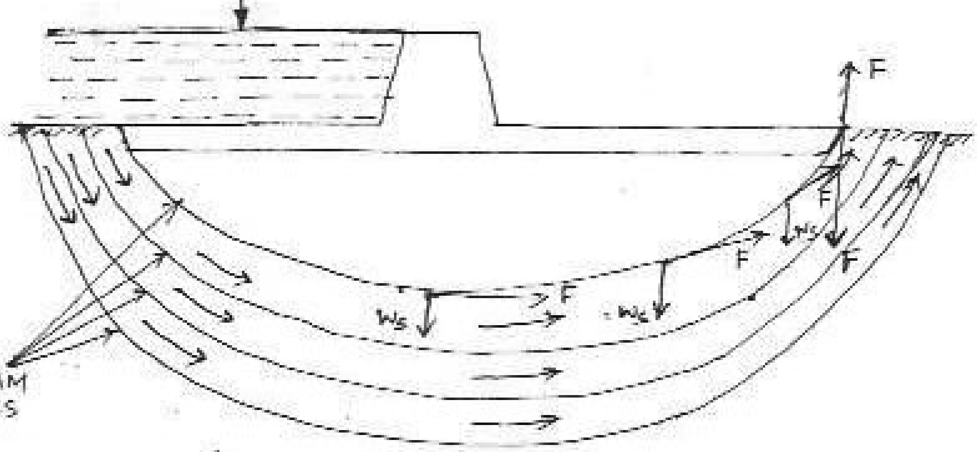
- The first streamline follows the bottom contour of the works and is the same as Bligh's path of creep. ~~similar to Bligh's creep path.~~
- The remaining streamlines follows smooth curve transiting slowly from the outline of the foundation to a semi-ellipse.

EQUIPOTENTIAL LINES

- Treating the d/s bed as datum and assuming no water on the d/s side, it can be easily stated that every streamline possesses a head equal to h_1 while entering the soil; & when it emerges at the d/s end into the atmosphere, its head is zero. Thus, the head h_1 is entirely lost during the passage of water along streamline.
- Further, at every intermediate point in its path, there is certain residual head (h) still to be dissipated in the remaining length to be traversed to the d/s end. This fact is applicable to every streamline, and hence, there will be points on different streamlines having the same value of residual head h. If such points are joined together, the curve obtained is called an equipotential line.

— Every water particle on line AB is having a residual head $h = h_1$ and on CD is having a residual head $h = 0$, and hence, AB and CD are equipotential lines.

(2)



- The seepage water exerts a force at each point in the direction of flow and tangential to the streamlines.
- This force (F) has an upward component from the point where the streamline turns upward. For soil grains to remain stable, the upward component of this force should be counterbalanced by the submerged weight of the soil grain.
- This force has the maximum disturbing tendency at the exit end, because the direction of this force at the exit point is vertically upwards, and hence full force acts as its upward component.

$F \propto dp/dl$

For the stability of soil grains, the submerged weight of soil grain should be more than the upward disturbing force. The disturbing force at any point is proportional to the pressure gradient at that point (dp/dl). This gradient of pressure of water at exit end, is called "Exit Gradient".

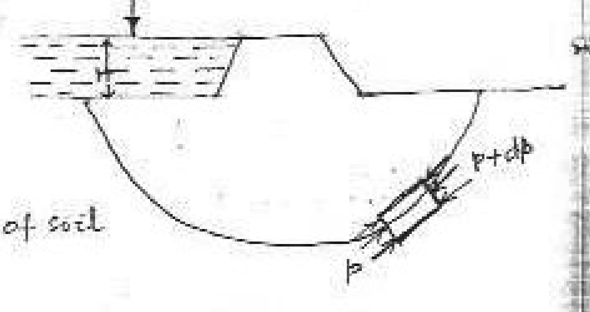
In order that the soil particles at exit remain stable, the upward pressure at exit should be safe. In other words, the exit gradient should be safe.

CRITICAL EXIT GRADIENT

The exit gradient is said to be critical, when the upward disturbing force on the grain is just equal to the submerged weight of the grain at the exit.

$$\boxed{\text{Exit gradient} = \frac{1}{4} \text{ to } \frac{1}{5} \text{ of critical exit gradient,}}$$

is ensured so as to keep the structure safe against piping.



The submerged weight (W_s) of a unit volume of soil is given as:

$$W_s = \gamma_w (1-n)(S_s - 1)$$

where, γ_w = unit weight of water.

S_s = Specific gravity of soil particles

n = porosity of soil materials.

For critical conditions to occur at the exit point,

$F = W_s$ where F = pressure gradient at that point = upward disturbing force on the grain

or $F = \frac{dp}{dl} = \gamma_w \frac{dh}{dl}$ $(\because p = \gamma_w h)$

$p + dp = \gamma_w (h + dh)$

where h = Residual head still to be dissipated, called hydrostatic excess head.

$\therefore \gamma_w \frac{dh}{dl} = \gamma_w (1-n)(S_s - 1)$

or $\boxed{\frac{dh}{dl} = (1-n)(S_s-1)}$ Under critical conditions.

$\frac{dh}{dl} \rightarrow$ rate of loss of head or gradient at exit end.

Under critical conditions, the critical exit gradient is equal to $(1-n)(S_s-1)$.

For most of the river sands, $S_s \simeq 2.65$ and $n \simeq 0.4$, then the value of critical exit gradient $= (1-0.4)(2.65-1)$

$= 0.6 \times 1.65 = 0.99 \simeq 1.0$

Exit gradient $= \frac{1}{4}$ to $\frac{1}{5}$ (Critical Exit Gradient)

$= \frac{1}{4}$ to $\frac{1}{5} * (1)$

or Exit gradient $= \frac{1}{4}$ to $\frac{1}{5}$; has to be provided for keeping the structure safe against piping.

CONCLUSIONS BY KHOSLA

→ Loss of head does not take place uniformly, in direct proportion to the creep length, as stated by Bligh. Infact, it depends upon the whole geometry of the figure, i.e; the shape of foundation, depth of impervious boundary & levels of u/s & d/s beds.

→ Safety against piping cannot be obtained by providing sufficient floor length, as stated by Bligh, but can be obtained by keeping the exit gradient well below the critical value. The exit gradient may not be safe even if the average hydraulic gradient of Bligh (i.e; $\frac{1}{C}$) is safe. $H_L = \frac{1}{C}$ Avg. Hydraulic gradient. $L \propto H_L$ $L = C H_L$.

DEPTH OF VERTICAL CUT-OFF

(3) Undermining of the floor starts from the d/s end of the d/s pucca floor, and if not checked, it travels u/s towards the weir wall. The undermining starts only when the exit gradient is unsafe for the sub-soil on which the weir is founded.

It is, therefore, absolutely necessary to have a reasonably deep vertical cut-off at the d/s end of the d/s pucca floor to prevent undermining.

The depth of this d/s vertical cut-off is governed by 2 considerations i.e.,

(i) maximum depth of scour; (ii) safe exit gradient.

Maximum Depth of Scour

While designing a weir, d/s cutoff from the maximum scoured depth consideration is, first of all, provided, and checked for exit gradient. If a safe value of exit gradient is not obtained, then the depth of cutoff is increased. The depth of cut-off is also governed & limited by practical considerations, as the execution of very deep cut off may be difficult or unpracticable at site.

A weir or a barrage may fail not only due to seepage (i.e; sub-surface flow) as stated by Bligh, but may also fail due to surface flow (i.e; when flood water flows over the weir crest) may cause scour, dynamic action; and in addition, will cause uplift pressures in the jump trough (if hydraulic jump forms on the d/s)

The maximum uplift due to this dynamic action (i.e; surface flow) should then be compared with the maximum uplift under steady seepage (i.e; sub-surface flow); and the maximum of the two chosen for designing the aprons and floors of the weirs.

IMPORTANCE OF KHOSLA'S THEORY

Due to the simplicity, Bligh's theory is still used for design of small works. A minimum practical thickness for the floor and a deep vertical cutoff at the downstream end is however, always provided, in addition to the requirements of Bligh's theory.
However, on major works, Bligh's theory should never be used, as it would lead to expensive & unsafe errorneous designs.

KHOSLA'S METHOD OF INDEPENDENT VARIABLES (for determination of pressures & exit gradient for seepage below weir or a barrage)

Why Independent Variables?

- Laplacian equation must be solved (to plot flownet), inorder to know as to how the seepage below the foundation of a hydraulic structure is taking place.
- This can be accomplished by,
  1. mathematical solution of the Laplacian equations.
  2. Electrical Analogy Method
  3. Graphical sketching by adjusting the streamlines & equipotential lines w.r.t boundary conditions.

  These are complicated methods and are time consuming.
- Therefore, for designing hydraulic structures such as weirs or barrages on pervious foundations, Khosla has evolved a simple, quick and an accurate approach, called "Method of Independent Variables".

What is Independent Variables?

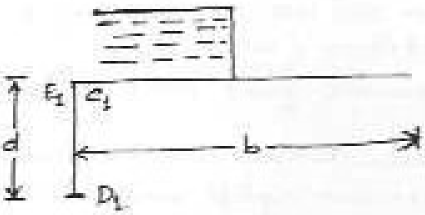
- In this method, a complex profile like that of a weir is broken into a number of simple profiles, each of which can be solved mathematically.
- Mathematical solutions of flownets for these simple standard profiles have been presented in the form of equations and curves, which can be used for determining the percentage pressures at the various key points.

The simple profiles which are most useful are :

(i) A straight horizontal floor of negligible thickness with a sheet pile line on the u/s and d/s end. (Fig. 1 & 2)

(ii) A straight horizontal floor depressed below the bed but without any vertical cut-offs. (Fig. 3)

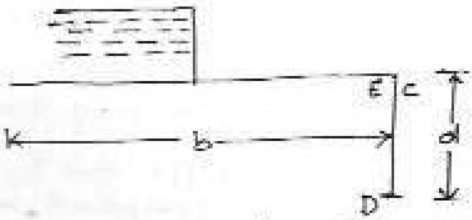
(iii) A straight horizontal floor of negligible thickness with a sheet pile at some intermediate point. (Fig. 4).



$\phi_{C_1} = 100 - \phi_E$

$\phi_{D_1} = 100 - \phi_D$

Fig. 1



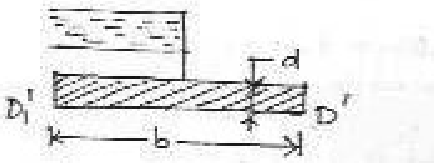
$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$$

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

where $\lambda = \dfrac{1+\sqrt{1+\alpha^2}}{2}$

$\alpha = \dfrac{b}{d}$ (respective)

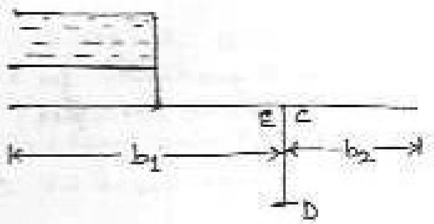
Fig. 2



$$\phi_D' = \frac{2}{3}(\phi_E - \phi_D) + \frac{3}{\alpha^2}$$

$$\phi_{D_1}' = 100 - \phi_D'$$

Fig. 3



$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1-1}{\lambda}\right)$$

Fig. 4

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$\phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1+1}{\lambda}\right)$$

where $\lambda = \dfrac{\sqrt{1+\alpha_1^2}+\sqrt{1+\alpha_2^2}}{2}$

$\lambda_1 = \dfrac{\sqrt{1+\alpha_1^2}-\sqrt{1+\alpha_2^2}}{2}$

$\alpha_1 = \dfrac{b_1}{d}$; $\alpha_2 = \dfrac{b_2}{d}$

[Khosla's simple profiles for a weir of complex profile]

Key points :-

1. E_1 & C_1 – Junctions of floor and the pile lines on either side ;
2. D_1 – Bottom of the pile line ;
3. D' & D_1' – Bottom corners in case of depressed floors.

The key points are the junctions of the floor and the pile lines on either side, and the bottom point of the pile line, and the bottom corners in case of a depressed floor.

The percentage pressures at these key points for the simple forms into which the complex profile has been broken is valid for the complex profile itself, if corrected for

(a) Correction for the mutual interference of piles;

(b) Correction for thickness of floor

(c) Correction for the slope of the floor.

(a) Correction for the Mutual Interference of Piles.

The correction C to be applied as percentage of head due to this effect, is given by

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

depth of effect pile — depth of influence pile

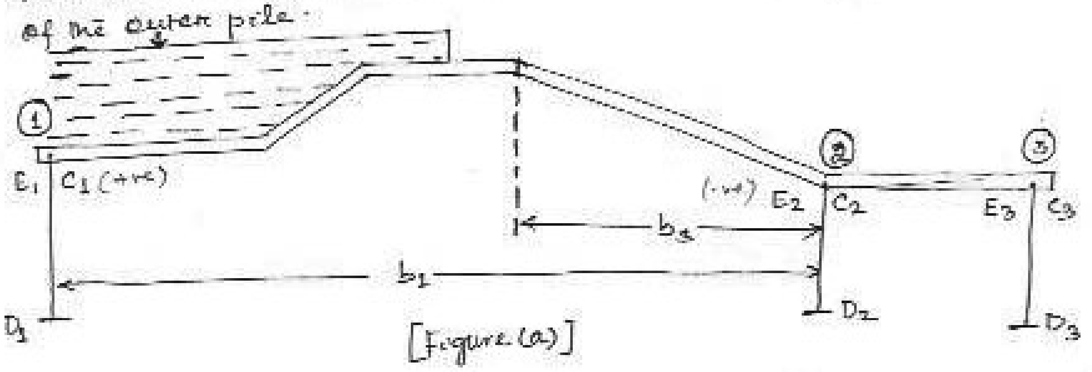
where b' = The distance b/w two pile lines.

D = The depth of the pile line, the influence of which has to be determined on the neighbouring pile of depth d. D is to be measured below the level at which interference is desired.

d = The depth of the pile on which the effect is considered.

b = Total floor length.

- This equation doesnot apply to the effect of an outer pile on an intermediate pile, if the intermediate pile is equal to or smaller than the outer pile and is at a distance less than twice the length of the outer pile. $d < 2lo$



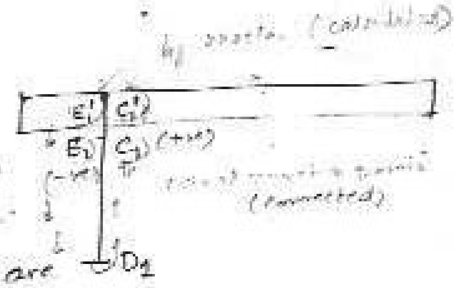
[Figure (a)]

Suppose in the above figure, we are considering the influence of the pile No. (2) on pile no. (1) for correcting the pressure at C_1. Since the point C_1 is in the rear, this correction shall be +ve. While the correction to be applied to E_2 due to pile No. (1) shall be −ve, since the point E_2 is in the forward direction of flow.

Similarly, the corrections at C_2 due to pile No. (3) is +ve, and the correction at E_3 due to pile No. (2) is −ve.

(b) Correction for the Thickness of Floor.

Percentage Pressures calculated by Khosla's equations or graphs shall pertain to the top levels of the floor. While the actual junction points E & C are at the bottom of the floor.



Hence, the pressure at the actual points are calculated by assuming a straight line pressure variation.

- The corrected pressure at E_1 should be less than the calculated pressure at E_1', the correction is to be applied for the point E_1 shall be -ve.

Similarly, the corrected pressure at C_1 is more than the calculated pressure at C_1', and hence, the correction to be applied at point C_1 is +ve.

(c) Correction for the slope of the floor.

A correction is applied for a sloping floor, and is taken as +ve for the down, and -ve for the up slopes following the direction of flow.

- The correction factor given is to be multiplied by the horizontal length of the slope divided by the distance between the two pile lines between which the sloping floor is located. This correction is applicable only to the key points of the pile line fixed at the start or the end of the slope.

Slope (H : V)	Correction factor
1:1	11.2
2:1	6.5
3:1	4.5
4:1	3.3
5:1	2.8
6:1	2.5
7:1	2.3
8:1	2.0

e.g., In figure (a), this correction is applicable only to point E_2. Since the slope is down at point E_2 in the direction of flow, hence, the correction shall be +ve & will be equal to the correction factor for this slope multiplied by b_s/b_1, where b_s & b_1 are shown in the figure. $C = f \times \frac{b_s}{b_1}$

Exit Gradient (G_E)

Exit gradient at the d/s end $(G_E) = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$

$$\text{where } \lambda = \frac{1 + \sqrt{1+\alpha^2}}{2}$$

$$\& \quad \alpha = \frac{b}{d}$$

From the curve of plate 11.2; for any value of α, the corresponding value of $\frac{1}{\pi\sqrt{\lambda}}$ can be read. Knowing H and d, the value of G_E can be easily calculated. The exit gradient so calculated must lie within safe limits.

Type of soil	Safe exit gradient
Shingle	$\frac{1}{4}$ to $\frac{1}{5}$ (0·25 to 0·20)
Coarse sand	$\frac{1}{5}$ to $\frac{1}{6}$ (0·20 to 0·17)
Fine sand	$\frac{1}{6}$ to $\frac{1}{7}$ (0·17 to 0·14)

NOTE:-

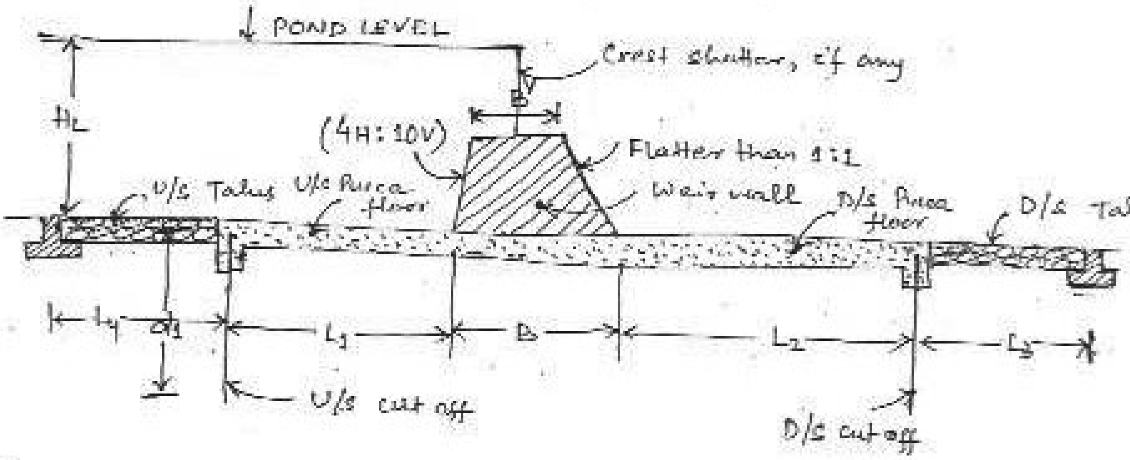
(1) The uplift pressure must be kept as low as possible consistent with the safety at the exit, so as to keep the floor thickness to the minimum.

(5) $G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$; it is obvious from this equation that if $d = 0$, G_E is infinite. Hence it becomes essential that a vertical cut-off at the d/s end must be provided.

<u>Design of a vertical Drop Weir on Bligh's Theory.</u>

- Many of the vertical drop weirs have been designed on Bligh's theory, & even though this theory has now been replaced by modern Khosla's theory, yet is still used at certain places and especially for minor works, owing to its simplicity.

1. Design of Pucca-floor and Apron.



Total length of the pucca floor of the weir (including twice the length of cut-off) } is designed in accordance with the equation $L = C.H_L$

and

Thickness of the floor is designed by using the equation:

$$t = 1.33\left(\frac{h}{G-1}\right)$$

The balance floor length = Total length − (d/s length + twice the cut-off length.)

The <u>balance floor length</u> is then provided under the crest & on the <u>u/s side</u>

FOR PROPER WEIR PORTION.

Various empirical formulas put forward by Bligh are given below.

(a) $L_2 = 2.21\,C\,\sqrt{\frac{H_L}{13}}$ for weirs having crest shutters.

(b) $L_2 = 2.21\,C\,\sqrt{\frac{H_L}{10}}$ for weirs having no crest shutters.

where H_L = total head loss

L_2 = length of the d/s pucca floor.

(c) $L_2 + L_3 = 18\,C\,\sqrt{\frac{H_L}{13} \times \frac{q}{75}}$ for weirs having crest shutters.

(d) $L_2 + L_3 = 18\,C\,\sqrt{\frac{H_L}{10} \cdot \frac{q}{75}}$ for weirs having no crest shutters.

where q = discharge intensity (in cumecs/m)

L_3 = Length of d/s loose stone talus.

(e) The length of u/s talus (L_4) may be kept half of length of d/s talus.

$$L_4 = \frac{L_3}{2}$$

FOR UNDERSLUICE PORTION.

For undersluice portion of weirs the following modified formulas for undersluice portion of weirs are used.

(i) $L_2 = 3.87\,C\,\sqrt{\frac{H_L}{13}}$ for 'undersluices' having crest shutters.

(ii) $L_2 = 3.87\,C\,\sqrt{\frac{H_L}{10}}$ " " no " "

(iii) $L_2 + L_3 = 27\,C\,\sqrt{\frac{H_L}{13} \times \frac{q}{75}}$ for 'undersluices' having crest shutters

(iv) $L_2 + L_3 = 27\,C\,\sqrt{\frac{H_L}{10} \times \frac{q}{75}}$ " " no " "

Design of weir wall

Top width of weir wall (B') is given as :

$$B' = \frac{H}{\sqrt{G-1}}$$

where B' = Top width of weir wall (1.5 – 1.8 m)

H = Head of water over the weir wall at the time of max. flood.

G = Sp. Gr. of floor material.

* Crest width should also be greater than by 0.6 m than the height of the crest shutters, if any.