Republic of Iraq
The Ministry of Higher Education & Scientific Research

University: Baghdad

College: Engineering

Department: Water Resources Engineering.

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DESIGN OF HYDRAULIC STRUCTURES

HYDRAULIC DESIGN OF STRUCTURES

CONCRETE DESIGN OF HYDRAULIC STRUCTURES

Course Objective
Improve the ability of student in designing the hydraulic structures. Calculating the **dimensions** and **reinforcement** of the hydraulic structures according to the available data.
Course Description
Studying the hydraulic structures and finding it is **dimensions**. Check the safety of the structures and finding the flow performance. Analysis and design of selected hydraulic structures and finding it is reinforcement.

Prerequisite module	Engineering Mechanics- Static, Strength of Materials, Structural Analysis, Design of Concrete Fluid Mechanics, Irrigation and drai

DESIGN OF HYDRAULIC STRUCTURES

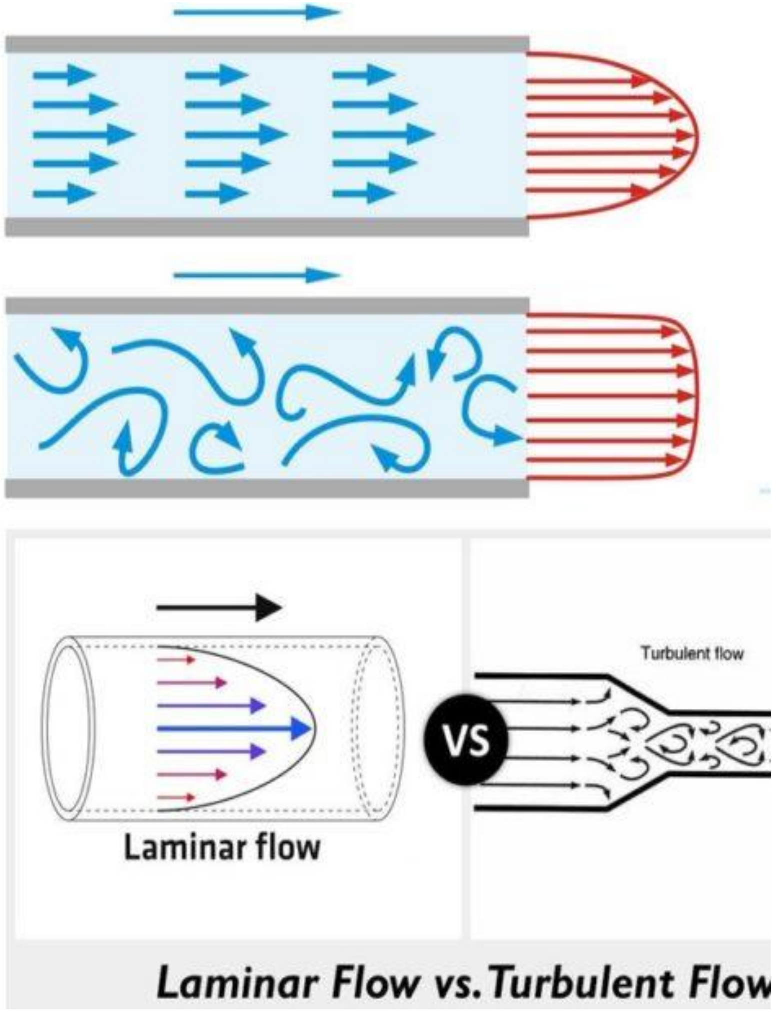
Textbook	**Varshney – Gupta – Gupta 1977 "Theory and Design of Irrigation Structures" 3rd. Edition Vol. II**
Reference Books	**1. Chow, V.T.,1959 “Open Channel Hydraulics”** **2. Davis, C.V., 1969 “Handbook of Applied Hydraulics” 3rd Edition.** **3. U.S.B.R. 1958 “Hydraulic Design of Stilling Basins and Bucket Energy Dissipaters** **4. Linsley and Franzini, 1972 “Water Resources Engineering” 2nd Edition.** **5. U.S.B.R. 1974 “Design of Small Canal Structures”** **6. FAO 1975 “Small Hydraulic Structures”.** **7. Varshney – Gupta – Gupta 1977 "Theory and Design of Irrigation Structures" 3rd. Edition Vol. II** **8. Punmia .B.C. & Pande B.B. Lal 1981 “Irrigation and Water Power”.** **9. Santosh Kumar Garg 1997 “Irrigation Eng. & Hyd. Str.”** **10. Larry W. Mays 2005 “Water Resources Eng.”** **11. R.K. Sharma & T.K. Sharma 2008 “Irrigation Eng.”.** **Dr.K.R. Arora 2009 “Irrigation, Water Power & Water Resources Eng.”.**

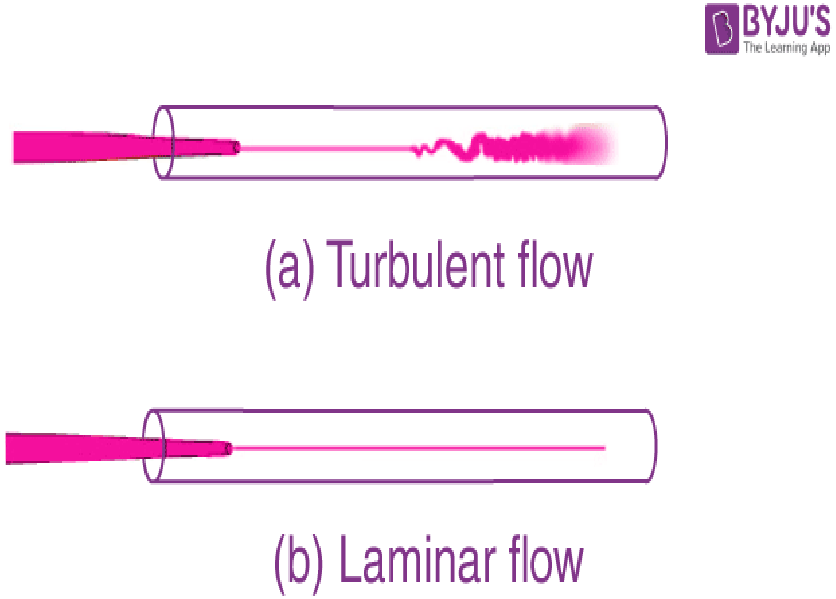
Course Assessment	Term Tests	Laboratory	Quizzes	Project	Final Exam
	15%		10%	5%	70%

DESIGN OF HYDRAULIC STRUCTURES

Course weekly Outline

week	Topics Covered	
1	Introduce basic definitions and introductory concepts of hydraulic structures and their use.	Introduction & Rigid Foundations
2	Introduce the definition, name, location and direction of regulator.	Rigid Foundations & Elastic Foundations
3	Introduce the hydraulic calculation of regulators (velocity and discharge).	Elastic Foundations, Design of aqueduct
4	Introduce the line of creep and up lift pressure theories (Bligh's creep theory and Lane's weighed line of creep method).	Design of aqueduct
5	Introduce the flow net (Khosla's theory / exit gradient, cut off depths and scouring depth)	Types of bridges, Design of slab bridges
6	Introduce the concrete floor thickness.	Design of deck grader bridges (concrete girders)
7	Introduce the transitions (kinds, properties, hydraulics, discharge equation, Mitra's method, Hind's method).	Design of deck grader bridges (concrete girders). (1st exam)
8	Introduce the energy dissipation (hydraulic jump, types and efficiency, type of flow D/S of gates and types of stilling basins).	Design of Box Culvert
9	Introduce protection of approaches U/S and D/S of concrete floors.	Concrete Design of Reinforced concrete culverts
10	Introduce gates (types, water pressure and forces on gates, design principle for sliding steel gates).	Concrete Design of Reinforced concrete culverts.
11	Introduce the closed regulating and conveyance structures (concrete pipes, reinforced concrete culverts, single and multiple barrels and siphons).	Concrete Design of circular tank
12	Introduce the closed regulating and conveyance structures (concrete pipes, reinforced concrete culverts, single and multiple barrels and siphons).	Concrete Design of circular tanks
13	Introduce the weirs (sharp and broad crested weirs).	Concrete Design of Rectangular tanks
14	Introduce the level control structures (canal outlet, canal escape, falls or drops).	Concrete Design of Rectangular tanks.
15	Introduce basic definitions and introductory concepts of hydraulic structures and their use.	Concrete Design of Barrages


Turbulent flow
VS
Laminar flow
Laminar Flow vs. Turbulent Flow


BYJU'S
The Learning App
(a) Turbulent flow
(b) Laminar flow

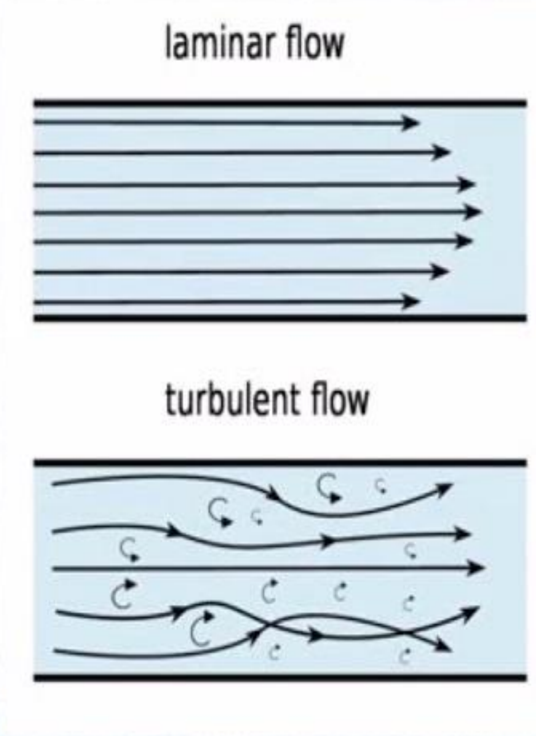
Laminar Flow and Turbulent Flow:

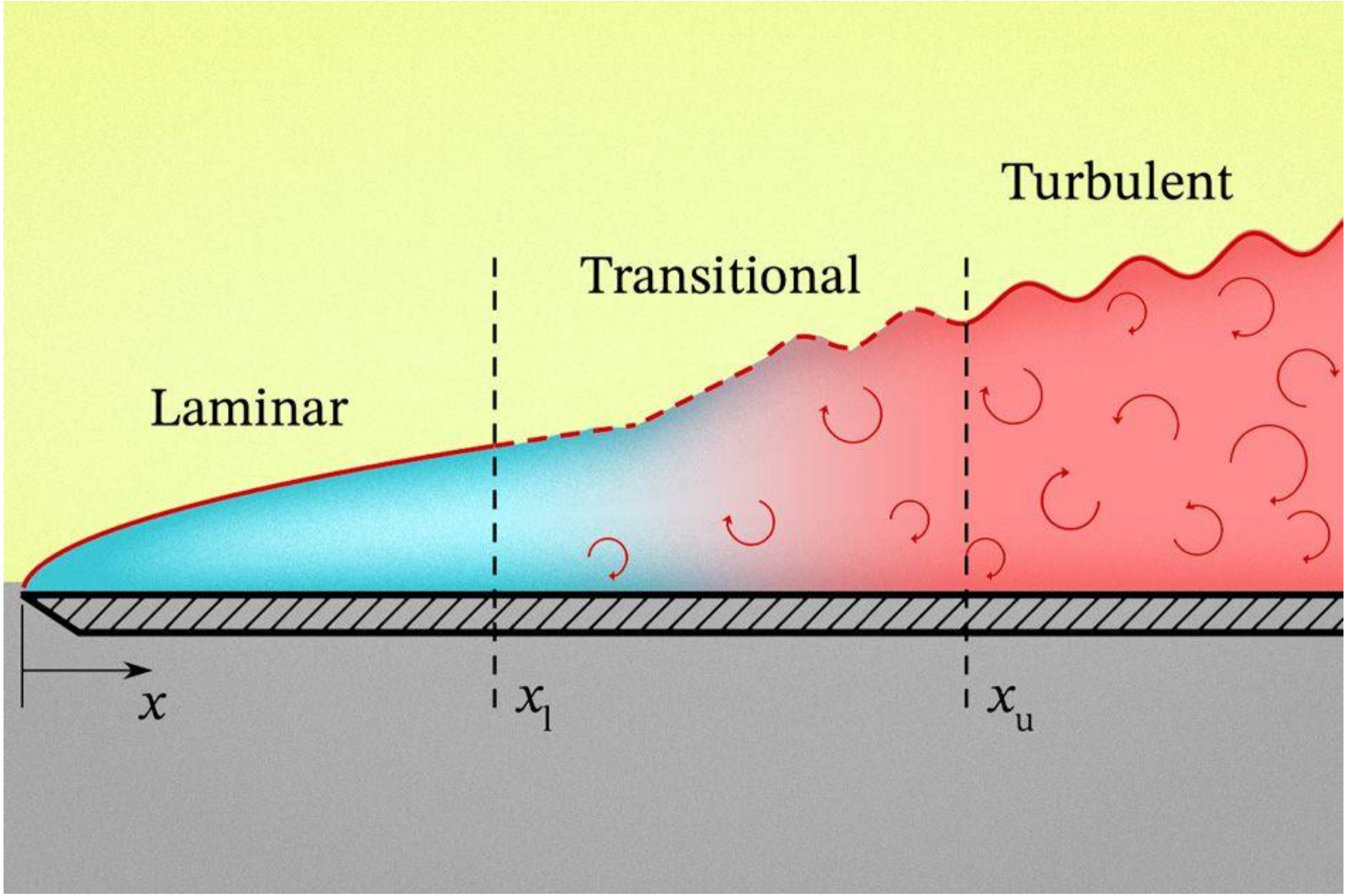
Laminar Flow:

- If the flow is smooth and if the layers in the flow do not mix macroscopically then the flow is called laminar flow. In laminar flow layers will glide over each other without mixing.

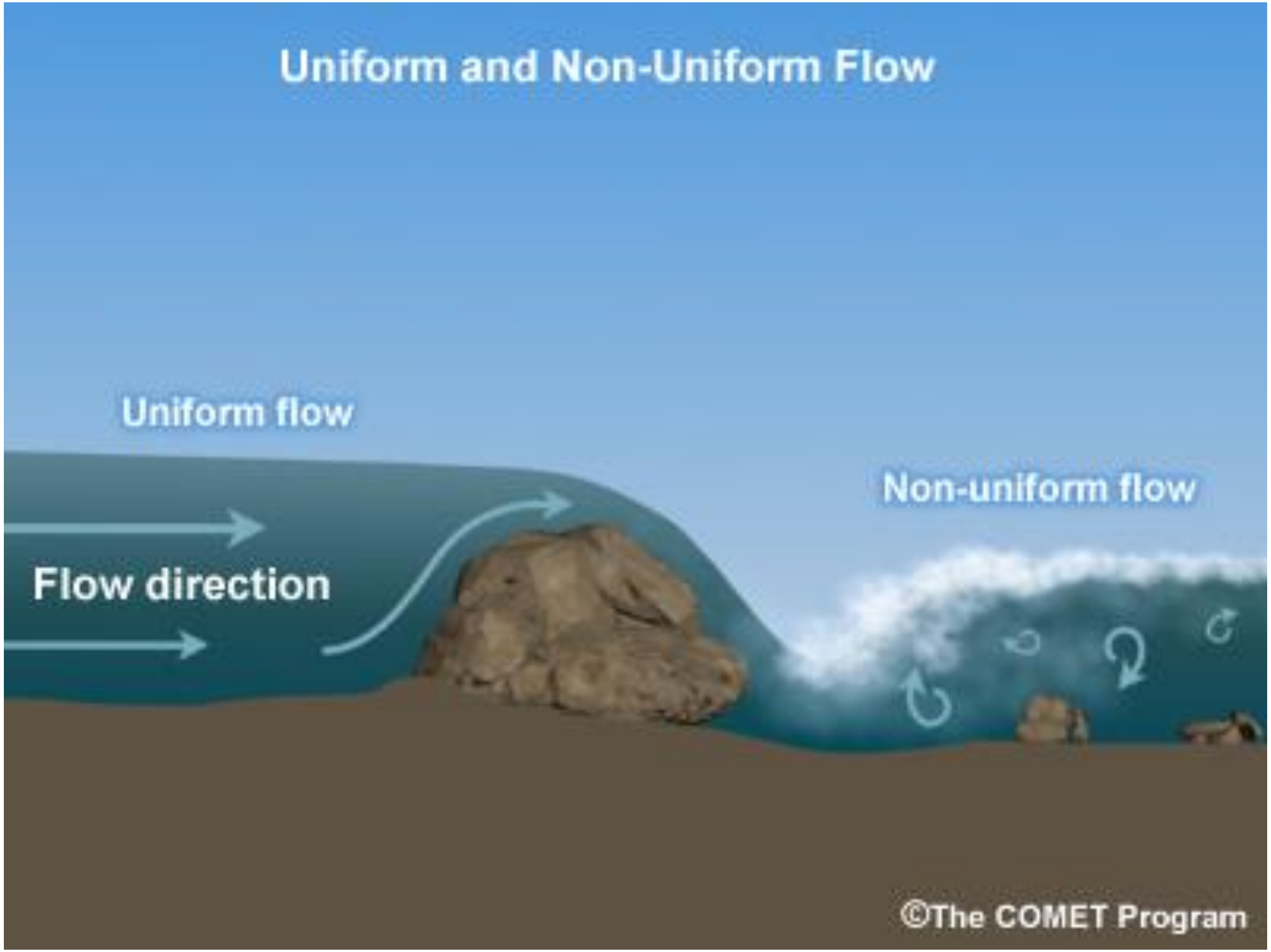
Turbulent Flow:

- In turbulent flow fluid layers mix macroscopically and the velocity/temperature/mass concentration at any point is found to vary over a time period.





Turbulent
Transitional
Laminar
x
x_l
x_u


Uniform and Non-Uniform Flow
Uniform flow
Non-uniform flow
Flow direction
©The COMET Program

Uniform Flow	Non-uniform Flow
The flow in which velocity at any given time does not change with respect to distance. $\left(\frac{\partial v}{\partial s}\right)_{t=c} = 0$	In this flow, velocity at any given time changes with respect to distance. $\left(\frac{\partial v}{\partial s}\right)_{t=c} \neq 0$

STEADY AND UNSTEADY FLOW:

Steady flow: the flow in which conditions at any point do not change with time is called steady flow.

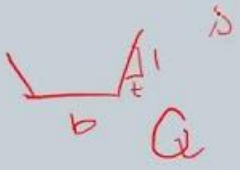
$$\boxed{\frac{\partial P}{\partial t} = 0, \frac{\partial V}{\partial t} = 0, \frac{\partial \rho}{\partial t} = 0,}$$

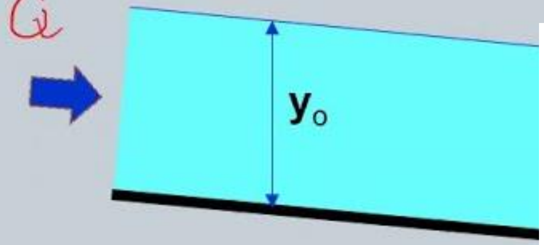
- ***Unsteady flow:*** the flow in which conditions at any point change with time, is called unsteady flow.

$$\boxed{\frac{\partial P}{\partial t} \neq 0, \frac{\partial V}{\partial t} \neq 0, \frac{\partial \rho}{\partial t} \neq 0,}$$

Definition of the Normal depth

The normal depth (y_o) is the actual channel water depth when the flow is uniform (i.e. when the flow depth and velocity do not change with distance)





$$Q = VA = \left(\frac{1.49}{n}\right) A R^{\frac{2}{3}} \sqrt{S} \quad [U.S.]$$

$$Q = VA = \left(\frac{1.00}{n}\right) A R^{\frac{2}{3}} \sqrt{S} \quad [SI]$$

Critical depth

Flow in an open channel is critical when the Froude number of the flow is equal to unity. Critical flow in a channel depends on the discharge and the geometry of channel section. For a rectangular section, the critical depth is given by

$$y_c = \sqrt[3]{\frac{Q^2}{gB^2}}$$

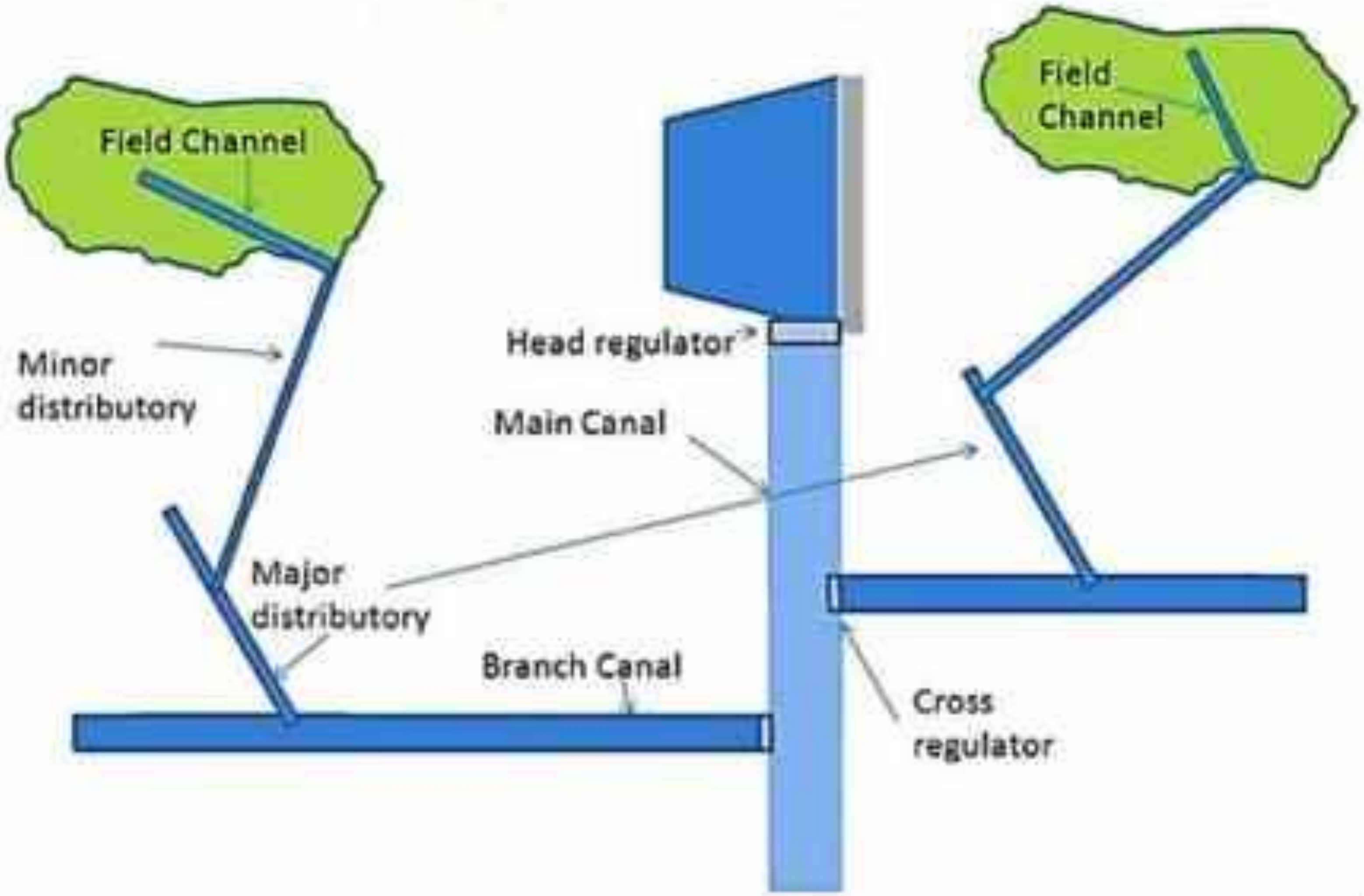
Where,

yc = The critical depth,

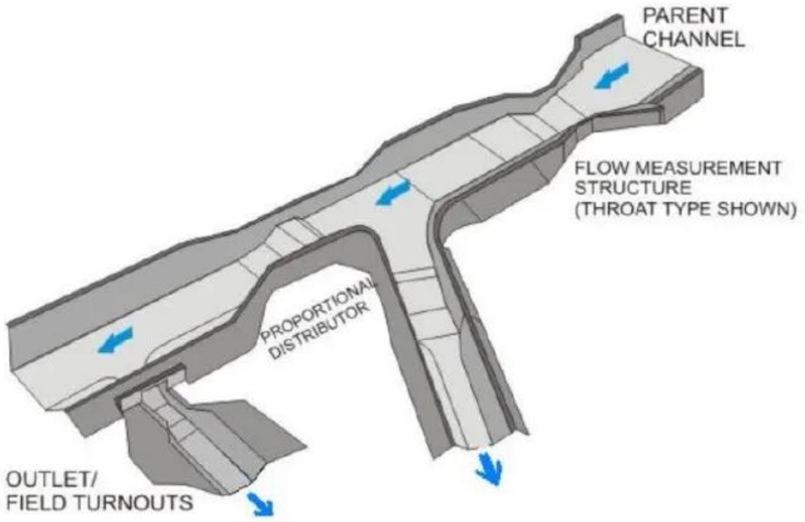
Q = The discharge and

B = The width of the channel.


20 Types of Irrigation Canal and Their Uses


Field Channel
Field
Channel
Head regulator
Minor
distributory
Main Canal
Major
distributory
Branch Canal
Cross
regulator

Canal Outlets



Introduction

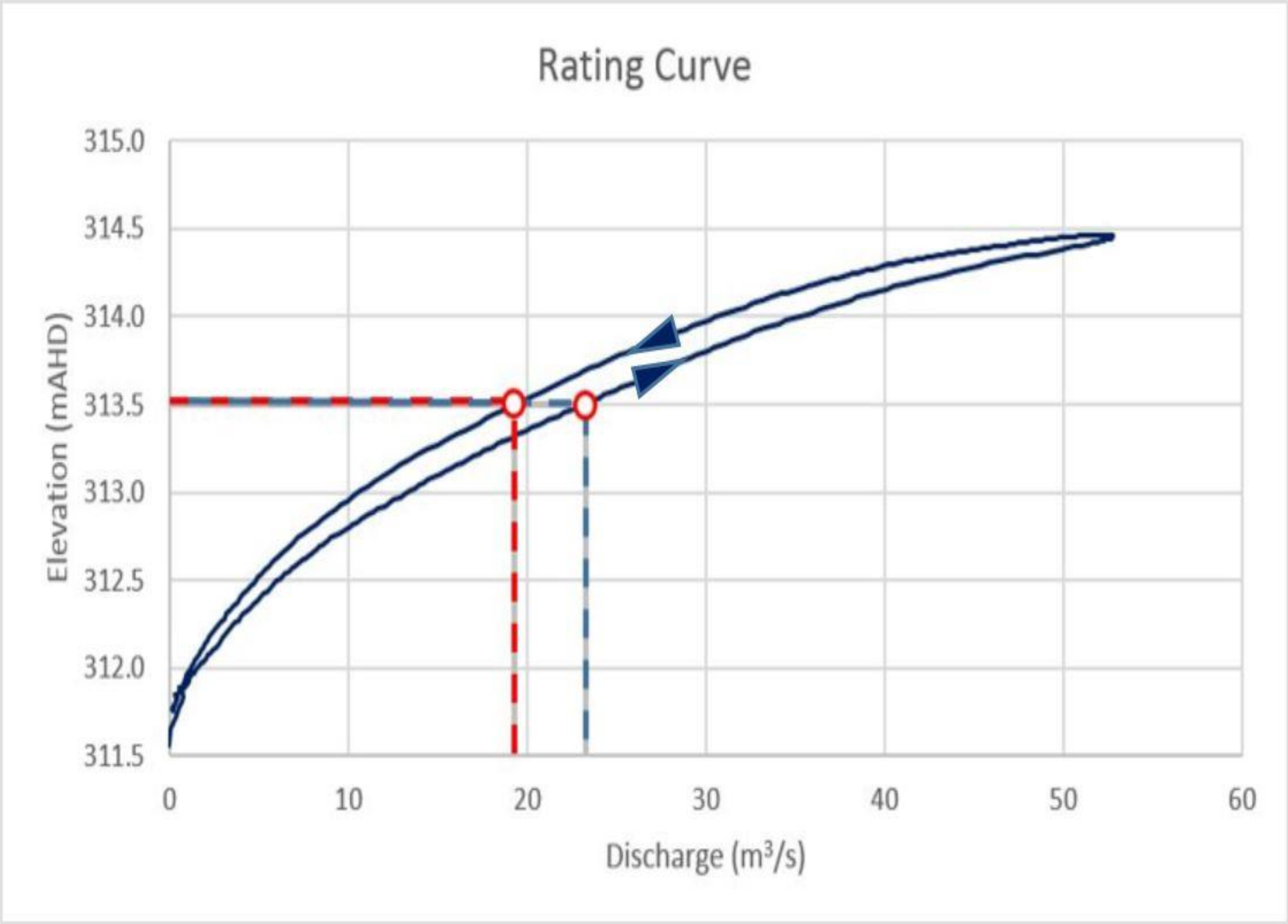
- **An outlet is a small structure which admits water from the distributing channel to a water course or field channel.**

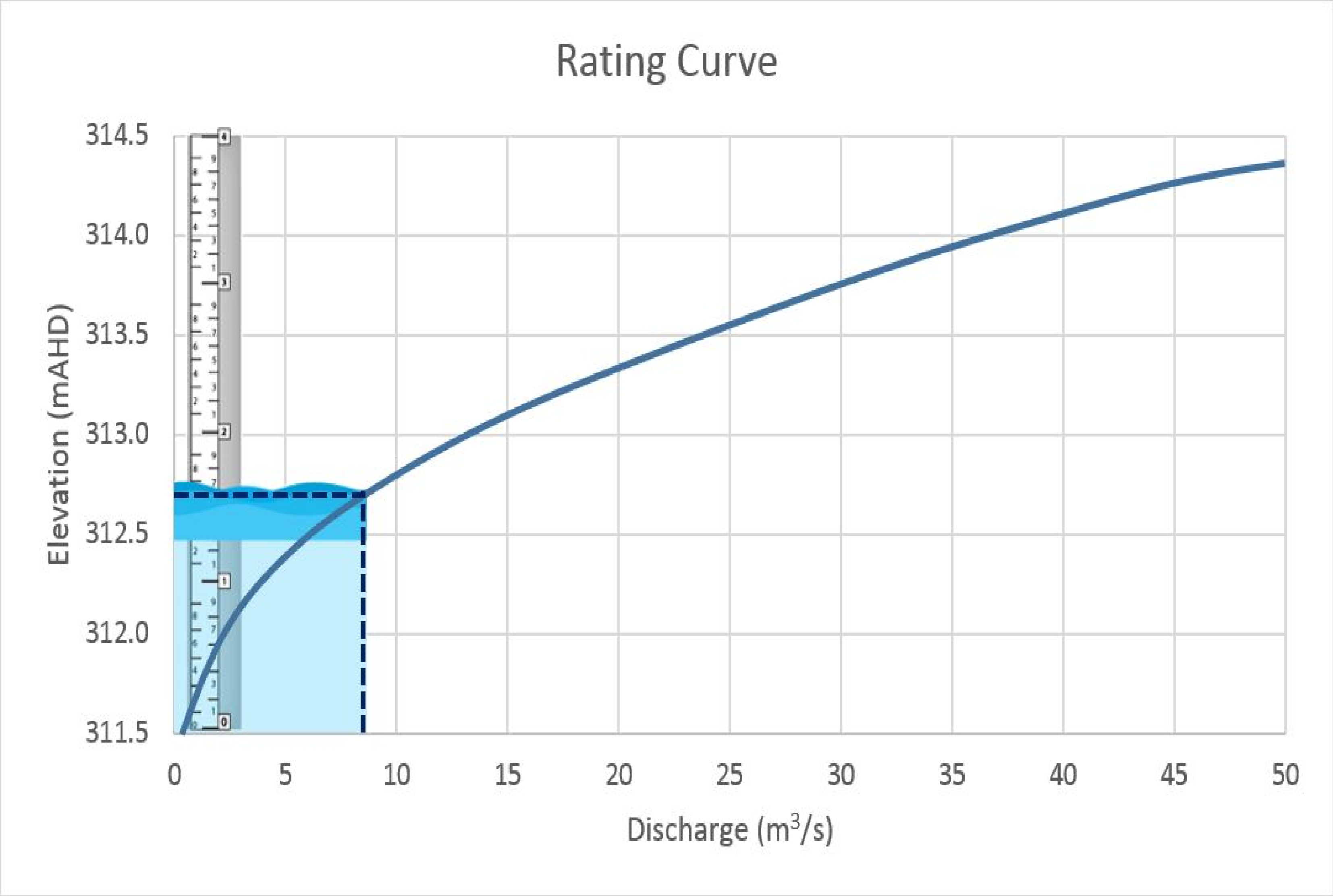
- **An outlet is a sort of head regulator for the field channel delivering water to the irrigation fields.**

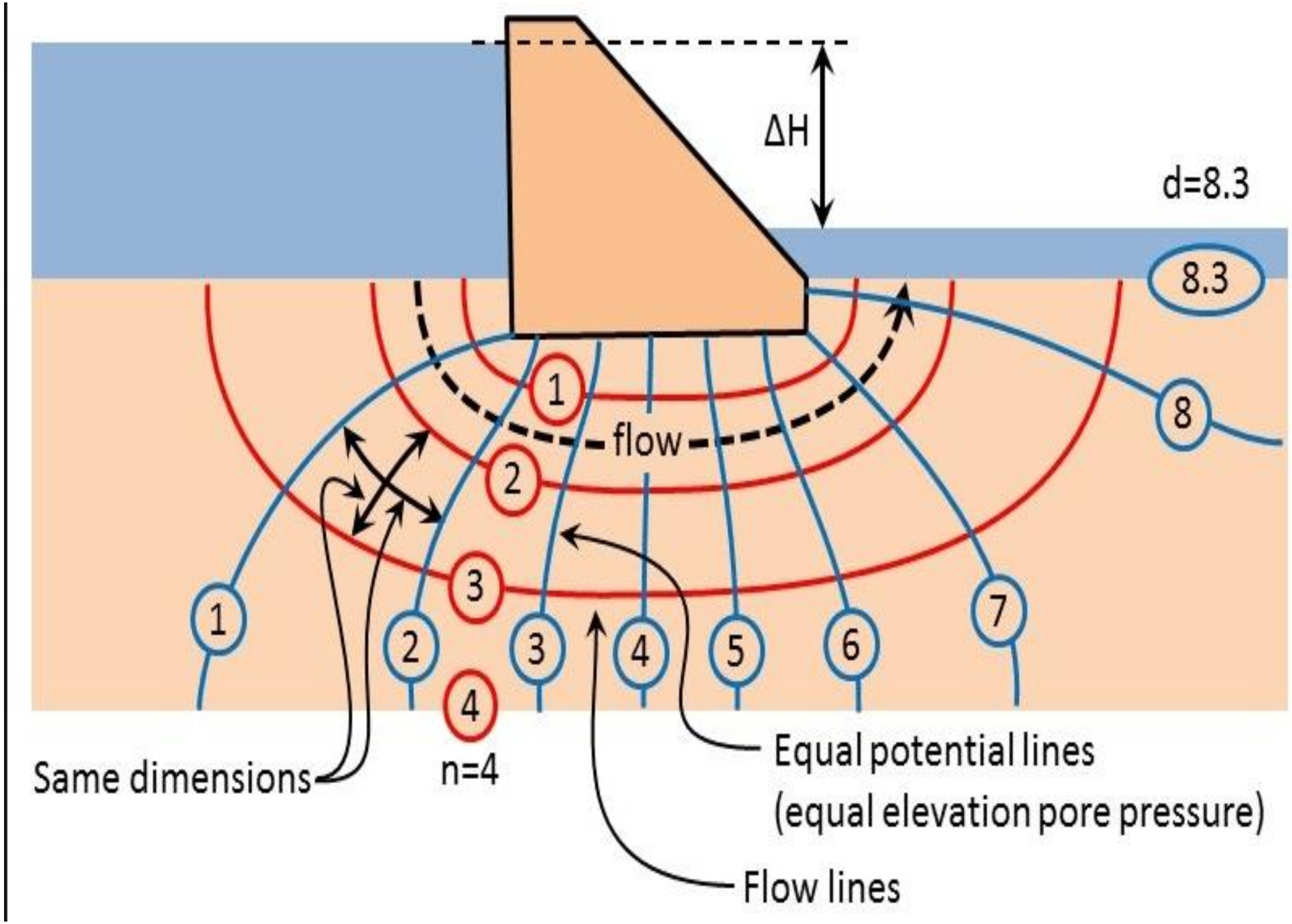
- **The responsibility of maintenance of the distributing channel and the whole canal network lies with Government, while that of the field channel lies with the farmer. The outlet is the connecting medium for the two.**

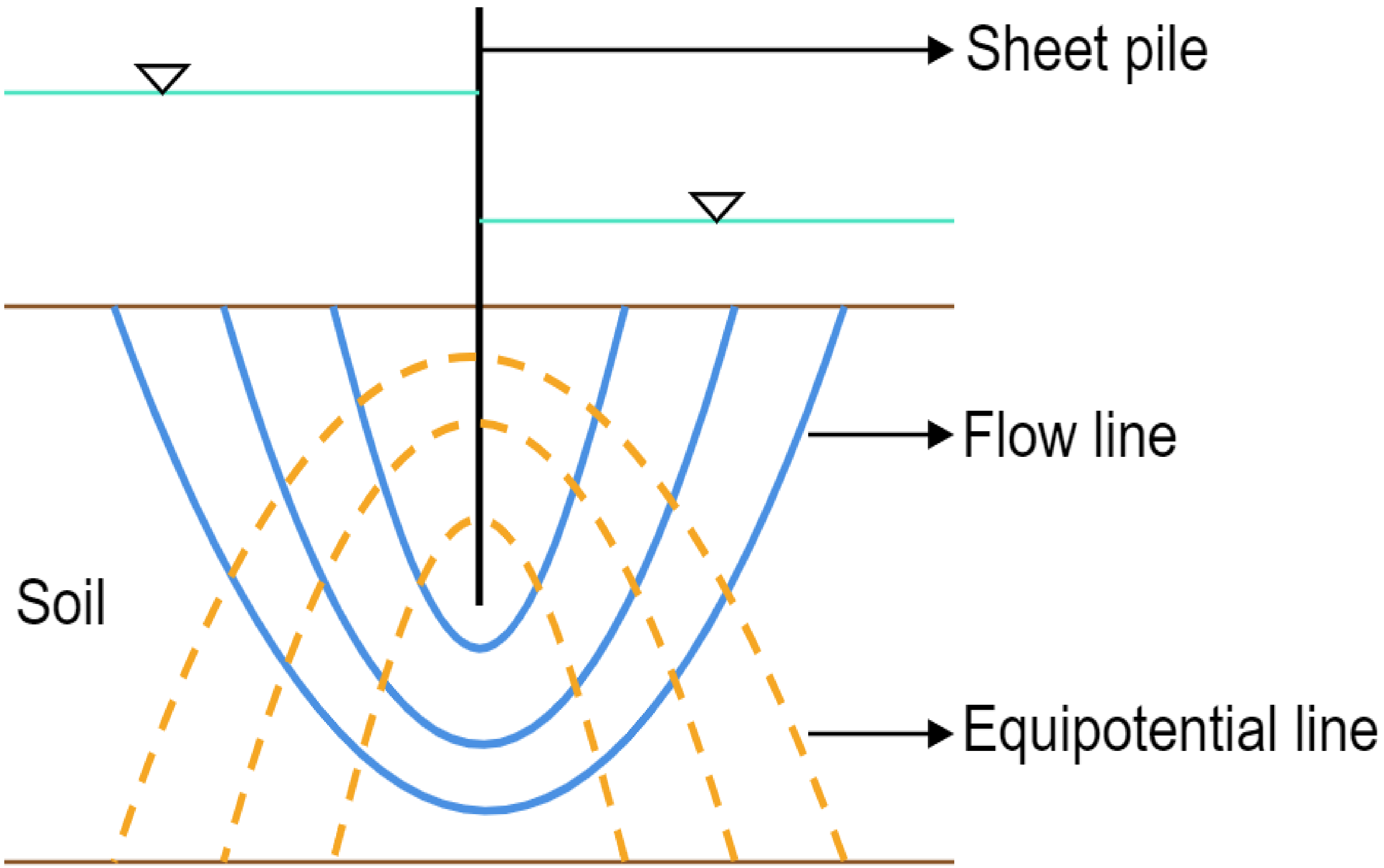


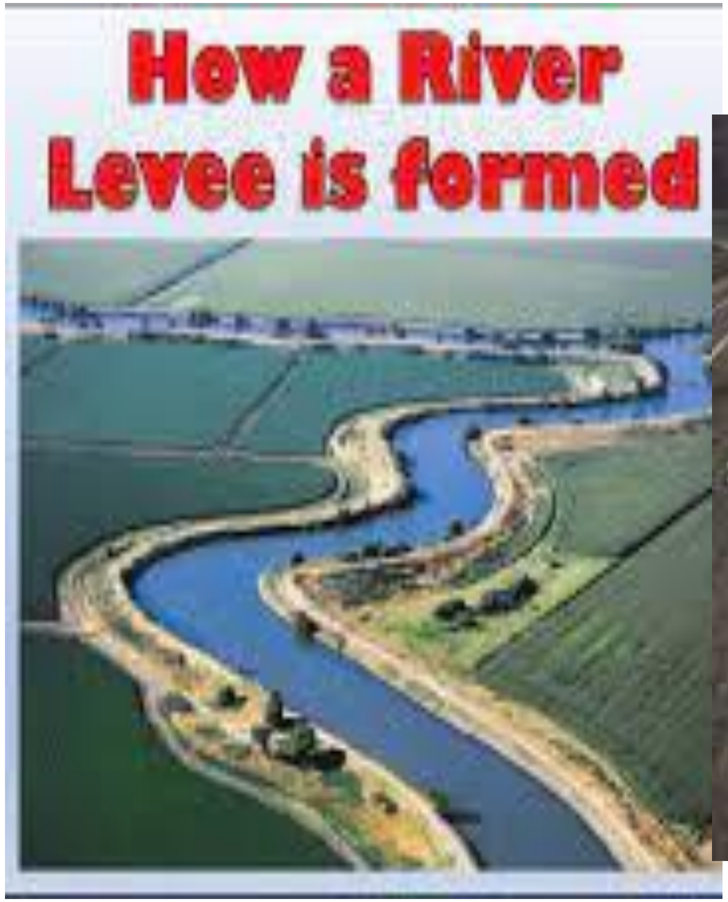
Barrage


Rating Curve
315.0
314.5
314.0
313.5
313.0
312.5
312.0
311.5
Elevation (mAHD)
0
10
20
30
40
50
60
Discharge (m³/s)


Rating Curve
Elevation (mAHD)
314.5
314.0
313.5
313.0
312.5
312.0
311.5
0
5
10
15
20
25
30
35
40
45
50
Discharge (m3/s)


ΔH
d=8.3
8.3
1
2
3
4
flow
1
2
3
4
5
6
7
8
Same dimensions
n=4
Equal potential lines
(equal elevation pore pressure)
Flow lines


Sheet pile
Flow line
Soil
Equipotential line

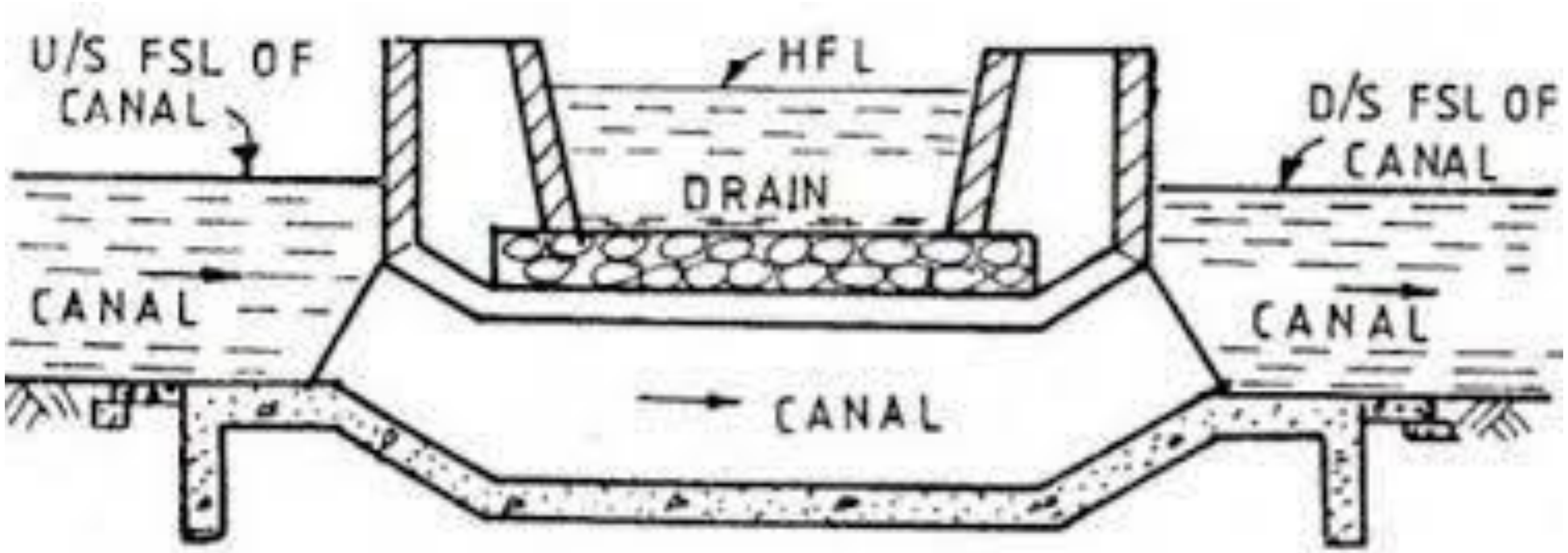

How a River
Levee is formed


gettyimages








U/S FSL OF
CANAL
HFL
D/S FSL OF
CANAL
DRAIN
CANAL
CANAL
CANAL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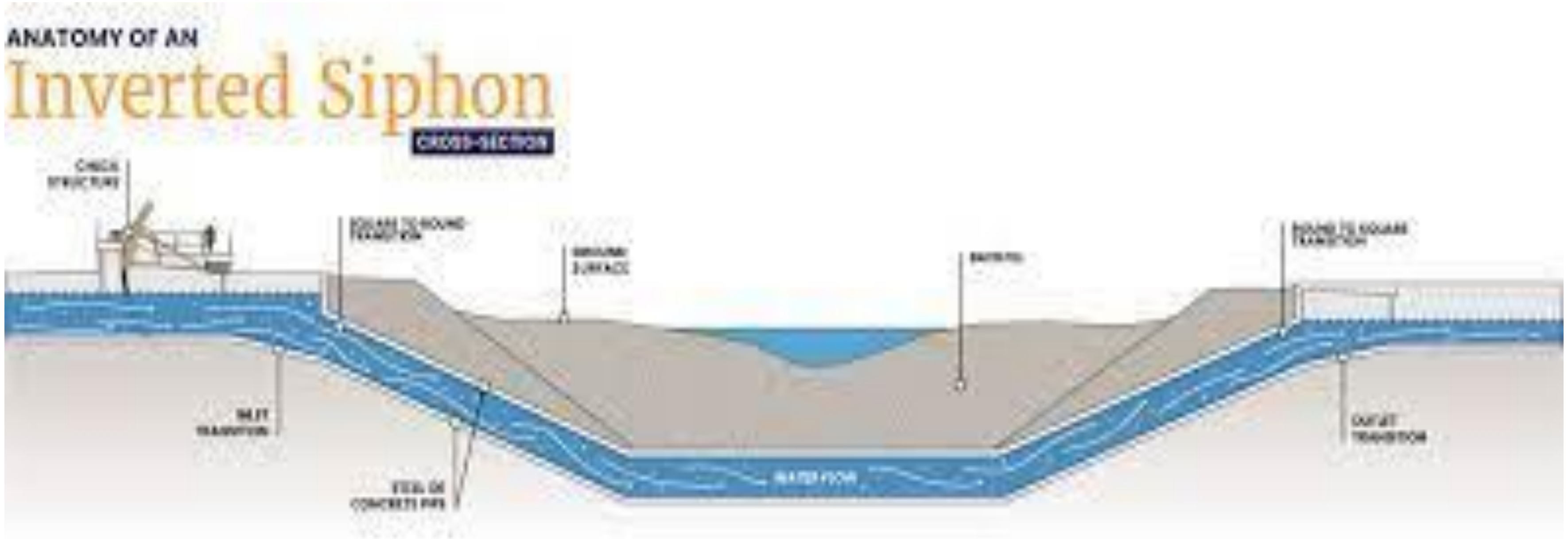
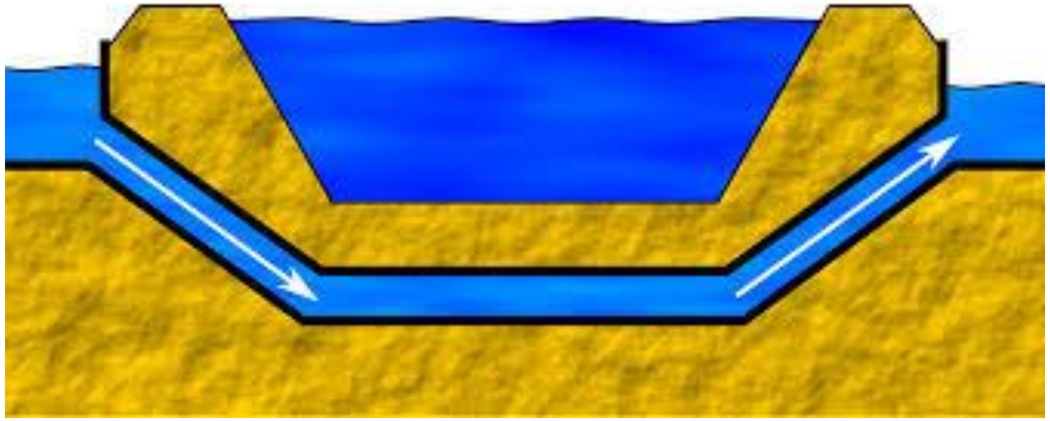
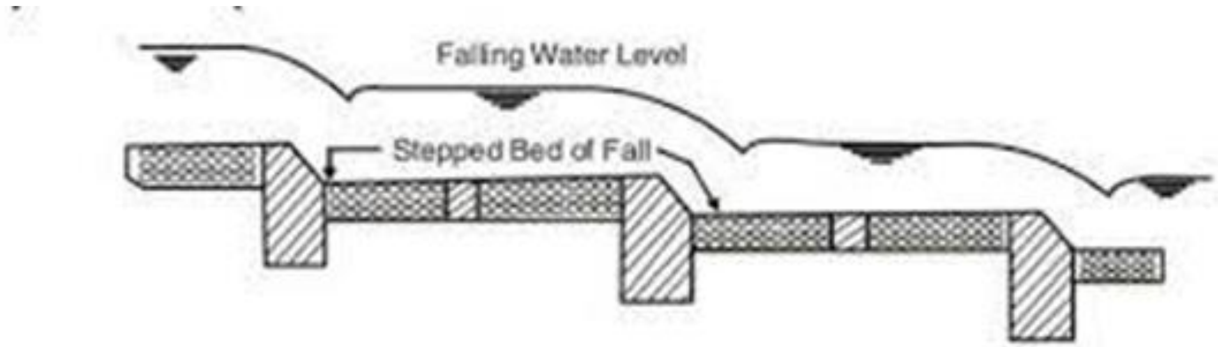
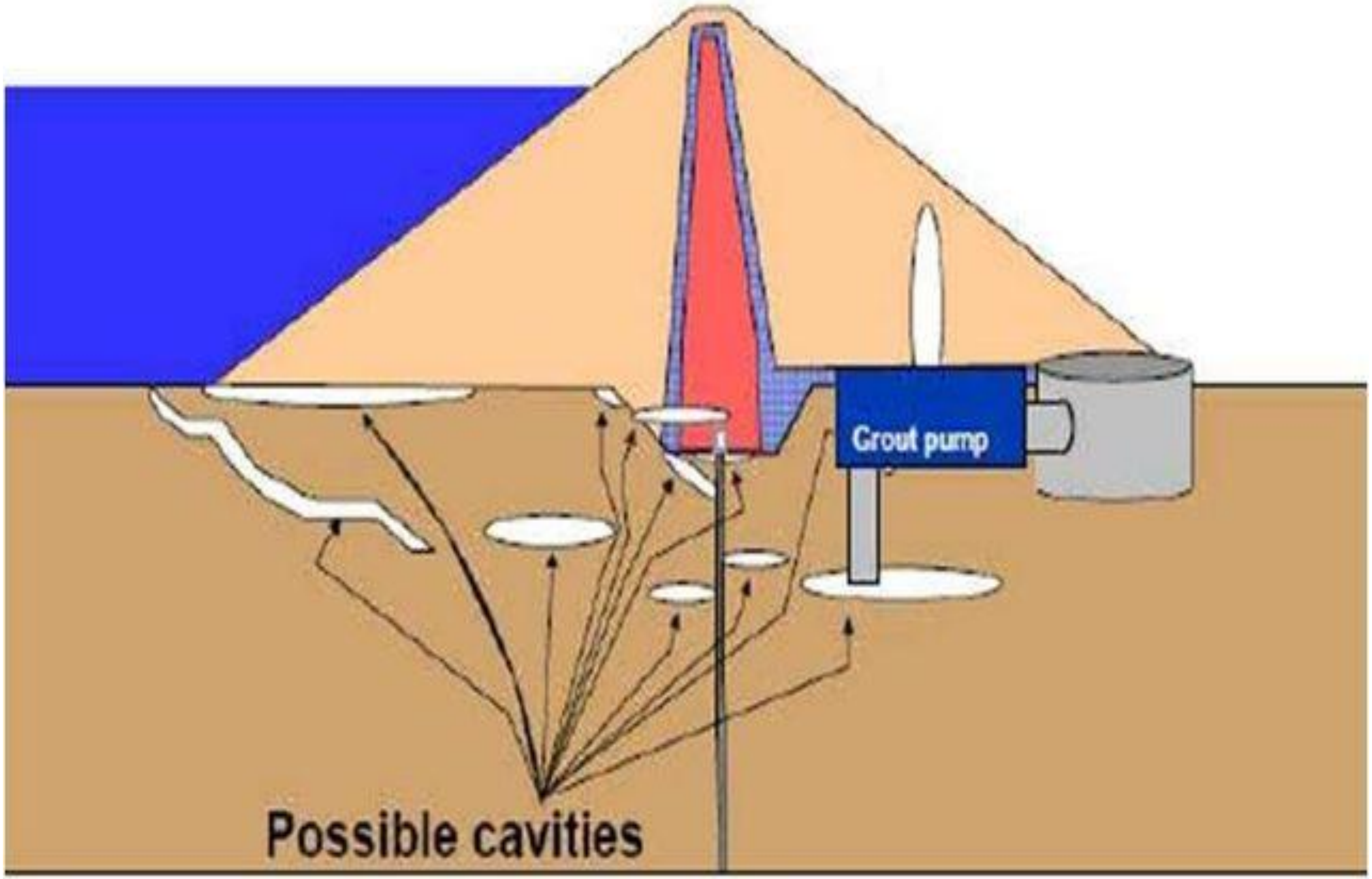
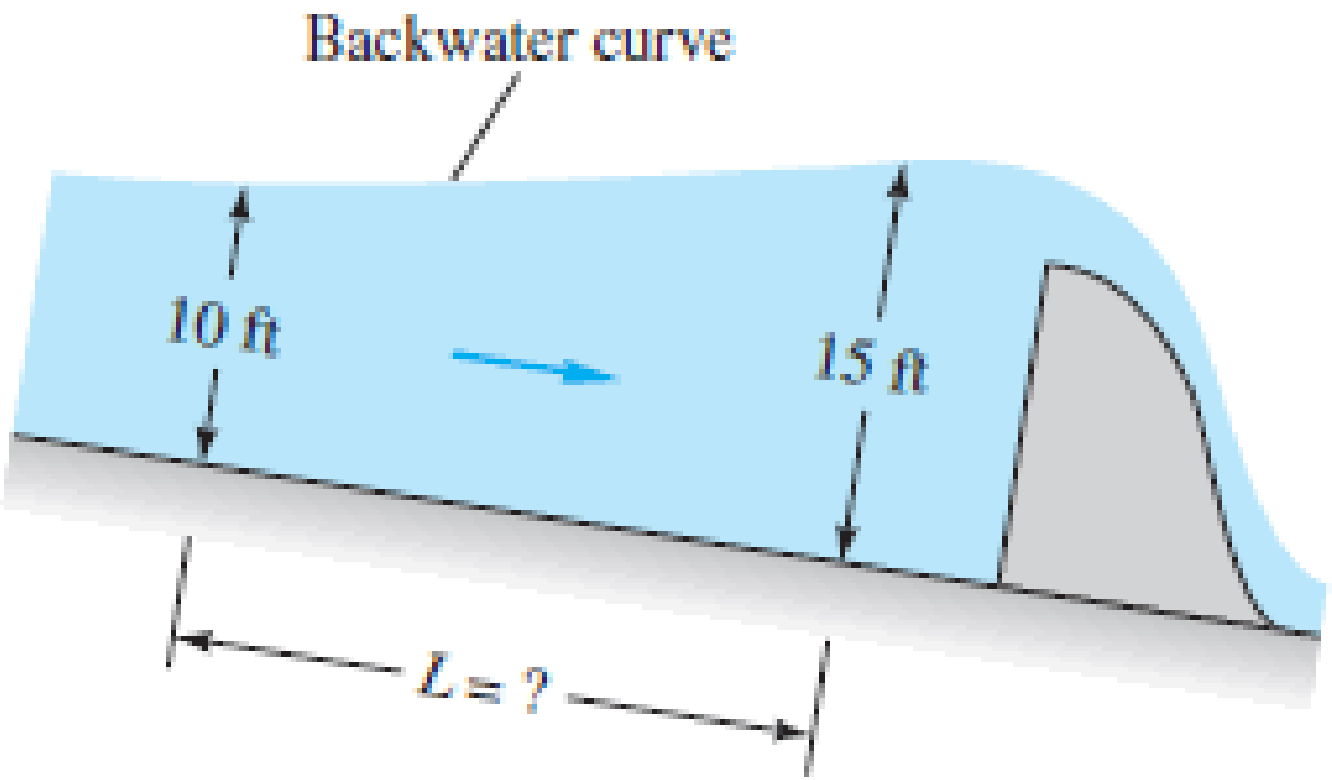
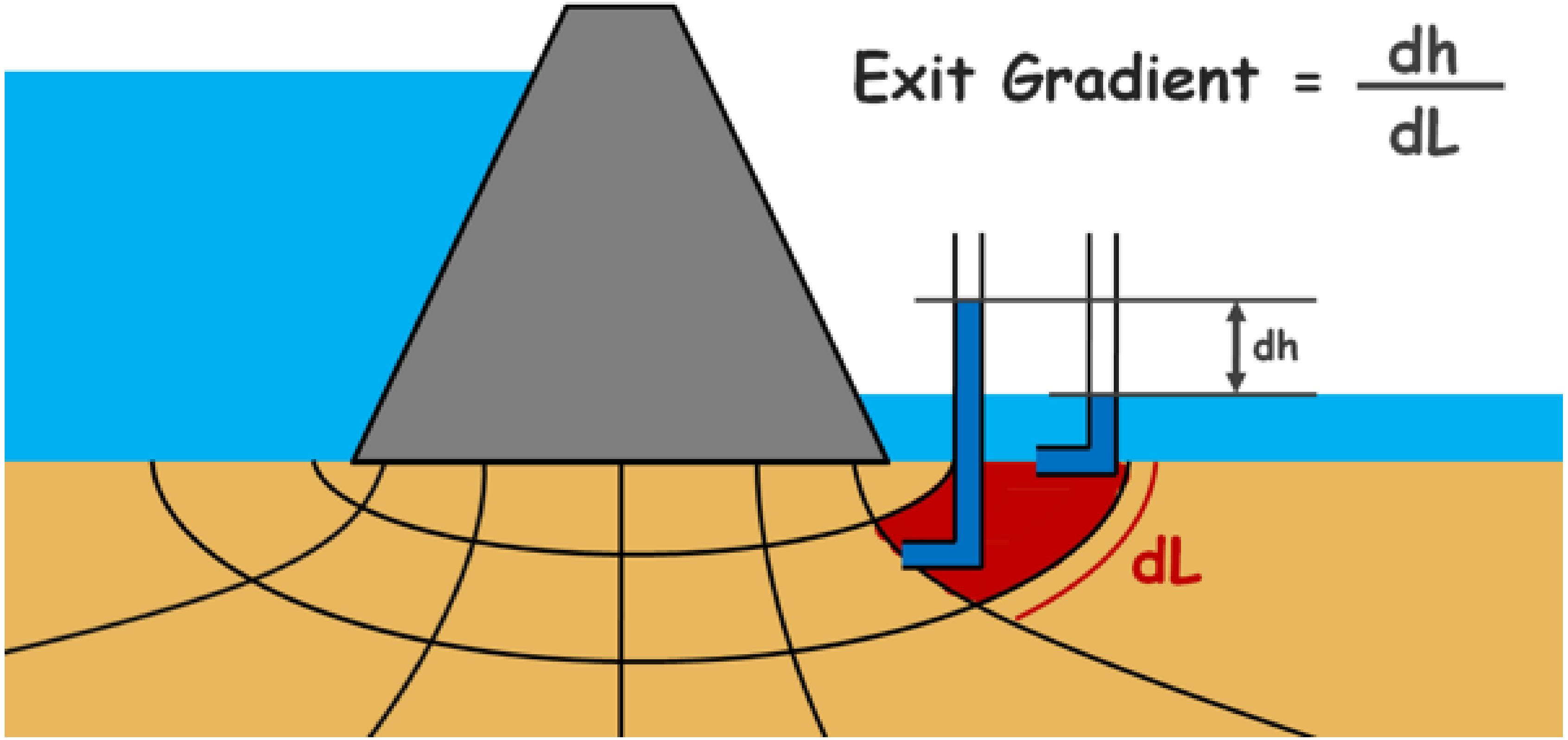

ANATOMY OF AN
Inverted Siphon
CROSS-SECTION




Falling Water Level
Stepped Bed of Fall


Grout pump
Possible cavities


Backwater curve
10 ft
15 ft
L = ?


Exit Gradient = $\frac{dh}{dL}$
dh
dL

DESIGN OF HYDRAULIC STRUCTURES

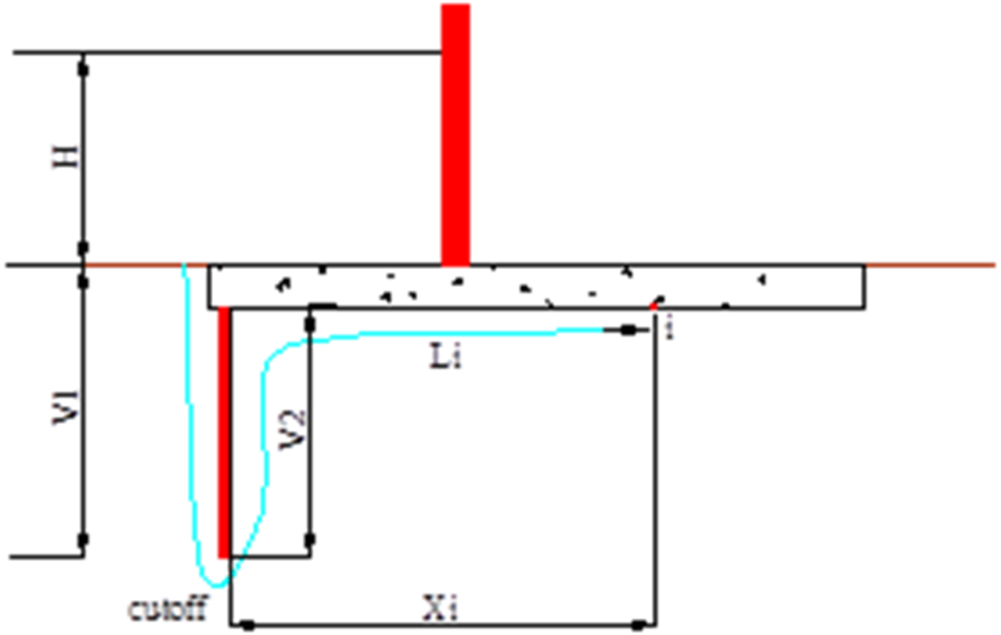
Lane’s Weighted Creep Method

كلية الهندسة /قسم هندسة الموارد المائية

المدرس : أ.م.د.أمين محمد صالح أمين

In calculating the length of seepage path, Lane assigns a weight of one to vertical creep distances, one-third to horizontal distances and intermediate values to inclined distances. The length of creep path is calculated as:

$$L_W = \frac{\sum H}{3} + \sum V = Cv\ \Delta h$$



L_w = weighted Creep Length

$\sum H$ = Cumulative horizontal length of percolation line.

$\sum V$ = Cumulative vertical length of percolation line.

Δh = Maximum Static head of water (upstream water level minus exit level) no flow

C_v = weighted creep ratio

Table (1). Recommended Weighted-creep Ratios   Ref.: USBR (Design of Small Dams)

Material	Ratio
Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel including cobbles	3.0
Boulders with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

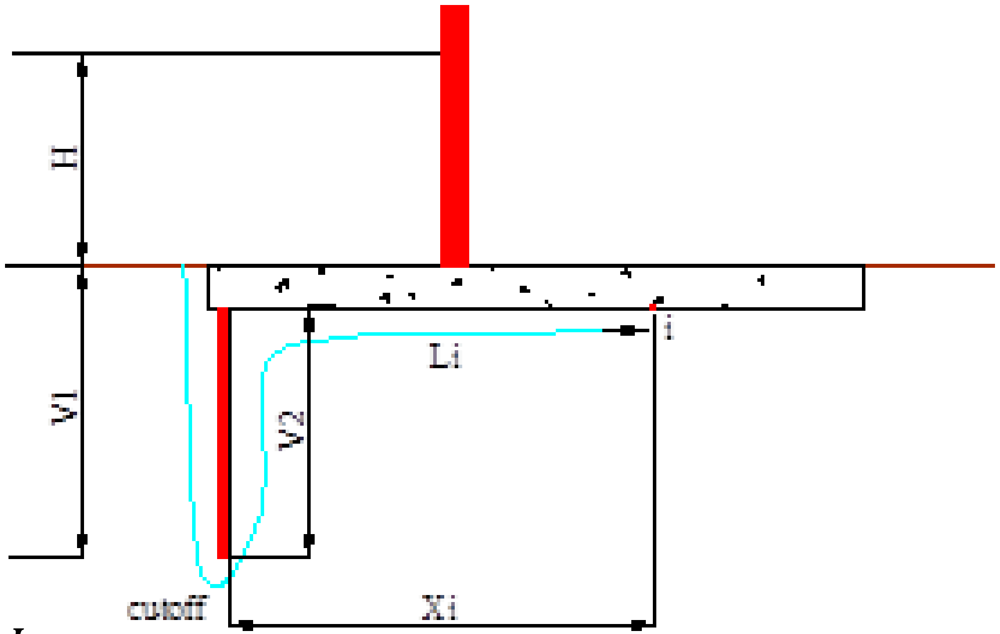
To ensure Safety against piping under the structure, L_w must not less than $C_v * \Delta h$.

$L_w \geq C_w * H$

The uplift pressure by this method is calculated for different points along the path as:

$$Pi = \frac{100\,(Lw - Li)}{Lw}\ and$$

$$Ui = \Delta h \times Pi$$



Pi = the Percentage of seepage pressure.

Li = Sum of the weighted creep length to point.

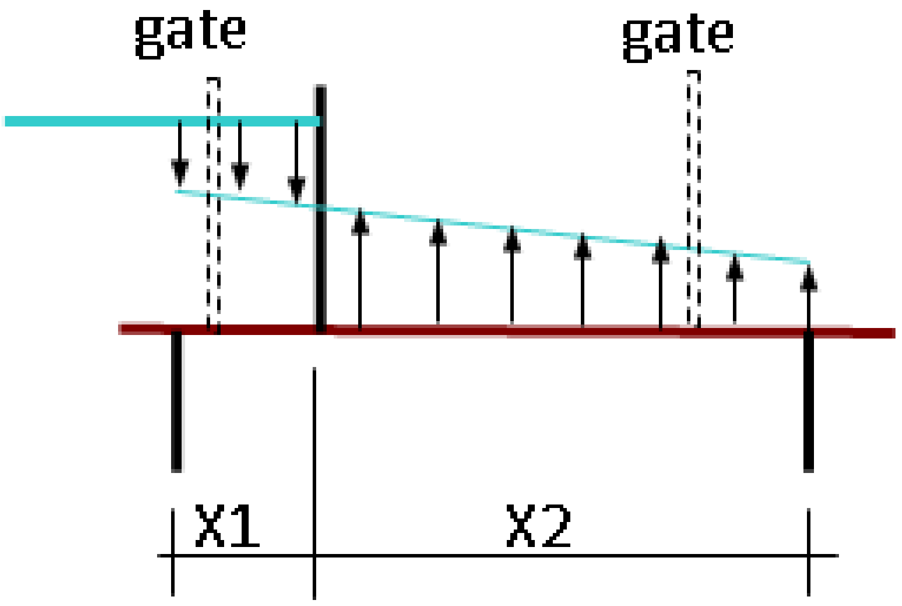
$$L_i = V_1 + V_2 + \frac{xi}{3}$$

Ui = uplift pressure at point

$$L_W = \sum L_V + \sum \frac{L_H}{3}$$

Location of Gate:

1. Moving U/S. (in creasing X_2) means increasing floor quantity.
2. Moving gate D/S. (reducing X_2) means reducing quantity of floor concrete.



Cutoff depth:

The cutoff can be designed according to depth of scour criteria,

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

R = scouring depth in (m)

f = silt factor obtained from the equation

d = median size of bed material particles

q = discharge per unit width in $m^3/s/m$

$$f = 1.76\sqrt{d_{mm}}$$

Minimum depth of U/S and D/S cut off.

Canal discharge or (capacity)	Min depth of cutoff below ground level
Up to 3 cumecs	1 – 1.25 m
3.1 to 30 cumecs	1.25 – 1.50
30.1 to 150 m^3/s	1.50 – 1.75 m
Above 150 m^3/s	Use 2 m

The depth of cutoff **D** is computed from

$$D = (XR - y)$$

Where X = factor of safety.

= 1.25 for upstream and = 1.5 for downstream

y = depth of water

The scour depth R may be also obtained from special charts as the one shown.

Example Problem

Given the following data for a regulator site: -

U/S conditions:

U/S W.L.	31.78 m
U/S B.L.	28.00 m
U/S canal B.W.	12 m
U/S side slope	1.5H: 1V
U/S Manning's (n)	0.021
U/S discharge	56 cumecs
Silt factor (f)	0.70

D/S conditions:

Water level at full discharge	31.25 m

Other data are the same as U/S canal.

Lane’s coefficient of creep C_w	8
Total length of concrete floor L_H	32.60 m

Other data are the same as U/S canal.

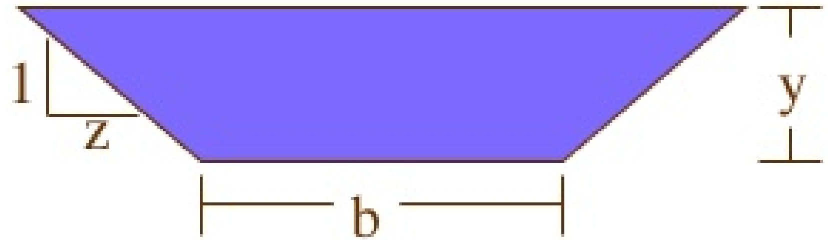
Lane’s coefficient of creep C_w 8

Total length of concrete floor L_H 32.60 m

Using the above information, solve the following: -

1. How much is the velocity of approach when the regulator gate is fully opened?

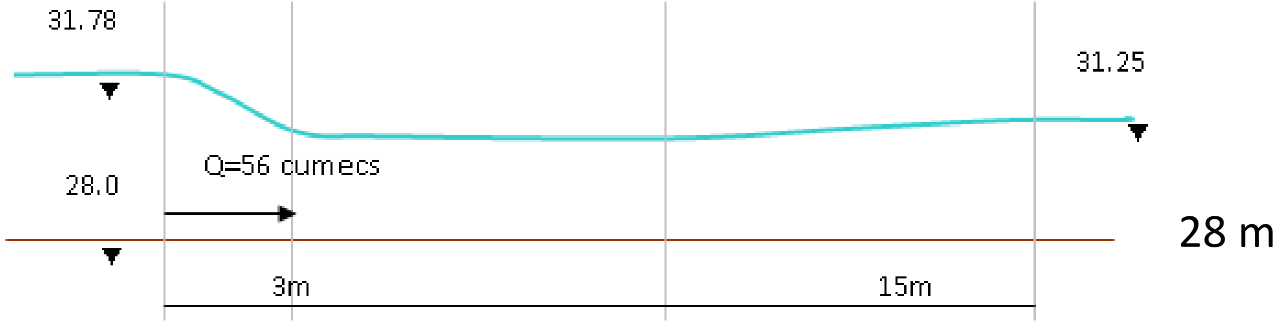
A = (B+Z*D)*D = (12+ 1.5*3.78)*3.78 = 66.79 m²



$$Va = \frac{56}{66.79} = 0.838\,\text{m/sec}$$

$$ha = \frac{Va^2}{2g} = \frac{(0.838)^2}{19.6} = 0.036$$

2. How much is the needed regulator width (S) for a fully opened regulator to pass the given discharge at the given water levels?



h = 31.78 – 31.25 = 0.53 m

ha ≈ 0.04 m

D = 31.25 – 28 = 3.25 m

$$Q = C * S * D\sqrt{2g(h + ha)}$$

$$56 = 0.9S(3.25)\sqrt{2 * 9.81(0.53 + 0.04)}$$

$$\Rightarrow S = 5.6\text{m}$$

3. Assuming a regulator opening of (6m) width (S), calculate the needed U/S and D/S cutoff depths.

$$q = \frac{56}{6} = 9.333 \quad m^3/\text{sec}.m$$

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3} \qquad R = 1.35\left(\frac{9.333^2}{0.7}\right)^{1/3} = 6.73 \text{ m}$$

$D_{D/S}$ = XR-Y = 1.5(6.73)-3.25 = 6.85 m

$D_{U/S}$ = XR-Y =1.25 (6.73)-3.78 = 4.63 m

4. Check by Lane's method to show whether your selected depths are acceptable or not?

$\therefore$ Selected U/S depth =4.60 m and D/S depth = 6.85 m

H= max. static head = 3.75 m

Needed $L_w = C_w * H$ = 8 * 3.78 = 30.24 m

Available L_w = $\frac{32.6}{3} + 2(4.63 + 6.85)$ =33.83 m Since this is more than needed, $\therefore$ design is accepted

5. Using a D/S and U/S cutoff depths of (10.0 m) and (5.0 m), respectively, plot the uplift pressure diagram showing values at important points.

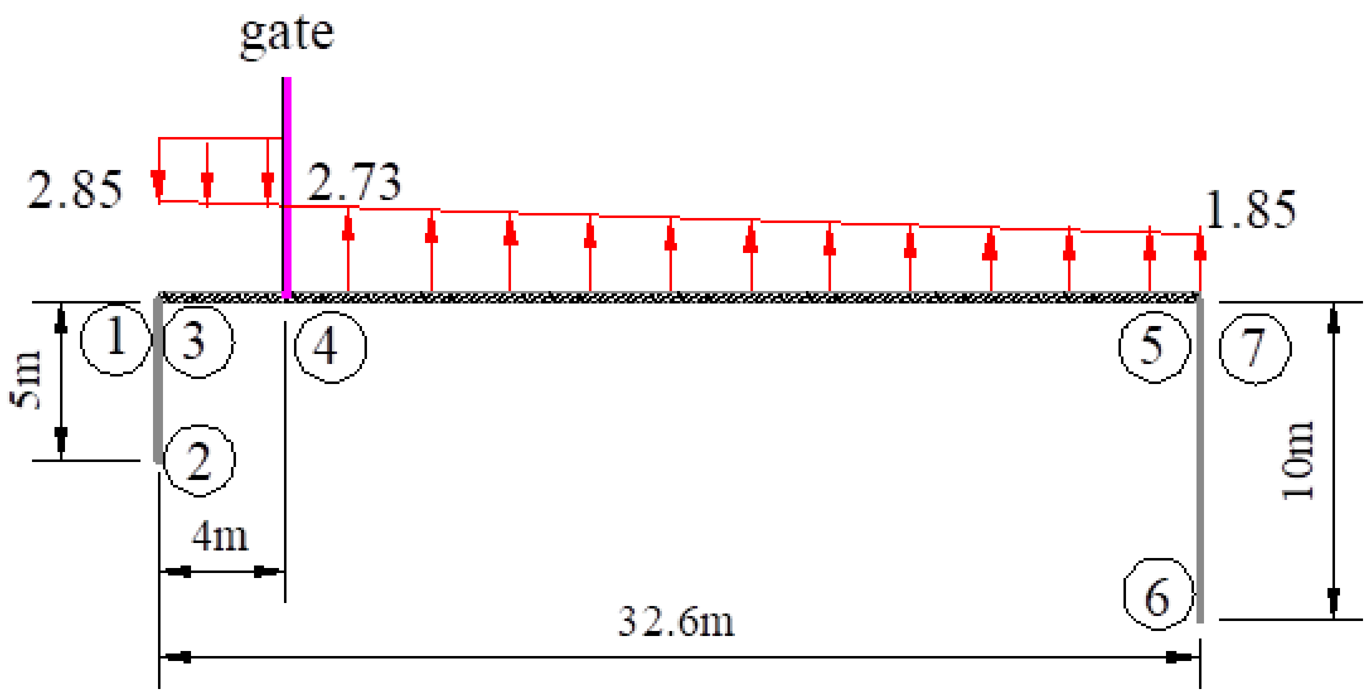
$$L_w = \frac{32.6}{3} + 20 + 10 = 40.86 \text{ m}$$

U.P. at 7 = zero

$$\text{At 5} = \left(\frac{3.78}{40.86}\right)(20) = 1.85 \text{ m}$$

$$\text{At 4} = \left(\frac{3.78}{40.86}\right)\left[20 + \left(\frac{32.6-4}{3}\right)\right] = 2.73\text{m}$$

$$\text{At 3} = \left(\frac{3.78}{40.86}\right)\left[20 + \left(\frac{32.6}{3}\right)\right] = 2.86 \text{ m}$$



6. Calculate the needed floor thickness at:

(a) U/S of gate.

(b) Under the gate.

(c) At the U/S side of the D/S cutoff.

$t_{max.}$ (under the gate) = $\frac{h_4}{G-1} = \frac{2.73}{2.4-1}$ =1.8 m ≈ 2 m

t at end = $\frac{h_5}{G-1} = \frac{1.85}{2.4-1} \approx 1.25\text{m}$

$t_{min.}$ at beginning = $\frac{t_{max}}{3} \approx 0.60\text{m}$

= 0.65(6)(0.31) =10.3 m3 /sec

7. If the gate width (S) is 6.0 m and the opining height is 0.31m; calculate the velocity through the gate and the total discharge Q, using full supply U/S W.L. (neglect ha).

If S = 6 m, D = 0.31 m, h_a is negligible

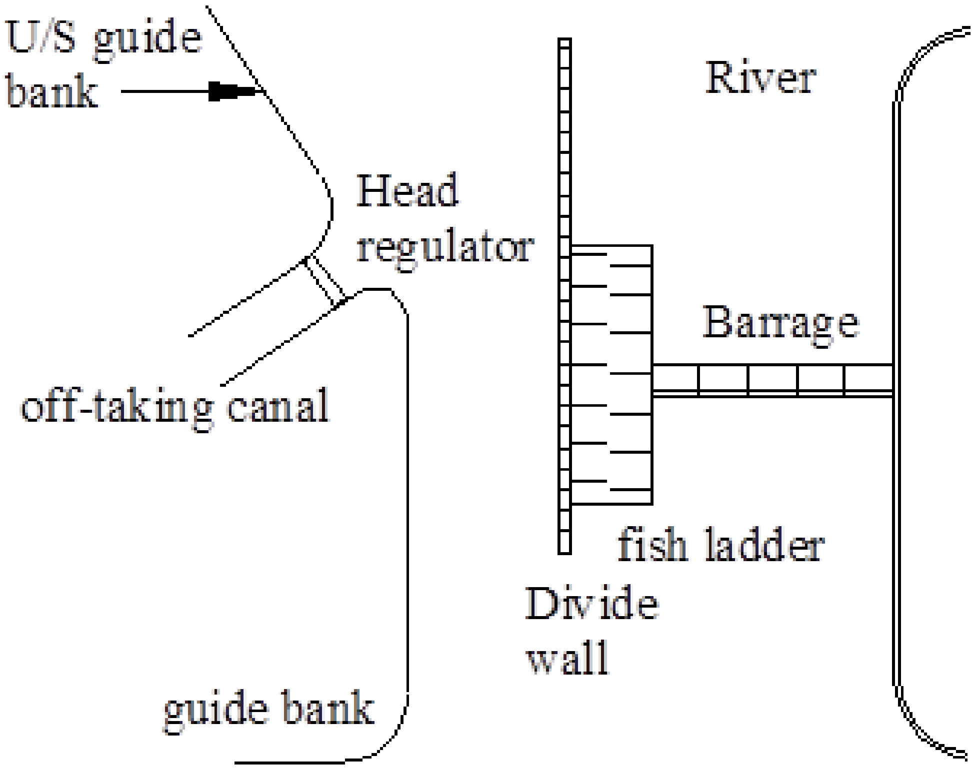
$$Q = CSD\sqrt{2gH} = 0.65(6)(0.31)\sqrt{2*9.81(3.78)} = 10.3 \text{ m3 /sec}$$

Velocity at vena contracts = $\frac{Q}{A} = \frac{Q}{0.31C_c(6)} =$

Component parts of diversion headwork

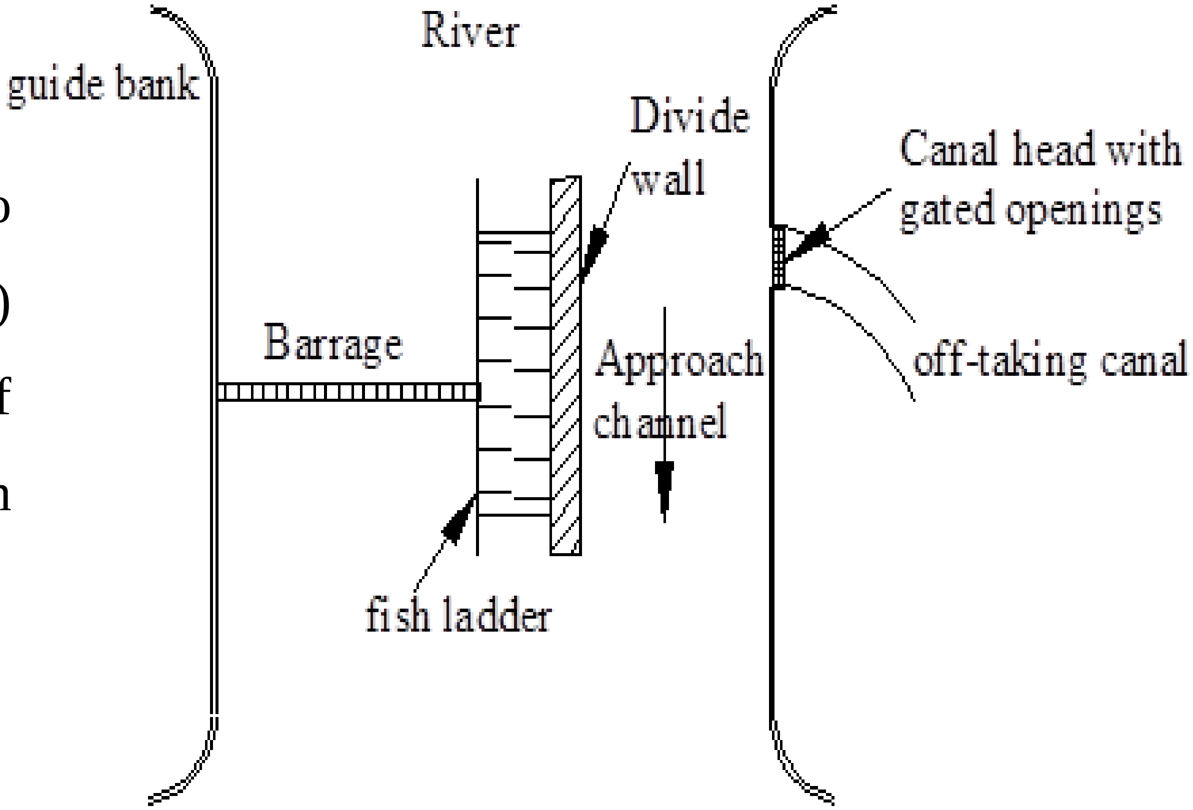
Diversion headwork or a River regulator: -

The works which are constructed at the head of the canal, in order to divert the river water towards the canal (off-taking canal) so as to ensure a regulated continuous supply of silt-free water with a certain minimum head into the canal.



The Barrage consists of the following parts: -

1. Weir or barrage.
2. Divide wall, dividing the river width into two portions; one is called the (weir or barrage portion) and the other portion from which canal takes off. If there are two canals, taking off from each bank, then there will be two divide walls.
3. Fish ladder.
4. Approach channel.
5. Canal head regulator.



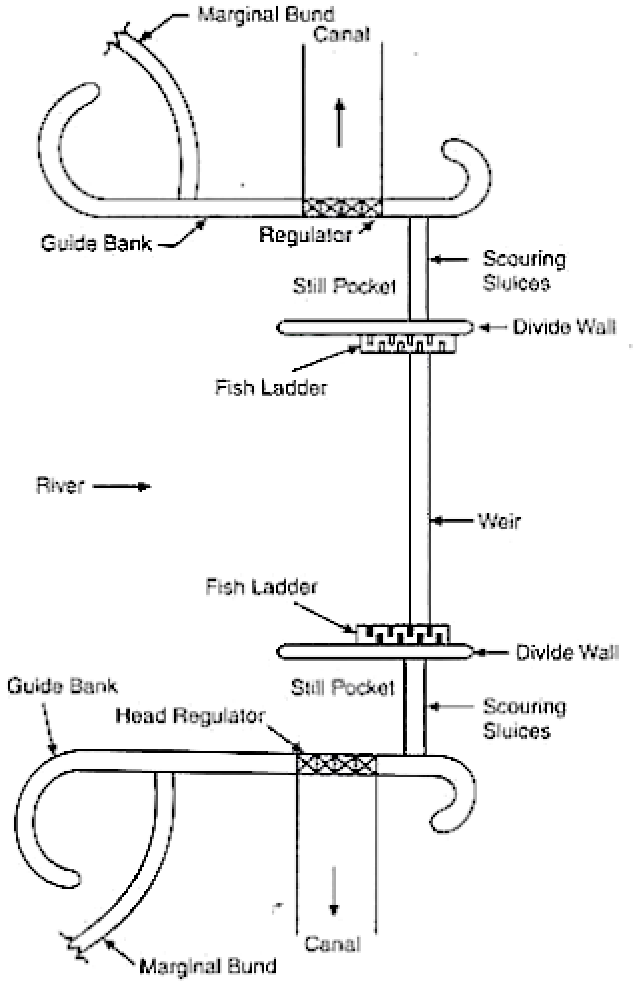


Fig. 12.1. Layout of headworks

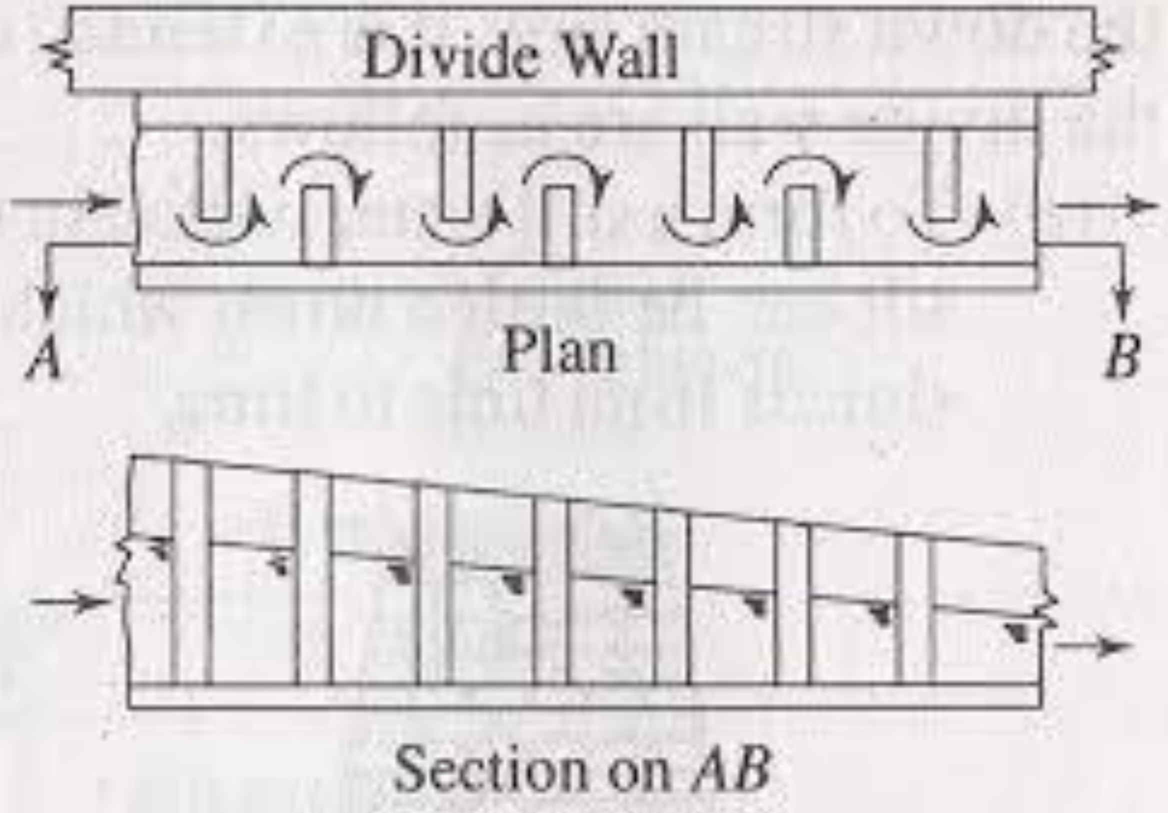
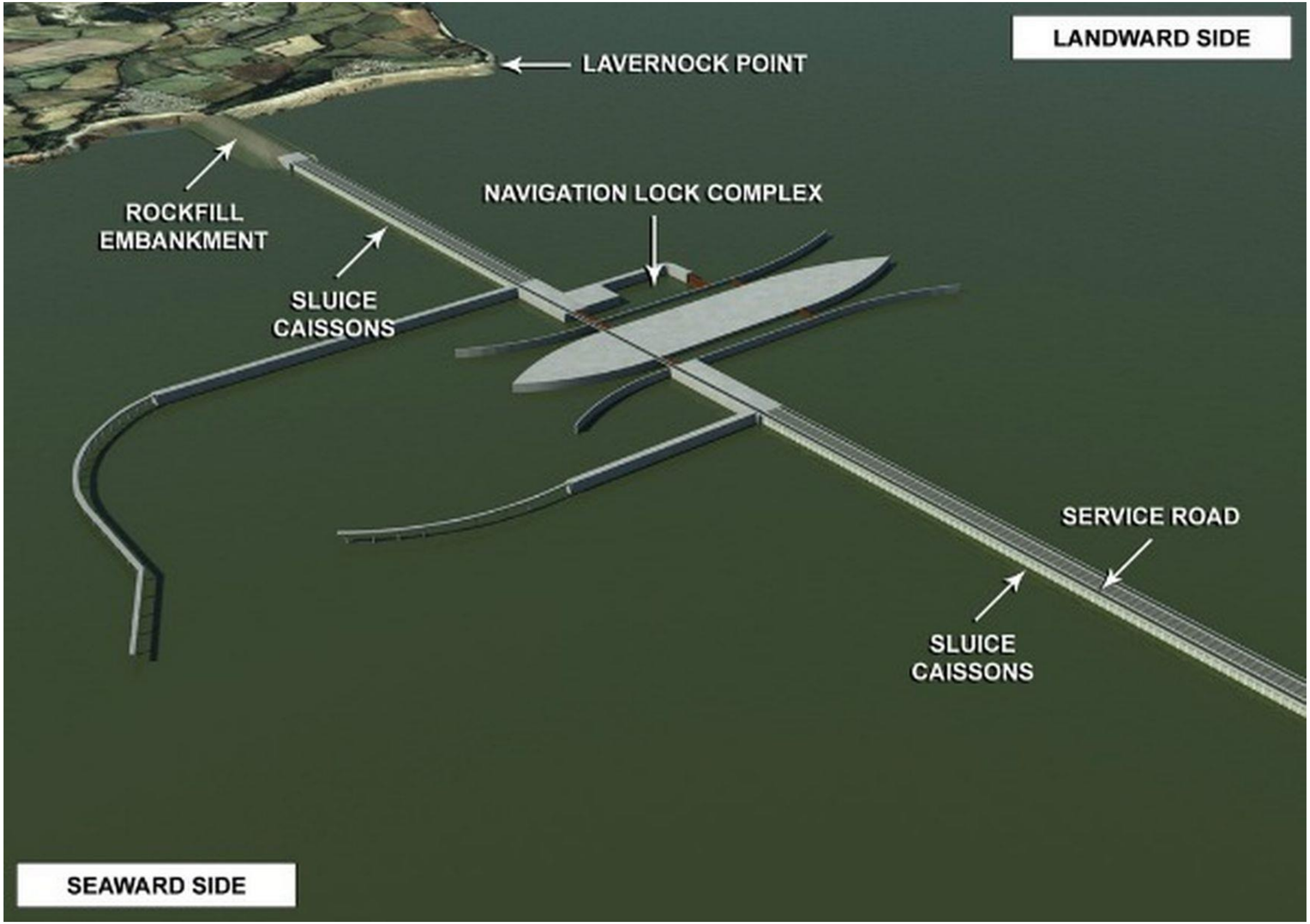


Fig: Fish Ladder


River Nile
Island
New barrage
Hydropower
plant
Navigation lock
Old barrage
River Nile


LANDWARD SIDE
LAVERNOCK POINT
ROCKFILL EMBANKMENT
NAVIGATION LOCK COMPLEX
SLUICE CAISSONS
SERVICE ROAD
SLUICE CAISSONS
SEAWARD SIDE

University of Baghdad
Collage of Engineering
Department of Water Resources

شكرا لاصغائكم

Khosla's Theory

Khosla's theory is the solution of Laplace equation by the method of independent variables. The results have been presented in the form of curves.

Khosla's arrived at the following important conclusions

a. The outer faces of the end sheet piles are much more effective than the inner ones and the horizontal length of the floor.
b. The intermediate sheet pile, if smaller in length than the outer ones, is ineffective except for local redistribution of pressure.
c. Undermining of floors start from the tail end. If the hydraulic gradient at exit is more than the critical gradient for the subsoil, the soil particles will move with the flow of water thus causing progressive degradation of the subsoil, resulting in cavities and ultimate failure.
d. If is absolutely essential to have a reasonable deep vertical cutoff at the downstream end to prevent undermining.

Khosla's Curves:

Let $\frac{b_1 + b_2}{d} = \alpha$

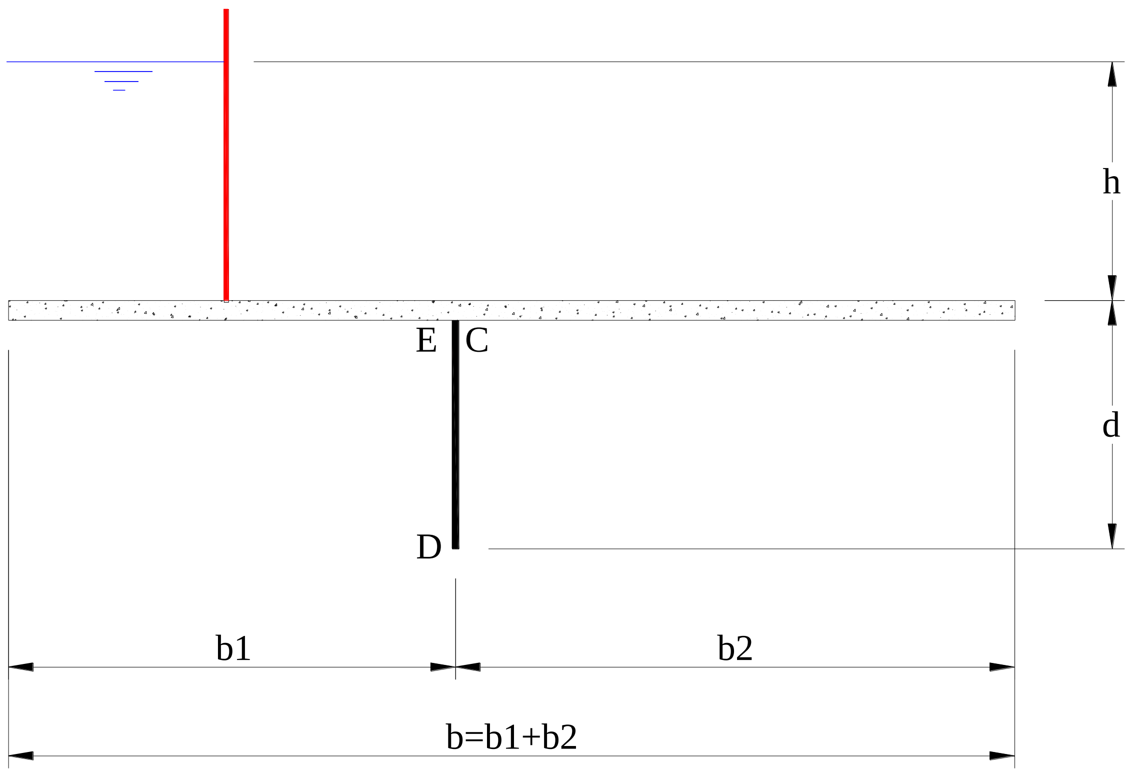
$b_1 + b_2 = b$

$$\sqrt{1 + \left(\frac{b_1}{d}\right)^2} = m$$

$$\sqrt{1 + \left(\frac{b_2}{d}\right)^2} = n$$

$$\lambda = \frac{m + n}{n}$$

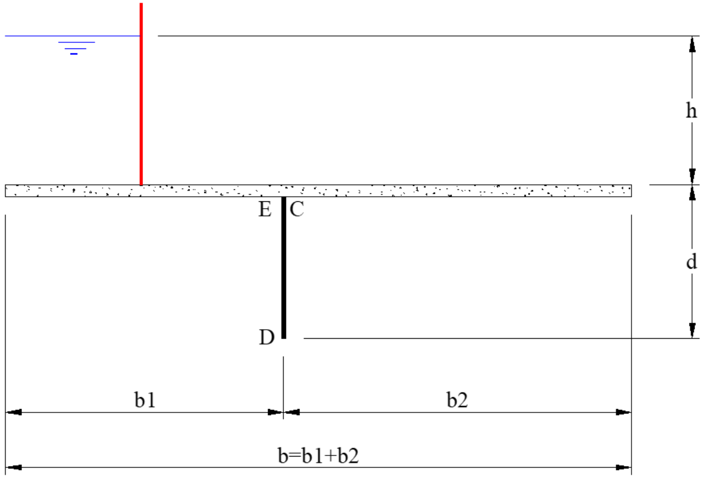
$$\lambda_1 = \frac{m - n}{2}$$



$$\phi_c = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$ also $$\phi_c = 0.318 * \cos^{-1}\left(\frac{m-n+2}{m+n}\right)$$

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right) \qquad \phi_E = 0.318 * \cos^{-1}\left(\frac{m-n-2}{m+n}\right)$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right) \qquad \phi_D = 0.318 * \cos^{-1}\left(\frac{m-n}{m+n}\right)$$



If pile is at end $b_2 = 0$, $b_1 = b$

$$m = \left(1 + \left(\frac{b}{d}\right)^2\right)^{1/2} \qquad n = 1$$

$$\lambda = \frac{m+1}{2}$$

$$\lambda_1 = \frac{m-1}{2}$$

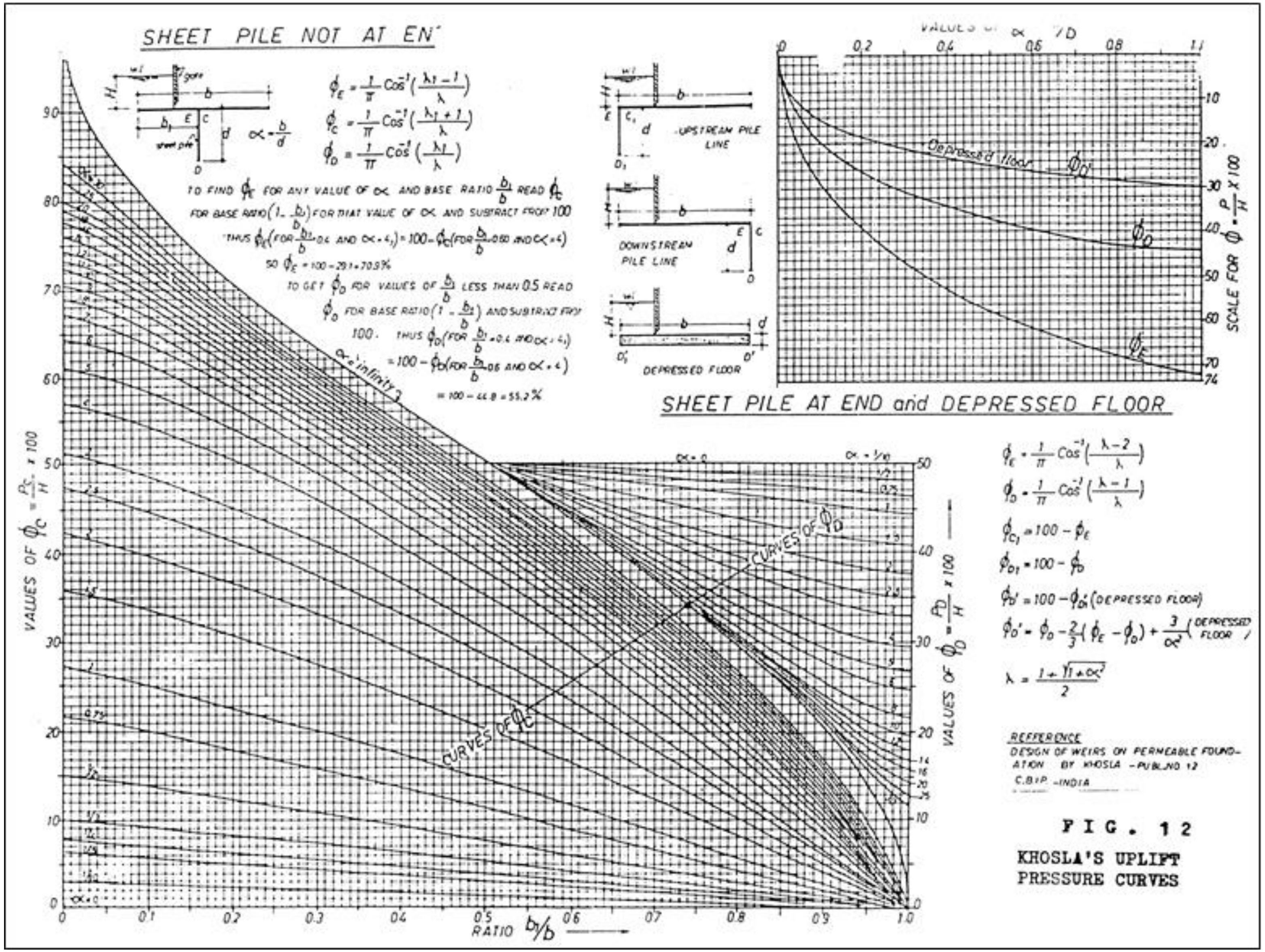


FIG. 12

KHOSLA'S UPLIFT PRESSURE CURVES

Using of Khosla’s Curves :-

The previous Figure (11) shows the mathematical solution of flownets. For Khosla’s simple standard profiles which have been presented in the from of curve (Fig 11); this Fig. is used for determining the percentage pressure at the various Key points the simple profiles which are most useful are: -

(1) A straight horizontal floor of negligible thickness with a sheet pile line on the U/ S end or D/S end.
(2) A straight horizontal floor depressed below the bed but without any vertical cut off.
(3) A straight horizontal floor of negligible thickness with a sheet pile line at some intermediate point.

Exit Gradient

Can be defined as local hydraulic gradient dh / dl at the exit end. According to Khosla, exit gradient is

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

Where: H = the head causing seepage. (maximum static head)

d = the depth of the downstream sheet pile below the top level of the floor.

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

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

If d = 0, $G_E = \infty$ and the floor will be unsafe against piping however long the floor may be (contrary to Bligh's theory). Hence there should always be a D.S. cutoff whether the floor is designed by Khosla's' theory or by Bligh's theory.

The length of the floor (**b**) has very little effect on $\mathbf{G_E}$ as compared to (**d**). Therefore, to reduce the exit gradient, it is much more economical to **increase (d)** than to increase (b). The value of critical exit gradient is generally 1 (for porosity 0.4) and assuming a factor of safety of 3 to 4, G_E = 0.33 or (the maximum permissible value is 0.4).

Note: All hydraulic structures on permeable foundation should be designed by **Khosla's theory or Lane’s method**. However small structures may by designed by Bligh's theory, but D/S. cutoff should always be provided and G_E, the exit gradient should invariably be checked. **Important structures may also be checked by the flow net analysis.**

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 the case of a depressed floor. The percentage pressure 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 :-

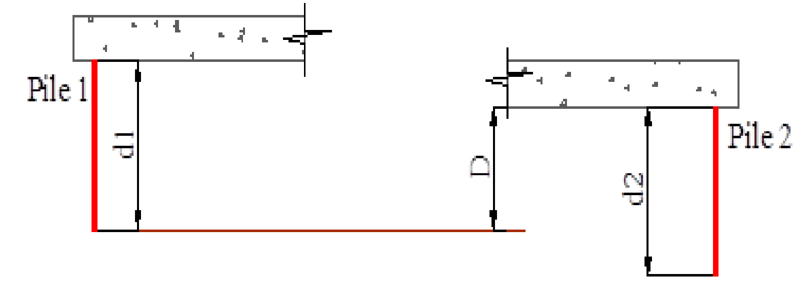
1. Correction for the mutual interference of piles.
2. Correction for the thickness of floor.
3. Correction for the slop of the floor.

These corrections are described below: -

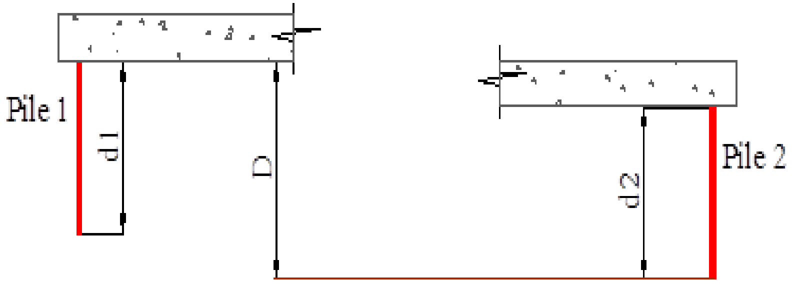
1. **Correction for the Mutual Interference of Piles: -**

Due to this effect is given by:

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



Effect of pile (1) on pile (2)



Effect of pile (2) on pile (1)

Where:b′= the distance between the two pile lines.

d = the depth of pile on which. The effect is can side red.

b = total floor length.

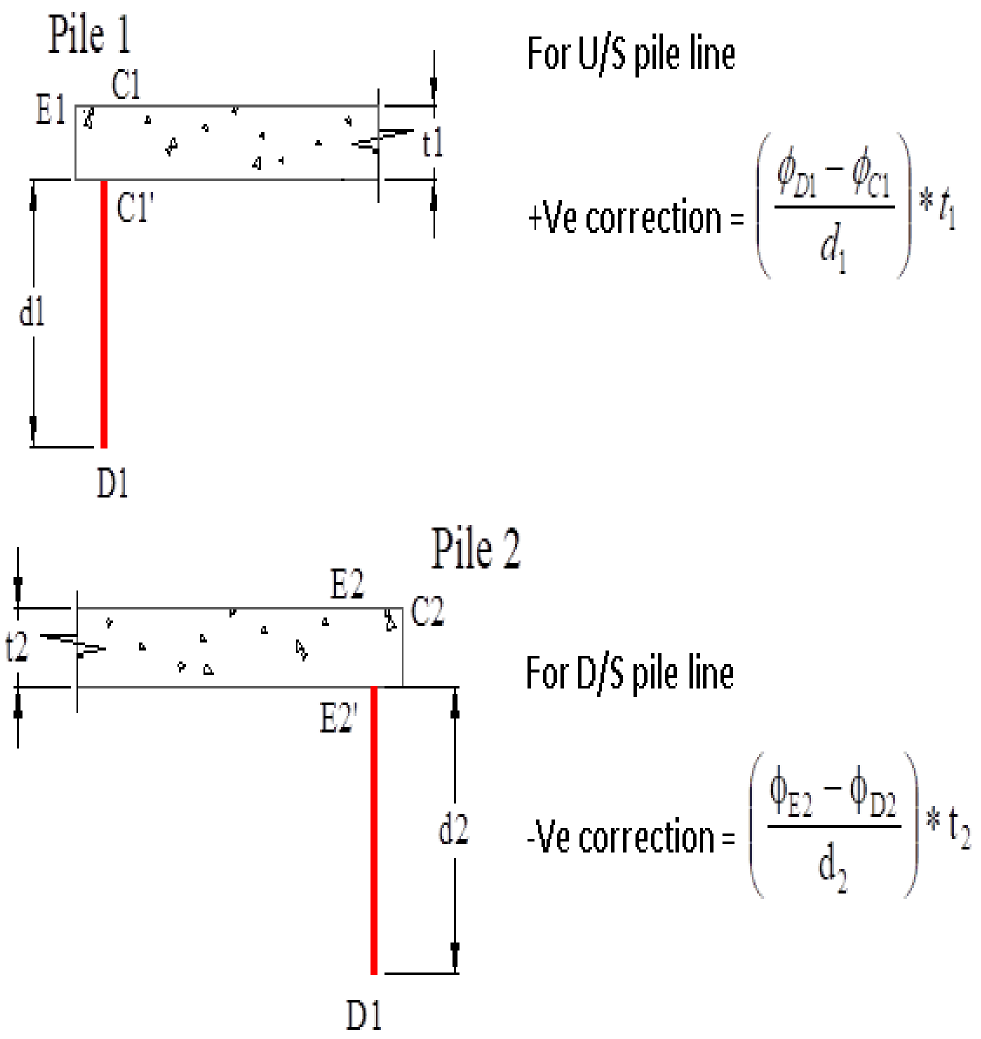
D = the depth of the influence of which has to be determined on the neigh boring pile of depth d. (as shown below)

This correction is (+Ve) for the points in the rear or backwater and (-Ve) for the points forward in the direction of flow.

2. Correction for the thickness of floor

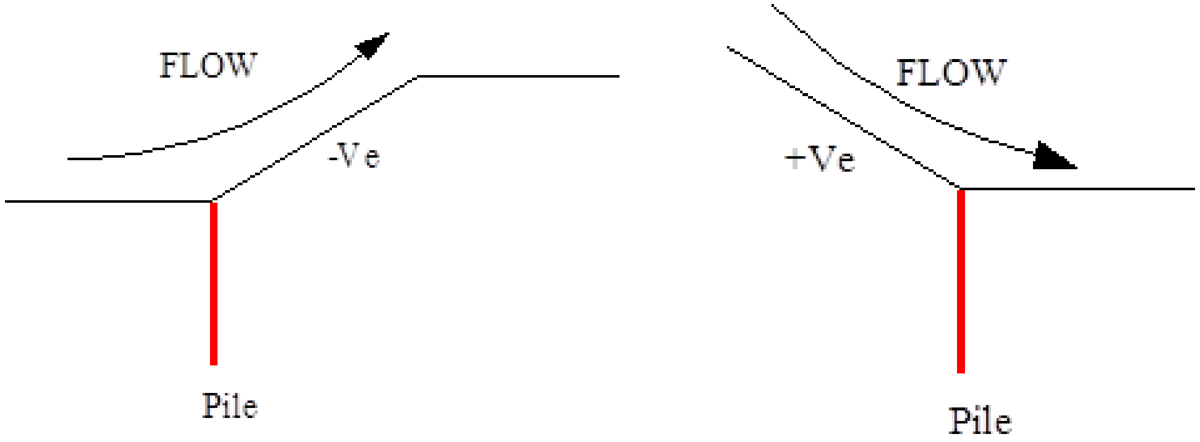
In the standard form profiles, the floor is assumed to have negligible thickness. Hence, the percentage pressure calculated by Khosla's graphs shall pertain to the top levels of the floor. While the actual junction points E and C are at the bottom of the floor. Hence, the pressures at the actual points are calculated by assuming a straight-line pressure variation.

Since the corrected pressure at E_2' should be less than the calculated pressure at E_2, the correction to by applied for the point E_2 shall by (-Ve) .Similarly, the pressuse calculated at C_1 is less than the corrected pressuse at C_1', and hence, the correction which applied at point C_1 is (+Ve) as shown below.



3. 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 fallowing the direction of flow.



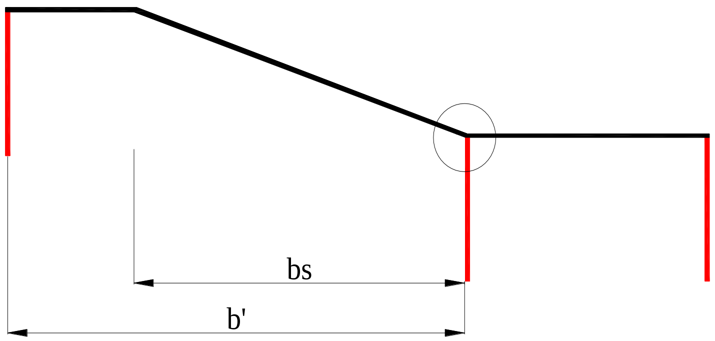
Values of correction for stab and slopes are given in the table below: -

b_s = horizontal length of the slope

b\` = distance between two pile lines between which the sloping floor is located.

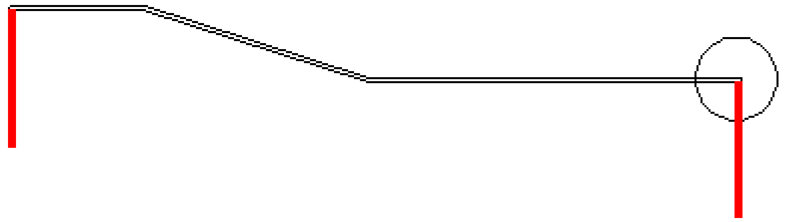
$$C = \left(\frac{b_s}{b'}\right) * \text{(correction value from the previous table}$$

Slope H: V	Correction factors
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



<u>Note:</u> This correction is applicably only to the Key points of the pile line fixed at the beginning or the ends of the slope.

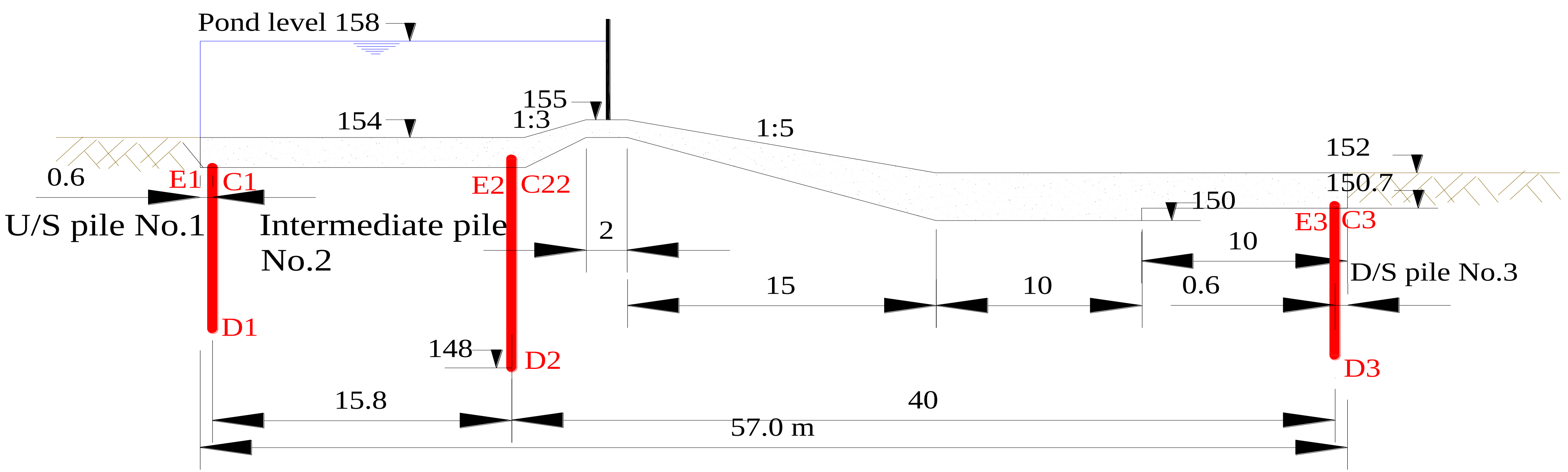
i.e. There is no correction of slope for this case



Example

Given: Q = 760 cumecs , Water way = 40 m, High flood level = 160 m U/S ,

D/S.W.L = 156.1 m, Pond level = 158 m, Silt factor (f) = 1.0



Required:

1. Calculate the depth of sheet pile and elevations.
2. Determine the percentage of pressure at key points.
3. Determine the exit gradient.
4. Plot the hydraulic grade line for pond level on U/S and no flow on D/s.
5. If safety factor F = 8; check horizontal floor length.

Solution:

1. **Depth of sheet pile: -**

Q = 760 m^3/sec, water way = 40 m

q = discharge intensity m^3/sec.m = 760 / 40 = 19 m^3/sec.m

Scouring depth $R = 1.35\left(\frac{q^2}{f}\right)^{1/3} = 1.35\left(\frac{19^2}{1}\right)^{1/3} = 9.6m$

D = XR $- y$

D = Depth of cutoff.

X = factor of safety coeff.

= 1.25 for upstream and = 1.50 for downstream

y = depth of water

D/S cutoff = 1.5 (9.6) = 14.4 m

Max. D/S W.L = 156.1 m

Bottom of sheet pile = 156.1-14.4 = 141.7 m (downstream cutoff level)

U/S cutoff = 1.25 (9.6) = 12 m

Level of bottom U/S cutoff = max. U/S.W.L -12

= 160 - 12 = 148 m

2. **Uplift pressure: -**

U/S pile line No.1:

Total length of floor = b = 57.0 m

Depth of U/S pile line = d= 154.0 – 148.0 = 6.0 m

$$\alpha = \frac{b}{d} = \frac{57}{6} = 9.5$$

$$\frac{1}{\alpha} = \frac{1}{9.5} = 0.105$$

From Khosla's curves

$$\phi_{E1} = 100\%$$

$$\phi_{C1} = 100\% - \phi_E = 100\% - 29\% = 71\%$$

$$\phi_{D1} = 100\% - \phi_D = 100\% - 20\% = 80\%$$

Equations may be used instead of Khosla's charts as follows

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) \qquad \lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + (9.5)^2}}{2} = 5.276$$

$$\phi_E = 28.67\%$$

$$\phi_{C1} = 100\% - \phi_E = 100\% - 28.67\% = 71.33\%$$

Also $\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{5.276 - 1}{5.276}\right) = 19.92$

$$\phi_{D1} = 100\% - \phi_D = 100\% - 19.92\% = 80.08\%$$

or

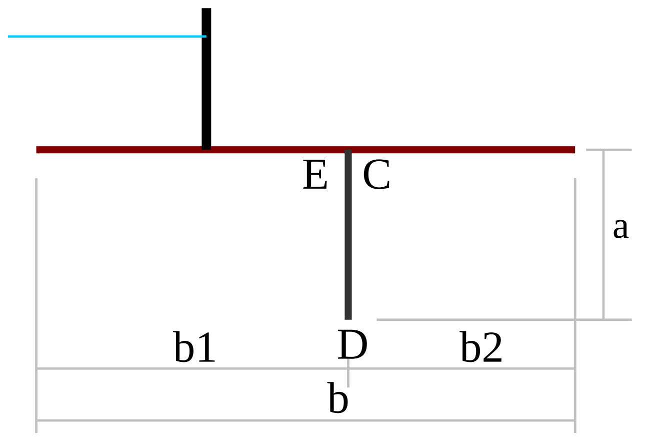
$$\phi_E = 0.318\cos^{-1}\left(\frac{m-n-2}{m+n}\right)$$

$$\phi_C = 0.318\cos^{-1}\left(\frac{m-n+2}{m+n}\right)$$

$$\phi_D = 0.318\cos^{-1}\left(\frac{m-n}{m+n}\right)$$

$$\sqrt{1+\left(\frac{b_1}{d}\right)^2} = m$$

$$\sqrt{1+\left(\frac{b_2}{d}\right)^2} = n$$



For Upstream pile

b_1 =zero and b_2 = b

$$m = \left[1+\left(\frac{b_1}{d}\right)^2\right]^{1/2} = 1$$

$$n = \left[1+\left(\frac{b_2}{d}\right)^2\right]^{1/2} = \left[1+\left(\frac{55.8}{6}\right)^2\right]^{1/2} = 9.354$$

$$\phi_{E1} = 0.318\cos^{-1}\left(\frac{1-9.354-2}{1+9.354}\right) = 100\%$$

$$\phi_{C1} = 0.318\cos^{-1}\left(\frac{1-9.354+2}{1+9.354}\right) = 70.96\%$$

$$\phi_{D1} = 0.318\cos^{-1}\left(\frac{1-9.354}{1+9.354}\right) = 79.88\%$$

The value of ϕ_{C1} must be corrected for three correction as below: -

(a). Correction due to effect of pile No. 2 on pile No. 1: -

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

Where: -

D = 153.0 – 148 = 5 m

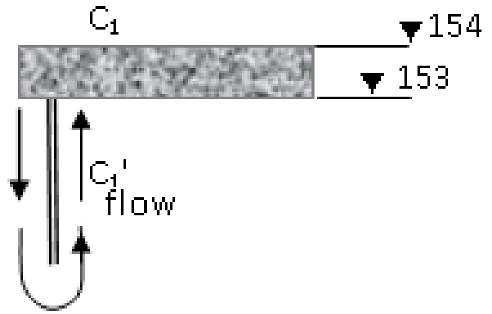
d = depth of pile 1 = 153.0 – 148.0 = 5 m

b′ = distance between two piles = 15.8 m

b = total floor length = 57.0 m

∴Correction $C = 19\sqrt{\frac{5}{15.8}}\left(\frac{5+5}{57}\right) = 1.88\%$ Since C_1 is in the rear direction of flow, hence the correction is +Ve

(b). Correction at C_1 due to thickness of floor.



Pressure calculated from curve is at C_1, but we want the pressure at C_1' The pressure at C_1' is more than at C_1 as the direction of flow is as shown, hence, the correction shall be +Ve and

$\left[\frac{\phi D_1 - \phi C_1}{d_1}\right] t_1 = \left[\frac{80\% - 71\%}{154 - 148}\right]$ ***(154-153) = 1.5 % +Ve**

(c). Correction due to slope at C_1 is nil.

∴Corrected $\phi_{C1} = 71\% + 1.88\% + 1.5\% = 74.38\%$

Intermediate pile line

d = 154.0 -148.0 = 6 m

b = 57.0 m

α = b/d= 57.0/6=9.5

Using Khosla's curves, we have b_1 in this case = 0.6+15.8= 16.4

m and b= 57 m

b_1/b = 16.4/57.0= 0.288 and 1-(b_1/b) =1-0.288= 0.712

ϕE_2 =100% - 30% =70% Where 30% is ϕ_C for a base ratio of 0.712 and α = 9.5

ϕC_2 =56% (for base ratio 0.288 and α =9.5)

ϕD_2 = 100% - 37% = 63% Where 37% is ϕ_D for a base ratio of 0.712 and α = 9.5

Correction for ϕE_2 : -

- Pile No. 1 will affect the pressure at E_2. Since E_2 is in the foreword direction of flow, this correction shall be –Ve. The amount of this correction is given by:

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

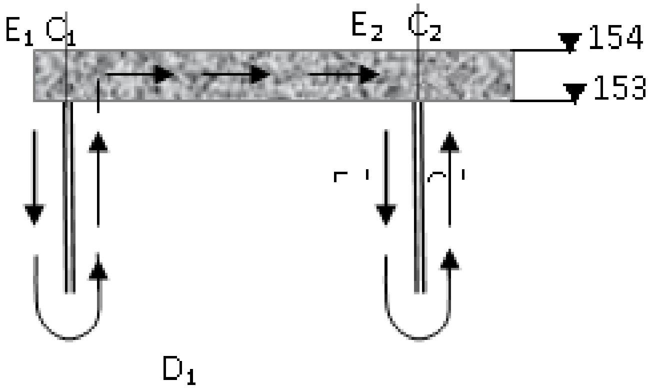
Where

D = 153.0 – 148 = 5 m

d = depth of pile No. 2 = 153.0 – 148.0 = 5 m

b′ = distance between two piles = 15.8 m

b = total floor length = 57.0 m



Correction $C = 19\sqrt{\frac{5}{15.8}}\left(\frac{5+5}{57}\right) = 1.88\%$ -Ve

- Correction at E_2 due to floor thickness

$$\left[\frac{\phi E_2 - \phi D_2}{d_2}\right] t_2 = \left[\frac{70\% - 63\%}{154 - 148}\right] *1 = 1.17\%$$

The pressure observed is at E_2 and not at E_2', and by looking at the direction of flow, it can be stated easily that the pressure at E_2' shall be less than at E_2 hence, this correction is negative.

Correction at E_2 due to slope is nil.

Hence, the corrected percentage pressure at E_2 = Corrected ϕE_2 = 70% - 1.88% - 1.17% = 66.95%

Correction for ϕC_2

Correction at C_2 due to pile interference. Pressure at C_2 is affected by pile No. 3. Since point C_2 is in the backward in the direction of flow, hence, this correction is +Ve. The amount of this correction is given by: -

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

D = 153.0 – 141.7 = 11.3 m

d = depth of pile No. 2 = 153.0 – 148.0 = 5 m

b' = distance between pile 2 and pile 3 = 40 m

b = total floor length = 57.0 m

$$C = 19\sqrt{\frac{11.3}{40}}\left(\frac{11.3+5}{57}\right) = 2.88\%$$ +Ve

Correction at C_2 due to floor thickness:

For the preview figure, it can be easily stated that the pressure at C_2' shall be more than that at C_2, and since the observed pressure is at C_2, this correction shall be +Ve and its amount is the same as we calculated for the point E_2 = 1.17%.

$$\left[\frac{\phi D_2 - \phi C_2}{d_2}\right] t_2 = \left[\frac{63\% - 56\%}{6}\right]$$ ***1=1.17%**

Correction at C_2 due to slope: -

Since point C_2 is at the beginning of a slope of 3:1, an up slope in the direction of flow, the correction is negative.

- Correction factor for 3:1 slop frame table = 4.5.
- Horizontal length of the slope = 3m
- Distance between two pile lines between which the sloping floor is located = 40 m

$\therefore$ Actual correction = 4.5(3/40) = 0.34% -Ve

Hence corrected ϕC_2 = 56% + 2.88% + 1.17% - 0.34% = 59.7%

D/S pile line:-

From Khosla's curves

d = 152.0 – 141.7 = 10.3

b = 57 m

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{10.3}{57.0} = 0.180$$

$\phi D_3 = 25\%$

$\phi E_3 = 37\%$

$\phi C_3 = \text{zero}\%$

Or by using the equations $\alpha = 5.534$ $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+(5.534)^2}}{2} = 3.312$

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{3.312-2}{3.312}\right) = 37\%$$

$$\phi_D = \frac{1}{\pi}\cos^{1-}\left(\frac{3.312-1}{3.312}\right) = 25.4\%$$

Correction for ϕ_{E3} :-

a. Correction due to effect of pile No.2 on pile No. 3 point E_3 is affected by pile No. 2 and since E_3 is in the forward direction of flow, hence this corrected is – Ve and its amount is given by: -

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

D = 150.7 – 148.0 = 2.7 m

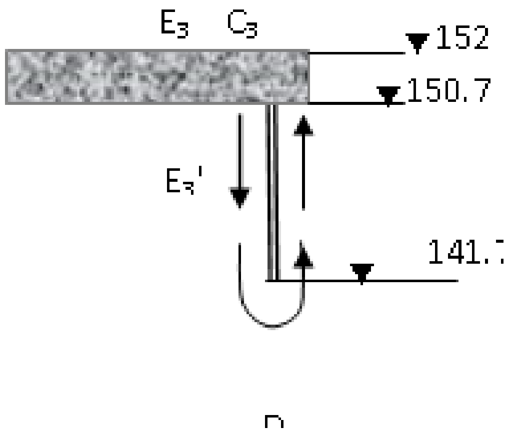
d = depth of pile No. 3= 150.7-141.7 = 9 m

b′ = distance between piles = 40 m

b = total floor length = 57.0 m

Correction $C = 19\sqrt{\frac{2.7}{40}}\left(\frac{9+2.7}{57}\right) = 1.013\%$ -Ve

(b) Correction due to floor thickness: -



Pressure at E_3' is less than at E_3 and since the pressure observed from curves is at E_3 this correction will be –Ve and its amount

$$\left[\frac{\phi E_3 - \phi D_3}{d_3}\right] t_3 = \left[\frac{37\% - 25\%}{152.0 - 141.7}\right]$$ ***1.3= 1.514% -Ve**

(c) .Correction due to slope at E3 =zero

Hence Corrected ϕE_3 = 37% - 1.013%- 1.514% = 34.47%

The corrected pressures at various key points are tabulated below: -

U/S pile No. 1	Intermediate pile No. 2	D/S pile No. 3
$\phi E_1 = 100\%$	$\phi E_2 = 66.95\%$	$\phi E_3 = 34.47\%$
$\phi D_1 = 80\%$	$\phi D_2 = 63.0\%$	$\phi D_3 = 25\%$
$\phi C_1 = 74.38\%$	$\phi C_2 = 59.71\%$	$\phi C_3 = 0\%$

Exit Gradient: -

Let the water be headed up to pond level, i.e. 158.0 m on the U/S side with no flow D/S.

The max seepage head = H = 158.0 – 152.0 = 6.0 m

The depth of D/S cutoff = d = 152.0 – 141.7 = 10.3 m

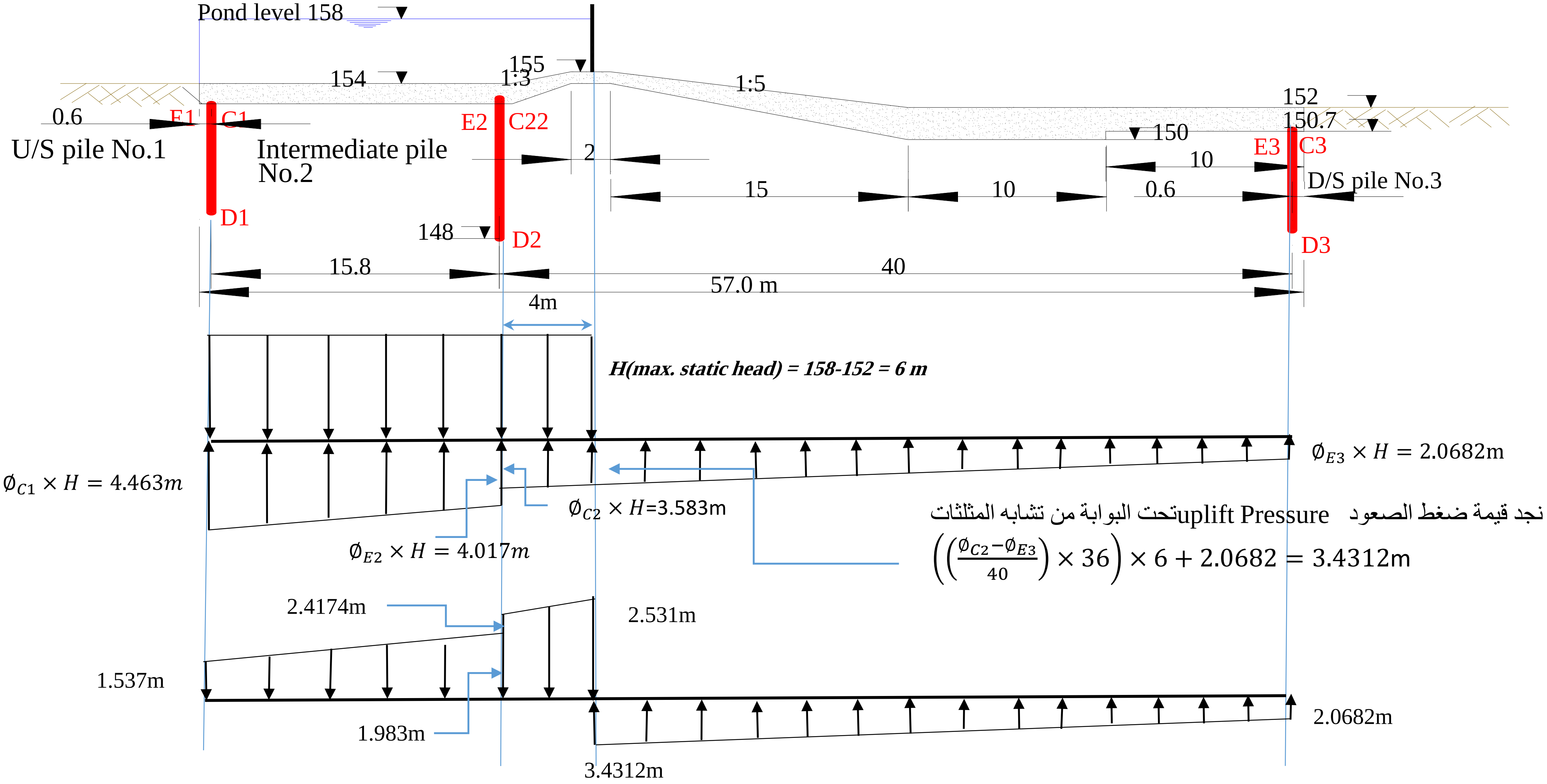
Total floor length = b = 57 m

$\alpha = \frac{b}{d} = \frac{57.0}{10.3} = 5.534$ $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+(5.534)^2}}{2} = 3.312$ Hence $G_E = \frac{H}{d} * \frac{1}{\pi\sqrt{\lambda}}$

$G_E = \frac{6.0}{10.3} * \frac{1}{\pi\sqrt{3.31}} = 0.102 = \frac{1}{9.8}$ and since safety factor $F = \frac{1}{G_E}$

Hence the exit gradient will be equal to 0.102, i.e. 1 in 9.8 which is very much safe.

Flow condition	**Head = U/S.W.L - D/S.B.L.**	Height (elevation of sub-soil H.G. line above datum)								
		U/S pile line			Intermediate pile line			D/S pile line		
		$\emptyset_{E1}$	$\emptyset_{D1}$	$\emptyset_{C1}$	$\emptyset_{E2}$	$\emptyset_{D2}$	$\emptyset_{C2}$	$\emptyset_{E3}$	$\emptyset_{D3}$	$\emptyset_{C3}$
Pond U/S & no flow D/S	(m)	100%	80%	74.38%	66.95%	63%	59.71%	34.47%	25%	0%
	6	6	4.8	4.463	4.017	3.78	3.583	2.068	1.5	0


Pond level 158
154
155
1:3
1:5
152
150.7
150
0.6
E1
C1
E2
C22
E3
C3
U/S pile No.1
Intermediate pile No.2
D/S pile No.3
2
15
10
10
0.6
D1
D2
D3
148
15.8
40
57.0 m
4m
H(max. static head) = 158-152 = 6 m
$\emptyset_{C1} \times H = 4.463m$
$\emptyset_{E3} \times H = 2.0682$m
$\emptyset_{C2} \times H$=3.583m
$\emptyset_{E2} \times H = 4.017m$
نجد قيمة ضغط الصعود uplift Pressure تحت البوابة من تشابه المثلثات
$\left(\left(\frac{\emptyset_{C2}-\emptyset_{E3}}{40}\right) \times 36\right) \times 6 + 2.0682 = 3.4312$m
2.4174m
2.531m
1.537m
1.983m
2.0682m
3.4312m

5. If factor of safety F = 8

Then $G_E = \frac{1}{8}$ $G_E = \frac{H}{d} * \frac{1}{\pi\sqrt{\lambda}}$

Where H = max. Static head

= Pond level (all gate closed and no water D/S)= 158.0 – 152.0 = 6m

d = D/S cutoff depth = 152.0 – 141.7 = 10.3 m

$$\frac{1}{8} = \frac{6}{10.3} * \frac{1}{\pi\sqrt{\lambda}}$$

$$\frac{1}{\pi\sqrt{\lambda}} = 0.215 \rightarrow \lambda = 2.20$$

And since $\alpha = \frac{b}{d}$

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

$$\Rightarrow \alpha = 3.25$$

Which is <57m

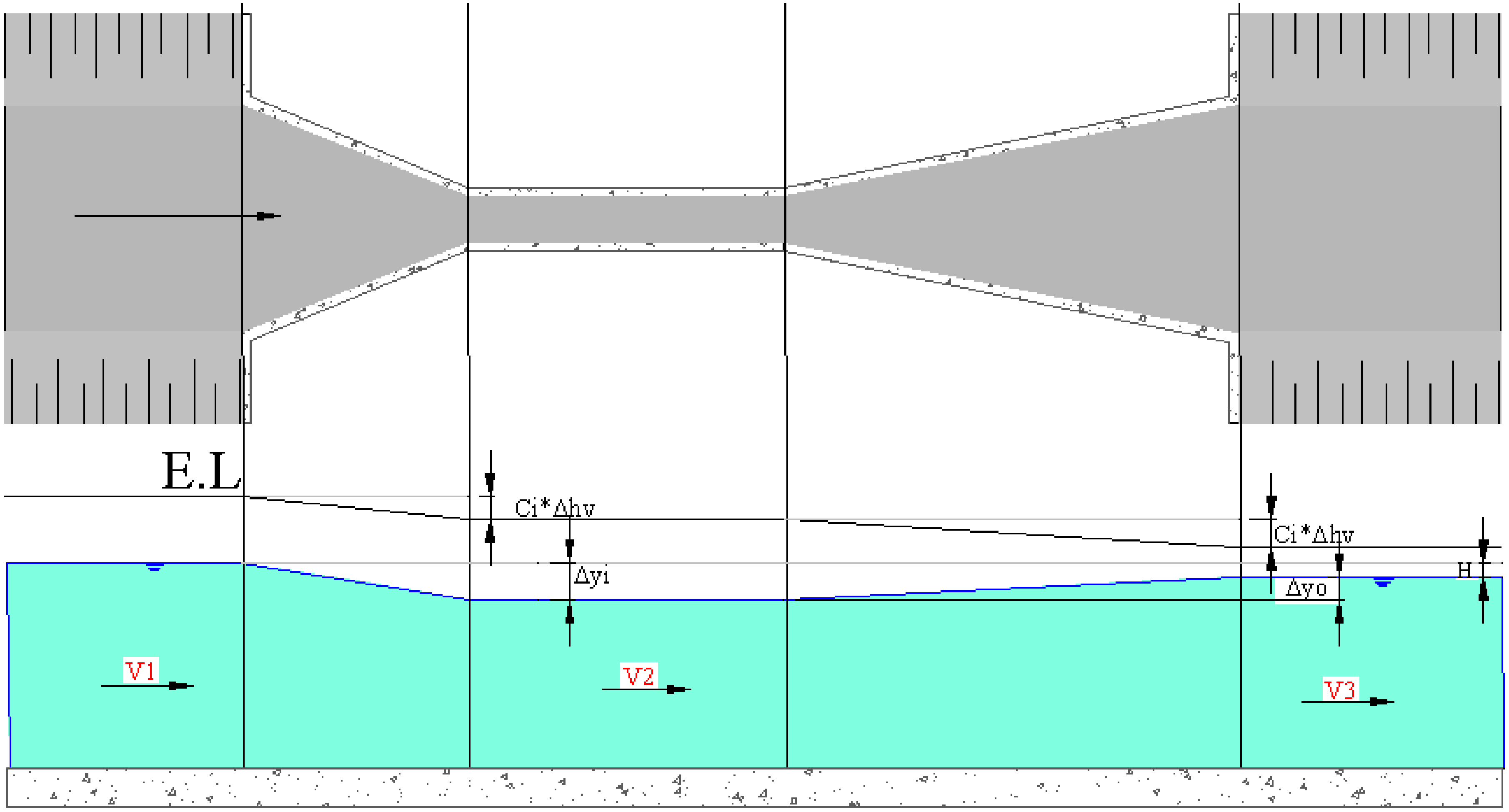
$b = 3.25(10.3) = 33.5m$ $\therefore$ O.K

Transitions

Transition (in a channel) is a structure designed to change the shape or cross-sectional area of the flow. The function of such a structure is:

1. To avoid excessive energy losses.
2. To eliminate cross-waves and other turbulence.
3. To increase the seepage path and thereby provide additional safety against piping.
4. To minimize canal erosion.
5. To retain earth, fill at the ends of structure

It should be noted that appreciable change in depth of flow generally occurs in all types of transitions



For inlet structures

The entrance velocity is less than the exit velocity; hence, the water surface must always drop at least a full difference between the velocity heads, plus a small conversion loss known as the inlet loss.

The drop ΔY_i in water surface for inlet structure may therefore be expressed as:

$$\Delta E = losses = c_i\left(\frac{V_2{}^2}{2g} - \frac{V_1{}^2}{2g}\right) \text{-------(1)}$$

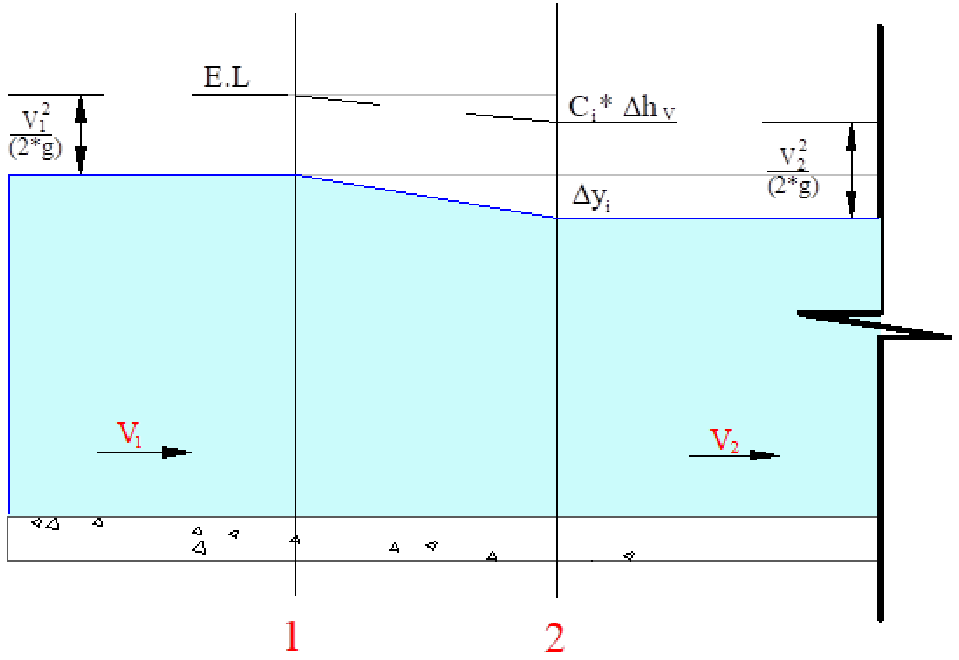
$$y_1 + \frac{V_1{}^2}{2g} = y_2 + \frac{V_2{}^2}{2g} + \Delta E$$

$$y_1 - y_2 = \frac{V_2{}^2}{2g} - \frac{V_1{}^2}{2g} + \Delta E \text{--------(2)}$$

$$\Delta y_i = \frac{V_2{}^2}{2g} - \frac{V_1{}^2}{2g} + c_i\left(\frac{V_2{}^2}{2g} - \frac{V_1{}^2}{2g}\right) \text{--------(3)}$$

$$\Delta Y_i = (1 + C_i) * \left(\frac{V_2^2 - V_1^2}{2g}\right)$$

$$\Delta Y_i = (1 + C_i) * \Delta h_V$$



C_i = Coefficient of entrance or the amount of losses in entrance (coefficient of inlet loss).

Δh_V = Difference in velocity heads.

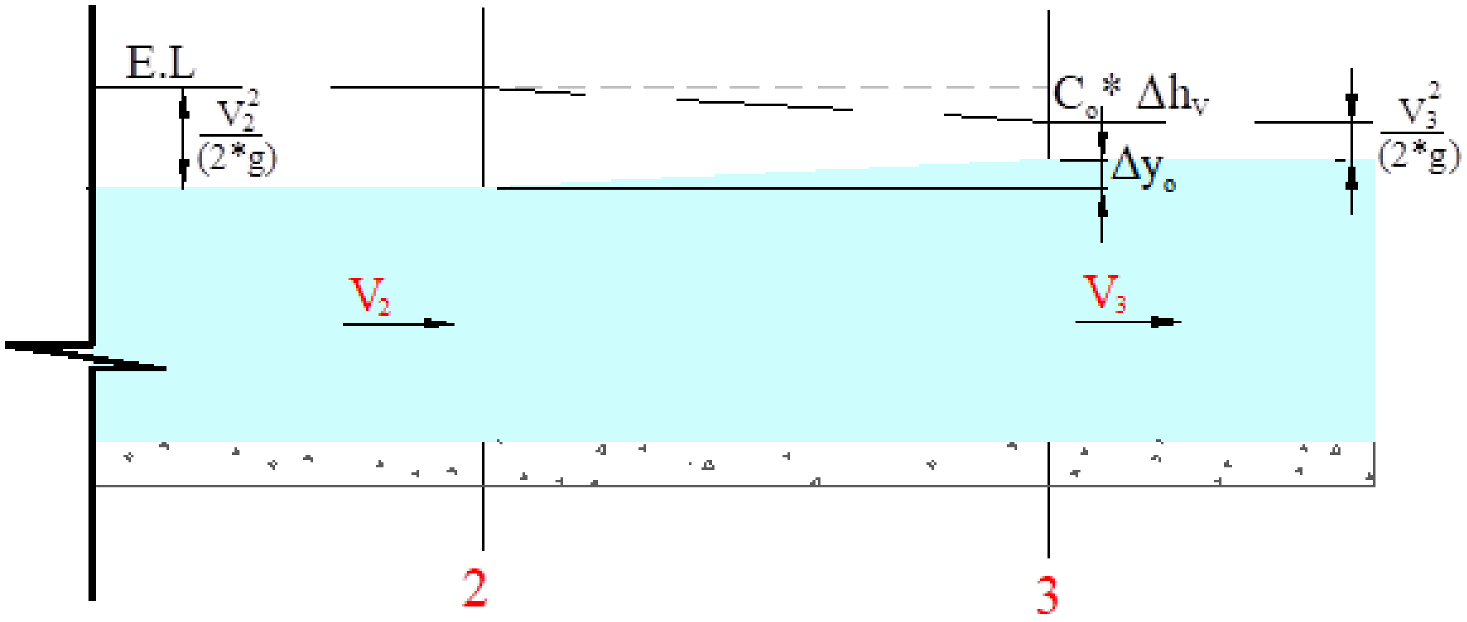
For outlet structures

The velocity is reduced, at least in part, in order to lift the water surface. This rise in water surface, known as the recovery of velocity head, is usually accompanied by a conversion loss known as the outlet loss. The rise ΔY_o in water surface for outlet structure may be expressed as:

$$\Delta Y_o = (1 - C_o) * \left(\frac{V_2^2 - V_3^2}{2g} \right)$$

$$\Delta Y_o = (1 - C_o) * \Delta h_V$$

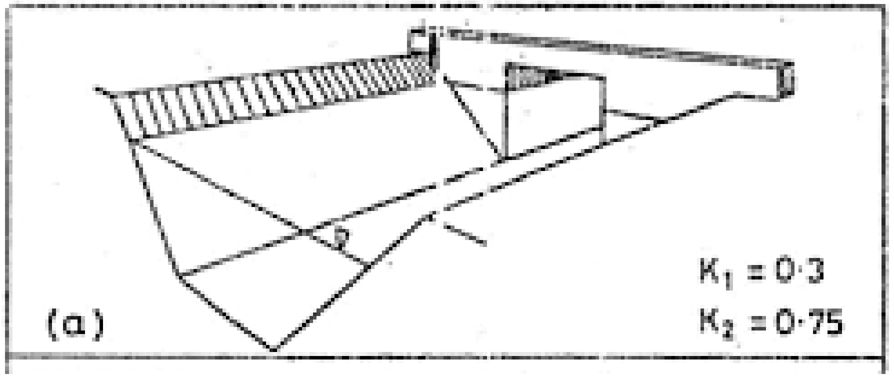
$C_o =$ Coefficient of outlet loss



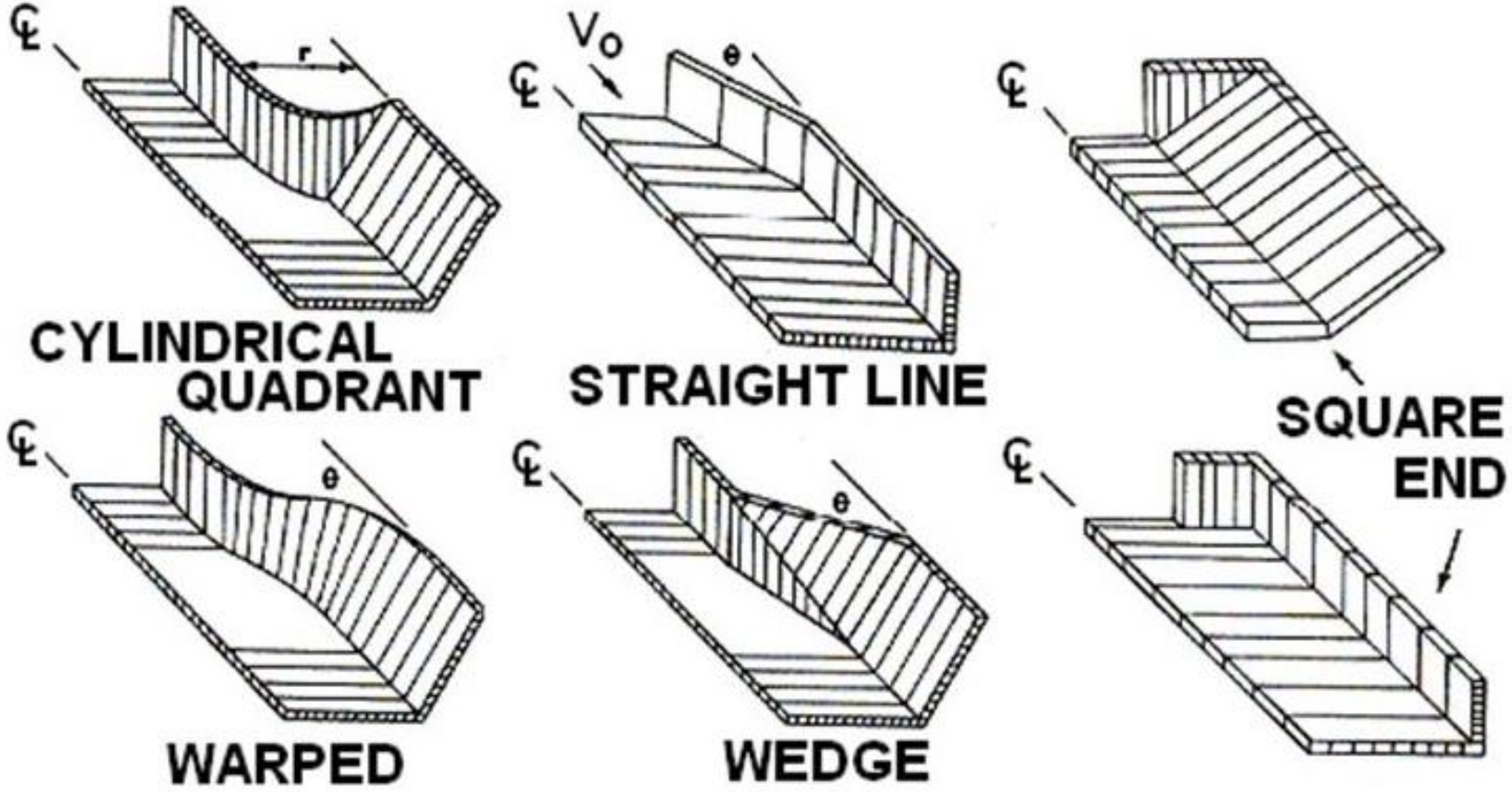
All transitions may be classified as either **inlet** (contraction) or **outlet** (expansion) transitions.

The various types of transitions most commonly used in subcritical flow are:

- Straight line headwall
- Broken back to rectangular or pipe opening
- Cylinder quadrant
- Straight warp to rectangular or pipe opening
- Streamline warp to rectangular opening



STRAIGHT LINE
HEADWALL



The figure shows, typical designs for channel transition from a rectangular cross section to a trapezoidal c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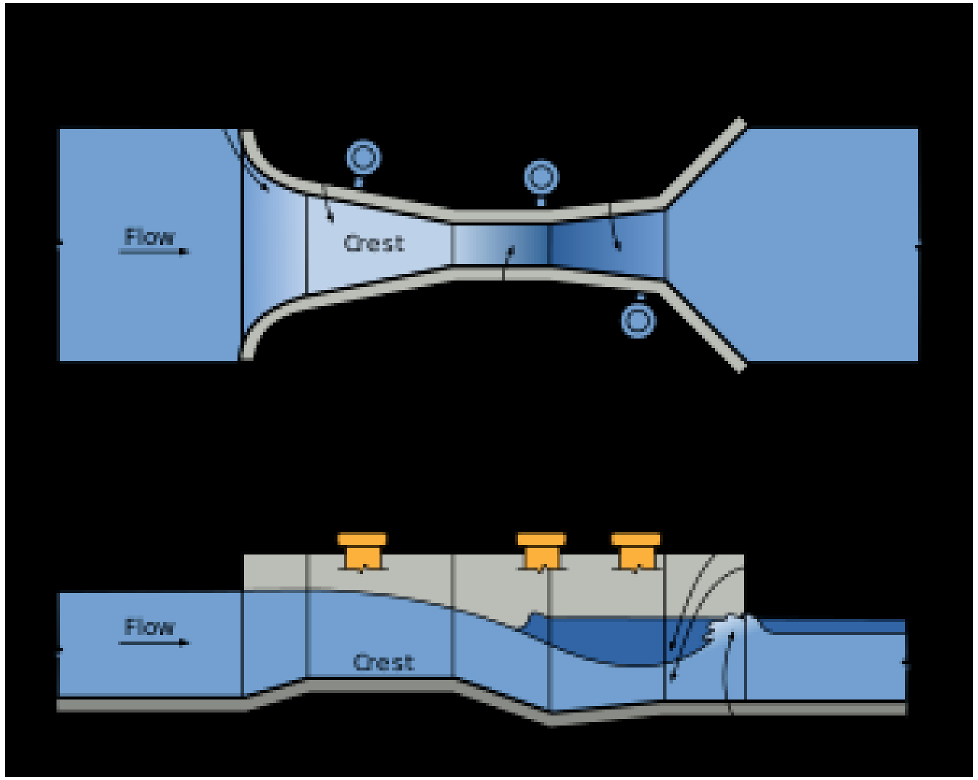
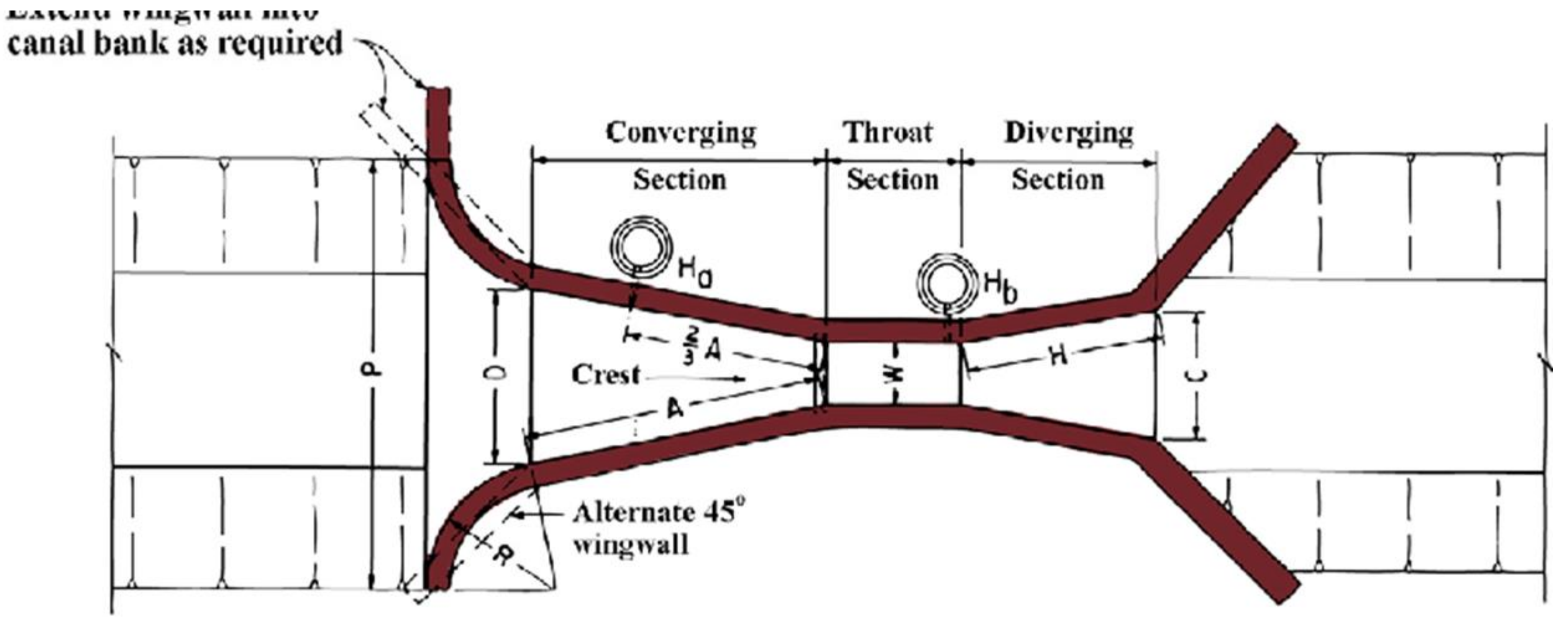
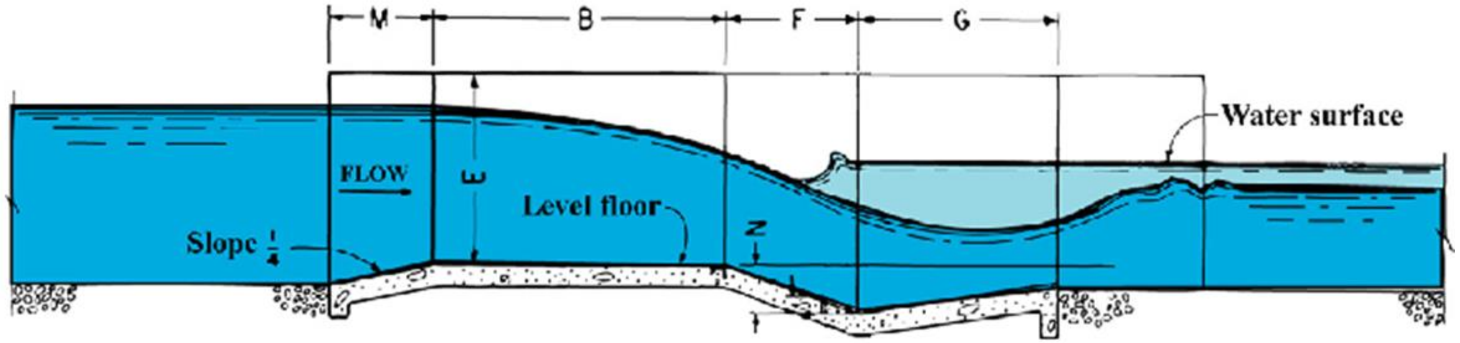

Flow
Crest
Flow
Crest


canal bank as required
Converging
Section
Throat
Section
Diverging
Section
Ha
Hb
T
2/3 A
Crest
A
W
H
C
D
P
Alternate 45°
wingwall
R

PLAN


M
B
F
G
Water surface
FLOW
E
Level floor
N
Slope 1/4

These are shown in ***figures 9.2 and 9.3*** together with their respective inlet and outlet loss coefficients. Table (2) gives a range of loss coefficients for pipes and rectangular sections under partial and full flow condition.

Table (2) Loss Coefficients for transitions

Transition type fig. 9.2	Pipe				Rectangular			
	Full Flow		Partial flow		Full Flow		Partial flow	
	Inlet	Outlet	Inlet	Outlet	Inlet	Outlet	Inlet	Outlet
a	0.5	1.0	0.4	0.8	0.5	1.0	0.3	0.75
b	0.4	0.7	0.3	0.6	0.4	0.7	0.3	0.6
c	*	*	*	*	0.3	0.6	0.25	0.5
d	0.2	0.4	0.2	0.4	0.25	0.5	0.2	0.4
e	0.2	0.4	0.1	0.2	0.2	0.4	0.1	0.2

* Not used for a pipe

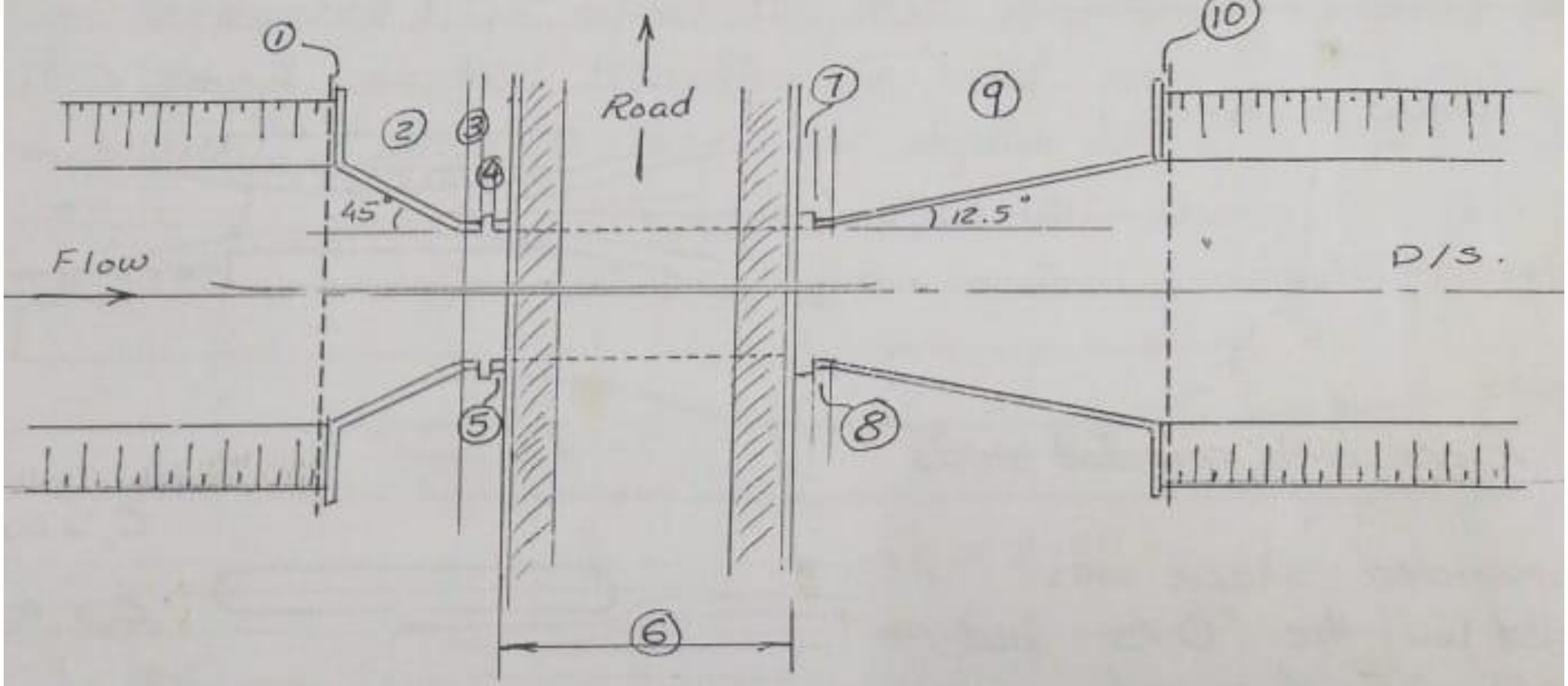
The following comments apply to the transitions shown in ***figure 9.2 and 9.3***:

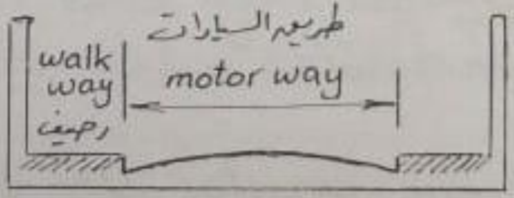
- The straight-line headwall transition is suitable for small short structures in watercourses and where head loss is not a problem. It is relatively cheap and easy to construct. See ***figure 9.2 (a).*** Both inlet and outlet transition can take this form. It is also used extensively on small pipe culverts flowing full as shown in ***figure 9.3.***
- For discharge ranges of 0.5 to 5 m^3/s the broken back (or dog leg) transition is used for inlets and outlets. ***See figure 9.2 (b)***. It is also suitable for transitions to pipes under pressure. ***See figure 9.3 (b).***
- The cylinder quadrant transition gives slightly lower loss coefficients then the broken back and is suitable for distributary canals. ***See figure 9.2 (c).***
- For discharge ranges of 2.5 to 5 m^3/s the straight warp transition is preferred on branch and distributary canals. ***See figure 9.2 (d).***

- Where canal discharges exceed 5 m^3/s the streamline warp transition is most suitable, especially for inlets. Construction is however more complicated and the transition longer than for the other types. For reasons of economy this transition is often paired with the straight-line warp as an outlet. It has been found that the most suitable convergence angle is about 1:4, or 14 degrees. ***See figure 9.2(e)***.

The selection of the most appropriate type of transition based solely on discharge is not practical, as will be shown below. In the end it is a balanced judgment between the competing considerations of head loss and cost. Each case must be considered on its merits and it is not possible to provide solutions of completely general applicability. Where the conservation of head is not of great importance the best solutions will usually be the straight-line headwall for the smaller flows and the straight warp for high flow. Where the minimizing of head loss is of importance (and this is the more usual situation) then the use of straight-line headwall should be limited to small flows and the straight warp and streamlined warp used for the higher flows. In opting for non-plane solution to the transition problem, the designer should have regard to the capability of the contractor to construct accurately the non-plane sections. Generally, this capability will be greater on the larger projects.

Practical Proportioning of Regulator's Dimensions :-





details No. 6

①- 0.2 to 0.3 m of floor excess (زيادة).

②- length of U/S transition.

③- about 0.5 m to 1.0 m of a straight reach.

④- 0.15 to 0.25 m groove (شق) for gate recess (تجويف)

⑤- about 0.3 to 0.5 m clearance (فسحة)

⑥- Bridge deck equal to :-

width of motorway (No. of lanes x 3.5) +

2 (width of walk (رصيف) ways , use 1.5 to 3.0 m , each) +

2 (thickness of handrails a parapets , about 0.25 m each)

⑦- D/S groove 0.2 to 0.25 m (D/S emergency groove)

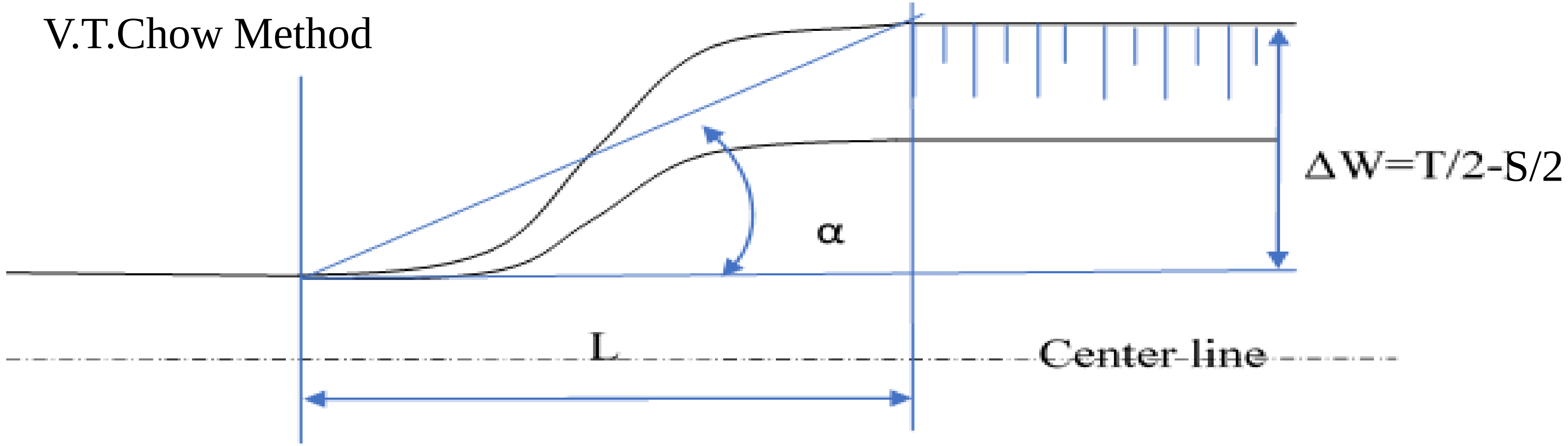
⑧- 0.5 m clearance

⑨- D/S transition.

⑩- 0.20 to 0.30 m of floor excess.

Calculations For Warped Transitions: -

V.T.Chow Method



1. Calculate the transition length $L = \Delta W cot(\alpha)\ where\ \alpha\ =\ 12.5°\ for\ D/S$
$=\ 12.5°\ to\ 45\ °for\ U/S.$

2. Divide the horizontal distance along the transition into equal whole number increment Δx such as 0 m, 5 m, 10 m, ……… etc.

3. Using the hydraulic equation $\Delta Y_i = (1+C_i)*\left(\frac{V_2^2 - V_1^2}{2g}\right)$ or $\Delta Y_o = (1-C_o)*\left(\frac{V_2^2 - V_3^2}{2g}\right)$

Calculate the total water surface drop ΔYi or the surface recovery ΔY_o .

Find *Δy* at intervals *Δx* from: *y=c x2*

4. The parabola is plotted as $y = c\,x^2$

at the middle of the transition $\frac{\Delta y}{2} = c\,\left(\frac{L}{2}\right)^2 \rightarrow find\ c$

this is done for L/2 then the same data are used for other half with x and y reversed.

5. If $\Delta h_V =$ difference in velocity head between any two sections, then:

$$\Delta h_V = \frac{\Delta y_0}{(1-c_0)} = \frac{{V_2}^2 - {V_1}^2}{2g} \quad \rightarrow \quad \frac{{V_2}^2}{2g} = \Delta h_V + \frac{{V_1}^2}{2g}$$

6. Find $V = \sqrt{2g\Delta h_V}$

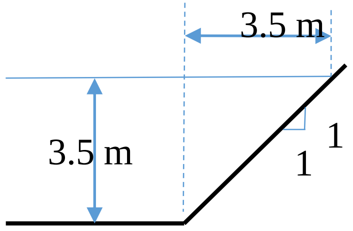
7. $A = \frac{Q}{V}$ (to find the area at any section).

8. $Estimate\ \dfrac{B+T}{2}\ or\ \dfrac{B}{2} + \dfrac{T}{2}$

9. $Depth\ of\ water\ y = \dfrac{A}{\left(\frac{B+T}{2}\right)}$

Example:

Calculate and plot to scale the required warped transition to a canal 20 m bed width, 3.5 m deep and 1:1 side slope leading from a regulator of opening S = 9 m , Q = 100 cumecs, Using the formula and the above curves calculate and plot the water surface profile.(c_0 = 0.2)

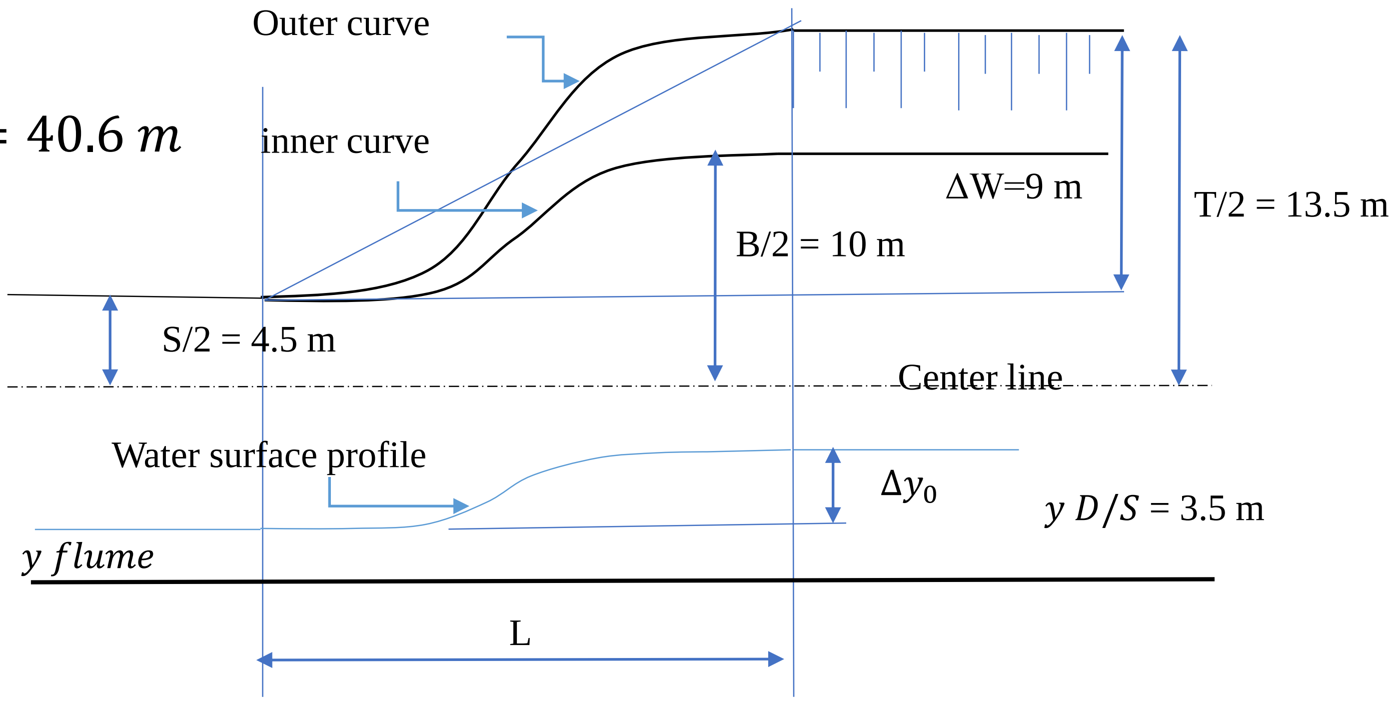


$$\Delta y_0 = (1 - c_0)\left(\frac{V_{flume}{}^2 - V_{D/S}{}^2}{2g}\right)$$

Solution:

$$L = \Delta W cot(\alpha) = 9 \times \cot(12.5°) = 40.6\ m$$

$$use \quad L = 40\ m$$



الان اصبح بالامكان رسم ال inner curve and outer curve باستخدام المعادلة $y = c\,x^2$ حيث ان L معلومة و y معلومة و منها يمكن استخراج قيمة c ثم احداثيات المنحني اما بالنسبة لسطح الماء فان L معلومة ولكن Δy_0 مجهولة ويجب حسابها كما موضح ادناه ومن ثم يمكن رسم ال water surface profile كما سبق شرحه بالمعادلة $y = c\,x^2$ اي بنفس طريقة رسم ال inner curve & outer cure

$$A_{D/S} = (B + ZD)D = (20 + 3.5) \times 3.5 = 82.25\, m^2$$

$$V_{D/S} = \frac{Q}{A_{D/S}} = \frac{100}{82.25} = 1.216\, m/s$$

$$\Delta y_0 = (1 - c_0)\left(\frac{{V_{flume}}^2 - {V_{D/S}}^2}{2g}\right)$$

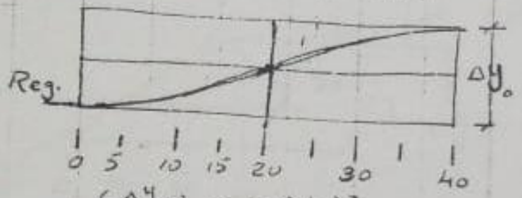
$$y_{D/S} - y_{flume} = (1 - 0.2)\left(\frac{{V_{flume}}^2 - {V_{D/S}}^2}{2g}\right)$$

$$3.5 - y_{flume} = 0.8 \times \left(\frac{{V_{flume}}^2 - 1.216^2}{2g}\right) \qquad \text{and } V_{flume} = \frac{Q}{A_{flume}} = \frac{100}{9 \times y_{flume}}$$

$$3.5 - y_{flume} = 0.8 \times \left(\frac{\left(\frac{100}{9 \times y_{flume}}\right)^2 - 1.216^2}{2g}\right)$$

By trial and error $y_{flume} = 3\, m$, $V_{flume} = 3.7\,\frac{m}{s}$, $and\ \Delta y_0 = 0.5\, m$

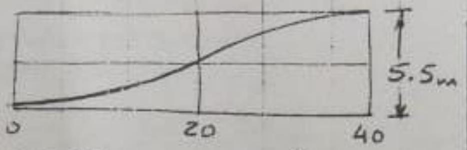
Water Surface profile



$(\frac{\Delta y_o}{2}) = C(\frac{L}{2})^2$

$(\frac{0.5}{2}) = C(20)^2 \longrightarrow C = 0.000625$

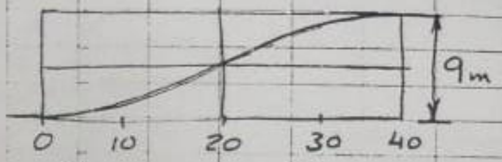
inner curve



$(\frac{5.5}{2}) = C(\frac{40}{2})^2$

$\longrightarrow C = 6.875 \times 10^{-3}$

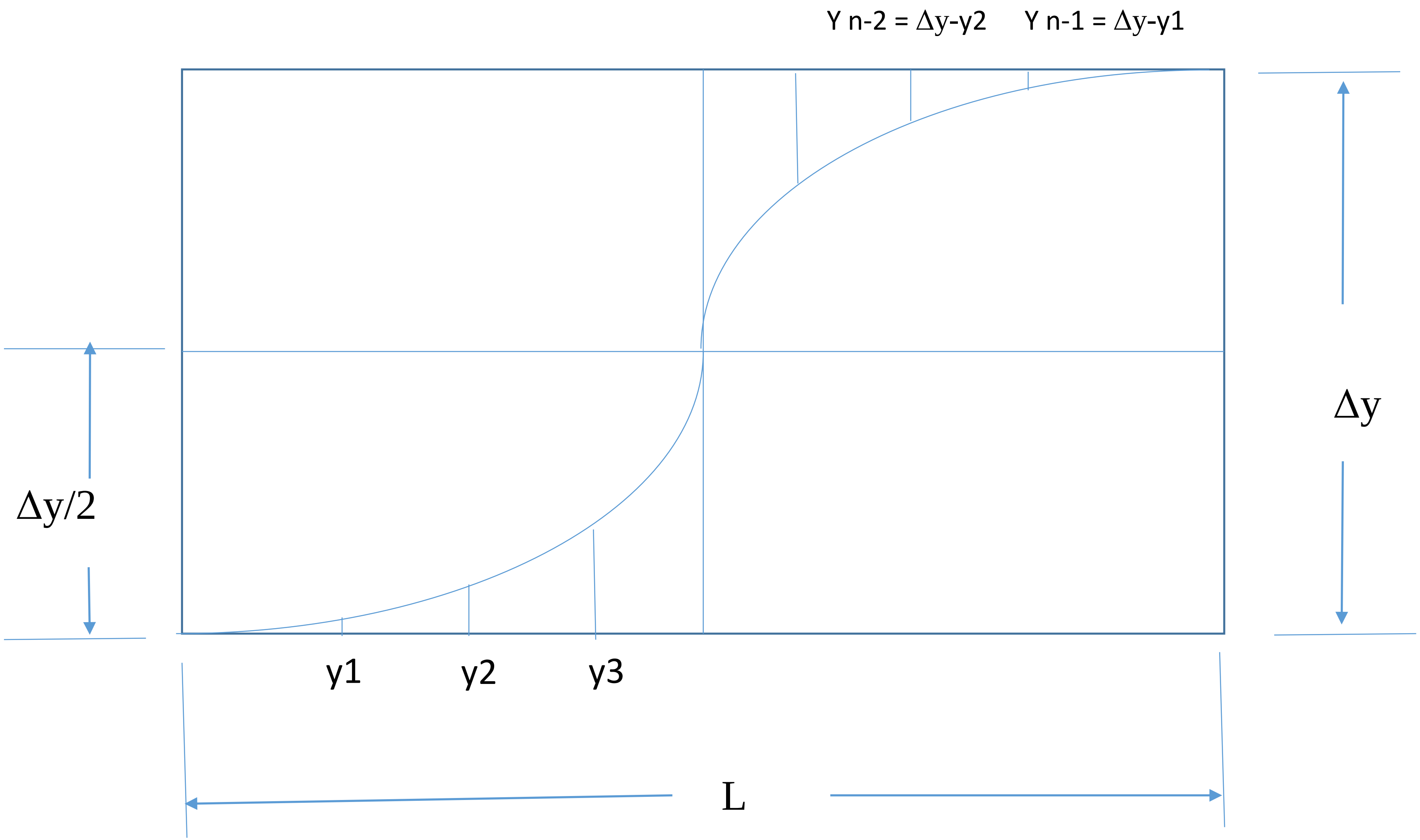
Outer Curve



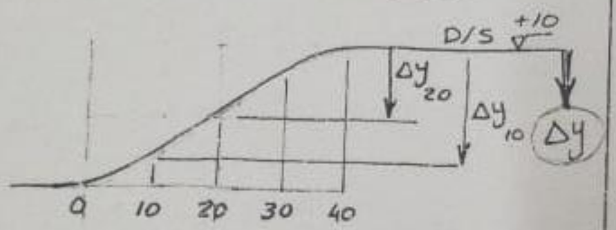
$(\frac{9}{2}) = C(\frac{40}{2})^2$

$\longrightarrow C = 0.01125$

dist. x	$\Delta y = 6.25 \times 10^{-4} X^2$	$\Delta y = 6.875 \times 10^{-3} X^2$	$\frac{B}{2} = \Delta y + 4.5$	$\Delta y = 0.01125 X^2$	$\frac{T}{2} = \Delta y + 4.5$	$(\frac{B+T}{2})$
Reg. 0	0	0	4.5	0	4.5	9.00
5	0.0156	0.172	4.67	0.28	4.78	9.45
10	0.0624	0.688	5.19	1.12	5.62	10.81
15	0.1406	1.547	6.05	2.53	7.03	13.08
الوسط → 20	0.250	2.750	7.25	4.50	9.00	16.25
25	(0.5 - 0.1406) = 0.3594	(5.5 - 1.547) = 3.953	8.45	(9 - 2.53) = 6.47	10.97	19.42
30	(0.5 - 0.0624) = 0.4376	5.5 - 0.688 = 4.812	9.31	9 - 1.12 = 7.88	12.38	21.69
35	(0.5 - 0.0156) = 0.4844	5.5 - 0.172 = 5.328	9.83	9 - 0.28 = 8.72	13.22	23.05
Canal 40	(0.5 - 0) = 0.500	5.5 - 0 = 5.50	10.00	9 - 0 = 9.00	13.50	23.50


Y n-2 = Δy-y2
Y n-1 = Δy-y1
Δy
Δy/2
y1
y2
y3
L

Water Surface



Δy_{total} الكلية محسوبة بين الـ flume والـ قناه خلف الـ D/S أما اذا اردنا Δy بين مقطعين اي القيم المدرجة بالعمود الثاني عندها نصحح المعادلة :-

$$\Delta y_{total} = (1-C_o)\left(\frac{V_f^2 - V_{D/S}^2}{2g}\right)$$

$$\Longrightarrow \frac{V^2_{at\,any\,sec.}}{2g} = \left(\frac{\Delta y_{at\,any\,sec.}}{1-C_o}\right) + \frac{V^2_{D/S}}{2g}$$

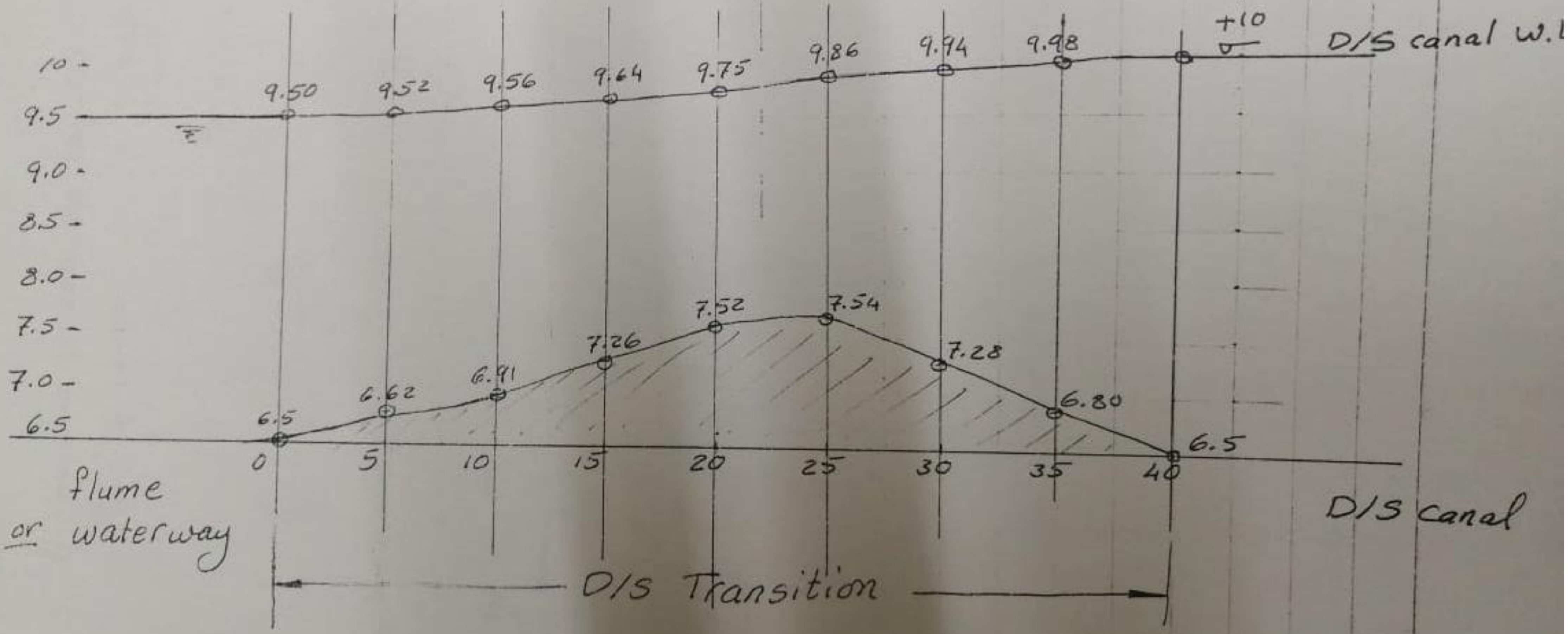
القيم تؤخذ من هذا العمود وتوضع تحت الجذر

تؤخذ قيم $\frac{B}{2}$ و $\frac{T}{2}$ من الجدول السابق

10 هي منسوب الماء في قناه الـ D/S المفروض انه تكون معطاه بالسؤال

على اعتبار ان Δy محسوبة من مستوى سطح الماء في الـ D/S كما موضح بالرسم.

station	Δy	$\frac{V^2_{at\,any\,sec.}}{2g} = \left(\frac{\Delta y_{at\,any\,sec.}}{1-C_o}\right) + \frac{V^2_{D/S}}{2g}$	$V = \sqrt{2g\left(\frac{V^2}{2g}\right)}$	$\left(\frac{B+T}{2}\right)$	$A = \frac{Q}{V}$	$y = \frac{A}{\left(\frac{B+T}{2}\right)}$	W.L $= 10 - \Delta y$	B.L $= (W.L - y)$
Reg. 0	0.500	$\left(\frac{0.500}{0.8}\right) + \left(\frac{1.216^2}{2g}\right) = 0.699$	3.702	9.00	27.01	3.00	9.50	6.50
5 m	0.484	$\left(\frac{0.484}{0.8}\right) + (\;=\;) = 0.679$	3.648	9.45	27.41	2.90	9.517	6.62
10 m	0.437	0.622	3.491	10.81	28.65	2.65	9.56	6.91
15	0.359	0.524	3.205	13.08	31.20	2.38	9.64	7.26
الوسط 20	0.250	0.388	2.757	16.25	36.27	2.23	9.75	7.52
25	0.141	0.25	2.215	19.42	45.14	2.32	9.86	7.54
30	0.0624	0.153	1.734	21.69	57.67	2.65	9.94	7.28
35	0.0156	0.095	1.364	23.05	73.32	3.18	9.98	6.80
40	0	0.075	1.216	23.50	82.25	3.50	10.00	6.50


10
9.5
9.0
8.5
8.0
7.5
7.0
6.5
9.50
9.52
9.56
9.64
9.75
9.86
9.94
9.98
+10
D/S canal W.L
6.5
6.62
6.91
7.26
7.52
7.54
7.28
6.80
6.5
0
5
10
15
20
25
30
35
40
flume
or waterway
D/S canal
D/S Transition

Design of Transitions

The flumed portion is gradually joined with normal section with smooth transitions. The flumed should be such that the velocity in flume ≤ 3 m/sec and the flow should remain subcritical to avoid the possibilities of hydraulic jump forming in the flume.

Transition when the water depth remains constant:

In case the water depth in the transitions and the flume remains constant, hyperbolic transitions as proposed by **Mitra** may be used.

Let B_c and B_f be the bed widths of the normal and flumed sections, respectively, and let B_x be the bed width at a distance (x) from flumed section. Let V_c, V_f and V_x be the mean velocities at the corresponding sections. The total length of the transition is L_f. The transition is designed on the basis that the rate of change of velocity per unit length of transition is constant.

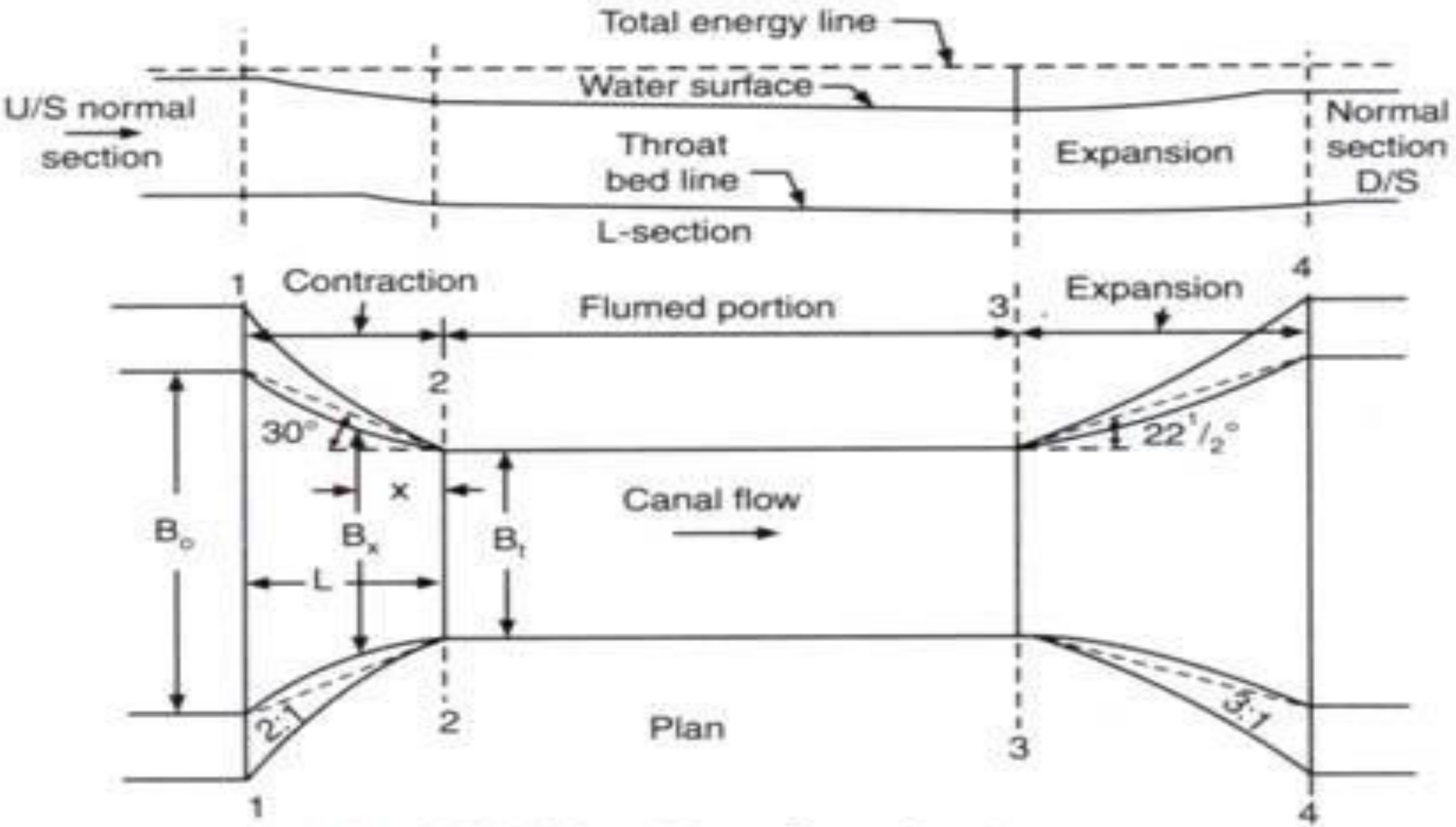


Fig. 19.25. Transition of canal water-way

Then: $$\frac{V_f - V_x}{x} = \frac{V_f - V_c}{L}$$

Since depth D is assumed to be constant, and letting Q be the discharge, then:

$$B_f \times V_f = B_x \times V_x = B_c \times V_c = \frac{Q}{D} = K$$, a constant.

Hence: $V_f = \dfrac{K}{B_f}$, $V_x = \dfrac{K}{B_x}$, $V_c = \dfrac{K}{B_c}$

$$\frac{\left(\frac{K}{B_f}\right) - \left(\frac{K}{B_x}\right)}{x} = \frac{\left(\frac{K}{B_f}\right) - \left(\frac{K}{B_c}\right)}{L}$$

$$B_x = \frac{B_c * B_f * L}{L * B_c - (B_c - B_f) * x}$$

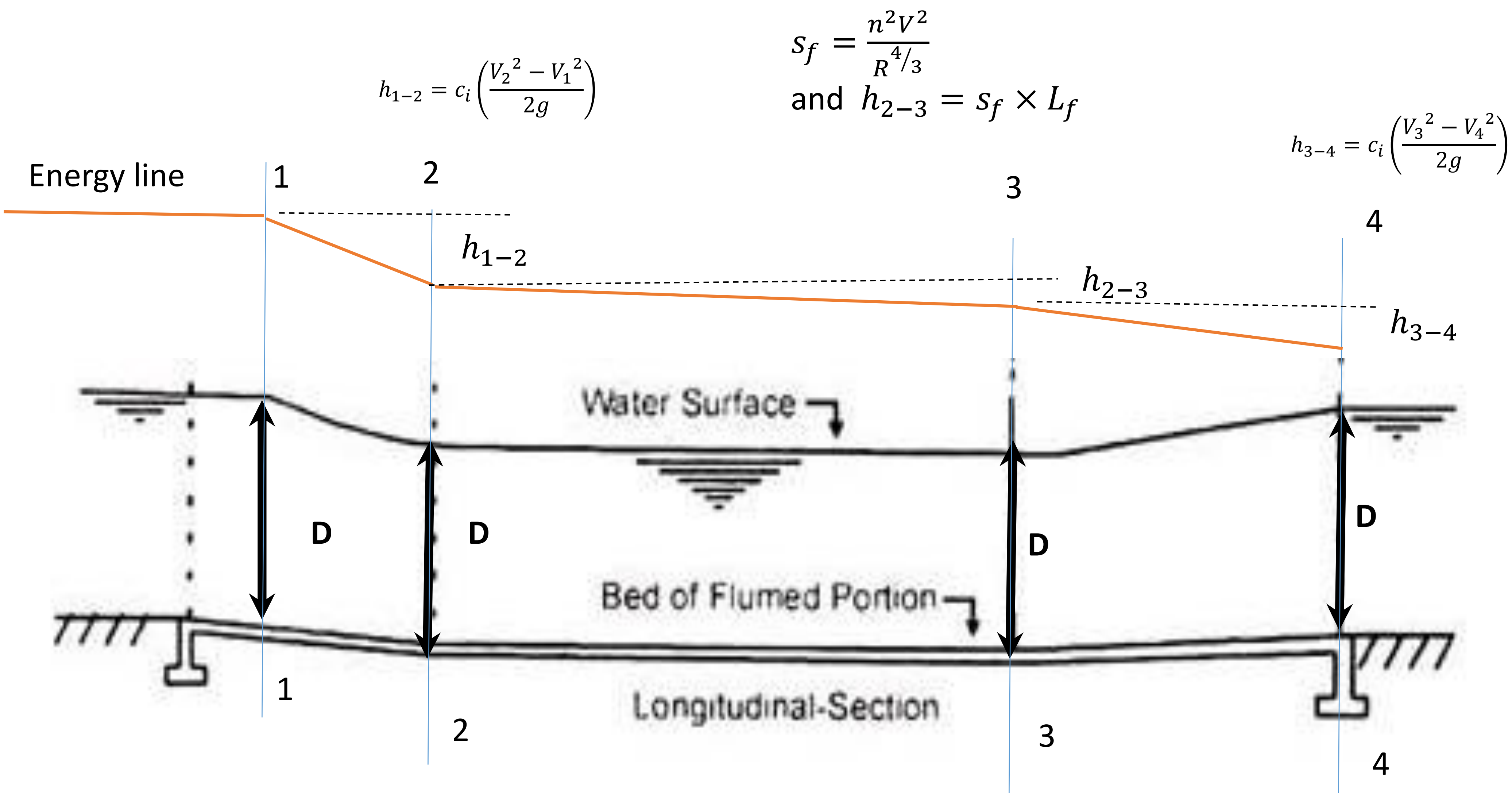
The above equation represents **Mitra** hyperbolic transition.

Example: Given the following data, design a canal transition using a constant waters depth method (Mitra's hyperbolic transitions method).

Canal discharge	30 cumecs
Canal bed with	20 m
Canal waters depth	1.50 m
Full supply level (F.S.L.)	251.50 (D/S canal w.l.)
Flume width	10.0 m
Length of the flumed portion.	74.50 m
U/S and D/S canal side slope	1½: 1
Manning roughness coefficient	0.016

$C_i = 0.2$, $C_0 = 0.3$

Providing 2:1 slope in contraction and 3:1 in expansion



$s_f = \frac{n^2V^2}{R^{4/3}}$
and $h_{2-3} = s_f \times L_f$
$h_{1-2} = c_i\left(\frac{V_2^{\ 2} - V_1^{\ 2}}{2g}\right)$
$h_{3-4} = c_i\left(\frac{V_3^{\ 2} - V_4^{\ 2}}{2g}\right)$
Energy line
1
2
3
4
h_{1-2}
h_{2-3}
h_{3-4}
Water Surface
D
D
D
D
Bed of Flumed Portion
Longitudinal-Section
1
2
3
4

Solution:

Length of contraction transition ((20-10)/2)*2=10 m

Length of expansion transition = ((20-10)/2)*3 =15m

Head loss and bed levels at different sections:

At section (4)

Area of section =(B+1.5D) D =(20+1.5x1.5) 1.5=33.75m^2

Velocity V=Q/A=30/33.75=0.89 m/s

Velocity head=(0.89)^2/2*9.81=0.043 m

Given B.L. =250.0 m

R.L. (reduced level) of waters surface = 250+1.50 (water depth)= 251.5 m

T.E.L. (Total energy line) = 251.5+0.043=251.543 m

At section (3)

Area of section =10*1.5=15 m^2

$V=30/15=2$ m/s

$V^2/2g=2^2/19.62=0.204$ m

Loss of head in expansion from section (3) to (4) $= c_0 \left(\frac{{V_3}^2 - {V_4}^2}{2g}\right) = 0.3 \times \left(\frac{2^2 - 0.89^2}{2g}\right) = 0.052$ m

T.E.L. at section 3 = 251.543 + 0.052 = 251.59 m

W.L. at section 3 = 251.595 - 0.204 = 251.391 m

B.L .at section 3 = 251.391 - 1.50 = 249.891 m

From section (3) to section (2) area and velocity are constant

$V = R^{2/3} S^{1/2}$; (S) is slope to be determined

$$2 = \frac{1}{0.016}\left[\frac{15}{10+2(1.5)}\right]^{2/3} \mathbf{S^{1/2}}$$ S =0.001024

Hence loss of head in the flume = flume length x slope =74.5*0.001024 = 0.0765 m

At section (2)

T.E.L =251.595+0.076=251.671 m

W.L0 =251.671-0.204=251.467 m

B.L. =251.467-1.50 =249.967 m

At section (1)

Loss of head in contraction transition from section 1to 2

$$= 0.2(\frac{V_2^{\,2}-V_1^{\,2}}{g^2}) = 0.2(\frac{2^2-0.89^2}{19.62}) = 0.035$$

T.E.L =251.671+0.035 =251.706 m

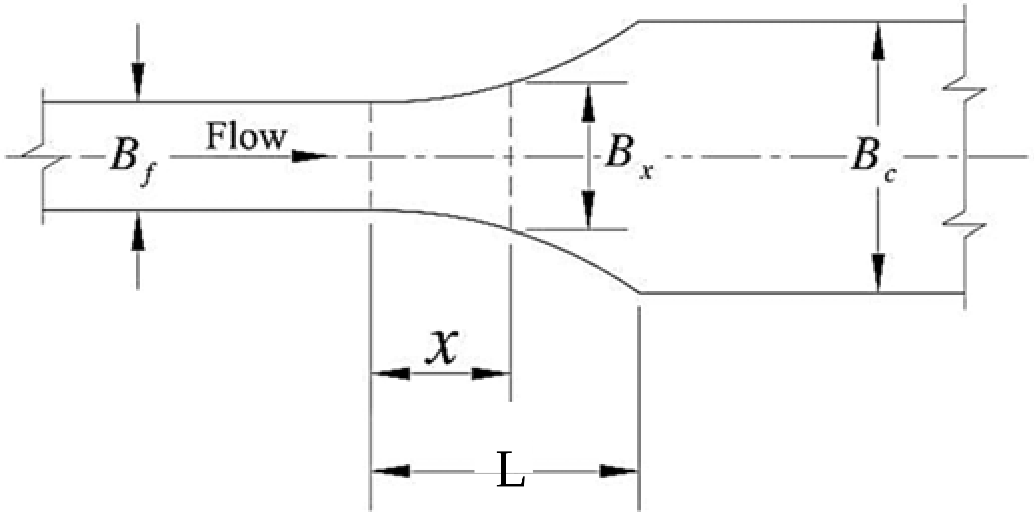
W.L. =251.706 - 0.043 = 251.663

B.L. =251.663-1.50 = 250.163 m

Transitions:

(a) Expansion Transition:

$$B_x = \frac{B_c . B_f . L}{L . B_c - (B_c - B_f)x}$$



Where: B_f =10 m, B_C = 20 m and L = 15 m

$$B_x = \frac{20\times10\times15}{15\times20-(20-10)x} = \frac{3000}{300-10x}$$

For different values of x, the following values of B_x are worked out in meters.

X	0	3	6	9	12	15 m
B_X	10.00	11.1	12.5	14.3	16.67	20.0

(b) Contraction Transition

In contraction transition the values of B_c, B_f and L are

$B_c = 20m$, $B_f = 10$ m and L=10 m

Substituting we get: -

$$B_x = \frac{2000}{200 - 10x}$$

For different values of x, the following values of B_x are given below in meters:

x	0	2	4	6	8	10m
B_x	10.0	11.1	12.5	14.3	16.67	20.0m

Hind's Method for Transitions Design

When Water Depth may also vary: -

This is general method and is applicable either when the depths in the flumed and unflawed portions are the same or when these depths are different.

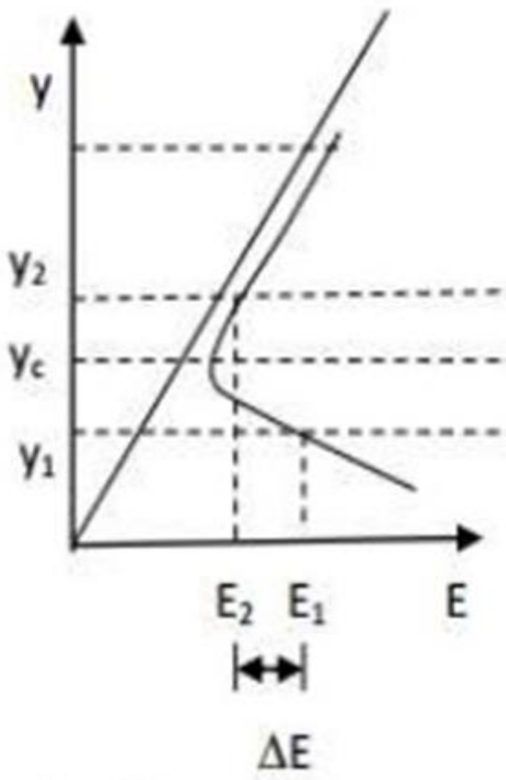
Hydraulic jump and its usefulness in Design of irrigation structures:

When a stream of water moving with a high velocity and low depth (i.e. super critical flow) strike another stream of water moving with a low velocity and high depth (i.e. subcritical flow), a sudden rise in the surface of the former takes place. This phenomenon is called “Hydraulic jump” and is generally accompanied by a large-scale turbulence, dissipating most of kinetic energy of super-critical flow. Such a phenomenon may occur in a canal below a regulating sluice, at the bottom of a spillway, or at place where steep channel slope suddenly turns flat.

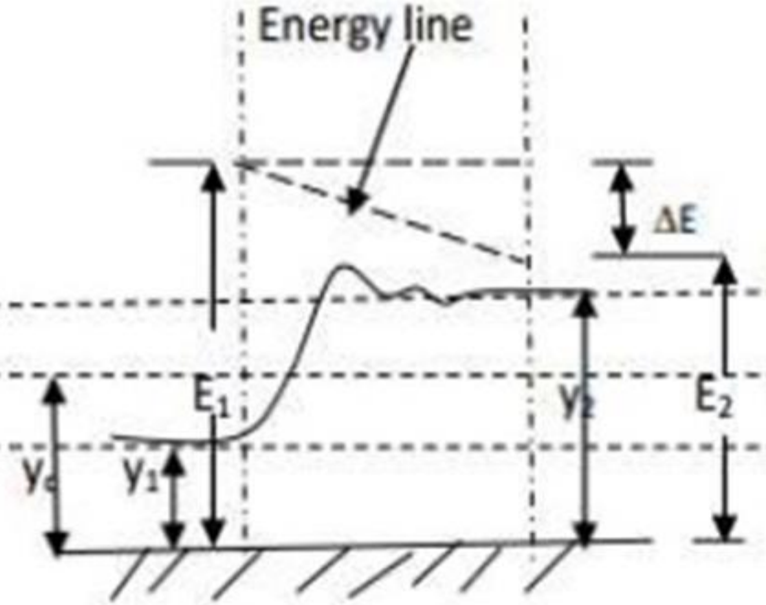
It may be noted that the depth before the jump is always less than the depth after the jump.

The depth before the jump is called the “initial depth” D_1 and the depth after the jump is called the “sequent depth” $D_{2.}$

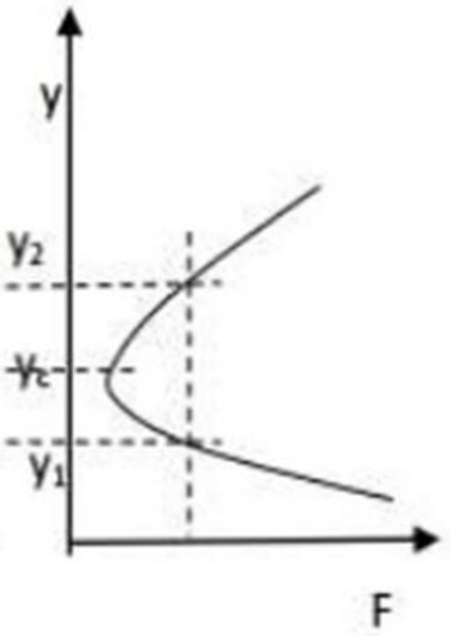
The” alternate depths” D_1 and $D_2{'}$ are two possible depths for the same specific energy.



Specific energy curve

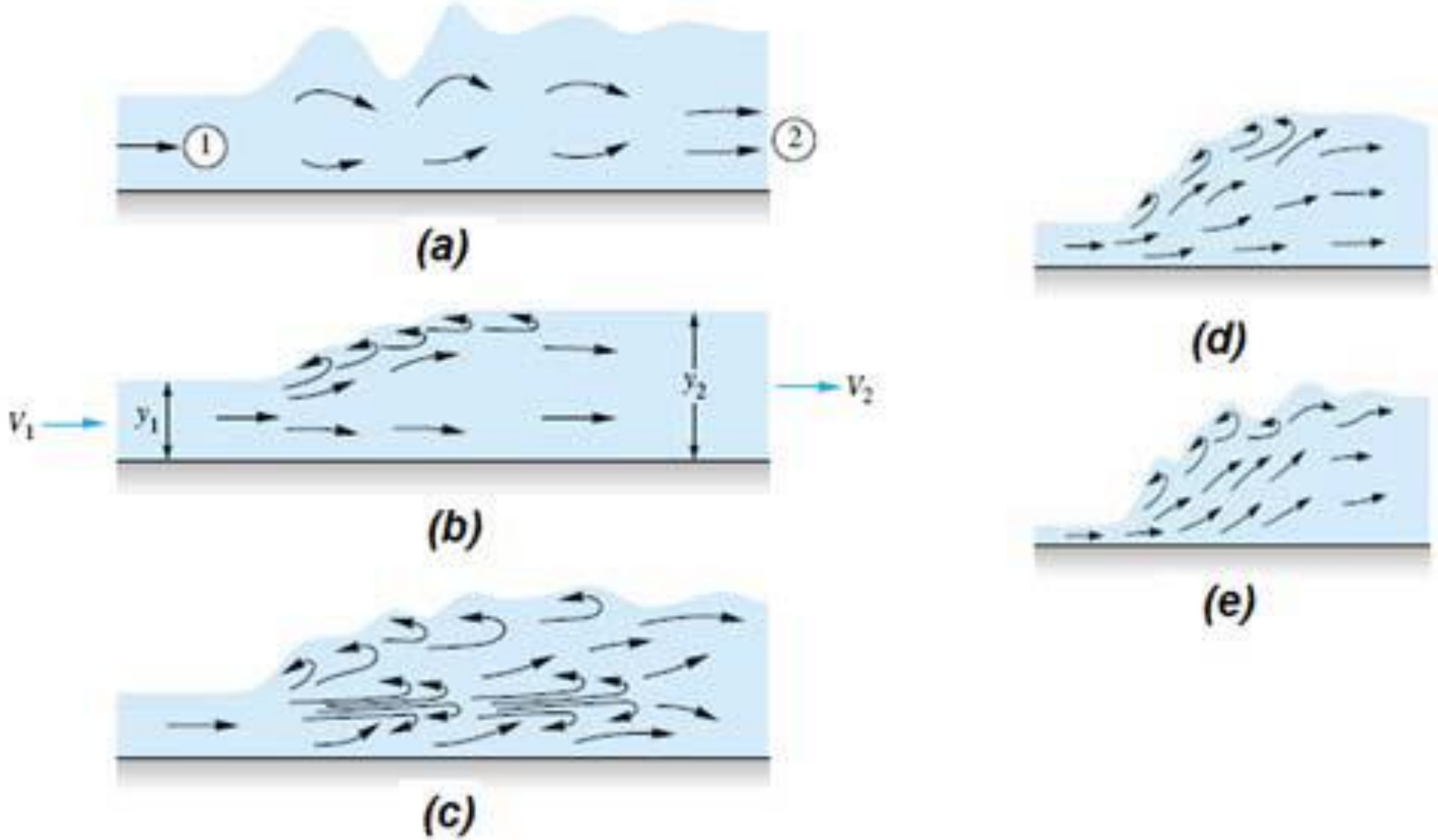


Hydraulic jump



Specific force curve

Types of Jumps: -



(a) Fr = 1.0 to 1.7: undular jumps;
(b) Fr = 1.7 to 2.5: weak jump;
(c) Fr = 2.5 to 4.5: oscillating jump;
(d) Fr = 4.5 to 9.0: steady jump
(e) Fr $\geq$ 9.0 : Strong jump

1) Fr*1* <1.0: Jump impossible, violates second law of thermodynamics.
2) Fr1 =1 ; No jump , critical flow
3) Fr1 = 1.0 to 1.7: Standing wave, or undular, jump about 4.y2 long; low dissipation, less than 5 percent.
4) Fr1 = 1.7 to 2.5: Smooth surface rise with small rollers, known as a weak jump; dissipation 5 to 15 percent.
5) Fr1 = 2.5 to 4.5: Unstable, oscillating jump, each irregular pulsation creates a large wave which can travel downstream for miles, damaging earth banks and other structures. Not recommended for design conditions. Dissipation 15 to 45%.
6) Fr1 = 4.5 to 9.0: Stable, well-balanced, steady jump; best performance and action, insensitive to downstream conditions. Best design range. Dissipation 45 to 70%.
7) Fr1 = >9.0: Rough, somewhat intermittent strong jump, but good performance. Dissipation 70 to 85%.

Equations used in Hydraulic jump:

$$\frac{q^2}{g} = \frac{1}{2} D_1 D_2 (D_1 + D_2)$$

$$d_2 = \frac{d_1}{2}\left(\sqrt{1+8F_{r1}^{\,2}} - 1\right)$$

This is a quadratic equation, the solution of which may be written as:

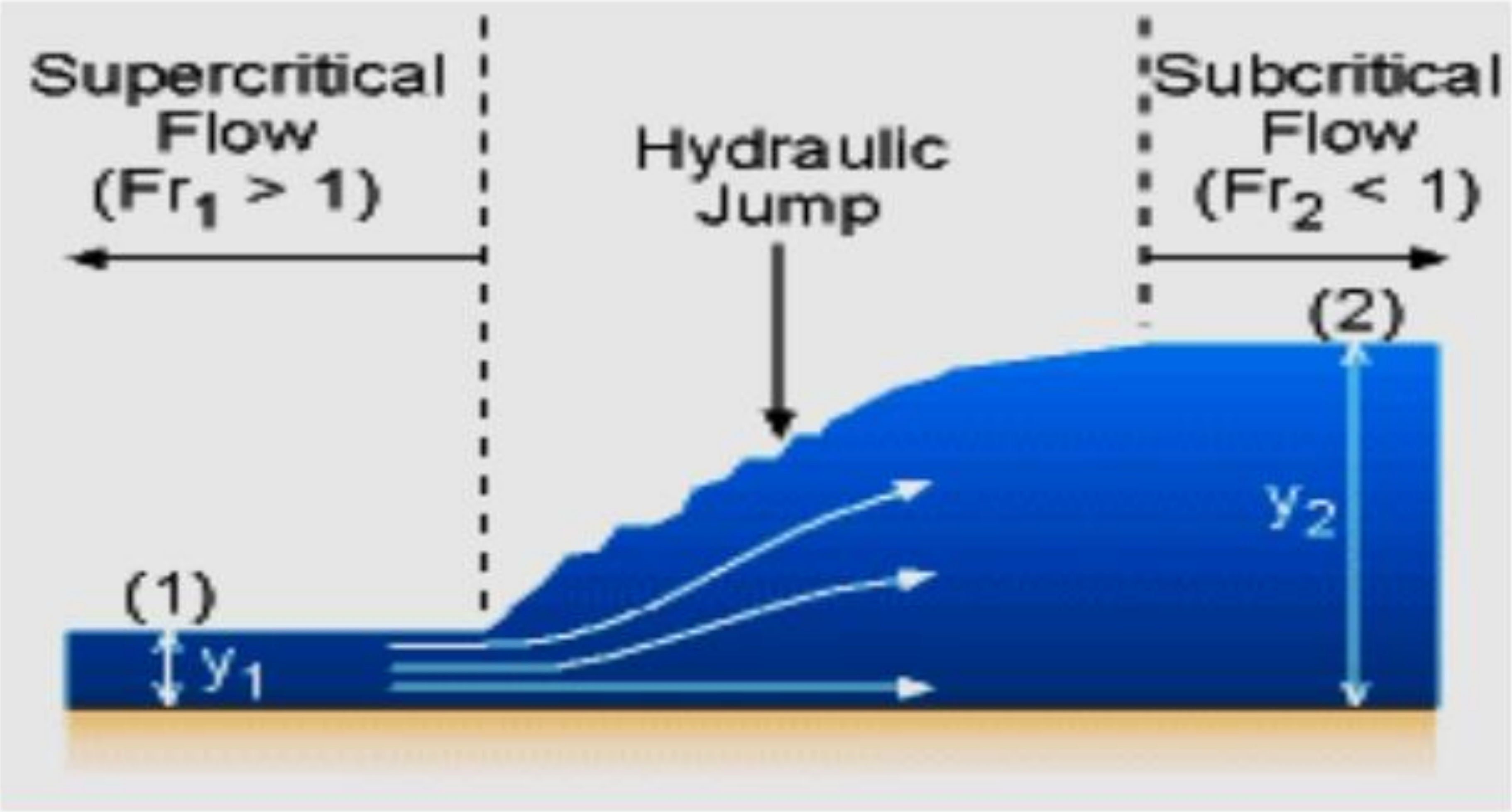
$$y_2 = -\frac{y_1}{2} + \sqrt{\left(\frac{y_1}{2}\right)^2 + \left(\frac{2q^2}{g\,y_1}\right)} \Rightarrow \frac{y_2}{y_1} = \frac{1}{2}\left(-1 + \sqrt{1 + \frac{8q^2}{g\,y_1^3}}\right)$$

$$y_1 = -\frac{y_2}{2} + \sqrt{\left(\frac{y_2}{2}\right)^2 + \left(\frac{2q^2}{g\,y_2}\right)} \Rightarrow \frac{y_1}{y_2} = \frac{1}{2}\left(-1 + \sqrt{1 + \frac{8q^2}{g\,y_2^3}}\right)$$

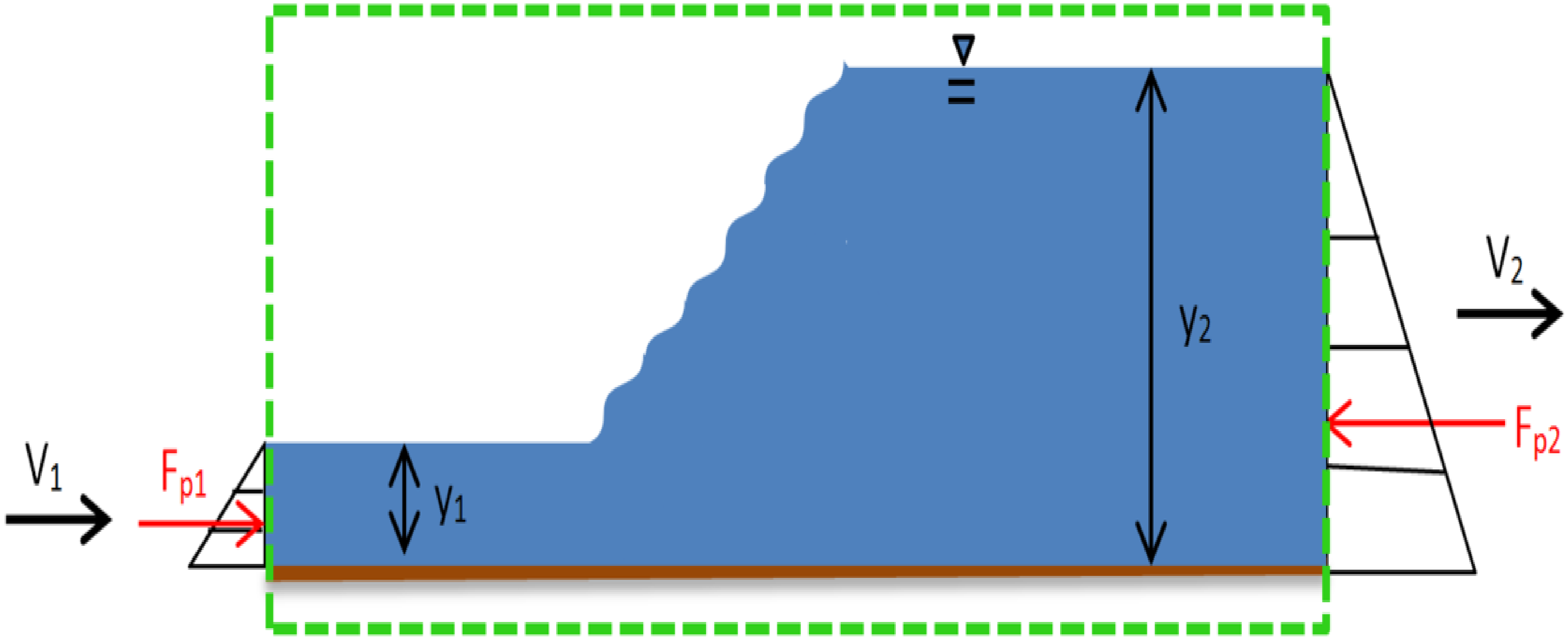
where y_1 is the initial depth and y_2 is called the conjugate depth. Both are called ***conjugate depths***.

These equations can be used to get the various characteristics of hydraulic jump.

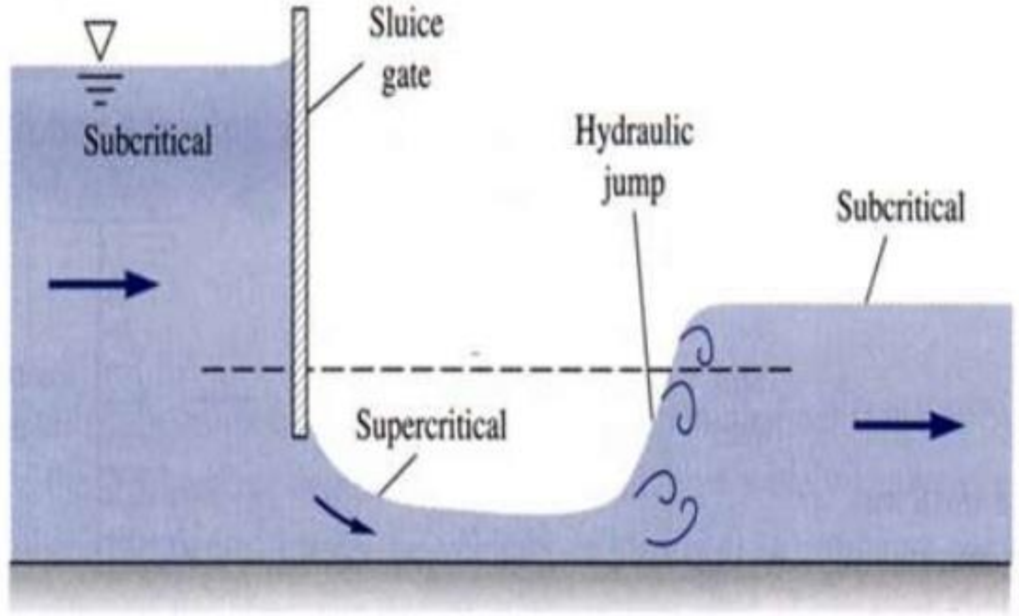
$$\Delta E = E_1 - E_2 = \left(D_1 + \frac{V_1^{\,2}}{2g}\right) - \left(D_2 + \frac{V_2^{\,2}}{2g}\right) \quad \text{or} \quad \Delta E = \frac{(y_2 - y_1)^3}{4 y_1 y_2}$$



Supercritical
Flow
(Fr1 > 1)
Hydraulic
Jump
Subcritical
Flow
(Fr2 < 1)
(1)
(2)
y1
y2



V_1
F_{p1}
y_1
y_2
V_2
F_{p2}



Flow under a sluice gate accelerates from subcritical to critical to supercritical and then jumps back to subcritical flow

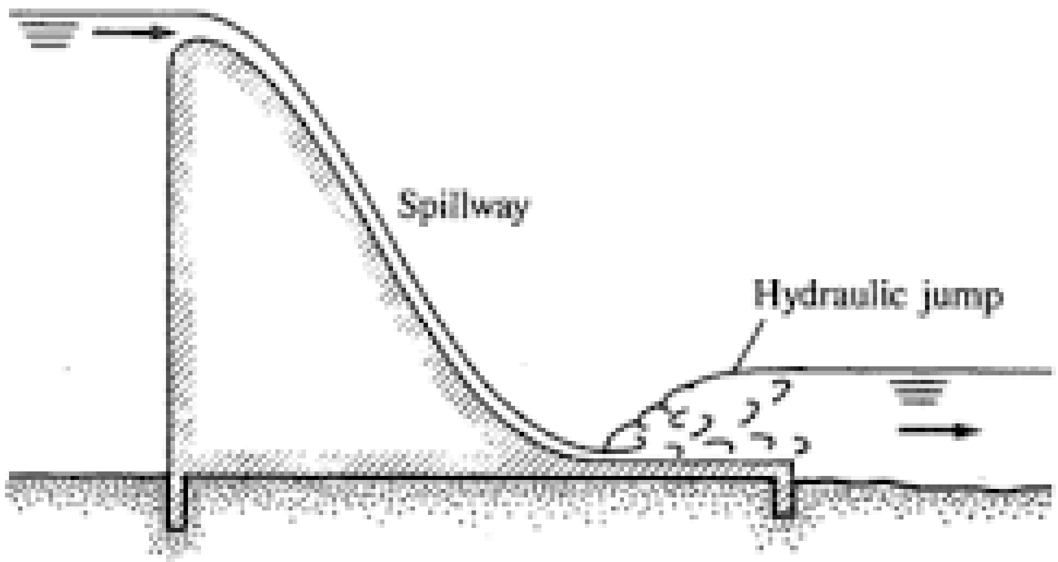
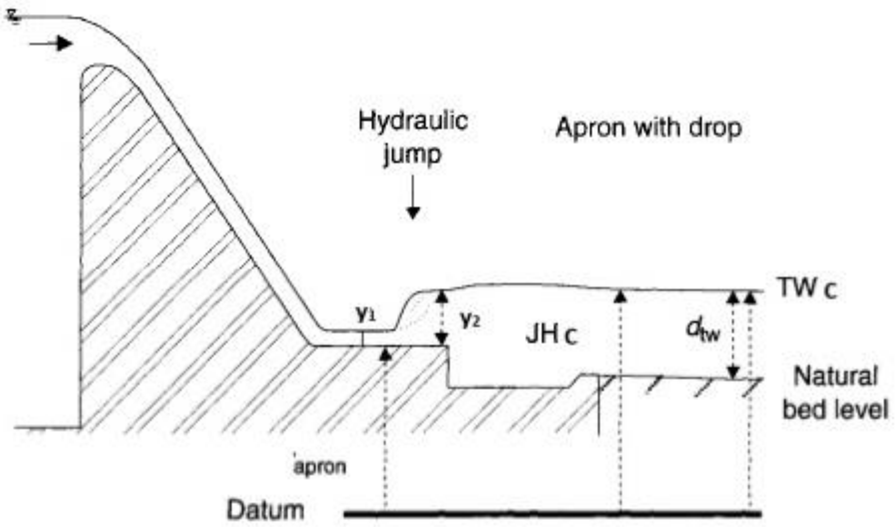
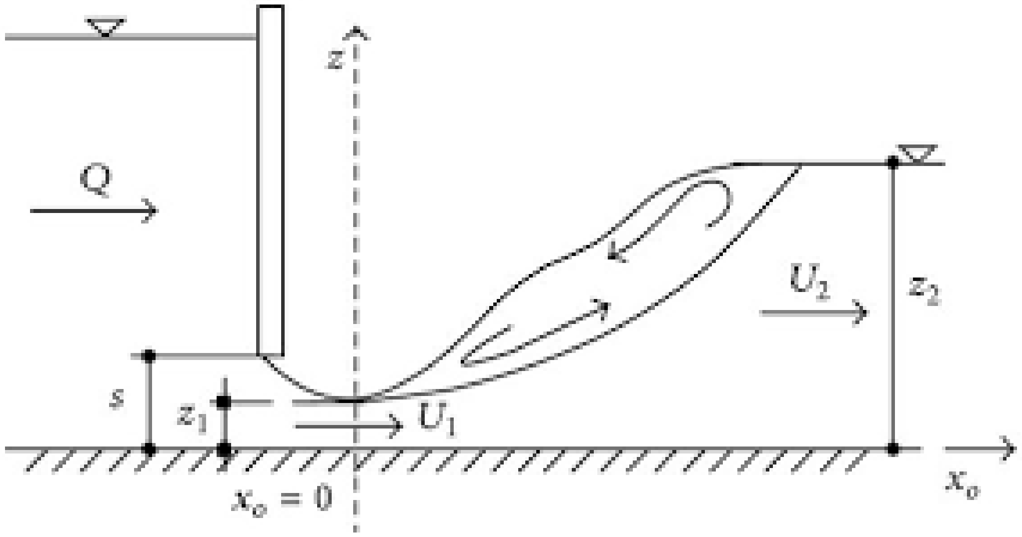


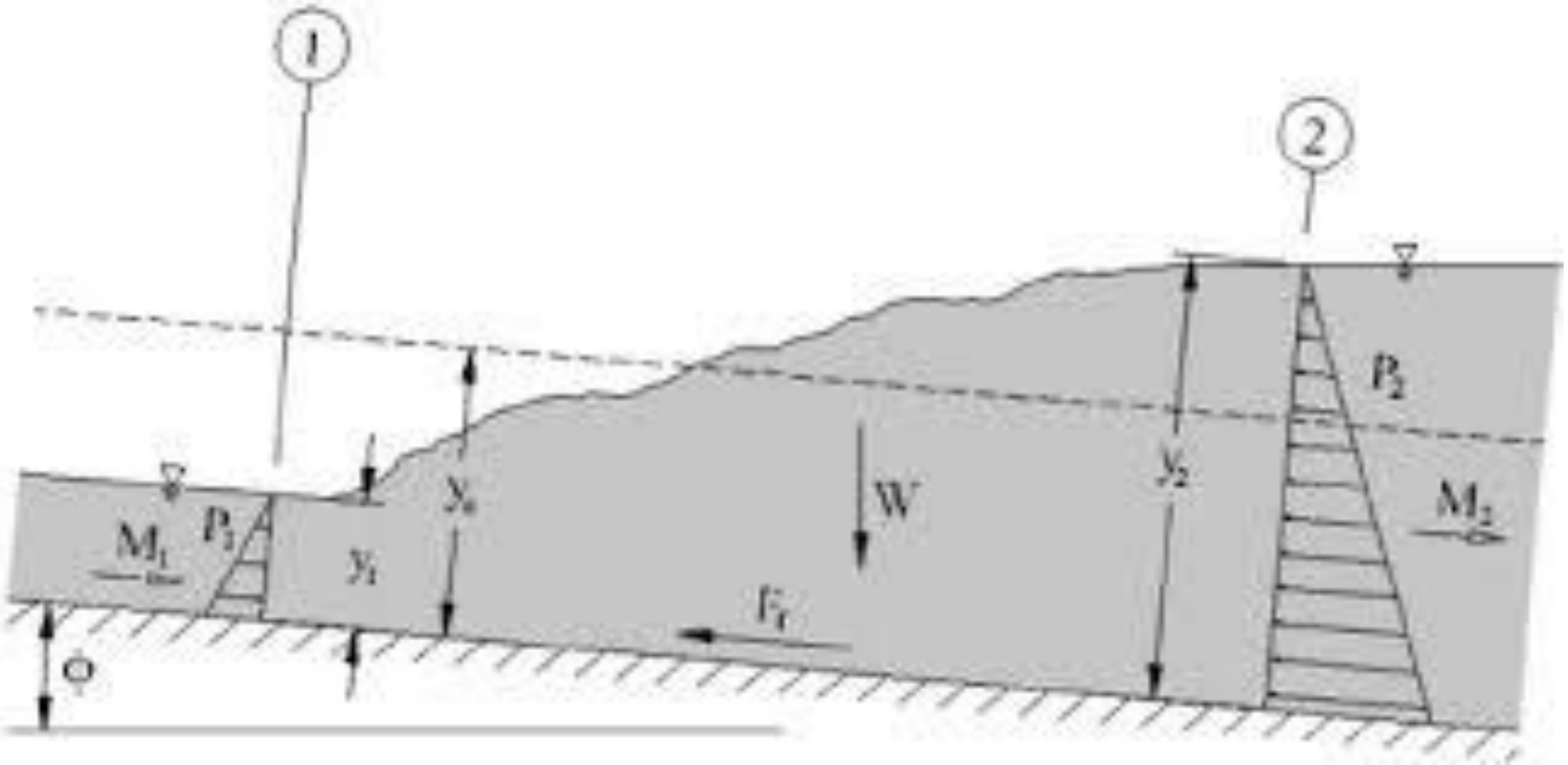
Figure 4-26 Spillway of dam and hydraulic jump



Hydraulic jump
Apron with drop
TW c
y1
y2
JH c
d_{tw}
Natural bed level
apron
Datum

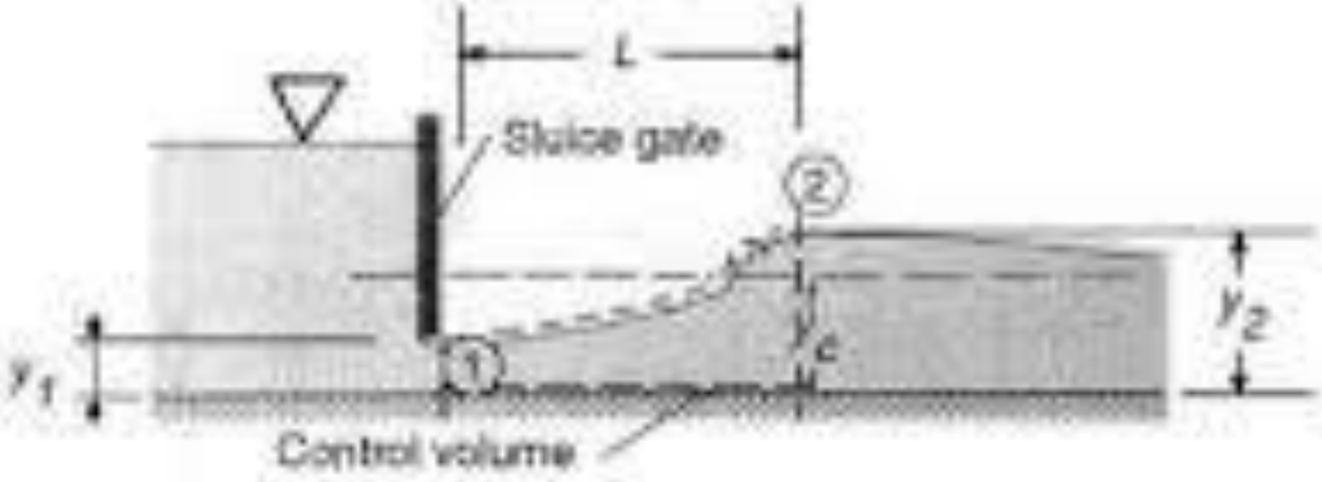


Q
z
s
z_1
U_1
U_2
z_2
$x_o = 0$
x_o



1
2
P1
P2
M1
M2
y1
y2
W
Ff
φ


L
V_1
V_2


L
Sluice gate
2
1
y_c
y_1
y_2
Control volume

Standard Stilling basins:

Stilling basins is a short length of paved channel placed at the end of any source at super critical flow. The aim of the designer is to make a hydraulic jump form within the basin, so that the flow is converted to sub critical flow before it reaches the exposed and unpaved riverbed D/S.

احواض التسكين القياسية:

و هي عبارة عن قناة بطول قصير مبطنة وموضوعة في نهاية اي مصدر من مصادر الجريان فوق الحرج. وهدف المصمم هو جعل القفزة الهيدروليكية تحدث في هذا الموضع المبطن او في هذا الحوض و بهذا يتحول الجريان الى تحت الحرج قبل ان يصل الى القناة الغير مبطنة.

وهناك انواع قياسية لاحواض التسكين يحوي كل منها على تراكيب معينة تساعد على زيادة وسرعة تشتيت طاقة الماء القادمة وفيما يلي تعاريف مختصرة لهذه التراكيب:-

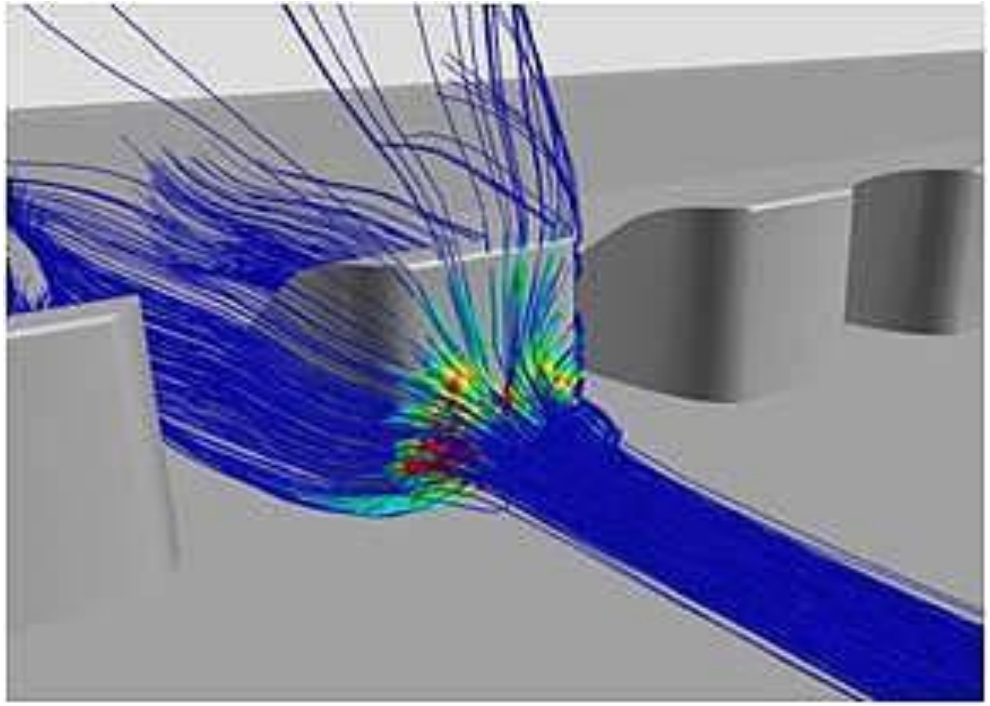
1) Chute blocks: -

Their function to separates the supercritical flow into lower and upper layers at the blocks.

Or to furrow the incoming jet and lift a portion of it from the floor.

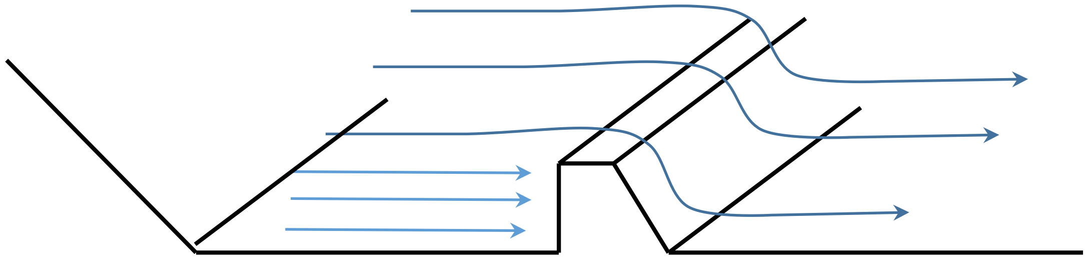
In this action the created turbulent of energy.

توضع في بداية الحوض وتقسم الماء الى سواقي ما بين البلوكات و يقذف الجرء الاخر من فوق البلوكات ليتداخل بشكل عشوائي فتضيع الطاقة.



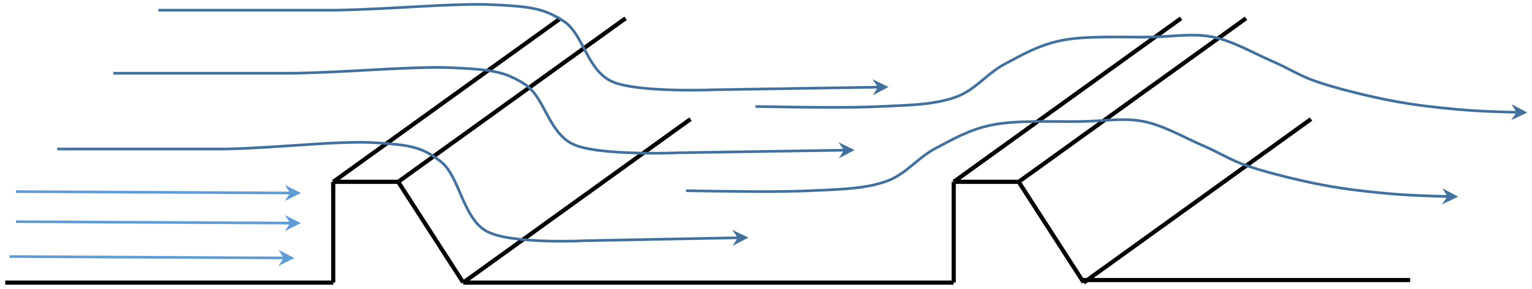
2) **Impact blocks (Baffle piers): -**

Are blocks placed in the intermediate position across the basin floor. Their function is to dissipate energy mostly by impact action, a big percentage of excessive energy is dissipated through impact with the blocks.

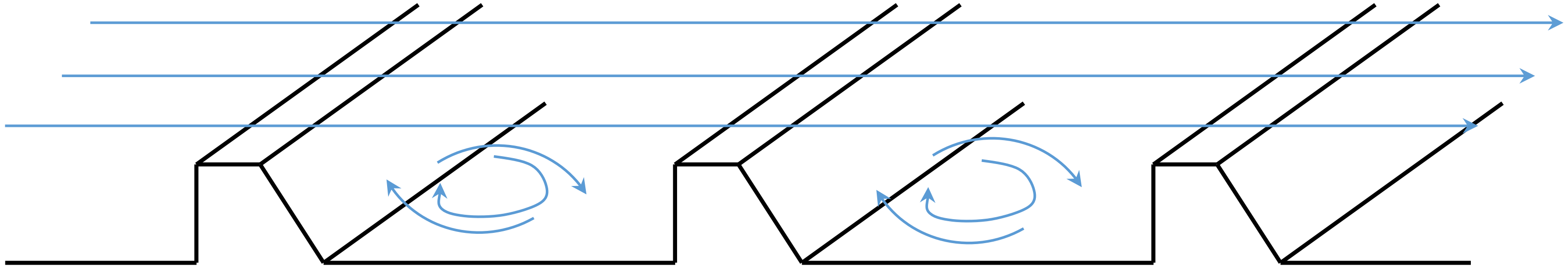


In super critical flow, if the blocks are continuous or near to each other, the supercritical flow may overshoot the blocks.

اذا كانت المسافة قريبة بين البلوكات فان الماء سينقذف من فوقها بدون ان تتشتتالطاقة وسيكون هناك ماء محصور بين البلوكات مما يجعل الطاقة الضائعة قليلة جدا. لذلك يجب ان يتم حساب المسافات بين البلوكات بدقة لتجنب هذه الحالة.



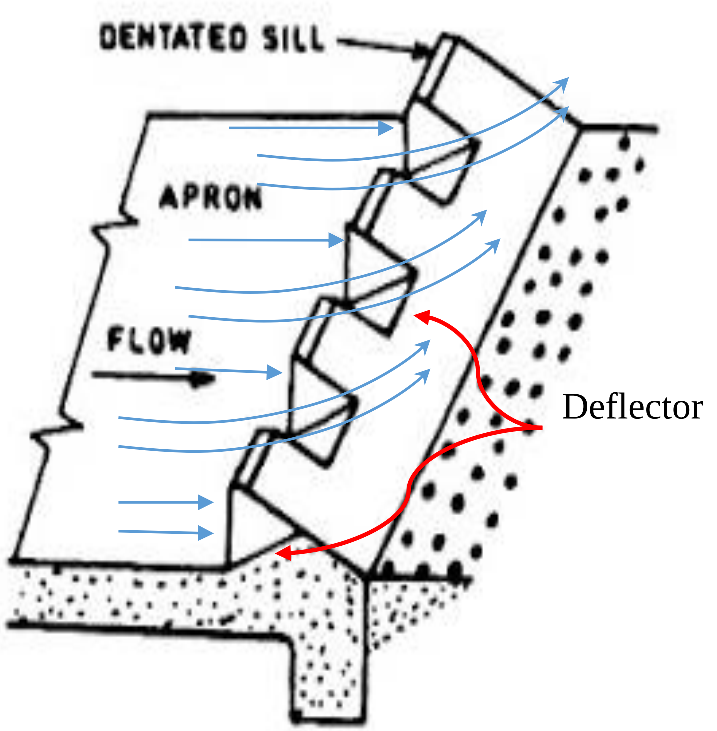
There is impact



There is no impact and roughness of whole structure is less

3) End Sills:

Either dentated or solid, is usually provided at the end of the stilling basin. Its function is to reduce further the length of the jump and to control scour. For large basins that are designed for high incoming velocities, the sill is usually dentated to perform of high- velocity jet that may reach the end of the basin

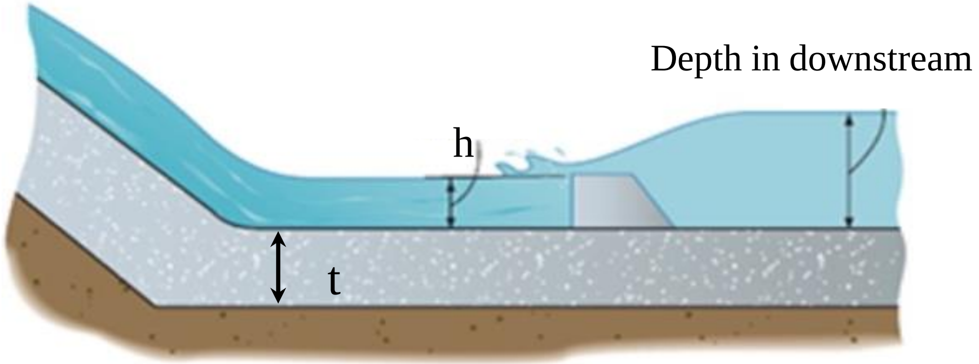


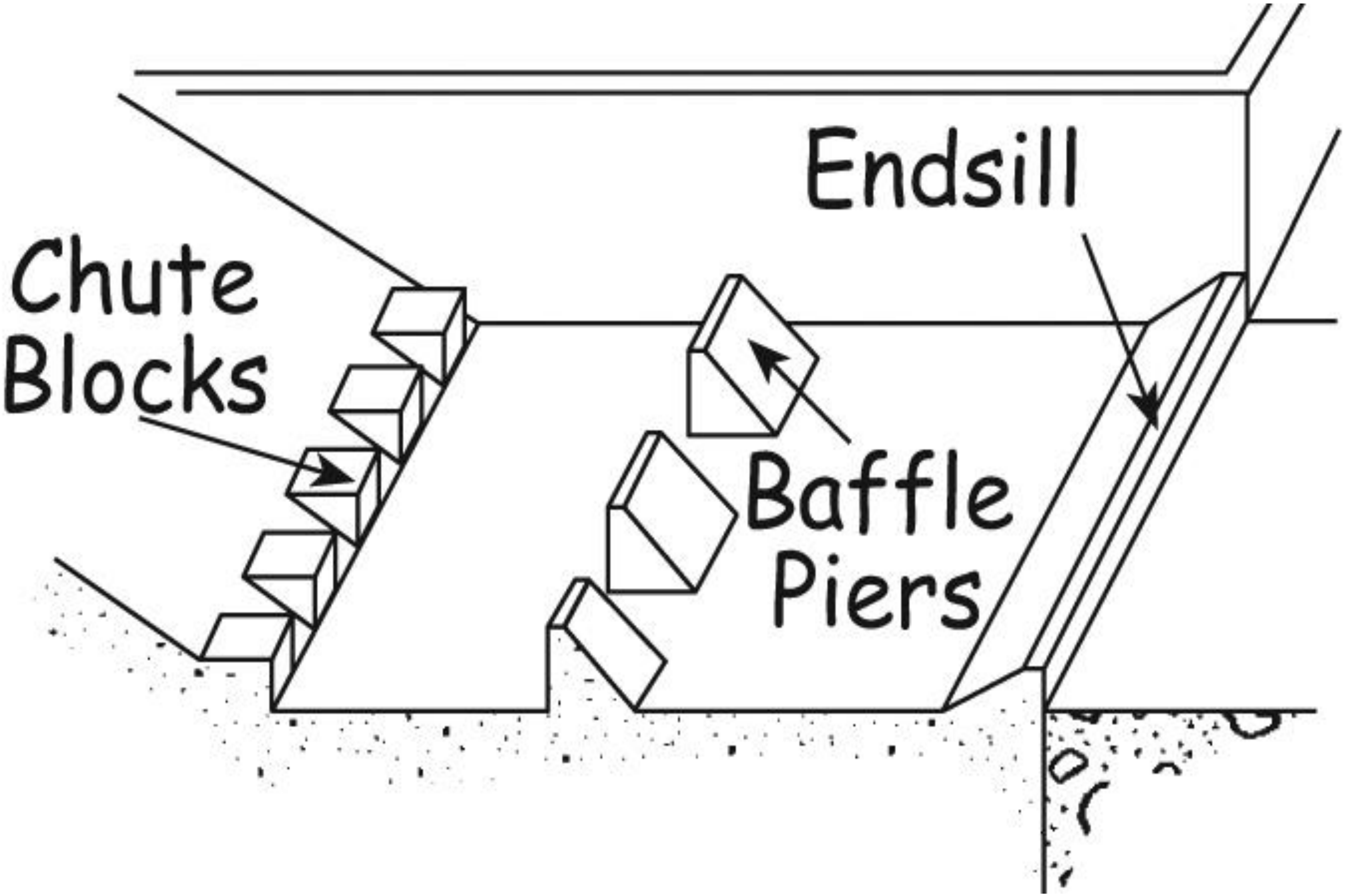
The effect of dentated sill is practically same as chute and impact blocks.

*Continuous sill has the following functions: -

1. AS impact wall.
2. It keeps the water at its U/S as a stilling pool.
3. The depth of water at its U/S help to reduce the floor thickness.

$$needed\ \ t = \frac{U.P.-h}{G-1}$$




Endsill
Chute
Blocks
Baffle
Piers

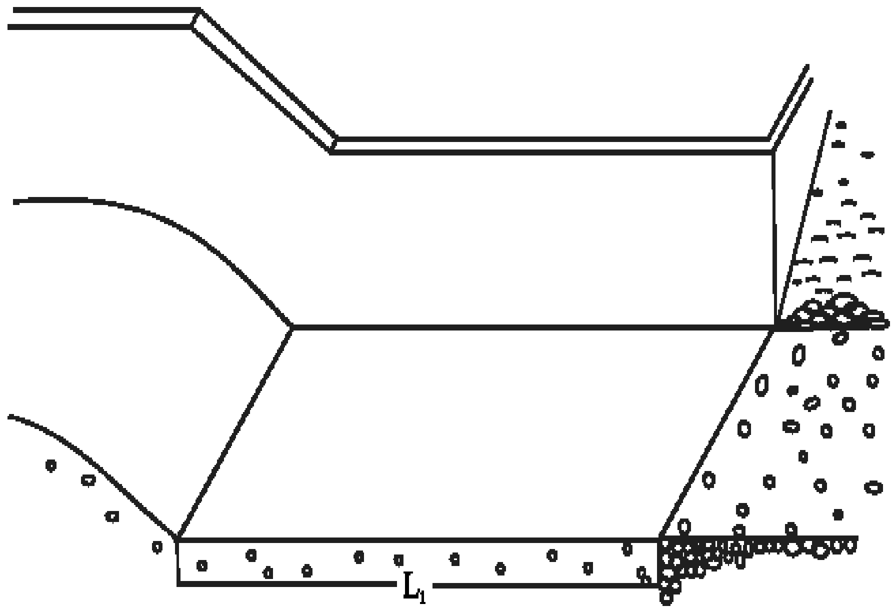
USBR Stilling Basins:

Using the U.S.B.R (U.S. Bureau of Reclamation) 1987, classification, there are five basic hydraulic jump type basins that are briefly described: -

USBR basin I:

In this type of the stilling basins the hydraulic jump is occurring on a flat floor without chute blocks, or sills. The basin length is equal to the hydraulic jump length. However, such a basin is usually not very practical because of its expensive length and its lack of control.

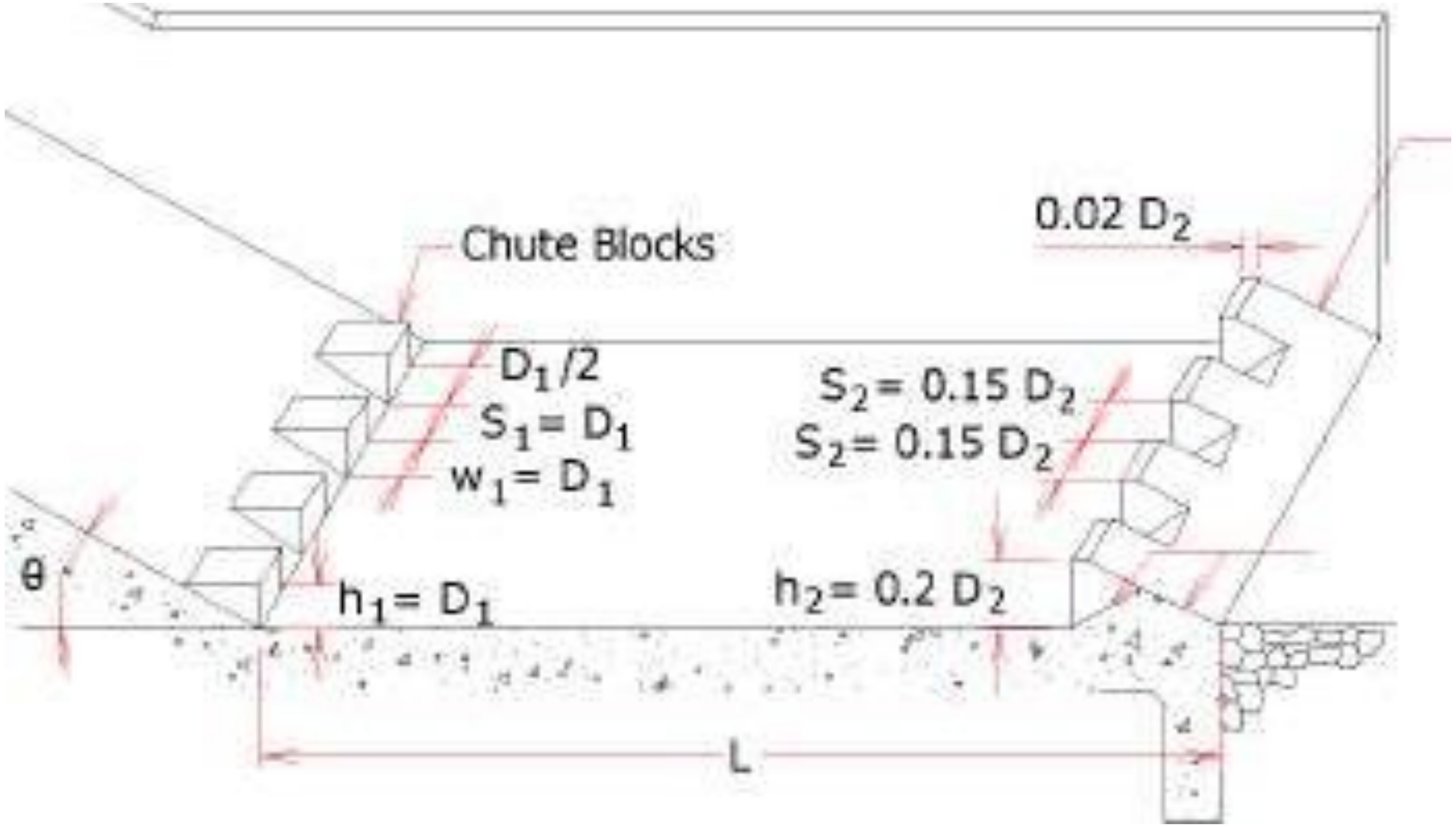
For **Froude number, less than 1.7**, no special stilling basin is required. **For Froude number between 1.7 and 2.5** the type I-basin also applies.



Type I basin dimensions

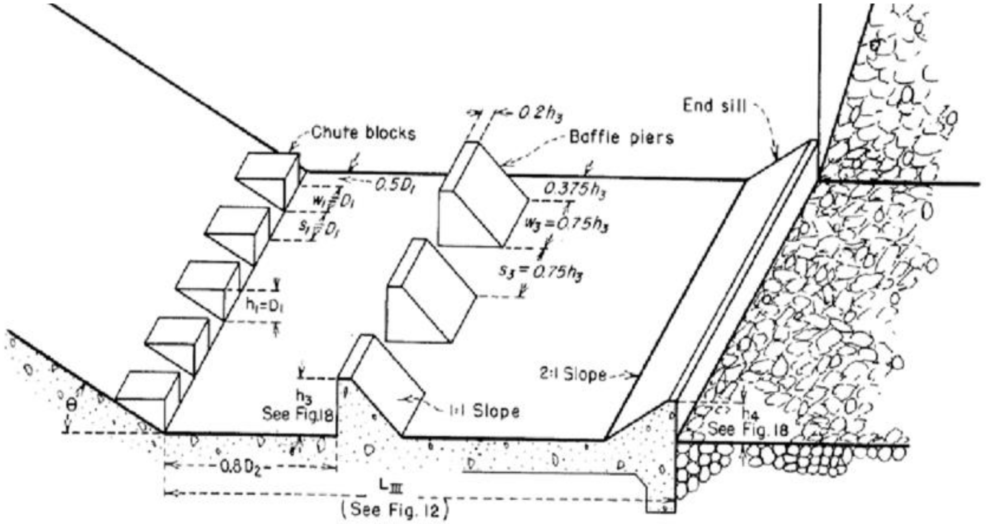
USBR basin II:

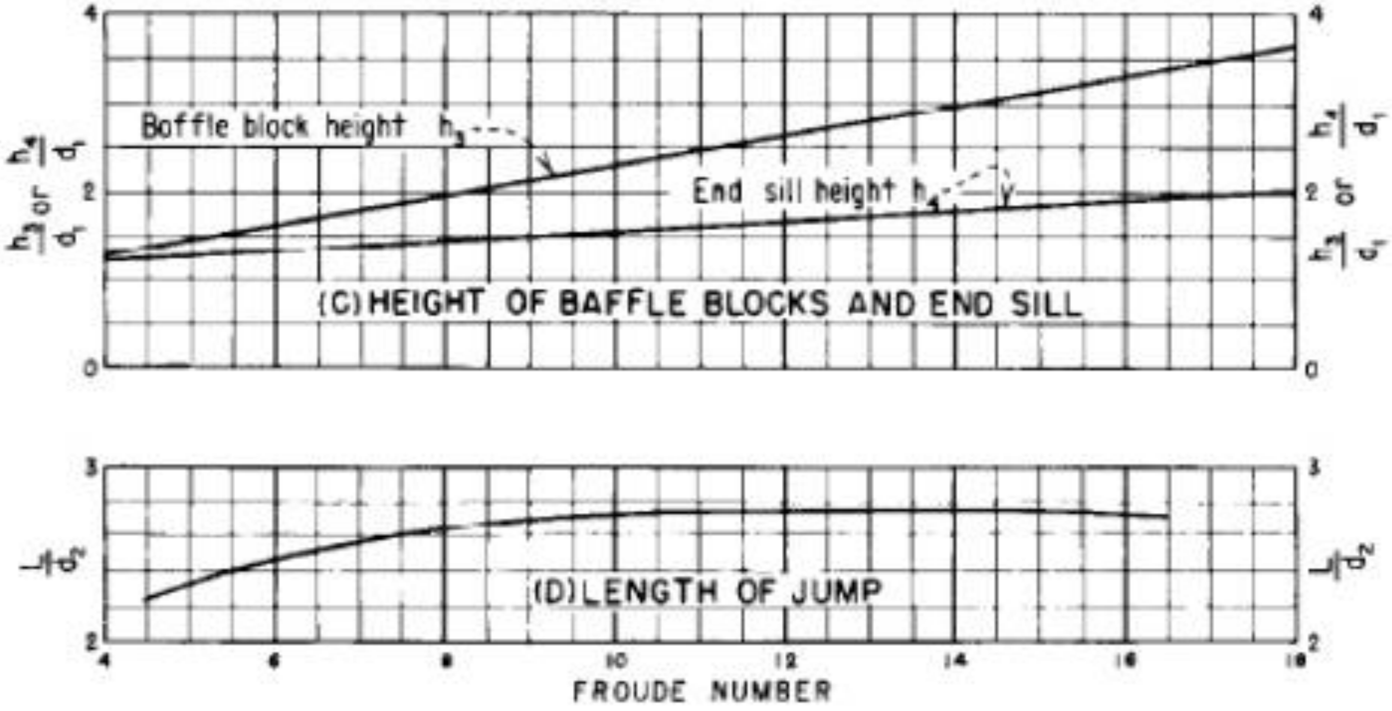
It is used when the incoming **velocities exceed 15 m/s and Fr >4.5**, and for high spillway or high head and large structures. The basin contains chute blocks at U/S end & a dentated sill near D/S end. No baffles piers are used because the relatively high velocities entering the jump might cause cavitation on piers.



USBR basin III:

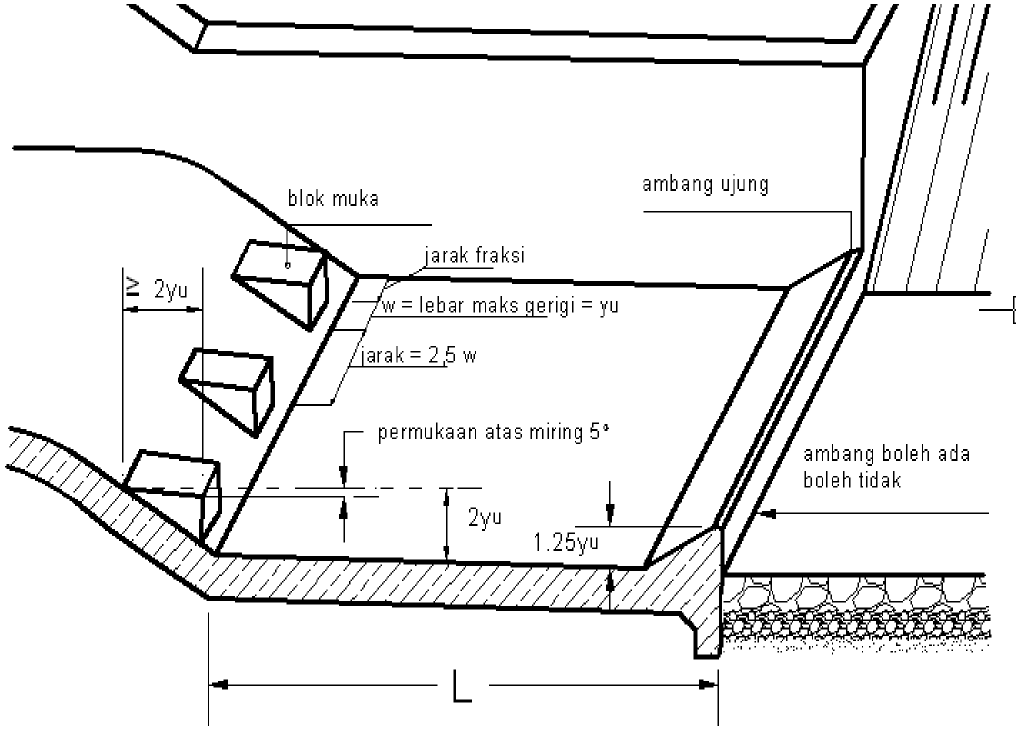
This basin may be used when the incoming **velocity do not exceed 15 m/s, and for <u>Fr >4.5</u>** but small structures. It is the same as no II but with additional impact blocks and continuous end sill.





USBR basin IV

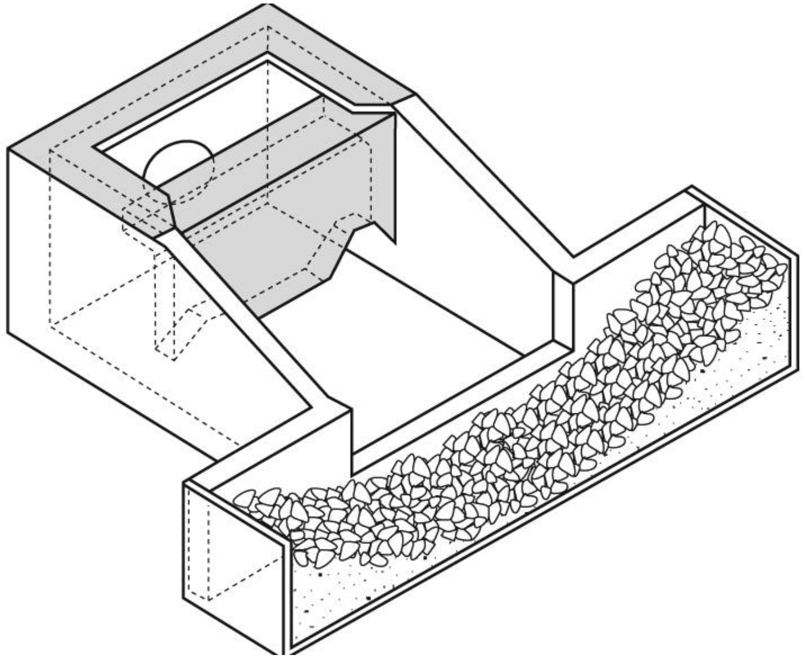
This recommended for use with jump of **Fr1 = 2.5 to 4.5 (Oscillating jump)** which usually occur on canal structures and diversion dams, this basin is applicable to rectangular cross section only, it is designed to eliminating the wave at its source.

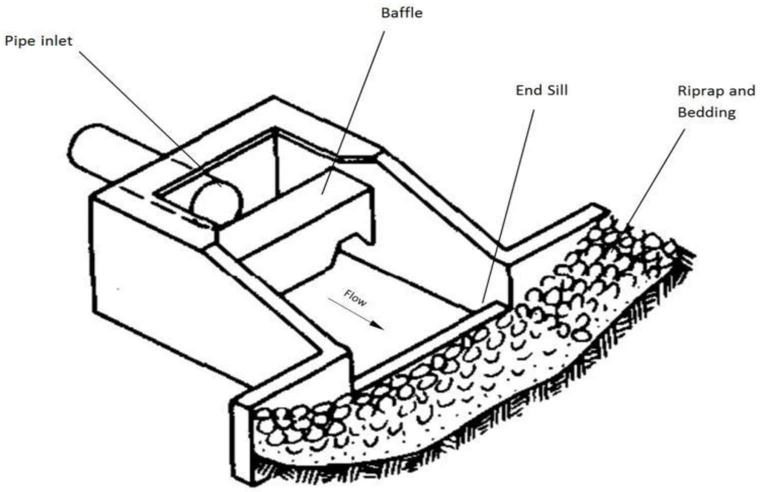


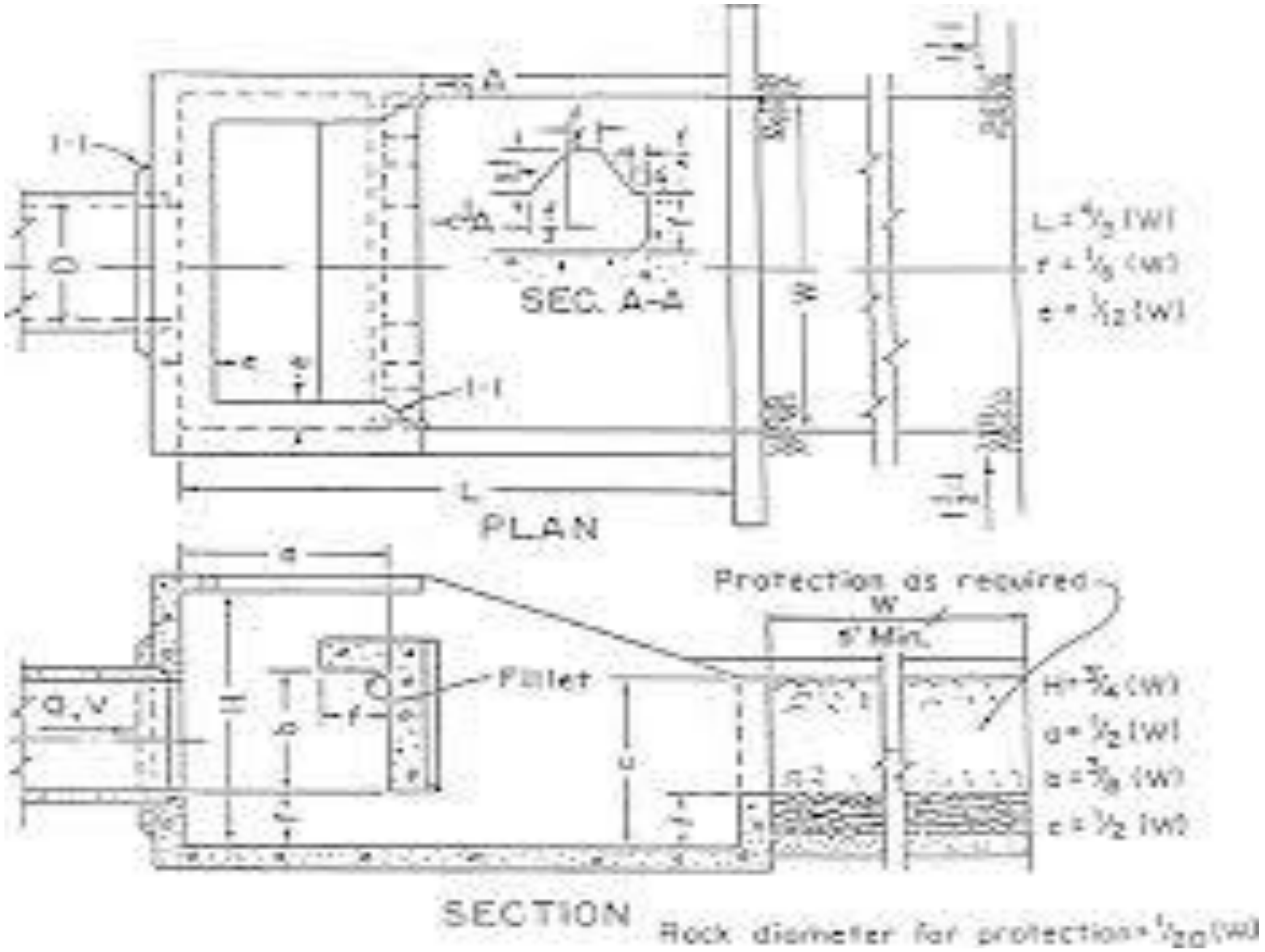
معظم الاحيان يستخدم طوف (Raft) فوق الماء لتقليل ذبذبة الماء (يضعون جذوع اشجاراو انابيب مربوطة بشكل افقي سوية وتربط وتوضع في القناة لتهدئة هذا النوع من القفزة العشوائية).هذا النوع من الاحواض لا يزيل خطورة القفزة (Oscillating jump) وانما يقللها.

USBR basin V:

It can be used for small structures and closed conduit structures, of energy is accomplished by the impact of the incoming jet on the vertical hunging baffle and by eddies which are formed from the change direction of the jet after it strike the baffle.



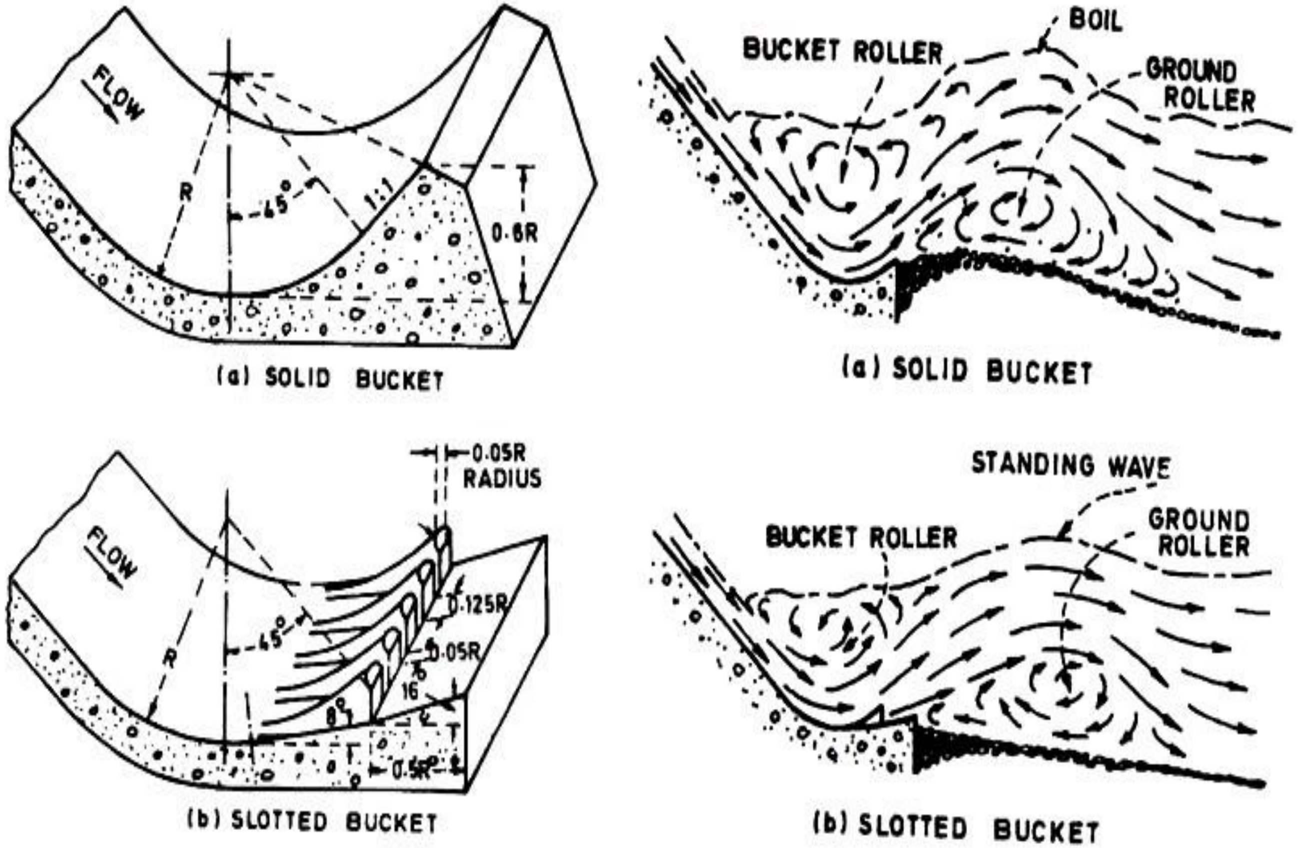


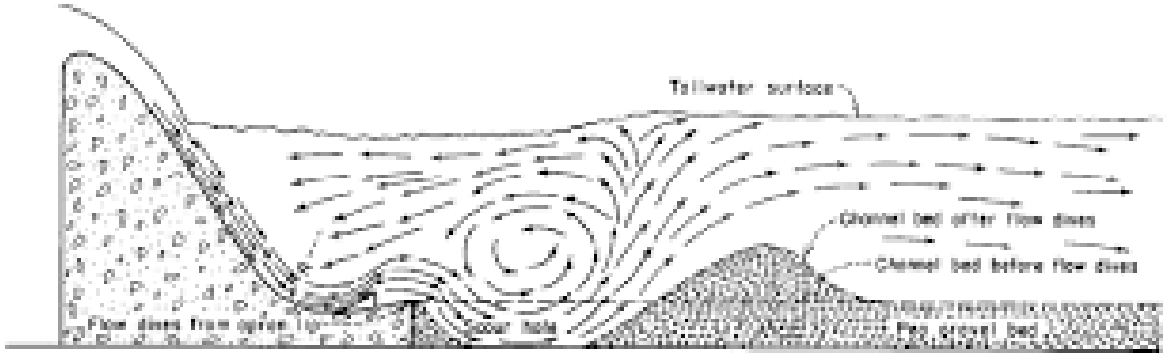


SEC. A-A
L = 4/3 (W)
f = 1/6 (W)
e = 1/12 (W)
PLAN
Protection as required
W
6" Min.
Fillet
H = 3/4 (W)
a = 1/2 (W)
b = 3/8 (W)
c = 1/2 (W)
Q, V
SECTION
Rock diameter for protection = 1/20 (W)

USBR Stilling basin VII:

This type of energy dissipator is suitable where the stream bed is composed of hard rock and the tail water depth is less than D_2 curve.





***S.A.F. Stilling Basin (Saint Anthony Falls)*:**

This is recommended for use on small structures such as small spillways, outlet works, and small canal structures. It has a particularly wide range of application conditions **($1.7 < Fr_1 < 17$)** and is very effective in shortening the jump. The reduction in basin length achieved through the use of appurtenances (baffle blocks, a sloping entrance and an end sill) designed for the basin is about 80%. This stilling basin was developed at the St. Anthony Falls Hydraulic Laboratory, University of Minnesota. The geometry and dimensions for a straight-sided basin are shown in Figures below. The design rules are summarized as follows:

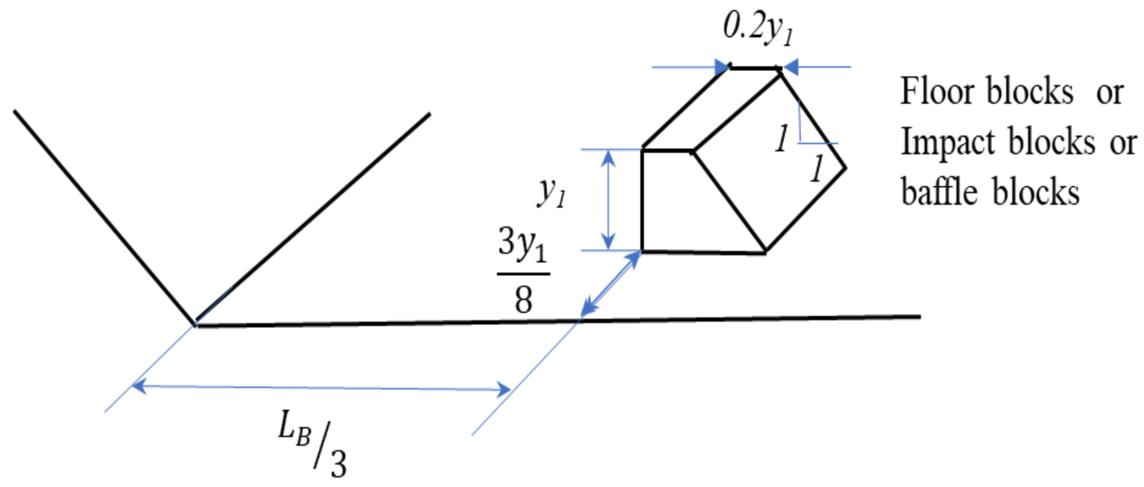
1. The stilling basin Length of basin (LB) is related to the incoming Froude number Fr_1 *by:*

$$L_B = \frac{4.5y_2}{{F_{r1}}^{0.76}}$$

2. *The height of the chute blocks and floor blocks is y_1, and the width and spacing are approximately $0.75y_1$.*

$$width = spacing = 0.75y_1$$

3. *The baffle piers are placed directly downstream of the openings between the chute blocks at a distance equal to $L_B/3$ from the ends of the chute blocks.*



4. No floor block (baffle pier) should be placed closer to the side wall than $\frac{3y_1}{8}$

5. The floor blocks (baffle piers) should occupy between 40 and 55% of the stilling basin width.

مجموع عرض البلوكات الارضية ككل = عرض البلوكة الواحدة ×عدد البلوكات

مجموع عرض البلوكات الارضية/عرض الحوض عند الوسط (منطقة البلوكات الارضية)×100 = (55-40)%

6. Hight of the floor blocks = y_1

and width of the floor blocks = $0.75 y_1 \left(\frac{B_2}{B_1}\right)$

حيث (B_1) عرض الحوض في المقدمة عند ال chute
و (B_2) عرض الحوض في الوسط عند ال floor blocks

7. The widths and spacing's of the floor blocks (baffle piers) for diverging stilling basins should be increased in proportion to the increase in stilling-basin width at the floor-block location.

8. The height of end sill is given by c = 0.07y2 , where y2 is the theoretical sequent depth corresponding to y1.

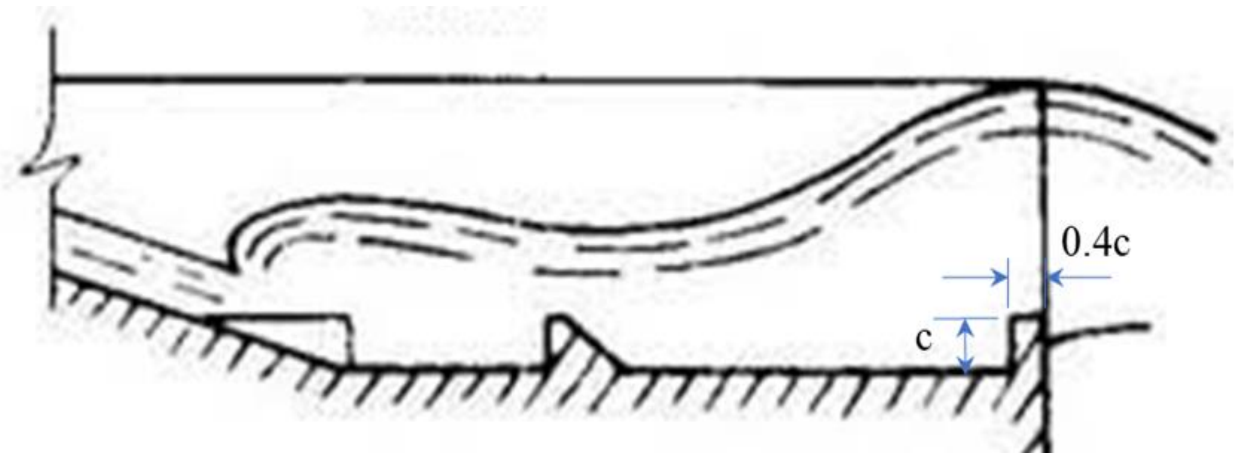
9. The depth of tail water above the stilling-basin floor is given by:

$$y_2' = \left(1.1 - \frac{F_{r1}^{\ 2}}{120}\right) y_2 \qquad for\ F_{r1} = 1.7 - 5.5$$

$$y_2' = 0.85 y_2 \qquad for\ F_{r1} = 5.5 - 11$$

$$y_2' = \left(1 - \frac{F_{r1}^{\ 2}}{800}\right) y_2 \qquad for\ F_{r1} = 11 - 17$$

10. The height of the side wall above the maximum tail water depth to be expected during the life of the structure should allow a free board of $Z = \frac{y_2}{3}$.



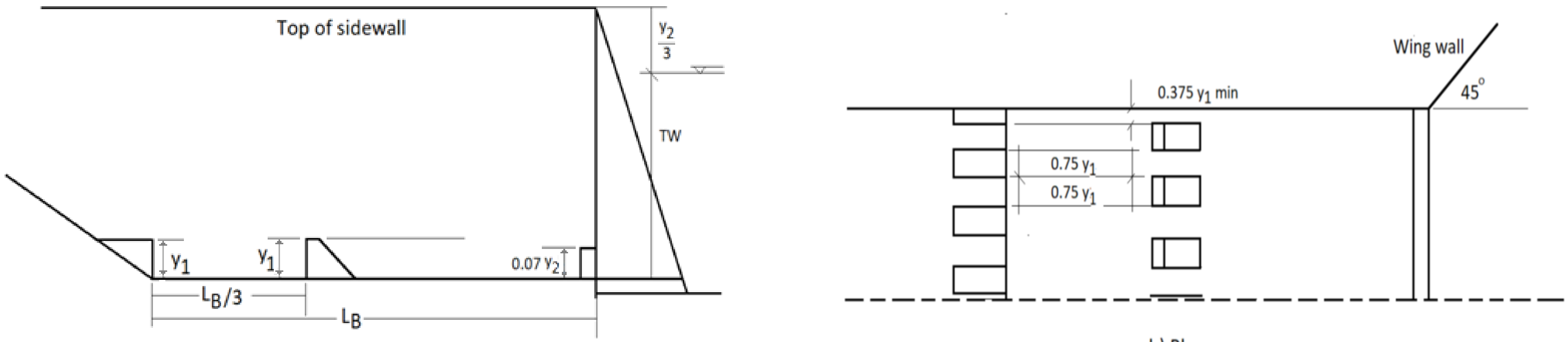
11. *Wing walls should be provided for the transition from the basin to the downstream channel equal in height to the stilling-basin side walls. The top of the wing wall should have a slope of 1 on 1.*

12. The wing wall should be placed at an angle of 45° to the outlet center line.

13. The stilling-basin side walls may be parallel (as in a rectangular stilling basin) or they may diverge as an extension of the transition side walls (as in a trapezoidal stilling basin).

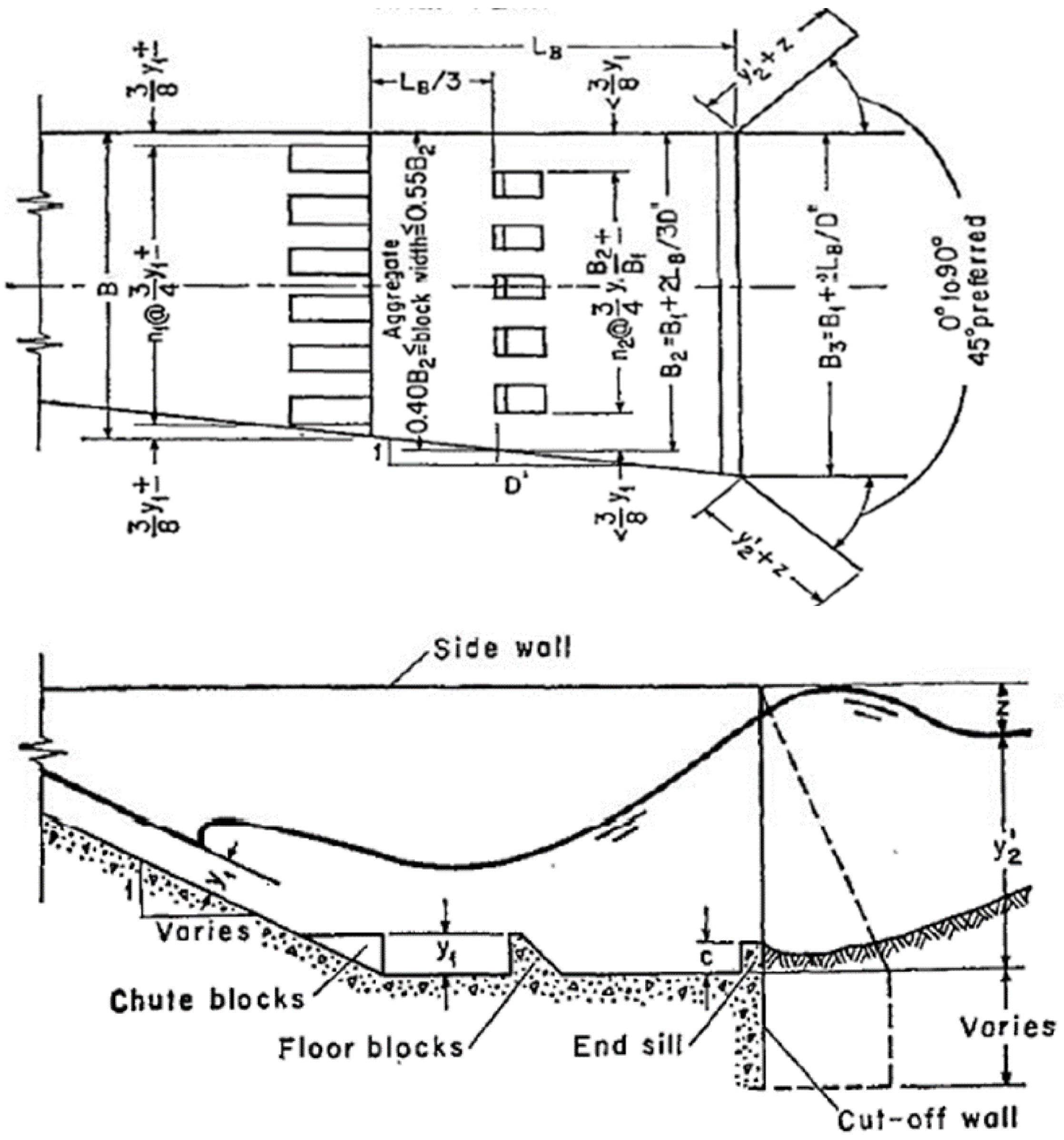
14. A cutoff wall of nominal depth should be used at the end of the stilling basin.

15. The effect of entrained air should be neglected in the design of the stilling basin



a) Longitudinal section

b) Plan



Side wall
Chute blocks
Floor blocks
End sill
Cut-off wall
Varies
Varies
0° to 90°
45° preferred

Example:

In order to dissipate energy below the spillway, it is proposed to form a hydraulic jump in the stilling basin. Due to this, the depth of flow changes from 1.0 m to 3.8 m. Calculate the discharge over the spillway, the critical depth and the head loss in the jump, the crest length of which is 110 m.

Solution:

By using the equation

$$\frac{D_2}{D_1} = \frac{1}{2}\left[\sqrt{1 + 8{F_{r1}}^2} - 1\right] \rightarrow \frac{3.8}{1} = \frac{1}{2}\left[\sqrt{1 + 8{F_{r1}}^2} - 1\right] \rightarrow F_{r1} = 3.02$$

$$F_{r1} = \frac{V_1}{\sqrt{gD_1}} \rightarrow 3.02 = \frac{V_1}{\sqrt{9.81 \times 1}} \rightarrow V_1 = 9.45\, m/sec.$$

$Hance\ the\ discharge\ over\ the\ spillway$

$$Q = V_1 A1 = 9.45(1)(110) = 1040\ \frac{m^3}{sec}$$

Also, you can solve the problem by using the relation:

$\frac{q^2}{g} = \frac{1}{2} D_1 D_2 (D_1 + D_2)$ to find q then find Q

$$q = VD = 9.45 \times 1 = 9.45 \frac{m^2}{sec}.$$

$$y_c = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{9.45^2}{9.81}} = 2.088\ m$$

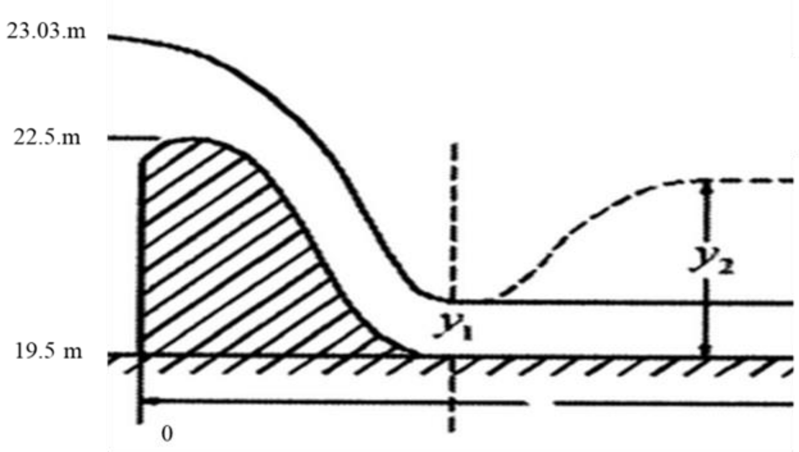
$$H_L = \frac{(D_2 - D_1)^3}{4D_1D_2} = \frac{(3.8-1)^3}{4(1)(3.8)} = 1.444\ m$$

Or

$$H_L = E_1 - E_2 = \left(D_1 + \frac{q^2}{2gD_1{}^2}\right) - \left(D_2 + \frac{q^2}{2gD_2{}^2}\right)$$

Example:

Design U.S.B.R. stilling basin for the following hydraulic structure:



Q= 5.15 m^3/sec

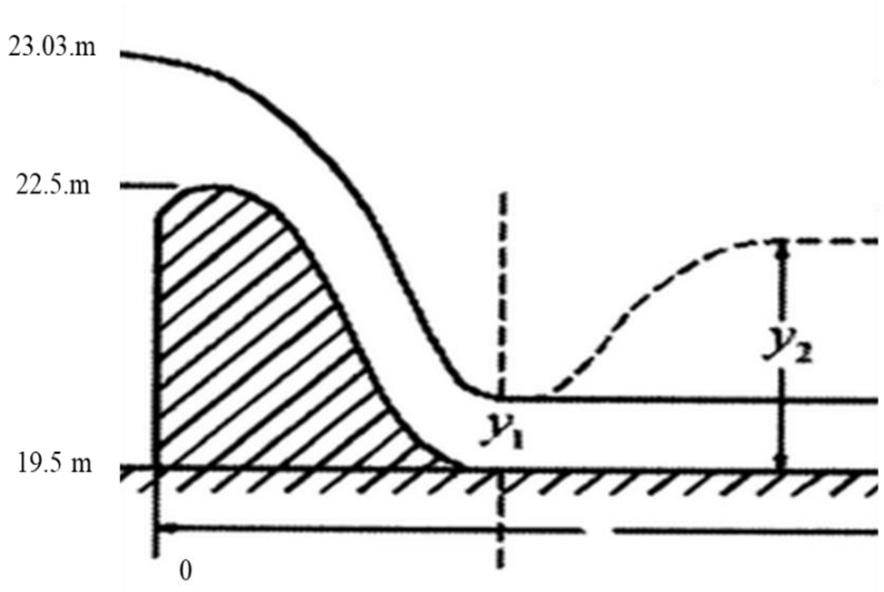
Crest width = 6 m

Neglect effect of approaching velocity

Solution:

$$q = \frac{Q}{B} = \frac{5.15}{6} = 0.858 \ m^3/s.m$$

The energy equation (Bernoulli) between (0 and 1)



$$Z_0 + y_0 + \frac{{V_0}^2}{2g} = Z_1 + y_1 + \frac{{V_1}^2}{2g}$$

$$3.53 = 0 + y_1 + \frac{\left({}^{q}/_{y_1}\right)^2}{2g} \rightarrow 3.53 = y_1 + \frac{0.858^2}{19.62\ {y_1}^2}$$

By trial and error $y_1 = 0.11\ m$

V_1 = 0.858/0.11 = 7.8 m/sec < 15 m/sec.

$$F_{r1} = \frac{7.8}{\sqrt{9.81 \times 0.11}} = 7.51 > 4.5$$

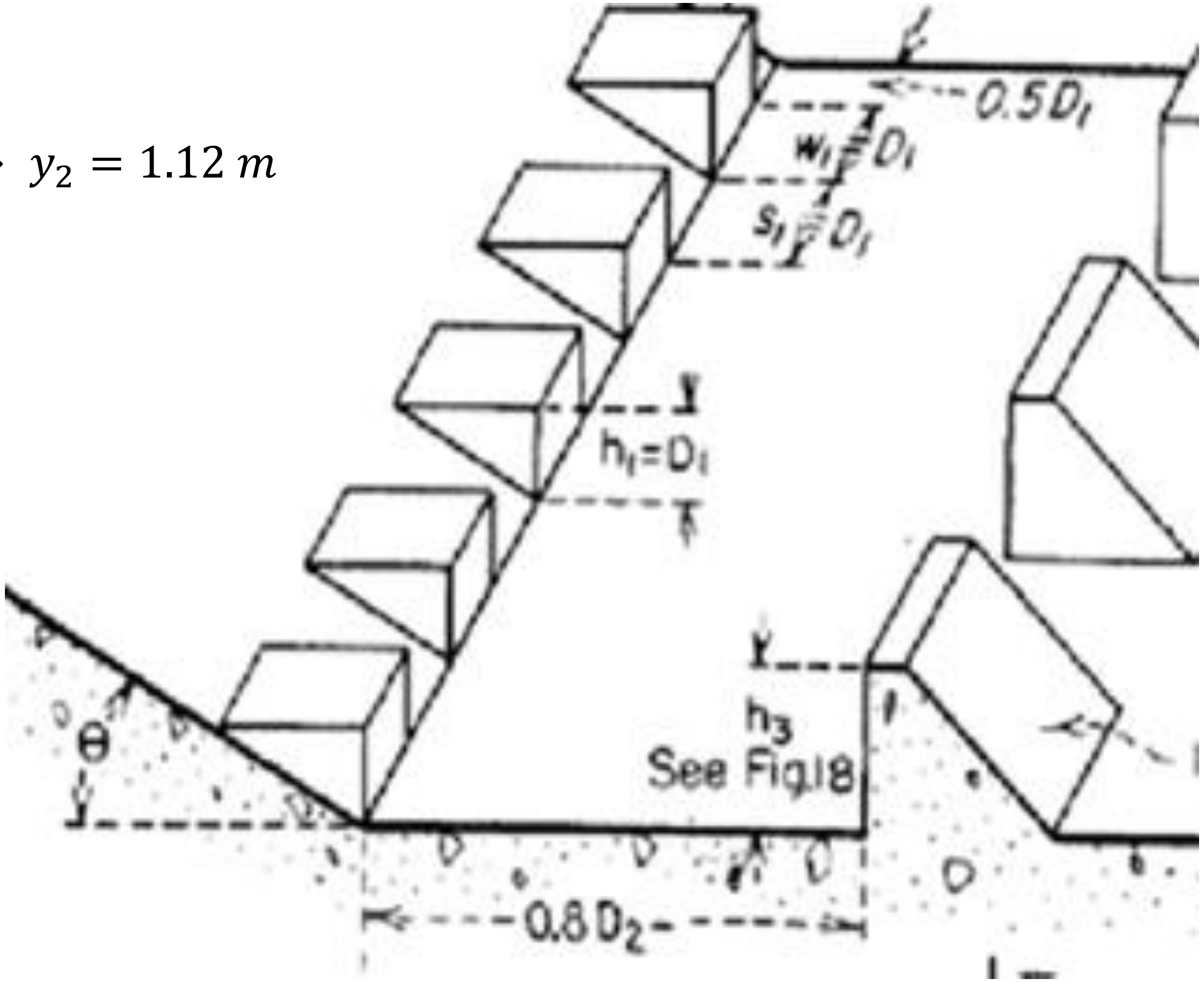
Use No. III basin

ندخل على المخططات الخاصة بالحوض الثالث سابقة الذكر و الموجودة بالمحاضرات

From third curve for $F_{r1} = 7.51$

$$\frac{L}{y_2} = 2.6$$

But $\frac{y_2}{y_1} = \frac{1}{2}\left[\sqrt{1 + 8{F_{r1}}^2} - 1\right] \rightarrow \ y_2 = 1.12\ m$



L =2.93 m   use 3 m

Chute blocks:

$h_1 = y_1 = 0.11\ m$

$w_1 = y_1 = 0.11\ m$

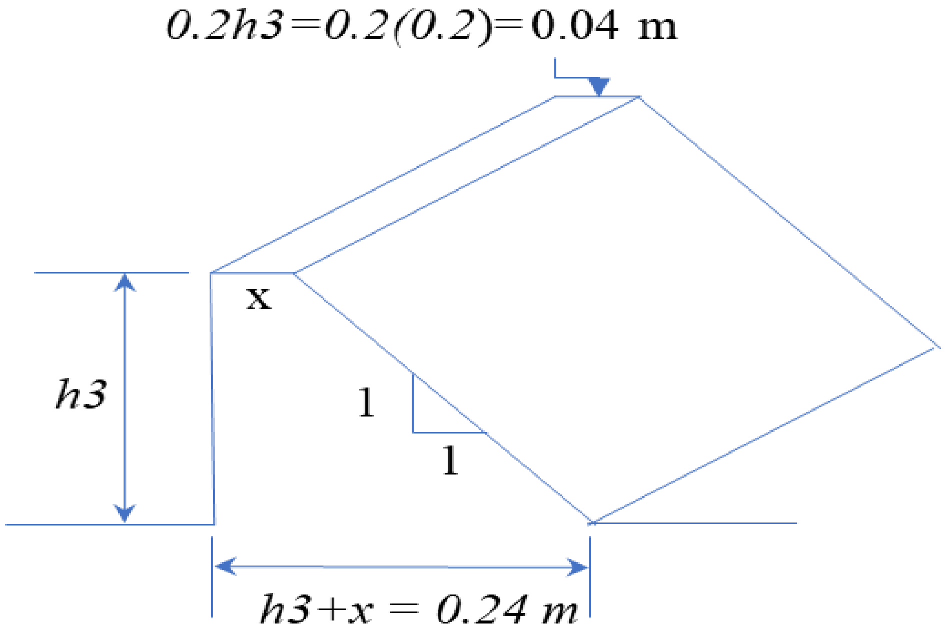
$s_1 = y_1 = 0.11\ m$

Baffle blocks:

From the second curve

$$\frac{h_3}{y_1} = 1.8$$

$$h_3 = 1.8 \times 0.11 = 0.2\ m$$

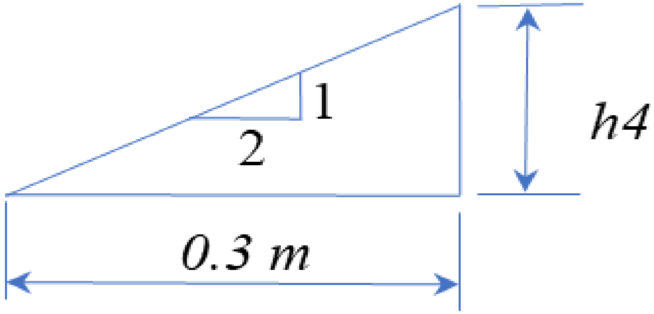


End sill:

From curve

$$\frac{h_4}{y_1} = 1.4$$

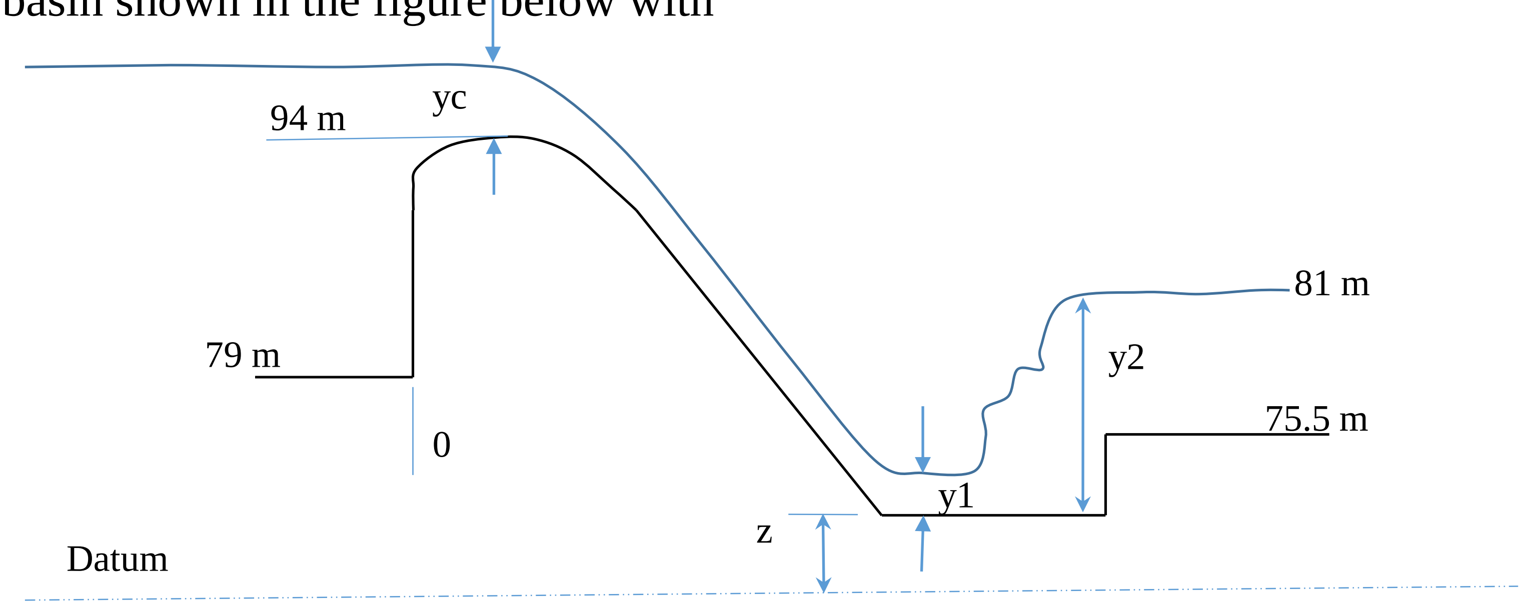
$$h_4 = 1.4 \times 0.11 = 0.154\ m\ \ use\ 0.15\ m$$



(ثم يرسم حوض التسكين و تؤشر الابعادعليه)

Example:

Determine the floor elevation for the stilling basin shown in the figure below with the following information's:



- Manning's n =0.03
- Average slope of the stream bed = 0.00375
- The design discharge for the spillway is = 566.8 m^3/sec.
- Spillway crest length is 30.5 m at elevation 94 m
- The approach channel floor elevation is to be at 79 m

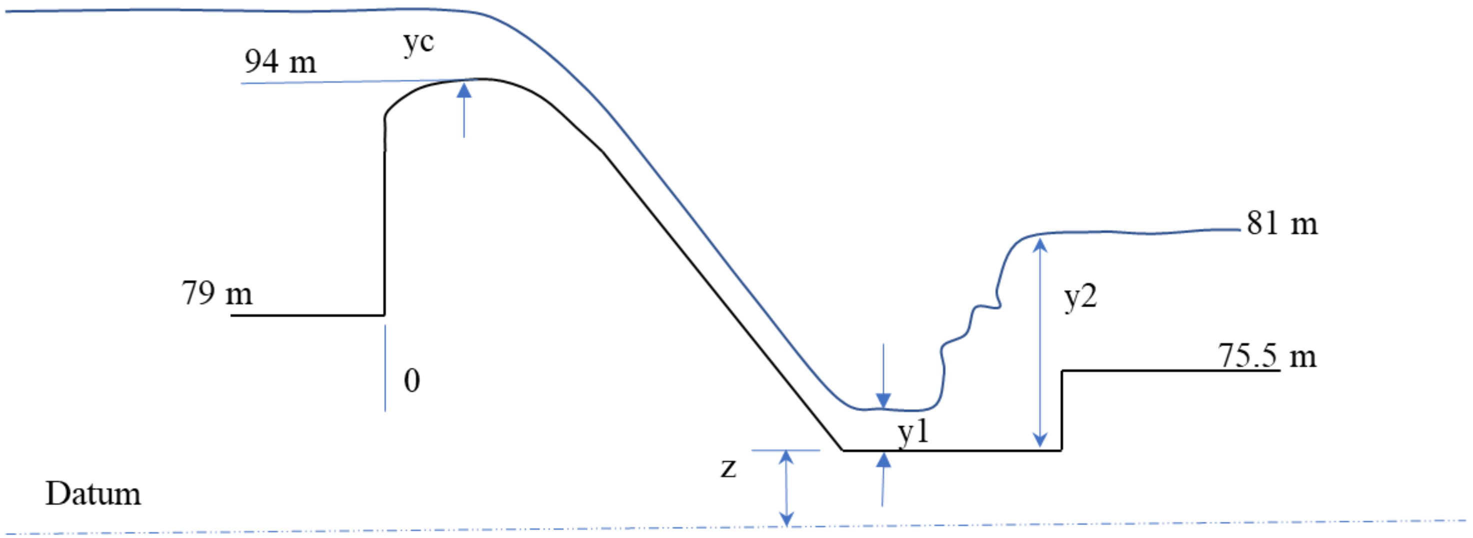
A hydraulic jump stilling basin having the same width as spillway crest is to be provided.

Required: Determine a suitable floor elevation for the hydraulic jump stilling basin.

Solution:

Neglecting the losses between section (0) and section (1)

$$94 + E_c = Z + E_1 \qquad 94 + \frac{3}{2} y_c = Z + y_1 + \frac{{V_1}^2}{2g}$$



$$94 + \frac{3}{2}\sqrt[3]{\frac{q^2}{g}} = Z + y_1 + \frac{\left(\frac{q}{y_1}\right)^2}{2g}$$

$$q = \frac{Q}{B} = \frac{566.8}{30.5} = 18.584 \ {m^3}/{s.m}$$

$$94 + \frac{3}{2}\sqrt[3]{\frac{18.584^2}{9.81}} = Z + y_1 + \frac{\left(\frac{18.584}{y_1}\right)^2}{19.62}$$

$$Z = 98.92 - y_1 - \frac{17.602}{{y_1}^2} \text{ ---------- } (1)$$

Using hydraulic jump equation $\frac{y_2}{y_1} = \frac{1}{2}\left[\sqrt{1 + \frac{8q^2}{g{y_1}^3}} - 1\right]$

$$y_2 = \frac{y_1}{2}\left[\sqrt{1 + \frac{8(18.574)^2}{9.81 \times {y_1}^3}} - 1\right]$$

$y_2 = \frac{y_1}{2}\left[\sqrt{1 + \frac{281.64}{{y_1}^3}} - 1\right]$-------------(2)

From geometry $y_2 + Z = 81 \rightarrow Z = 81 - y_2$ --------- (3)

Substituting equations (1) and (2) in equation (3), we get:

$98.92 - y_1 - \frac{17.602}{{y_1}^2} = 81 - \frac{y_1}{2}\left[\sqrt{1 + \frac{281.64}{{y_1}^3}} - 1\right]$-------------(4)

By trial and error $y_1 = 0.82\ m$

Z = 71.9 m stilling basin elevation

To select type of stilling basin

The Froude number at sec (1-1) is $F_{r1} = \frac{V_1}{\sqrt{gy_1}} = \frac{q}{y_1\sqrt{gD_1}} = \frac{18.584}{0.82\sqrt{9.81 \times 0.82}} = 8 > 4.5$

The incoming velocity is $V_1 = \frac{q}{y_1} = \frac{18.584}{0.82} = 22.66\ m/sec$ which is > 15 m/sec

So type II Basin is recommended

Design Principle for Sliding Steel Gates:-

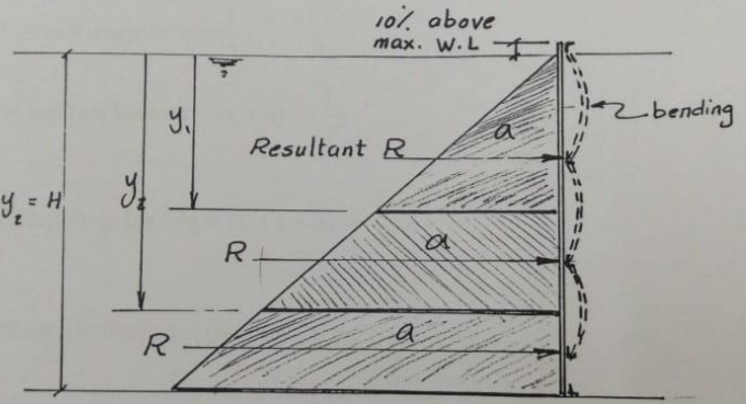
There are two methods for gate design.

1. By dividing the total load (pressure) exerted on the gate into equal parts.
2. By dividing the gate height into equal parts

The first method (which is more economical) usually the gate is closed with maximum head at its U/S side as shown.

(1) The gate will act as a slab simply supported on two walls.

(2) The total load was not uniformly distributed, but it varies from zero to $\gamma_w H$.



(3) The loading may be equally divided into (n) parts; each beam is located such that it carries a total W equal to the calculated area.

(4) The total load transmitted to a stiffener member is equal to the area of loading member .For the upper triangle the resultant will act at ($^1/_3 y_1$) from the base of the triangle and ($^2/_3 y_1$) from the top, so the stiffener beam will placed at that position, but for other portions of division the resultant will assumed to act at the middle and that will be the position of beam.

(5) For large span, you need to use stiffener beams to prevent bending moment. Beams may be of different shaps such as: T, I, L, ⊏,...

(6) For gate design, you have to select .the beam size and shape first then design the plate (or slab) between each two beams.

Design Procedure: -

(1) Total load (or area of loading) $A = \frac{1}{2}\gamma_w H^2$

$\gamma_w = 1 \text{ton}/\text{cu.m} \Rightarrow A = \frac{1}{2}H^2$

Sub area to be carried by each member **a** = $\frac{A}{n} = \frac{H^2}{2n}$

Area of top triangle $= a = \frac{1}{2}y_1^2$

Area of top two loading $= (a + a) = \frac{1}{2}y_2^2$

Area of top three loading $= (a + a + a) = \frac{1}{2}y_3^2$

Area of top (**n**) loading $= (a + a + \ldots + a_n) = \frac{1}{2}H^2$

So first you have to assume (**n**) then calculate $a = H^2/2n$ after that find:

$$a = \frac{1}{2}y_1^2$$

$$2a = \frac{1}{2}y_2^2$$

$$3a = \frac{1}{2}y_3^2$$

$$na = \frac{1}{2}y_n^2 = \frac{1}{2}H^2$$

(2) Find the height of each area h1 , h2 , h3 ,.......hn

$h_1 = y_1,\ h_2 = y_2 - y_1,\ h_3 = y_3 - y_2,\ h_n = y_n - y_{n-1}$

(3) Locate the stiffener beams at centers of each sub area except the top one which is located at $^{2}/_{3}y_1$ from top.

(4) Add one beam at top and another at the bottom (this is recommended for large regulators).

(5) Bending Moment Calculations: -

Loading on each stiffener beam is equal to $\omega = \gamma_w a$ in tons /m ;

W =wL

For simply supported beam from two ends with uniform load distribution.

$$B.M. = \frac{Wl}{8} \text{ or } \frac{wl^2}{8} \text{ in(ton} - \text{m)}.$$

(6) Knowing f_s (steel stress) use

$$z = \frac{B.M.}{f_s} \text{ in}(L^3) \quad , \quad Z = \frac{I}{C} \longrightarrow \frac{B.M.}{f_s} = \frac{I}{C}$$

Where: -

z = section modulus = I / C (for rectangular section)

f_s = steel stress (working stress) $\longrightarrow f_s = \frac{B.M. * C}{I}$

مهم $Z = \frac{B.M.}{f_s}$

(7) Find the proper size of (R .S.J Rolled Steel joist) L, T, or I beam to fit the required (z) as given in the steel section table.

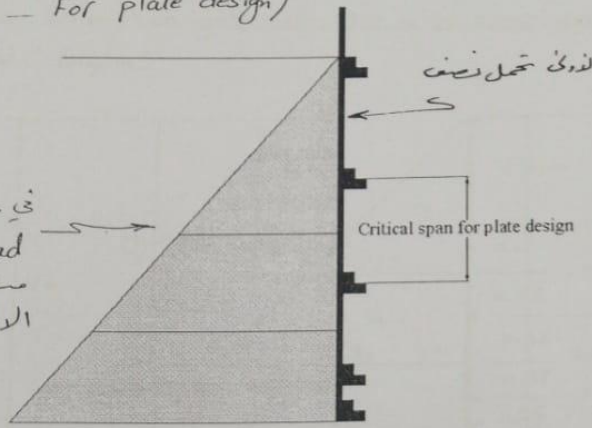
(8) Readjust the size of the beam such that the composed plate girder gives the same z .

(والنقطه الأخيره هذه تحتاج ان يكون الطالب قد درس موضوع ال
steel structure .)

(9) For plate design:

Between each two joists there is a steel plate; plate design should be made for the one that have the biggest bending moment.

($B.M. = \frac{WL}{10}$ — For plate design)



ال plate الأولى تحمل نصف الوزن .

في حالة تقسيم ال load الى أقسام متساوية الى الطريقة الأولى للتصميم .

So to get maximum B .M., The span (L) should be the longer one, but since the first one from top is the longer one and it hold half the load so the second one from top is the one that hold the total load W and have the longer length compared with the other.

For plate design use the second span from top to calculate plate thickness since this span will subject to max. B .M.

Practical Procedure for Design of Gates :-

There are three main elements :

1- R.S.J (Rolled Steel Joist) size and No.
روافد - الواح حديد - فولاذ / مدلفنة - مستوى

2- plate thickness.

3. Operating method عملية الفتح والغلق

* Proper choice of joists depends on cost of gate as calculated from several designs

* ملاحظة :

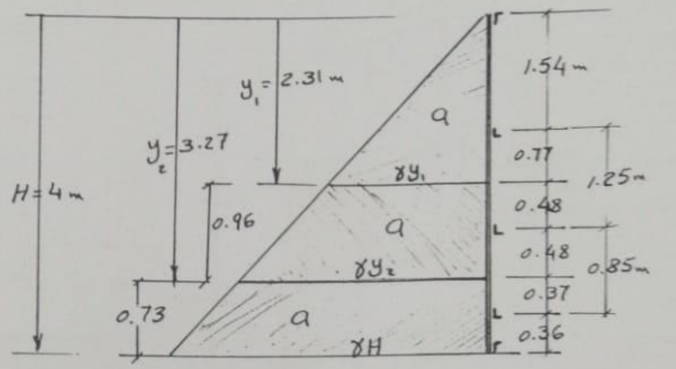
اذا كان عدد الروافد (beams) كبير عند ذلك حجم ال beam يكون صغير وال plate thicknes قليل . أما اذا كان العدد قليل فان حجم ال beam يكون كبير وسمك البوابة كبير ايضا .

Example :

Design a 4.0 m high gate with 3.0 m of clear span.

$f_s = 14,000$ ton/m^2

solution :



Assume 3 divisions to the pressure area.

$A = \frac{1}{2}\gamma_w (4)^2 = 8$ tons

$\frac{A}{n} = a = \frac{8}{3} = 2.67$ each

area of top triangle $a = \frac{A}{n} = 2.67$ m^2 $= \frac{1}{2} y_1^2$

$\therefore y_1 = \sqrt{2 \times 2.67} = 2.31$ m

area of triangle of height $y_2 = \frac{1}{2} y_2^2 = 2a = 2.67 \times 2$

$\therefore y_2^2 = 5.34 \longrightarrow y_2 = 3.27$ m

$\frac{1}{2} y_3^2 = 3 \times 2.67 \Longrightarrow y_3 = 4.00$ m

Each joist will carry a load equal to ($\omega = \gamma_w a = 2.67$ ton/m)

ال load لـ joist يعتبر unit load لأنه لمتر واحد طول

For simply supported joist : $M = \frac{wl^2}{8} = \frac{2.67(3)^2}{8}$

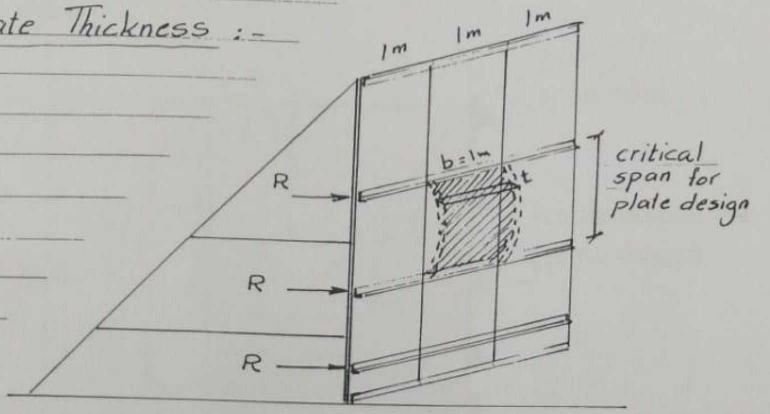
$\therefore M = 3$ ton.m

This B.M. is the same on any of the three joists.

$$Z = \frac{B.M.}{f_s} = \frac{.3 \text{ ton-m}}{14,000 \text{ ton/m}^2} = 2.14 \times 10^{-4} \text{ m}^3$$
$$= 2.14 \times 10^{2} \text{ cm}^3$$

then choose an angle or an I beam from steel tables.

Plate Thickness :-



Total weight/m can be considered as $W = \gamma_w a = 2.67$ ton

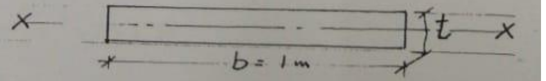
$$B.M. = \frac{Wl}{10} = \text{moment for the continuous plate}$$

$$B.M. = \frac{2.67\,(1.25)}{10} = 0.33 \text{ ton-m}$$

$$Z = \frac{B.M.}{f_s} = \frac{0.33}{14,000} = 23.5 \times 10^{-6} \text{ m}^3$$

$b = 1.0$ m

t or d = ?



$$Z = \frac{I}{c} = \frac{\frac{bd^3}{12}}{\frac{d}{2}} = \frac{bd^2}{6}$$ (section modulus for rectangular section.)

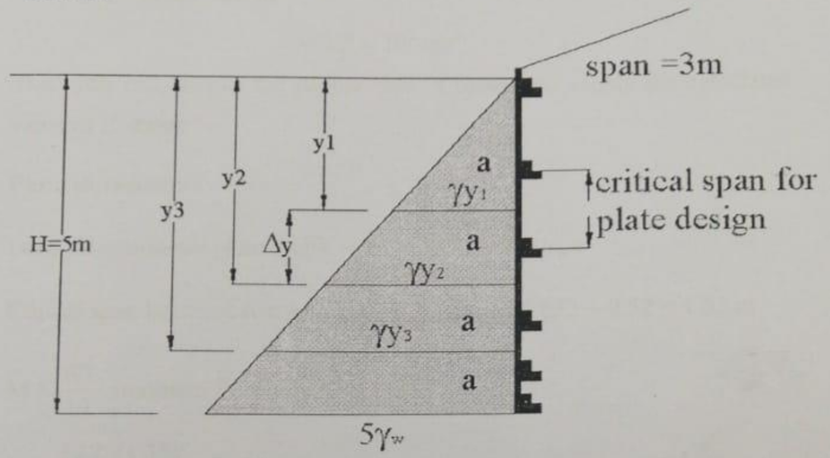
$$23.5 \times 10^{-6} = \frac{(1)\,t^2}{6}$$

$$\therefore t = \sqrt{\frac{23.5 \times 10^{-6} \times 6}{1 \text{ m}}} = 0.12 \text{ m}$$ → use standard 0.5" plate

Example: -

Design a 5 m high gate with 3 m of clear span. Assuming that the loading triangle is divided into four parts. Use f_s = 14,000 ton /m^2

Solution



n = 4 (given)

Area of triangle $A = \frac{1}{2}\gamma_w (5)^2 = 12.5 \text{ton}$

$a = \frac{A}{n} = \frac{12.5}{4} = 3.125$ ton carried by each part.

- area of top triangle $a = A/n = 3.125 = ½\, y_1^2$

$\therefore y_1 = \sqrt{2(3.125)} = 2.5\text{m}$

-area of triangle of height $y_2 = 2a = \frac{1}{2} y_2^2 = 2(3.125)$

$y_2 = \sqrt{2(2 * 3.125)} = 3.54\text{m}$

$½\, y_3^2 = 3a = 3(3.125)$

$y_3 = \sqrt{6(3.125)} = 4.33\text{m}$

$\Delta y = 3.54 - 2.5 = 1.04$

Each joist will carry a load equal to $w = \gamma_w a = 3.125 \text{ton/m}$ i.e., (unit load because it is for 1 m length)

For simply supported joist $M = \frac{wl^2}{8} = \frac{3.125(3)^2}{8} = 3.52 \text{ton} - \text{m}$

This **B.M**. is the same on any of the three joists.

Using $f_s = 14{,}000 \text{ ton}/\text{m}^2$

$$Z = \frac{B.M}{f_s} = \frac{3.52 \text{ton} - \text{m}}{14{,}000 \text{ton}/\text{m}^2} = 2.5 \times 10^{-4} \text{m}^3$$

$$= 2.5 \times 10^2 \text{ cm}^3$$

Then you can choose the proper size of beam that satisfy the calculated value of Z above.

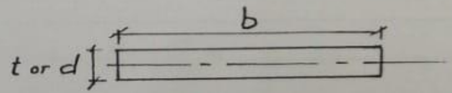
Plate thickness: -

Load on one-meter plate width $= W = 3.125 \text{ton} = \gamma_w a$

Critical span between two joists $= \frac{1}{3} y_1 + \frac{1}{2} \Delta y = 0.833 + 0.52 = 1.35 \text{ m}$

$M = \frac{WL}{10}$ (moment for continuous plate)

$$M = \frac{3.125(1.35)}{10} = 0.422 \text{ton} - \text{m}$$



$$Z = \frac{I}{C} = \frac{(\frac{bd^3}{12})}{(\frac{d}{2})} = \frac{bd^2}{6}$$

Where:-

Z is the section modulus for rectangles section

d is the plate thickness

b is taken as(1 m)

$$Z = \frac{bt^2}{6} = \frac{B.M}{f_s}$$

$$Z = \frac{t^2}{6} = \frac{0.422}{14{,}000} \qquad \text{(for b = lm)}$$

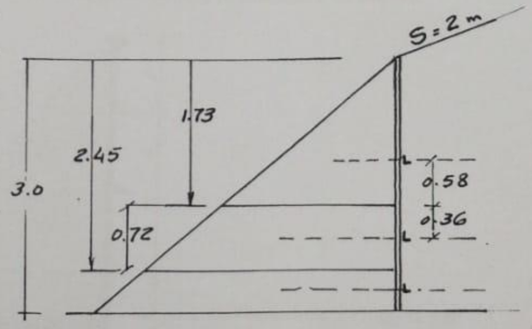
$$\therefore t = \sqrt{\frac{6(0.422)}{14{,}000}} = 0.0134 \text{m} \cong 13.45 \text{mm}$$

use **14 mm** plate

Example :

Calculate the required plate thickness of a gate of 2.0 m span and 3.0 m height with the loading triangle divided into three parts . Use $f_s = 14,000$ ton / m^2

$\gamma_w = 1$ ton / m^3



Area of triangle =

$\frac{3\times3}{2} = 4.5$

$\frac{4.5}{3} = 1.5$ ton

carried by each part.

$\frac{1}{2} y_1^2 = 1.5 \longrightarrow y_1 = \sqrt{2\times1.5} = 1.73$ m

$\frac{1}{2} y_2^2 = 3 \longrightarrow y_2 = \sqrt{2\times3} = 2.45$ m

$\Delta y = 2.45 - 1.73 = 0.72$ m

load on one meter plate width $W = 1.5$ ton

Critical span between two joists $= 0.58 + 0.36$

$= 0.94$ m

$B.M_{plate} = \frac{Wl}{10} = \frac{1.5(0.94)}{10} = 0.141$ ton-m

$Z = \frac{bt^2}{6} = \frac{B.M.}{f_s}$ (b= 1m)

$Z = \frac{t^2}{6} = \frac{0.141}{14,000}$

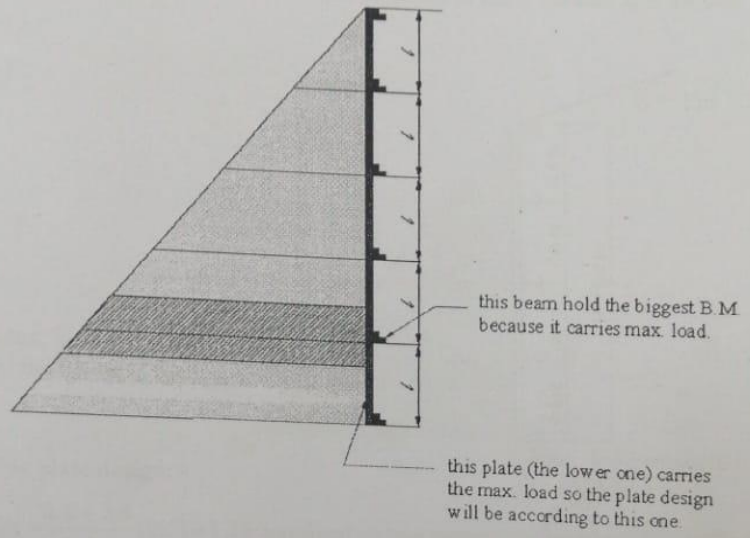
$\longrightarrow t = \sqrt{\frac{6\times0.141}{14,000}} = 0.0078$ m

~ 8 mm plate

Second Method of Gate Design: -

In this method the height is divided into equal parts and space the stiffener beams a accordingly then the size of each one will depend on the unit load **(w)** the size of the biggest beam is used through out.

There is a waste in R. S. J. size, plate design is the same as before



- الطريقه الثانيه للتصميم تعتمد على تقسيم الارتفاع الكلي للبوابه الى
- اقسام متساويه عندها يكون التصميم على ال plate التي تحمل
- اكبر load ومعرضه لاكبر .B.M.

لذا وحسب ما موضح بالرسم اعلاه فان تصميم ال plate يكون على الأخيره (بالاسفل).

- اما بالنسبه لل beam فان التصميم يكون على ال beam الذي يحمل
- اكبر .B.M

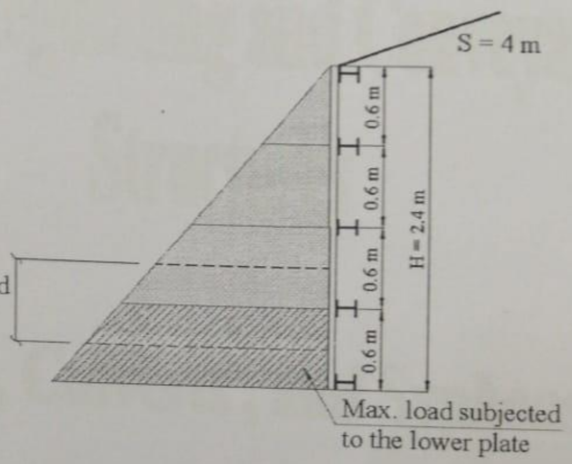
$max.\ B.M. = \frac{W\,l}{cons.}$ وبما ان $l = const.$ إذن نعتمد على

- اكبر W وهي الأخيره اي انه ال beam ما قبل الأخير يحمل load كامل واكبر ما يمكنه (كما موضح بالرسم).

Example

Design a 2.4 m height sliding steel gate with 4 m of clear span. The height is equally divided with 5. I beams uniformly spaced at 0.6 m center to center ...Calculate the required plate thickness in (mm) and the required section modules Z of the I beam subjected to max. loading $f_s = 14{,}000$ ton /m^2

Sol



For plate design: -

$$W = \frac{1.8+2.4}{2}(0.6) = 1.26 \text{ ton (load on one meter plate width)}$$

$$M_{plate} = \frac{wl}{10} = \frac{1.26(0.6)}{10} = 0.076 \quad \text{Ton -m}$$

$$Z = \frac{bt^2}{6} = \frac{M}{f_s}$$

$$\frac{t^2}{6} = \frac{0.076}{14{,}000} \Rightarrow t = 6\times10^{-3} = 0.006\text{m} = 6\text{mm}$$

For beam design: -

$$\text{(W) on I = beam subjected to max load} = \frac{1.5+2.1}{2}(0.6) = 1.08\text{ton}$$

$$B.M = \frac{wl^2}{8} = \frac{1.08(4)^2}{8} = 2.16$$

$$Z_{beam} = \frac{M}{f_s} = 1.543\times10^{-4}\text{m}^3$$

Closed Regulation and Conveyance Structures:-

They are structures constructed in line of canals, convey waters from one location to another across various existing obstructions along the canal route, they are of different types as follows: culverts, inverted siphons, aqueducts, … etc.

Pipes:

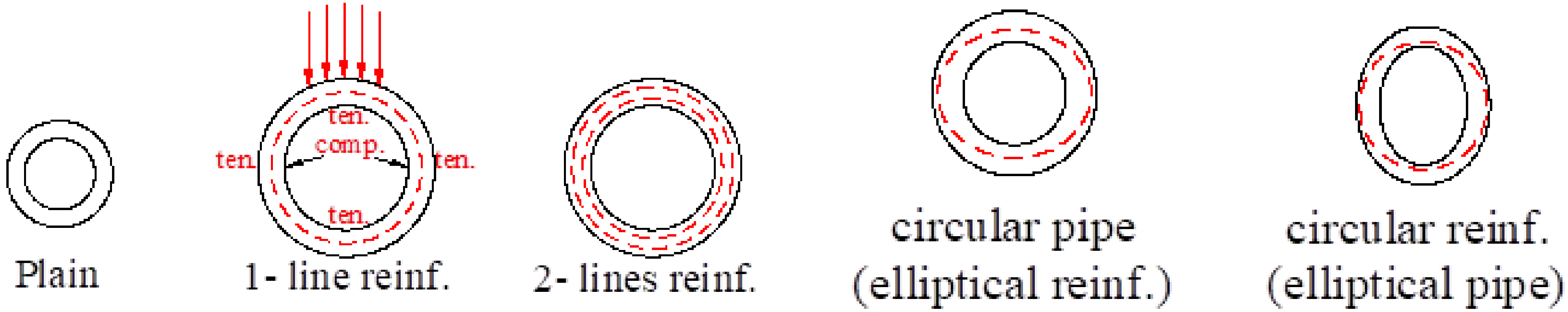
Used as closed conduits for trans porting (conveying) a large amount of waters for irrigation purposes, usually such pipes are circular and made of metal, asbestos cement, plastic, plain or reinforced concrete (plain or corrugated or tiles usually for drainage).

Plain concrete pipes under load may by use up to diameters of 30cm.

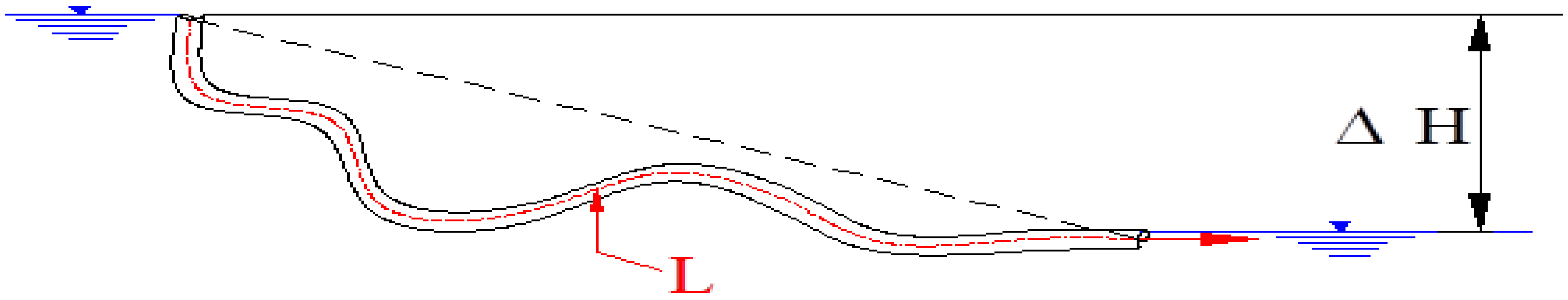
Types of concrete pipes

- Precast plain concrete pipe.
- Precast reinforced concrete pipe.
- Precast prestressed concrete pipe.
- Asbestos cement pipe

Shape and Reinforcement:



•In closed conduits (ΔH) is actual vertical difference between supply and discharge ends.
•(L) is the actual length of conduit along its cents line.



Formulas used for pipes could by the same as those of open channels, i.e. Manning’s formula for circular pipe of diameter D

$$A = \frac{\Pi D^2}{4} \;;\quad P = \Pi D$$

$$\therefore \quad R = \frac{A}{P} = \frac{D}{4}$$

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

$$V = \frac{0.40}{n} \; D^{2/3} \; ((\frac{\Delta H}{L})^{1/2} \ldots\ldots\ldots\ldots\ldots\text{metric}$$

$$\left[Q = AV = \frac{\pi}{10n} D^{8/3} (\frac{\Delta H}{L})^{1/2} \right]$$

(n) Value is same as for open channels of the same material.

- Losses in pipes (**Minor losses –Major losses**)

In a pipe system whether it is along pipe line on just a culvert the following are the expected losses expressed as a proportion of the total velocity head $\frac{V^2}{2g}$ and this ratio is called (k).

ΔH = Sum of all losses due to velocity, entrance, ………etc.

$= h_1 + h_2 + h_3 + \ldots\ldots\ldots\ldots\ldots + h_8$

where: -

$h_1 = K_1 \left(\frac{V^2}{2g}\right)$ Velocity head inside the pipe (it is the losses due to conversion of static head to velocity form).

$h_2 = K_2 \left(\frac{V^2}{2g}\right)$ Entrance losses which is given by U.S.B.R. as :-

0.5 for square edged inlet flush with vertical walls

0.1 for roundest inlets, radius r where $\frac{r}{D} \leq 0.15$

0.15 for grooved or socket ended pipes

0.16 for bell mouth and rounded entrance

0.20 for projecting conc. Pipes

0.85 for projecting steel pipes

$h_3 = K_3 \left(\frac{V^2}{2g}\right)$ Friction losses (Manning)

To find K_3: -

$$V = \frac{1}{n}(R)^{2/3}\left(\frac{\Delta H}{L}\right)^{1/2} \qquad \frac{V^2}{2g} = \frac{1}{n^2}(R)^{4/3}\left(\frac{h_3}{L}\right)\frac{1}{2g} \qquad h_3 = \left(\frac{n^2 L 2g}{(R)^{4/3}}\right)\frac{V^2}{2g} \qquad \text{So} \quad K_3 = \left(\frac{2gn^2 L}{R^{4/3}}\right)$$

$h_4 = K_4 \left(\frac{V^2}{2g}\right)$ Bends losses

$h_5 = K_5 \left(\frac{V^2}{2g}\right)$ Gradual expansion or contraction losses .

$h_6 = K_6 \left(\frac{V^2}{2g}\right)$ Sudden expansion or contraction losses.

$h_7 = K_7 \left(\frac{V^2}{2g}\right)$ Fitting losses

$h_8 = K_8 \left(\frac{V^2}{2g}\right)$ Exit losses K_8 = exit loss coeff. Which is taken as 1.0 for most outlets.

$$\Delta H_{total} = K_1 \left(\frac{V^2}{2g}\right) + K_2 \left(\frac{V^2}{2g}\right) + \ldots\ldots\ldots\ldots\ldots + K_8 \left(\frac{V^2}{2g}\right)$$

$$\Delta H_{total} = \Sigma K \left(\frac{V^2}{2g}\right) = \Sigma K \left(\frac{Q^2}{A^2}\right) \cdot \frac{1}{2g}$$

where (K) is actual amount of losses.

Culverts (Single and multiple barrels):

Culverts are used to pass the flow under an obstruction.(roads, railroads, … etc.). The flow through culverts is complex and depends on all the design variables such as:

Size, shape, length, slope, roughness, depths of head and tail waters.

Box sections are used for large culverts to pass the flow under an obstruction.

For most canal culverts where head loss is to be kept to a minimum, the culvert is assumed to flow full and the discharge is a function of the difference between head water and tailgates, if this difference is ΔH and the culvert is of depth D than the discharge is given by:-

$$\left[2g\Delta H = \left(K_1 + K_2 + \frac{2gn^2L}{R^{4/3}} \right) \frac{Q^2}{A^2} \right]$$ which is the general culvert equation

For the equation above: -

K_1 = inlet loss coeff.

K_2 = exit loss coeff.

R = hydraulic Radius in (m).

n = Manning roughness cuff.

L = length of culvert (m).

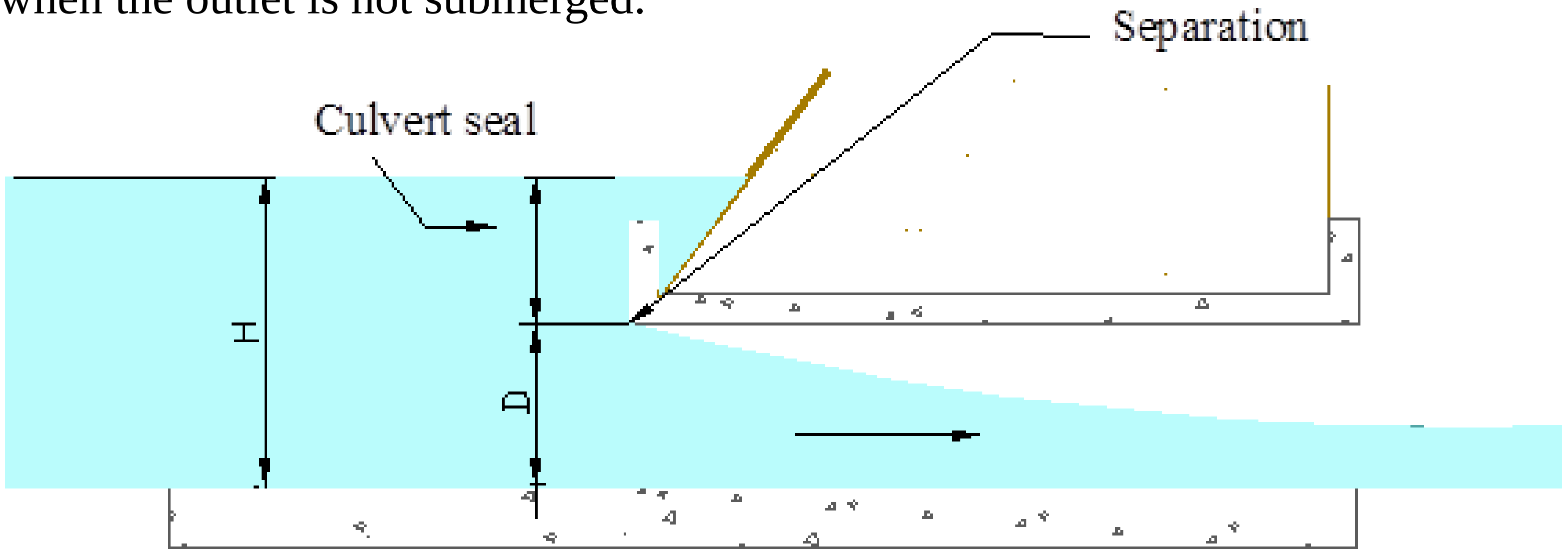
A = area of culvert (m^2)

Flow through culverts

- The entrance will not be submerged if the headwater is less than a critical value H^* , while the outlet is not submerged

$1.2D \leq H^* \leq 1.5D$ where D = culvert height

A culvert with a square edge at the top of the entrance will not flow full even if the entrance is below head water level when the outlet is not submerged.



- For practical purposes, culvert flow may be classified into 6 types of flow within 2 groups.

GROUP (A): -

Free surface flow (inlet and outlet) throughout (neither end submerged).

Case 1: -

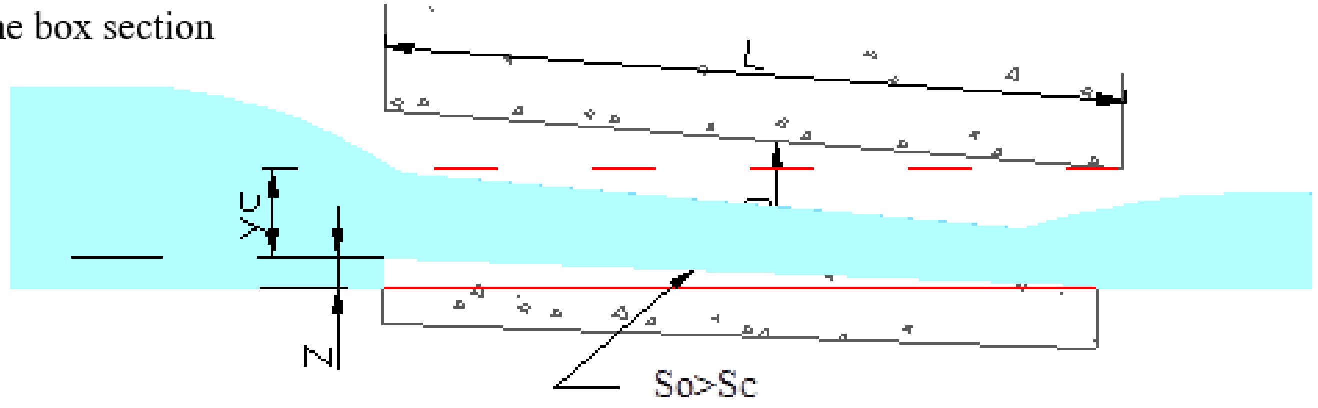
Critical depth at inlet (inlet control)

$H < 1.2D$ $y_t < y_c$

Culverts on supercritical slopes, inlet not submerged, free outlet, control at inlet, flow is supercritical

In this case discharge is independent of slope, roughness, length, outlet type, shape and size of the barrel. It depends entirely on the inlet geometry and the headwater elevation

$$Q = Bg^{1/2}\left(\frac{H}{1.5}\right)^{1.5}$$ where **B** is the width of the box section



Case 2: -

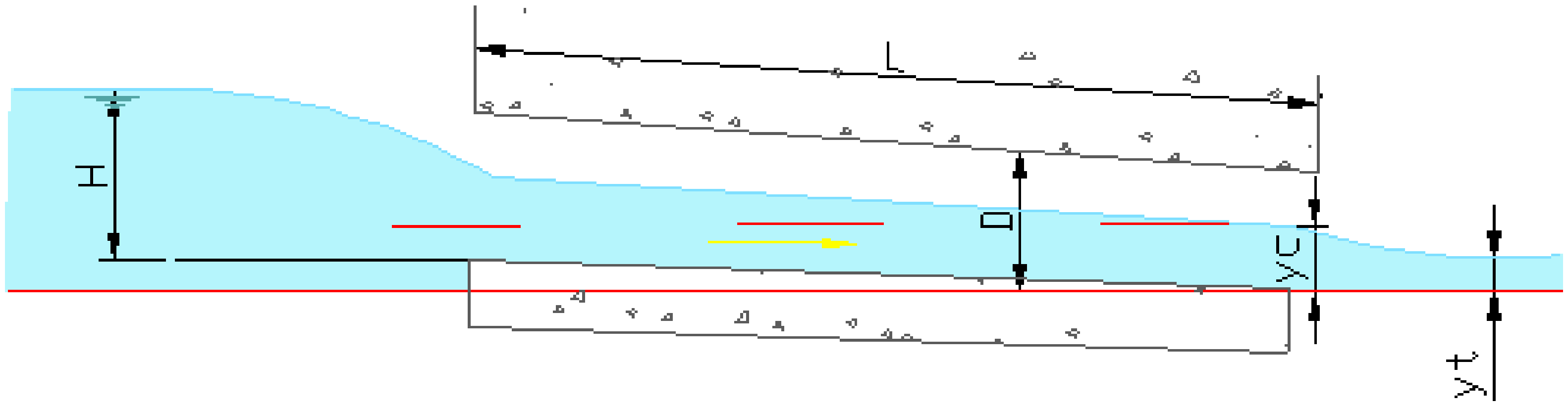
Critical depth at outlet (outlet control)

$H < 1.2D$ $y_t < y_c$

Case 2: -

Critical depth at outlet (outlet control)

$H < 1.2D$ $\qquad$ $y_t < y_c$



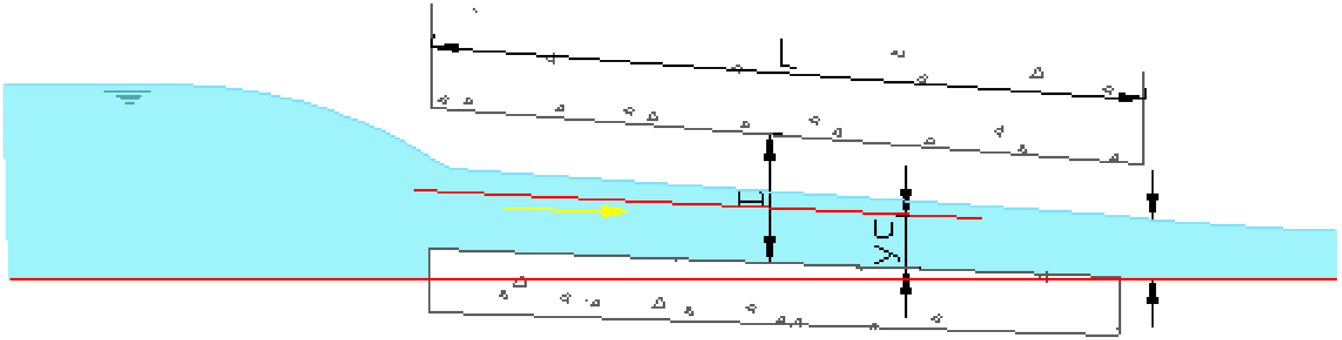
Culverts on subcritical or horizontal slope, hence the control section is at the outlet.

Discharge depends on inlet geometry, headwater elev., shape, size of barrel, roughness, slope and length.

Case 3: -

Sub critical flow case

$H < 1.2D$ $y_t > y_c$



Culverts on subcritical slopes, it does not flow full

GROUP B: -

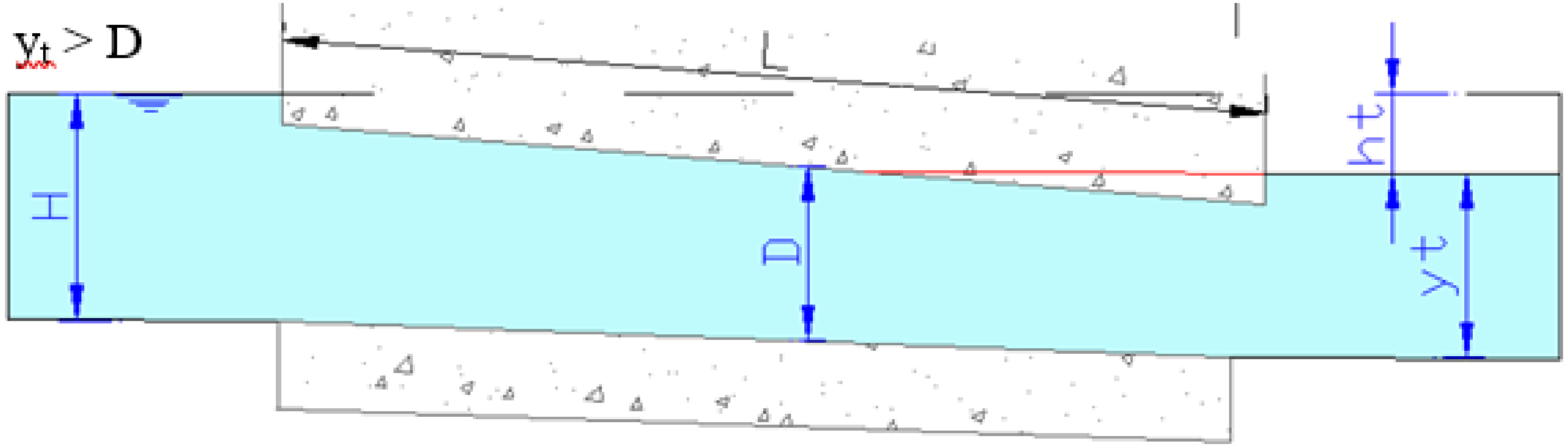
U/S. end of culvert always submerged.

Case 4: -

Inlet and outlet are submerged. It is the most economical case, which is usually used in design. The conduit is flowing full

$H > D$

$y_t > D$



Submerged outlet $H > 1.2D$

full flow $Y_t > D$

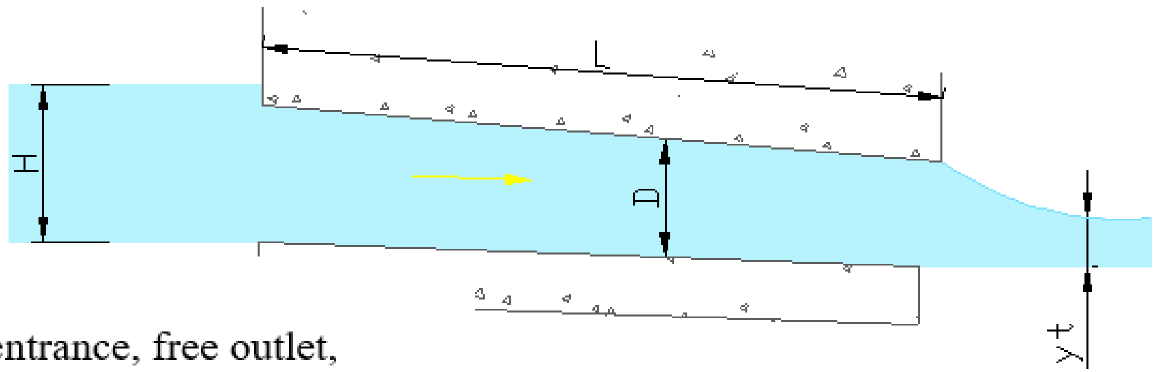
Case 5: -

Submerged inlet, full flow, free outlet, culverts on mild (sub critical) or horizontal slopes

$H > 1.2D$

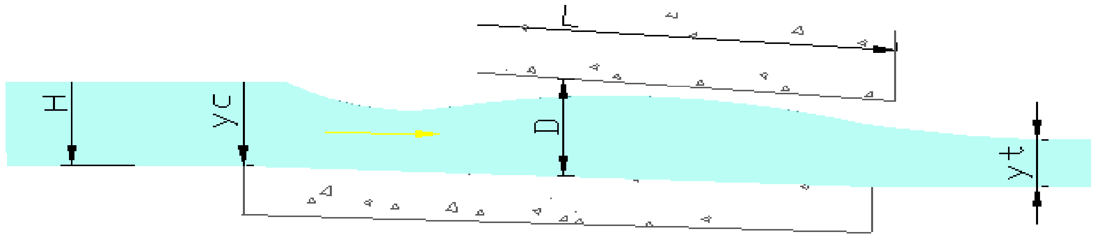
$y_t < D$

In this case the culvert is ***hydraulically long.***



Case 6: -

Partly full flow, submerged inlet, Rapid flow case at entrance, free outlet, hydraulically short, control at inlet. Orifice flow



The flow is analogous to a sluice and the equation of discharge becomes

$$Q = C_d.B.D.(2gH)^{1/2}$$

$$C_d = 0.42 + 0.05\frac{H}{D} \qquad \text{For} \quad 1.2 < \frac{H}{D} < 4 \ \text{in meters system}$$

- For design purposes; the designer usually ensures that both inlet and outlet are submerged. Therefore Case (4) will be selected.
- For Iraqi conditions use $\frac{H}{D} \geq 1.2$ To ensure inlet submergence. The greatest economy results from designs based on the culvert flowing full. This requires careful attentions to the inlet design, slope and roughness.
- In Iraq the topography is relatively flat and head losses have to be minimized. So flow condition of case (4) is applicable.

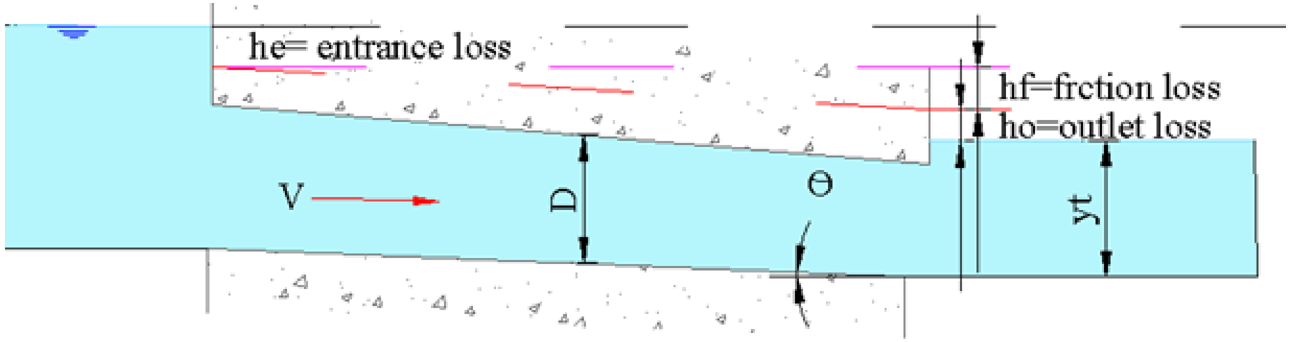
For this type the discharge-head relation ship is given by:

For circular pipes:

$H + L \sin\theta = y_t + H_t$

$H + L \sin\theta = y_t + (h_o + h_f + h_e)$ thus

$2g\,(H - y_t + L \sin\theta) = (K_1 + K_2 + \lambda L)\dfrac{Q^2}{A^2}$



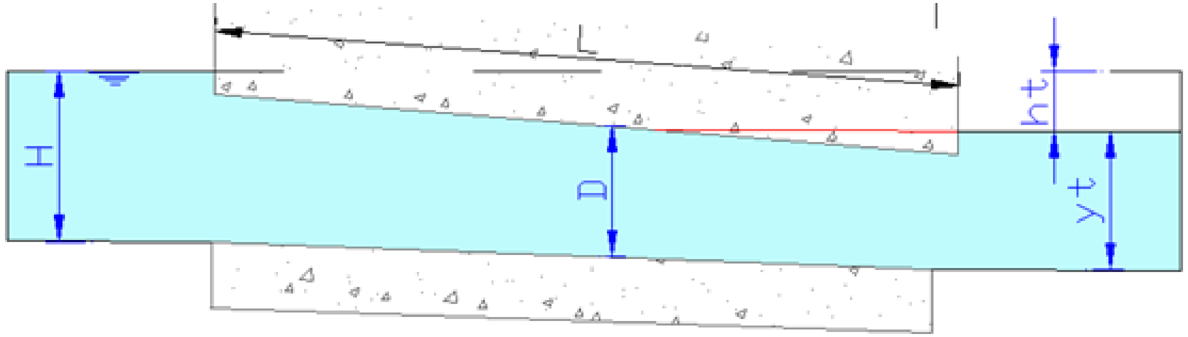
Values of (K_1) for inlets are given by U.S.B.R. tables.

(K_2) are taken as 1.0 for most out let

(λ) Friction factor in pipes

For box section: -

$$2g.\ \Delta H = (K_1 + K_2 + 2g\, n^2\, L / R^{1.33})\ \frac{Q^2}{A^2} \quad \text{----------metric}$$



$K_1 = 0.16$ for round entrance

$= 0.5$ for square edge entrance

$K_2 = 1.0$

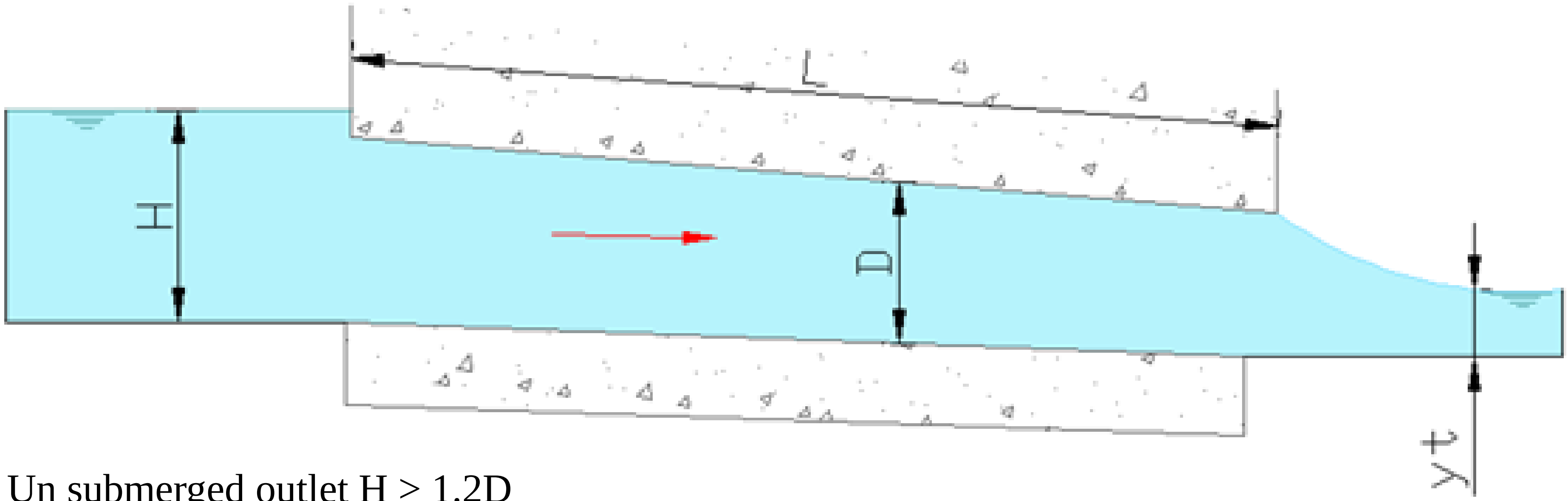
R = hydraulic Readies

Submerged outlet H > 1.2D

full flow $Y_t > D$

The equation for flow through a culvert as given above assumes that head losses are to be kept to a minimum and culvert will flow full without outlet submerged. In this case the discharge of the culvert is independent of the culvert slope. (This is the usual case in flat topography as in Iraq). To ensure inlet submergence, the head H at the inlet should be greater than 1.2D where D is the depth.

- The greatest economy results from design based on the culvert flowing full.
- Discharge for a free outlet and full flow is greater than for the submerged case, but of coarse, the loss is also greater.



Un submerged outlet $H > 1.2D$

full flow $Y_t < D$

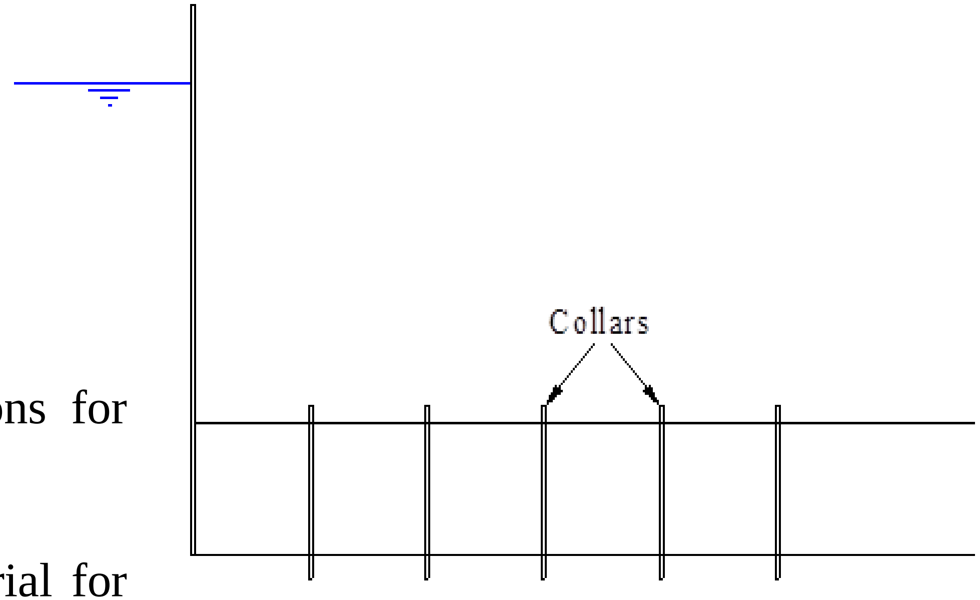
- For discharges up to about 2.5 m3/ sec. Pipes can be used, but for larger discharges a box section is preferred

Gated Culverts:

Are small regulators and are different from usual culverts by the addition of gate.

For gated culverts length of percolation line should be checked according to weighed cuff. of creep (Lane's cuff.) with a min. value of 2.5

Length of vertical parts of the callers should be not less than 1/4 the length of line including, circular, square, rectangular oval (parabolic or elliptical shape) or any other practical shape.

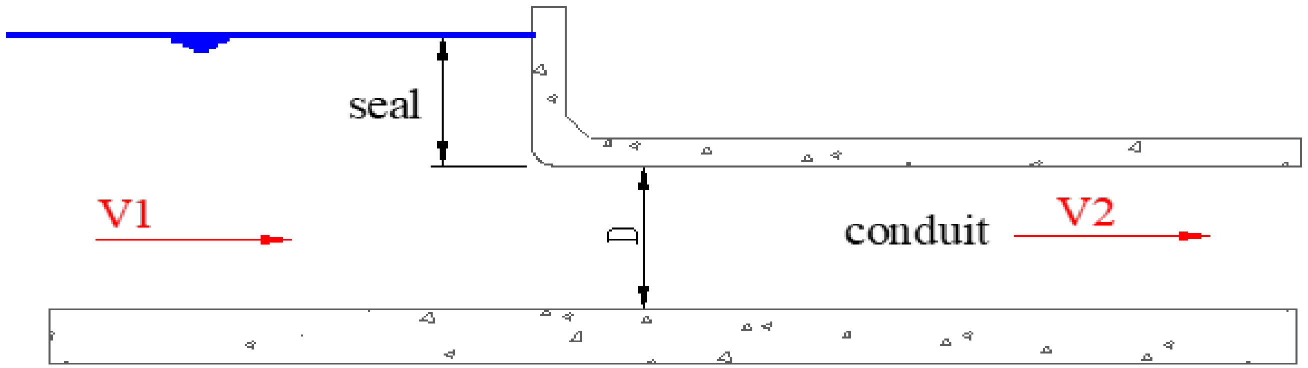


1. Great care should be taken in designing proper transitions for gated culverts.
2. Great care should be given to the choice of smooth material for long pipelines to reduce the friction losses.
3. Efficient care should be taken in hydraulic design of road culverts.

Conduit Inlet Seal:

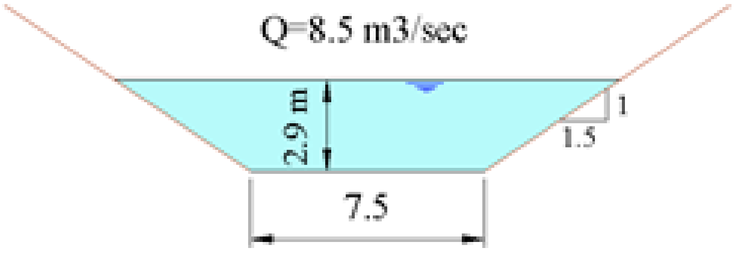
To ensure full flow condition with proper performance of the conduit , a minimum in let seal should by provided with suitable submergence at out let for straight conduits

$$Seal = 1.5\,\Delta hv = 1.5\left(\frac{V_2^{2}}{2g} - \frac{V_1^{2}}{2g}\right)$$



Example 1:-

Design a canal box culvert required to pass discharge of 8.5 cumecs under a road regarding the following information:



<u>Canal</u>

Canal bed width	7.5 m
Water depth	2.9 m
Side Slope	1.5H : 1V
S	10 cm / km
Ground level	38 m
Manning n	0.015
U/S.W.L	37.73 m
D/S.W.L	37.47 m

Road

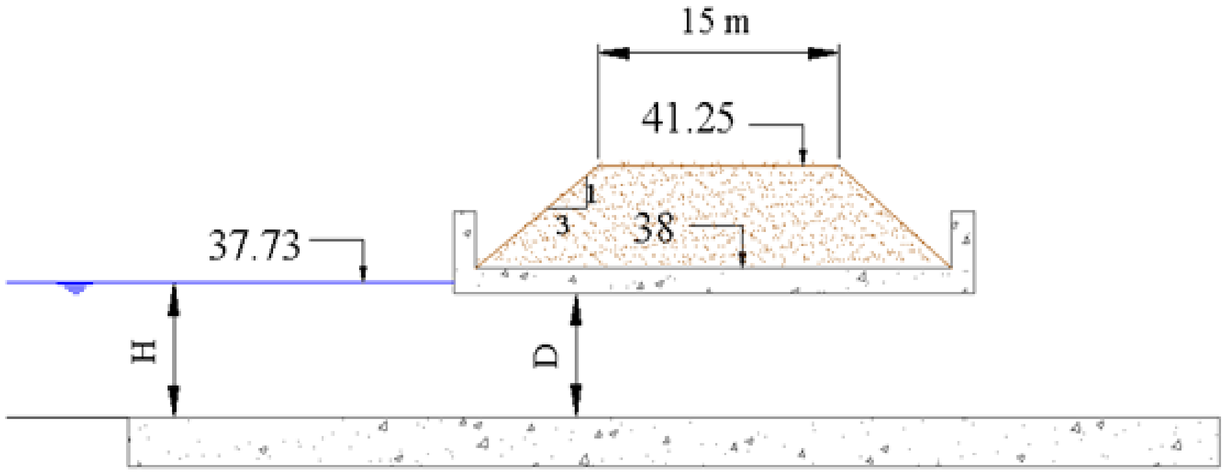
Top level of road 41.25 m

Top Width of road 15 m

Side Slope 3H: 1V

Using $K_1 = 0.5,\ K_2 = 1.0$

Assuming Full Flow Conditions (Sub merged entrance)



Sol.

L= 15+ $\left[(41.25-38)\times3\times2\right]$ =34.5m

To ensure submerged entrance $\dfrac{H}{D}\geq1.2$

$\dfrac{2.9}{1.2}=D\Rightarrow$ D = 2.42m (max .allowable value)Use D = 2.40m

Assuming Square section with one cell

$\Delta H_{total}=U/S.W.L-D/S.W.L$ = 37.73 - 37.47 = 0.26 m

Using the general culver equation

$$\Delta H=\left[K_1+K_2+\frac{2gn^2L}{R^{4/3}}\right]\frac{Q^2}{A^2}*\frac{1}{2g}$$

$$H_L=\left[0.5+1.0+\frac{2\times 9.81(0.015)^2(34.5)}{R^{4/3}}\right]\frac{(8.5)^2}{A^2}*\frac{1}{2\times 9.81}$$

$$A=2.4\times 2.4=5.66\text{m}^2$$

$$R=\frac{A}{P}=\frac{D^2}{4D}=\frac{D}{4}=\frac{2.4}{4}=0.6\text{m}$$

$$\therefore R^{4/3}=0.506$$

$$\therefore H_L=0.20$$

H_L calculated $\neq \Delta H_{total}$ available $\quad i.e., 0.2\neq 0.26$

So you have to reduce D

Use D = 2.25 , A= 2.25 2.25 = 5.0625 m2 , R = $\frac{D}{4}$ = 0.5625 m

Which gives H_L = 0.26 = 0.256 in O.K. if H_L is smaller; increase D or use tow openings.

Note

If the computed losses H_L are greater than the difference in U/S and D/S canal water surface , the pipe will probably cause If back water exist , the pipe size should be in creased .

$$\textbf{Because} \quad \mathbf{H_L = f} \frac{L}{D}\frac{V^2}{2g} \quad ; \ \text{i.e } H_L \ \alpha \frac{1}{D}$$

Example 2

A lined canal with Q = 5 m^3 /sec, depth of flow = 1.4 m, slope 17.7 cm/km, side slopes 1.5 :1,

Manning's n = 0.015 and bed width = 2.4 m, U/S canal W.L = 11.40 m, U/S canal bed level = 10 m

- Design a canal culvert 45 m long to pass a normal full supply discharge of 5 m^3/sec with 0.1 m head loss.
- Assume a square box section $D \times D$ is to be used with a rounded entrance so that $K_1 = 0.16$

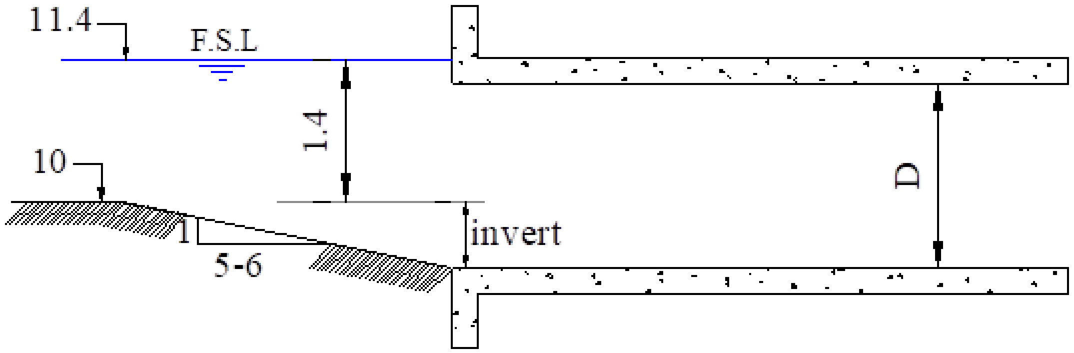
Sol.

Using the general culvert equation with R = D /4 for a single box section

$$2g.\Delta H_{total} = \left[K_1 + K_2 + \frac{2gn^2L}{R^{4/3}}\right]\frac{Q^2}{A^2}$$

$$2g\ (0.1) = \left[1.16 + \frac{2\times 9.81(0.015)^2(45)(4)^{1.333}}{D^{1.333}}\right]\frac{25}{D^4}$$

$$\Rightarrow D = 2.13\text{m} \cong 2.15\text{m}$$



To ensure inlet submergence the box should

be dropped so that $\frac{H}{D} \geq 1.2$

i.e $\frac{H}{2.15} \geq 1.2$

So H = 2.58 m and depth of invert below U/S bed level = 2.58 – 1.4 =

1.18 m say 1.2m , which is too much.

The alternative is to use 2 square box section then solving the a above equation with Q = 2.5 cumecs which gives D = 1.55 m and culvert should be dropped by 0.46 say 0.5m

- If canal discharge is 6 cumecs, U/S normal W.L. is 1.53 m, then the head loss is:

$$2g\Delta H = \left[1.16 + \frac{2 \times 9.81(0.015)^2(45)(4)^{1.333}}{(1.55)^{1.333}}\right]\frac{3^2}{(1.55)^4}$$

H = 0.148m which cause a slight backwater at the culvert entrance of about 50 mm which is not a serious problem.

Box Culverts Flowing Partly Full or Culverts dose not flow full: -

In such a case it is the same as the (flow in an open channel)

$$H_L = (K_1 + K_2)* \frac{V^2_{Culver} - V^2_{U/Scanal}}{2g}$$

Where: K_1 = inlet loss coeff.

K_2 = outlet loss coeff

From this equation we can find out the velocity V in the culvert.

Example: -

Design a rectangular box culvert flowing partially full as a bridge crossing in a lined canal carrying discharge of 5 cumecs. The length of rectangular bridge crossing is 10m …

The overall head loss is to be taken for the design of the structure is 0.1m. Assume that the outlet transition will be similar in shape to the inlet transition. The canal parameters are given below: -

Bed width 2.4 m

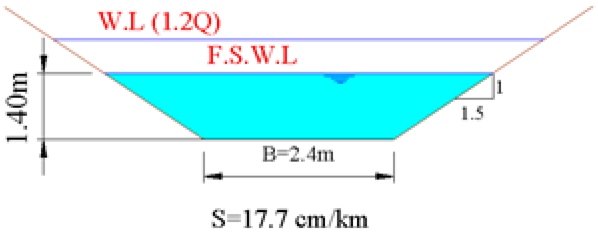
Depth of flow 1.4 m

Longitudinal slope 17.7 cm /Km

Side slopes 1.5 (H): 1 (V)

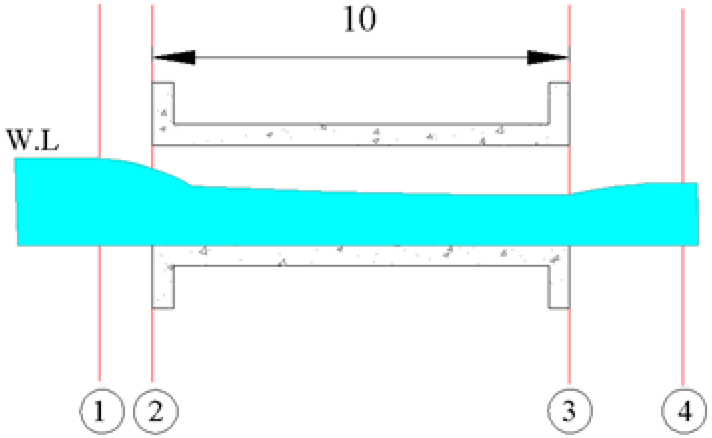
Manning's (n) 0.015

U/S canal B.L. 10 m



Canal cross section

U/S and D/S



Straight line head wall transition $K_1 = 0.3$ and $K2 = 0.75$

Solution:

The structure will be designed for 1.2 Q

Discharge Q = 5 cumecs

$1.2\ Q = 1.2\ (5) = 6\ m^3/sec$

For this discharge calculate depth of flow and velocity in the canal

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

$$A = (B + Z\,y)\,y = (2.4 + 1.5\,y)\,y$$

Wetted perimeter $P = (B + 2y\sqrt{1+z^2})$

$$= (2.4 + 2y\sqrt{1+1.5^2})$$

Hyd. Radius $R = \frac{A}{P} = \frac{(2.4+1.5y)y}{(2.4+2y\sqrt{1+1.5^2}}$

$$6 = \frac{1}{0.015}(2.4+1.5y)y\left[\frac{(2.4+1.5y)y}{(2.4+2y\sqrt{1+1.5^2}}\right]^{2/3} \times 0.000177^{\,1/2}$$

by trial and error Y_{canal} 1.53333 m

$$V_{canal} = \frac{Q}{A} = \frac{6}{(2.4+1.5(1.53)1.53} = 0.83\,\mathrm{m/sec}$$

Given $h_L = (K_1 + K_2)(\frac{V^2 - V_1^{\,2}}{2g}) = 0.10$

$$H_L = (0.3 + 0.75)\left(\frac{V^2 - 0.83^2}{2 \times 9.81}\right) = 0.10$$

Velocity in box section $V = 1.6$ m /sec

Velocity head in the canal $= \dfrac{V^2}{2g} = \dfrac{0.83^2}{2 \times 9.81} = 0.035m$

Total energy line at sec. (1) = bed level $+ y_1 + \dfrac{V_1^2}{2g}$

$= 10 + 1.530 + 0.035$

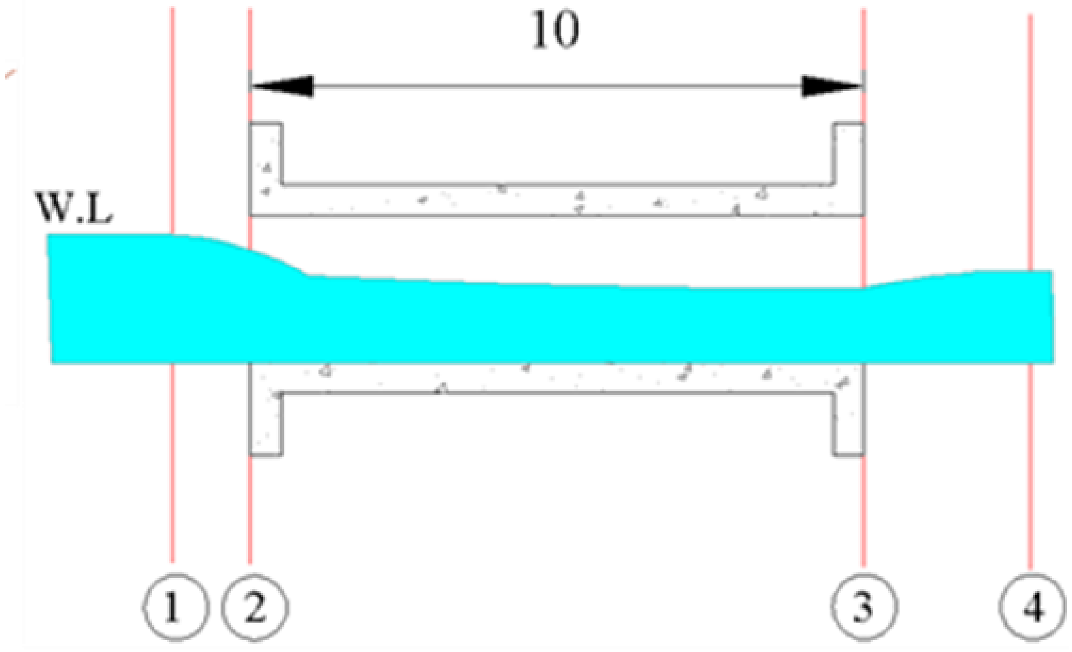
$= 11.565$ m

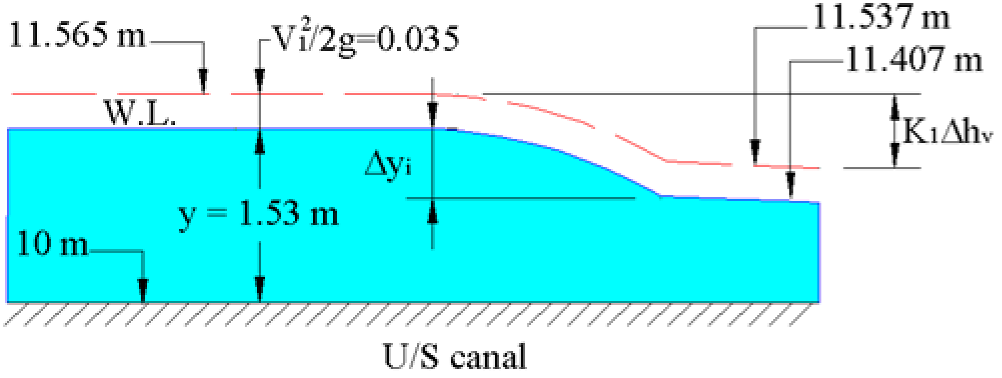
Velocity head in culvert $= \dfrac{V^2}{2g} = \dfrac{(1.6)^2}{2 \times 9.81} = 0.13$m

Total energy line at sec. (2) = $11.565 - K_1 \left(\dfrac{V^2 - V_1^2}{2g}\right)$

$= 11.565 - 0.3\,(0.13 - 0.035)$

$= 11.537$ m





Water Surface level at sec. (2) $= 11.537 - \dfrac{V^2}{2g}$

$= 11.537 – 0.13$

$= 11.407m$

$\therefore$ depth of water y in culvert $= 11.407 – 10.0 = 1.407m$

Suppose width of box section = B

$\therefore$ discharge $Q = B*Y*V$

$$\text{Or } B = \frac{Q}{V*y} = \frac{6}{1.6(1.407)} = 2.66m$$

Thus the width of the box is more than the bed width of the canal which is 2.40 m. It is not good practice to increase the bed width from 2.40 to 2.66 m.

$\therefore$ use B = 2.4m

New depth of flow in the box will by given by: -

$$Y = \frac{6.0}{2.4(1.6)} = 1.56m$$

∴ Floor level of the box = 11.407 – 1.56

= 9.85 m

Bed level of the canal = 10.0 m

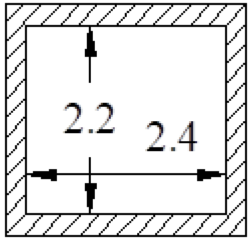
∴ The floor of the box will be depressed by 0.15 m

Allowing 0.60 m minimum clearance in the culvert section, the box height

= 1.56 +0.6 = 2.16 m $\cong$ 2.20 m

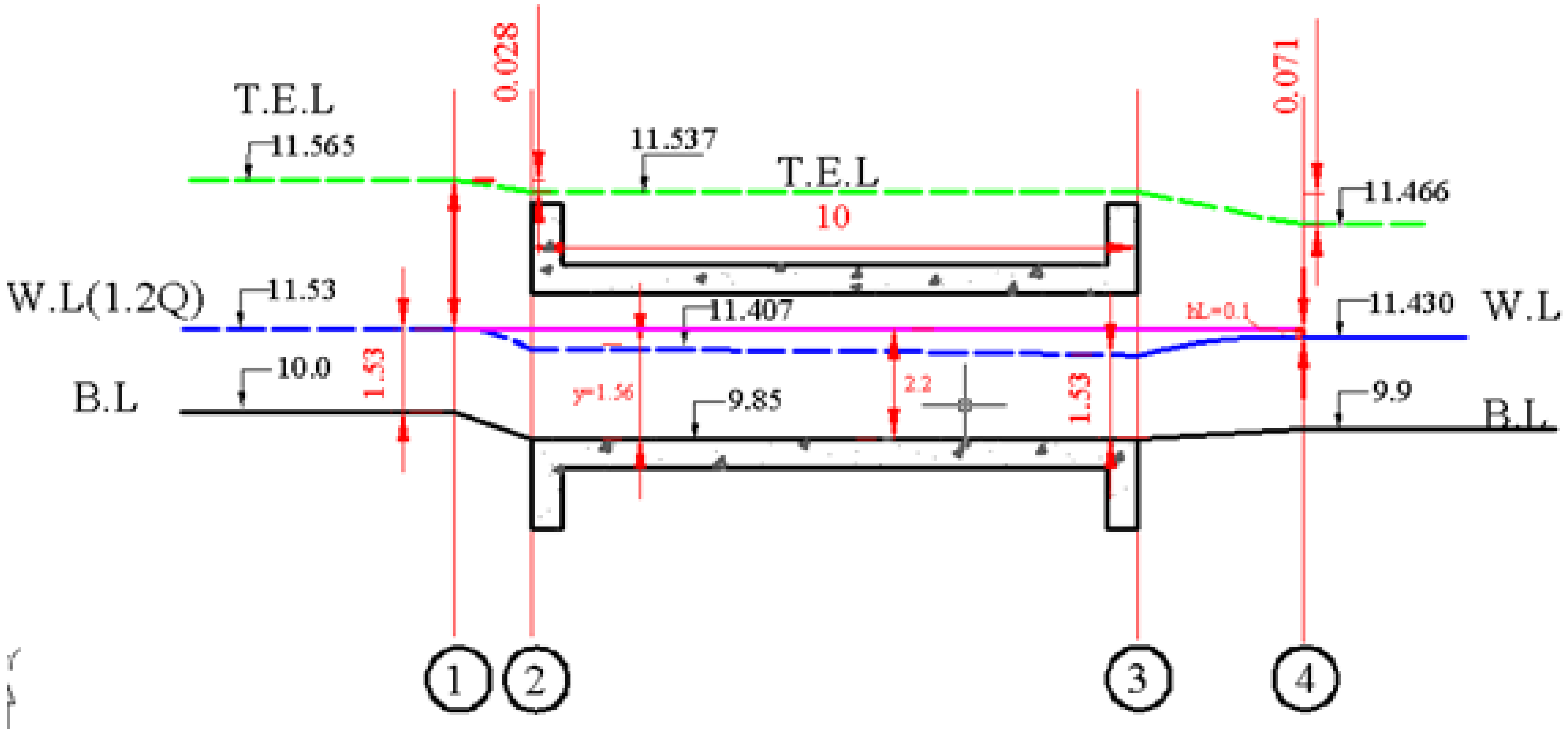
Box culvert size: - Box width 2.40 m

Box height 2.20 m



Note:

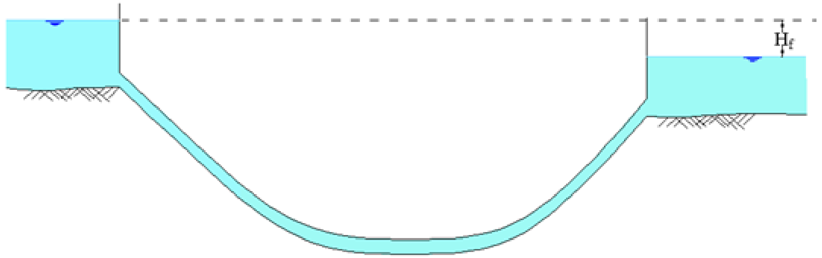
If the floor of the box were made level with the canal bed, the depth of flow U/S of the box section would have to increase to pass the discharge through the contraction, (backwater), so it is better to depress the floor of the box culvert.



T.E.L
11.565
0.028
11.537
T.E.L
10
0.071
11.466
W.L(1.2Q)
11.53
11.407
hL=0.1
11.430
W.L
10.0
1.53
B.L
y=1.56
9.85
2.2
1.53
9.9
B.L
1
2
3
4

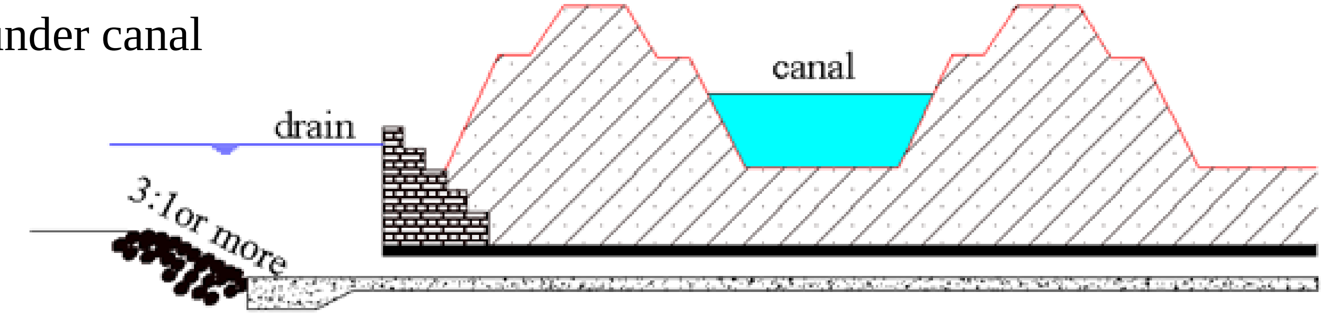
SIPHON / SYPHON

A siphon is a closed conduit designed to run full and under presser. Inverted siphons are used to convey canal water by gravity under roads, railroads, other structures, various types of drainage channels, and depressions.



Inverted siphons usually used for water supply to.

1. Cross deep valleys.
2. Pass drainage water under roads.
3. Pass drainage water under canal



As a structure, Same design and hydraulic principles of culverts are used for siphones except for the following points.

Design of syphons

1- Calculate size and water losses by the given method $H_f = \sum K\left[\frac{V^2}{2g}\right]$

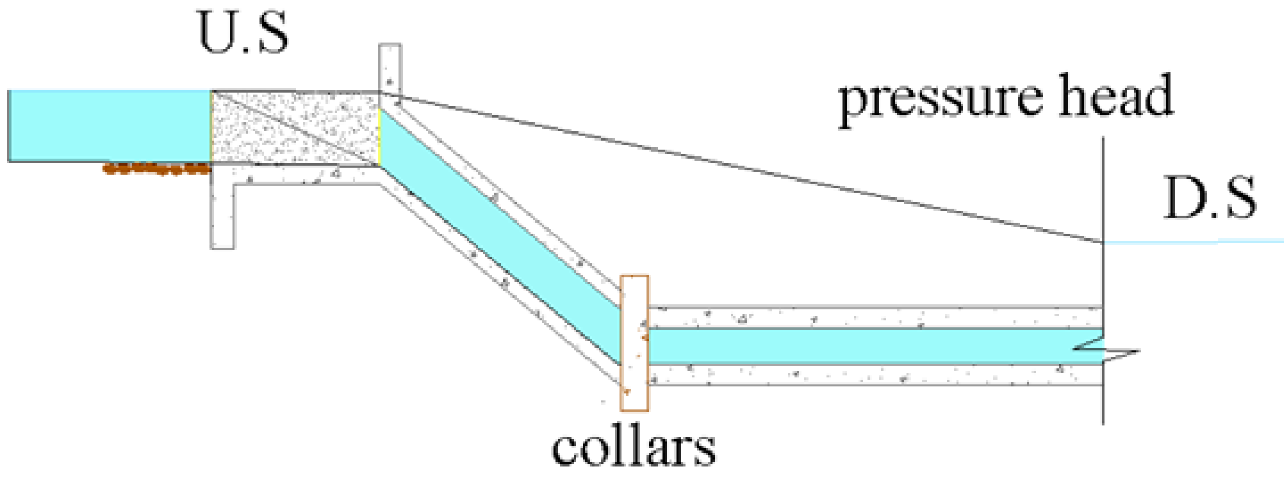
2- Floor length (equals to th pipe or culvert length) is determined by site conditions. (L_W) is determined by floor length plus any needed vertical cutoofs.

3- Transitions at inlet and outlet are determined according to the allowed H_f .For small H_f , transitions are needed to reduce losses at entrance and exit.

4- Riprap or pitching protection is needed when H_f is high and soil resistance is low.

Structural details

5- Structural details.



6- Hydrostatic pressure should be considered and pipe permeability should be low enough to prevent water leakage.

$V = Ki$ Darcy’s equation $$V = K_{conc.} \cdot \frac{H}{t_{conc.}}$$

Where: V = Velocity

K = Permeability

$i = H/t$ = hydraulic gradient

So to reduce seepage, from conc.: pipe you have to increase t_c (thickness of the pipe wall) or reducing the head H or using another material for the pipe with smalls K

7- Pipe slopes should not by steeper than ½ nor flatter than 1/200.

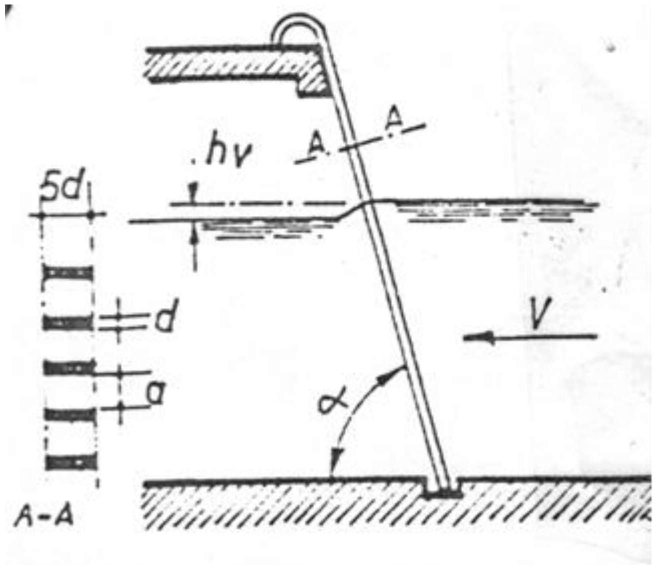
8- Earth cover to the top of a siphon barrel should not by less than 1.0 m where a siphon passes under a cross drainage channel.

For siphons under roads, L.L has to by considered and the siphon barrel has to by strong enough to resist vertical forces on, top of it.

9- Finally (same as in culvert design) If the computed losses Hf are greater than the difference in U/S and D/S canal water surface , the siphon will probably cause backwaters in the canal U/S from the siphon . If backwaters exist , the pipe size should be increased or the canal profile revised to provide adequate head.

On the contrary if the computed losses are appreciably less than the difference in U/S and D/S canal water surface it may by possible to decrease the size of pipe.

Losses in siphons: -



1. Entrance losses $(h_e) = K_1 \left(\frac{V_c^2}{2g} - \frac{V^2}{2g}\right)$

2. Outlet losses $(h_o) = K_2 \left(\frac{V_c^2}{2g} - \frac{V^2}{2g}\right)$

3. Bend losses $(h_b) = K_3 \left(\frac{V_c^2}{2g}\right)$

4. Friction losses $(h_f) = S_f * L$

   Where h_f = friction slope from Manning's equation or any other uniform flow equ.

5. Screen losses

   When the trash rack structure consists of racks of bars, the losses will depends on the bar thickness, depth and spacing. An average approximation can be obtained from the following Equation of screens with rectangular bars.

$$h_s = 2.42 \sin \alpha \left(\frac{d}{a}\right)^{4/3} . \frac{V^2}{2g}$$

$$h_s = 2.42 \sin \alpha (\frac{d}{a})^{4/3} . \frac{V^2}{2g}$$

Where:

h_s = head loss in (m)

α = Angle of inclination (degree)

d = thickness of bare (mm)

a = spacing of bare (mm)

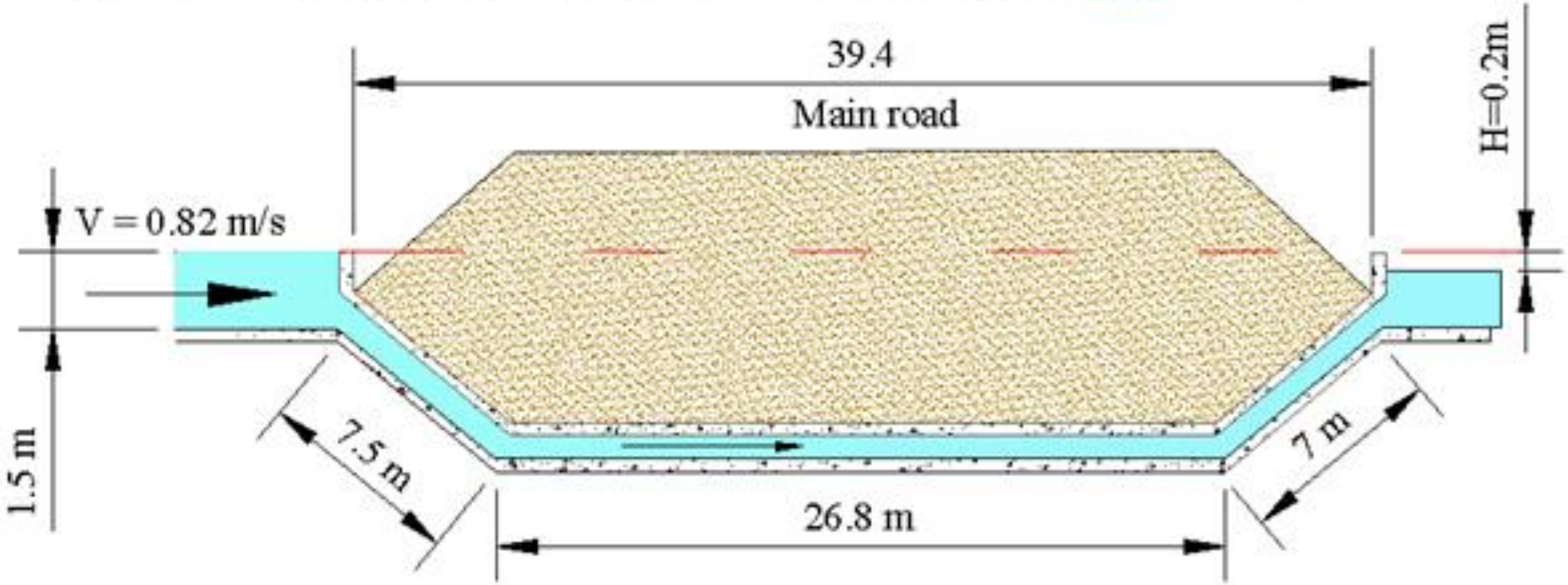
v = velocity of flow U /S the rack (m /sce)

$\Delta H_{total} = h_{entrabce} + h_{out\ let} + h_{bends} + h_{friction} + h_{screen}$ + any other losses which is considered as 10 % extra safety

i.e. Use $(1.1\ \Delta H_{total})$

Example (1): -

Design an Inverted Syphon required to pass canal discharge of 4 m^3/sec under a main road with 0.2 m head loss. The velocity in the canal is 0.82 m/sec and depth of water is 1.5m. Safety screens are provided at entry and exit. The inverted siphon has 22.5º elbows at each end. The site dimensions are shown in the figure. Use Manning's $n_{conc.} = 0.013$



Solution: -

Total head loss = U/S. W.L. – D/S. W.L.

$$H = 0.2 \text{ m (given)}$$

This total head loss is required to meet losses in the various components of the inverted siphon which are given below: -

a. Head loss due to friction in the barrel $= \dfrac{2gn^2L}{R^{4/3}}\left(\dfrac{V^2}{2g}\right)$

b. Head loss at entry and exit $K_1 = 0.2, K_2 = 0.3$

c. Head loss in the two screens $K = 0.2$

d. Head loss in the two elbows $K = 0.05$

For discharges up to about 2.5 m^3/sec, pipes can be used in the siphons, but for larger discharges a box section is preferred.

Use square box section, each side = d m

Area of box $A = d^2 \quad m^2$

Perimeter $P = 4*d \text{ m}$

Hydraulic Radius $R = \frac{A}{P} = \frac{d^2}{4d} = \frac{d}{4}$

Velocity in the canal $V = 0.82$ m/sec

Velocity head $\frac{V^2}{2g} = \frac{(0.82)^2}{2*9.81} = 0.034\text{m}$

Discharge in the barrel $Q = 4$ m³/sec

Velocity of flow (say) = V_1

$Q = AV_1 \longrightarrow V_1 = \frac{Q}{A} = \frac{Q}{d^2} = \frac{4}{d^2}$ m/sec.

Velocity head $\frac{V_1^2}{2g} = \frac{1}{2g}\left[\frac{4}{d^2}\right]^2 = \frac{16}{2gd^4}$

Length of the barrel L = 7.5 +26.8 +7.0 =41.3 m

a) Friction loss in the barrel $= \frac{2gn^2L}{R^{1.33}}\left(\frac{V_1^2}{2g}\right)$

$R = d/4$, $n = 0.013$

$$\therefore \text{ Friction loss} = \frac{n^2.L.4^{1.33}}{d^{1.33}}.V_1^{\,2}$$

$$= \frac{(0.013)^2(41.3)(4)^{1.33}}{d^{1.33}}\left(\frac{16}{d^4}\right) = \frac{0.709}{d^{5.33}} \qquad \ldots(1)$$

b) Entry and exit losses $= (0.2+0.3)\left(\frac{V_1^{\,2}}{2g} - \frac{V^2}{2g}\right)$

$$= 0.5\left(\frac{16}{2gd^4} - 0.034\right)$$

$$= \left(\frac{0.4077}{d^4} - 0.017\right) \qquad \ldots(2)$$

c) Head loss in two screens $= (0.2+0.2)\frac{V^2}{2g}$

$$= 0.4(0.034) = 0.0136 \qquad \ldots(3)$$

d) Head loss in two elbows $= 2 * 0.05\left(\frac{V_1^2}{2g}\right)$

$$\therefore \text{Head loss} = \frac{2 * 0.05 * 16}{2 * 9.81 * d^4} = \frac{0.0816}{d^4} \qquad \ldots(4)$$

Total head loss = (1)+(2)+(3)+(4)

$$= \frac{0.709}{d^{5.33}} + \frac{0.4077}{d^4} - 0.017 + 0.0136 + \frac{0.0816}{d^4}$$

$$= \frac{0.709}{d^{5.33}} + \frac{0.4077}{d^4} - 0.0034$$

This should be equal to total head loss = 0.20 m

$$\therefore \frac{0.709}{d^{5.33}} + \frac{0.4077}{d^4} - 0.0034 = 0.2$$

Solving by trial and error $\rightarrow$ d = 1.45 m

$\therefore$ Required square box size = 1.45*1.45 m

Example (2)

Given a single barrel single with the following data:-

Diameter of barrel D (m)

Length of barrel L = 90

Discharge through the barrel Q = 20 cumecs

Friction factors (in Darey – Weisbach formula) F = 0.013

Coeff of bend loss (2- bends) = 0.10

Coeff of bend loss in expansion at out let = 0.20

Coeff of head loss in contraction at inlet = 0.1 0

Friction loss in pipe (h_f) = 0.16 m

Determine afflux. Neglect velocity head

Sol

Total head loss in siphon = $\Delta H = 0.1\left(\frac{V^2}{2g}\right) + 2(0.1)\left(\frac{V^2}{2g}\right) + 0.2\left(\frac{V^2}{2g}\right) +$

$f\left(\frac{L}{d}\right)\left(\frac{V^2}{2g}\right)$

$$\Delta H = \left[0.1 + 0.2 + 0.2 + 0.013\left(\frac{90}{D}\right)\right]\frac{Q^2}{A_2}\cdot\frac{1}{2g}$$

$h_f = 0.16 = f\left(\frac{L}{D}\right)\left(\frac{V^2}{2g}\right)$

$= 0.013\left(\frac{90}{D}\right)\left(\frac{Q^2}{A^2}\cdot\frac{1}{2g}\right)$

Using box section

$Q > 2.5\ m^3/sec$

So $A = D \times D = D^2$

$$0.16 = 0.013\left(\frac{90}{D}\right)\left[\frac{20^2}{D^4}\cdot\frac{1}{2g}\right]$$

$$0.16 = 0.013\ (\frac{90}{D})\left[\frac{20^2}{D^4} . \frac{1}{2g}\right]$$

$$\Rightarrow D = 2.7\ m$$

$$\Rightarrow \text{Vel.}\ = \frac{Q}{A} = \frac{20}{(2.7)^2} = 2.74 m/sec$$

$$\Delta H \quad = 0.5\ 9\frac{V^2}{2g}) + 0.16$$

$$= 0.5(\frac{2.74}{2g}) + 0.16$$

$$\Delta H = \text{Afflux} = 0.35\ m$$

Weir:-

Any barrier(حاجز اة عائق) in an open channel over which flow takes place serves as a control which has a consistent relation between head and discharge so (it is an overflow structures built to measure the rate of flow of water).

Generally, it is of two types:

1. Sharp crested weirs
2. Broad crested weirs

The reason(السبب او الغرض) for using a weir in discharge measurement of a clear overfall weir depends on one factor only, namely, on the water depth over the sill(العتبة). Thus, the probable error of a discharge-observation will be directly dependent on the error of one-measurement only; whereas in the case of a regulator three separate measurements are to be performed in order to calculate one single discharge: -

1. The U/S water depth
2. The D/S water depth
3. The area of the openings,

For the same reason, it is much more accurate to measure the discharge by free over fall weir than by submerged one, because the second case needs to measure U/S and D/S depths to find the submerged ratio. Both kinds of weirs i.e. (sharp or broad) are either suppressed or contracted.

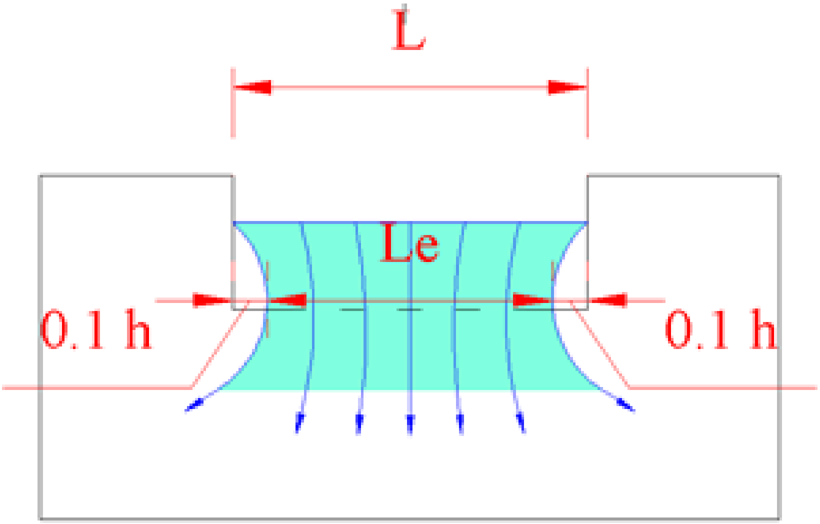
Suppressed قمعي weir: -

The sides of a rectangular weir are coincidentمتعاقد with the sides of the approach channel and extend D/S from the weir, the sheet of water (nappe) leaving the weir crest does not contract laterally.

Contacted Weir: -

The sides and crest of a rectangles, trapezoidal, or V- notch weir are far enough away from the sides and bottom of the approach channel, the nappe will fully contract laterally at the ends and vertically at the crest of the weir.

- A contracted weir is some times used to give a greater head for relatively low discharges in wide channels.
- The contracted weir is similar to the uncontracted in all respects except that its length is less than the width of the a approach channel; i.e., the effect of the end contractions must be considered



Where

L_e = effective weir length

L = weir length

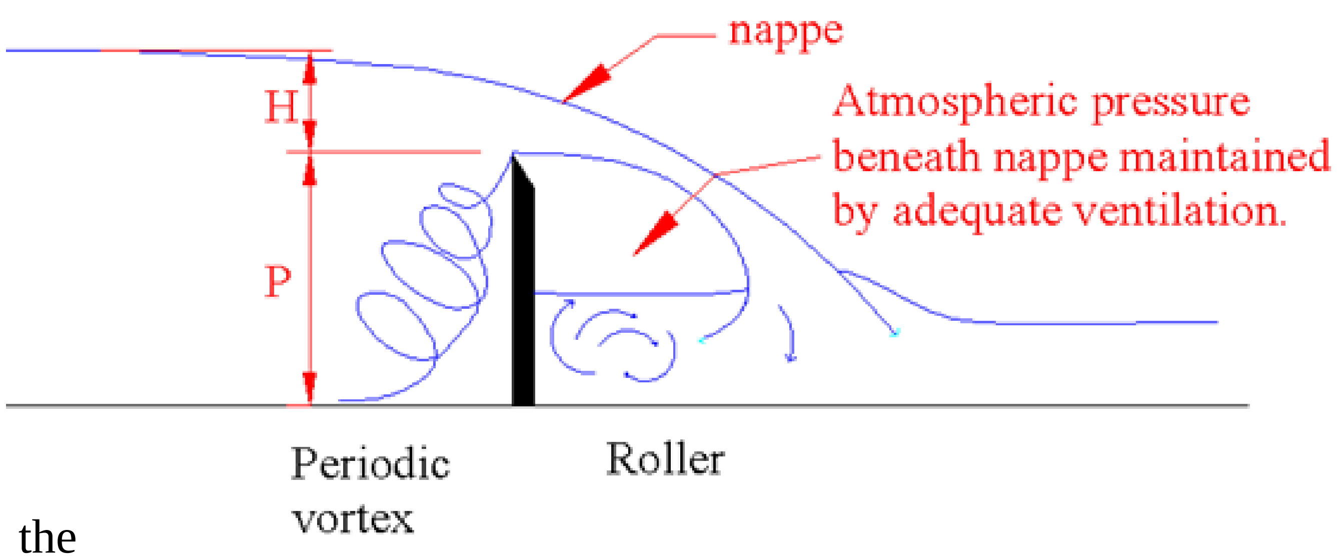
n = number of end contractions

h = head of water over the weir crest.

$$L_e = L - \frac{nh}{10}$$

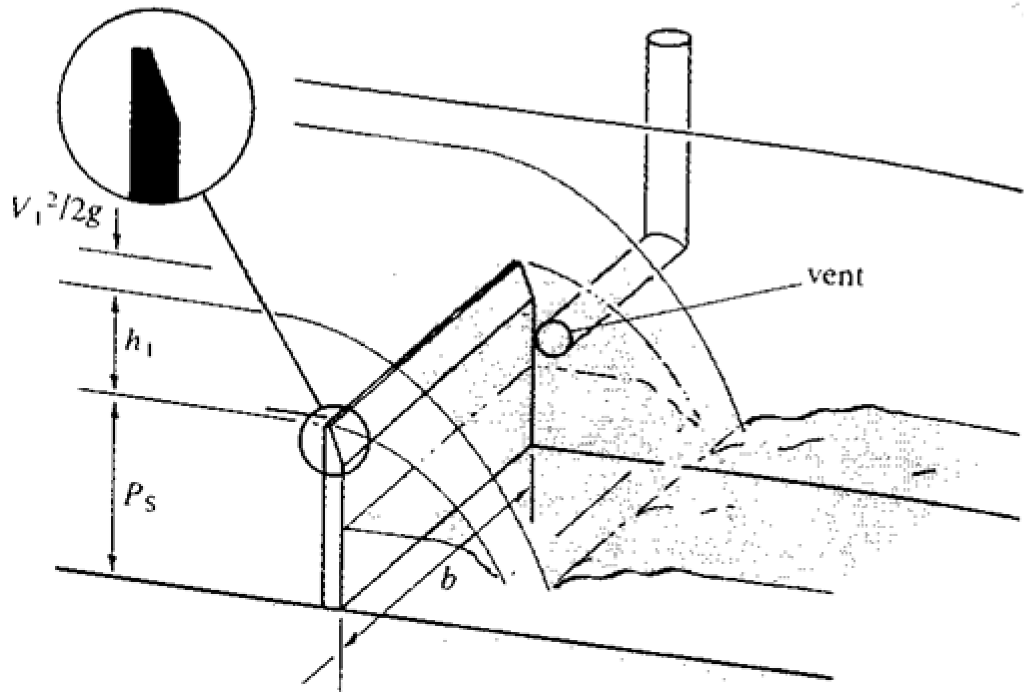
Sharp Crested Weirs

Actual weir flow is shown the figure

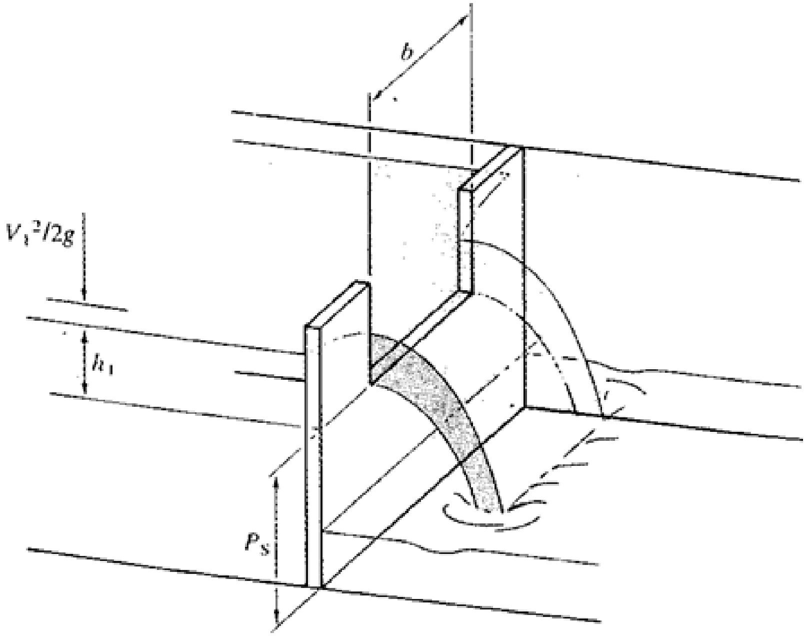


The geometry of the weir determines both the discharge coeff. and the exponent in the formula

$Q = C\ L\ h^n$



Types of sharp (حاد)crested weirs: -



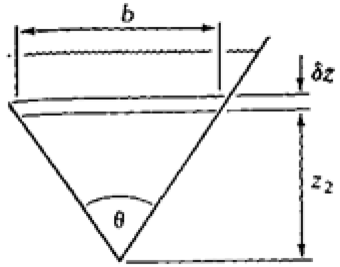
(1) Rectangles sharp crested weirs: - are used for large flows.

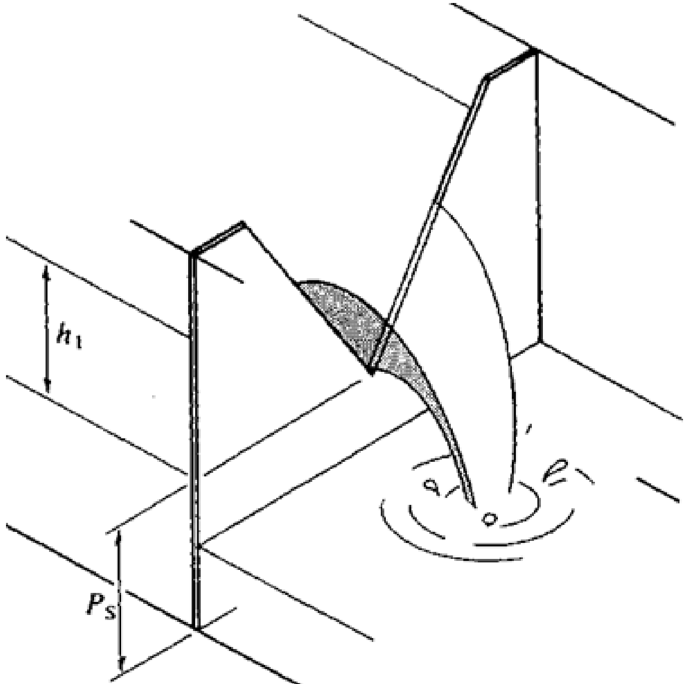
$$Q = 1.83\,(L - 0.2h)\,h^{3/2} \qquad \text{metric system}$$

(2) Triangular weir

This is preferred to a rectangular for measuring low discharges

$Q = 1.41\,h^{2.5}$ metric

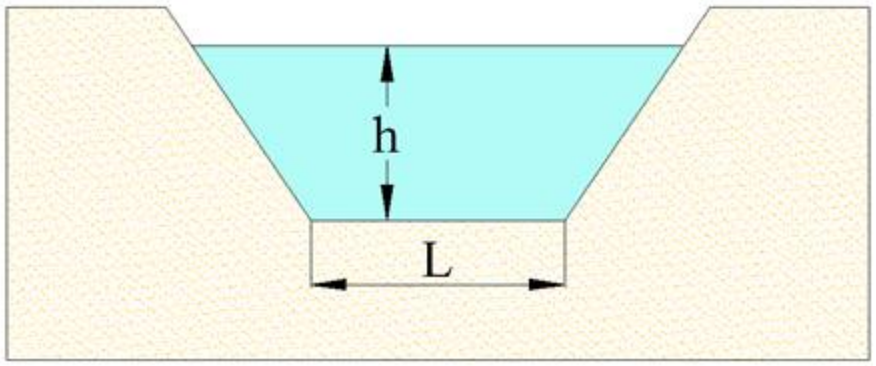




- No need for nappe ventilation.
- It is easy to measure h even for low discharges .
- h_a may be neglected without any appreciable error (خطا ملموس).

(3) Cipollitti trapezoidal weir:-

It is actually a combination of two weirs i.e., a rectangles and triangular weir



$$Q = 1.86\ L\ h^{3/2}$$

Board Crested Weir: -

A weir having a wide crest is known as a broad crested weir

- If the width of the weir B < 0.5H then the weir is considered as narrow crested weir and the discharge over it may by found by using equation for sharp crested weir.
- If the width of the crest of the weir B > 0.5H then it is behaves as a broad crested weir

There are two types of broad crested weirs: -

(1) With a sharp corner at the U/ S end.

(2) With a round corner at the U/S end..

Critical depth (h_c) will takes place over the weir crest. In other words, the discharge over a broad crested weir is maximum when the critical depth of flow occurs over the surface of the weir crest.

$h_c = 2/3\ H$

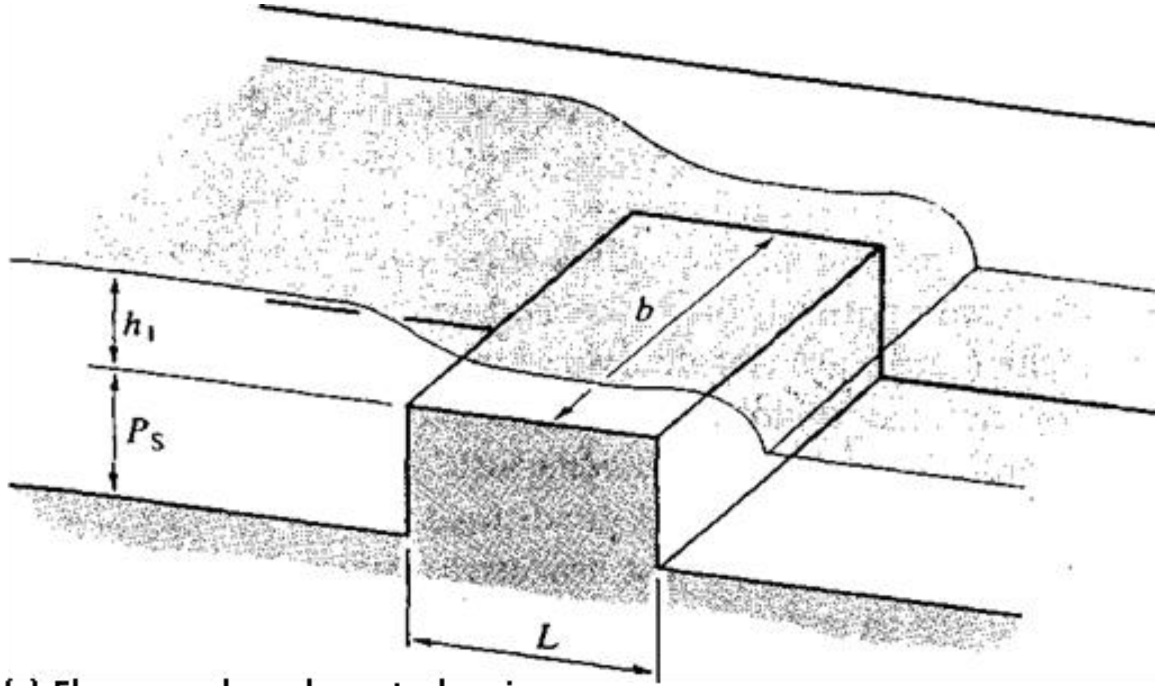
Where **(H)** is the head of water over the weir crest at the U /S side.

- The broad crested weir equation in general is :-

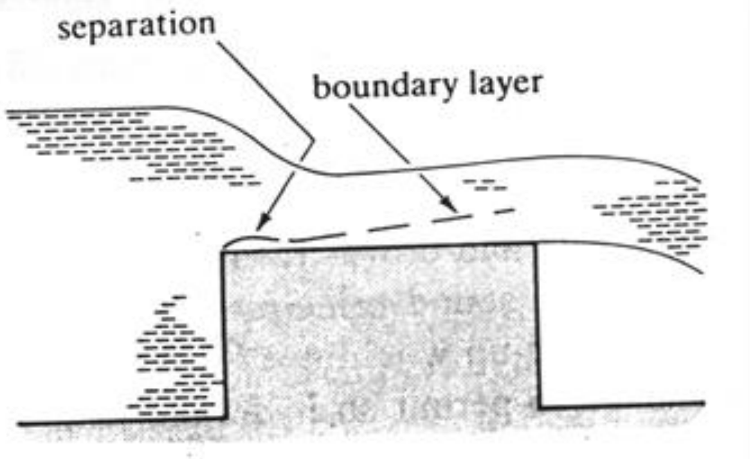
$Q = 1.7 C_d\ L\ H^{3/2}$

where :

C_d = discharge coeff. with value varies from (0.85 to 1.00)



(a) Flow over broad crested weir

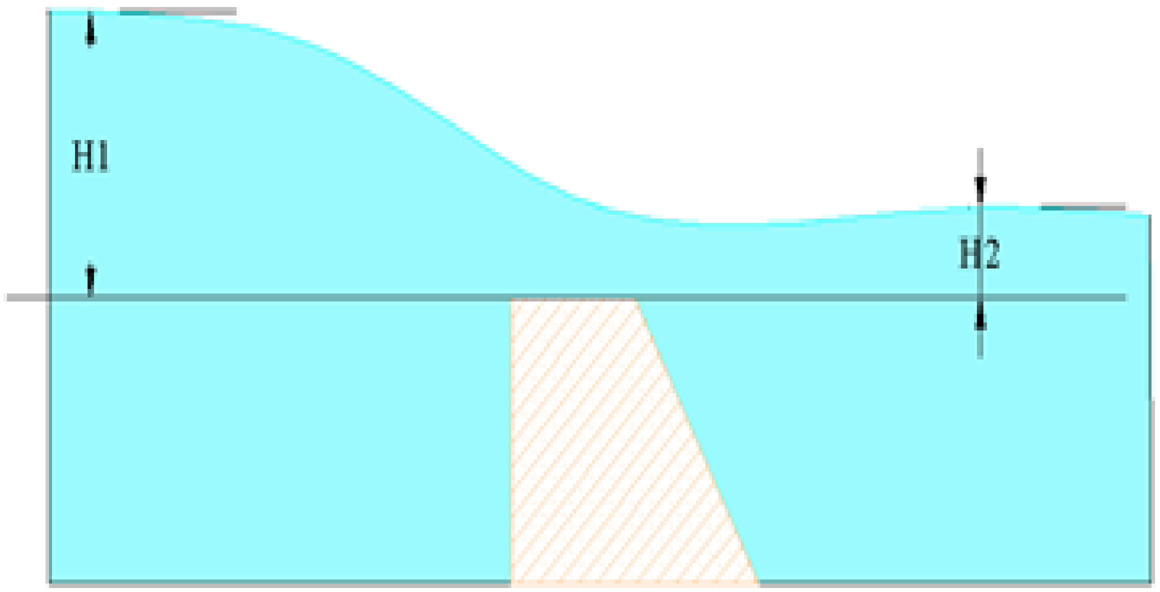


(b) Effect of boundary layer on flow

Submerged Weir:

When the water level on the D/S of the weir is above the crest level then the weir is said to be submerged weir.

Submerged weirs have larger discharging capacity as compared with freely discharging weirs



Submergence ratio = $\frac{H_2}{H_1} \times 100\%$

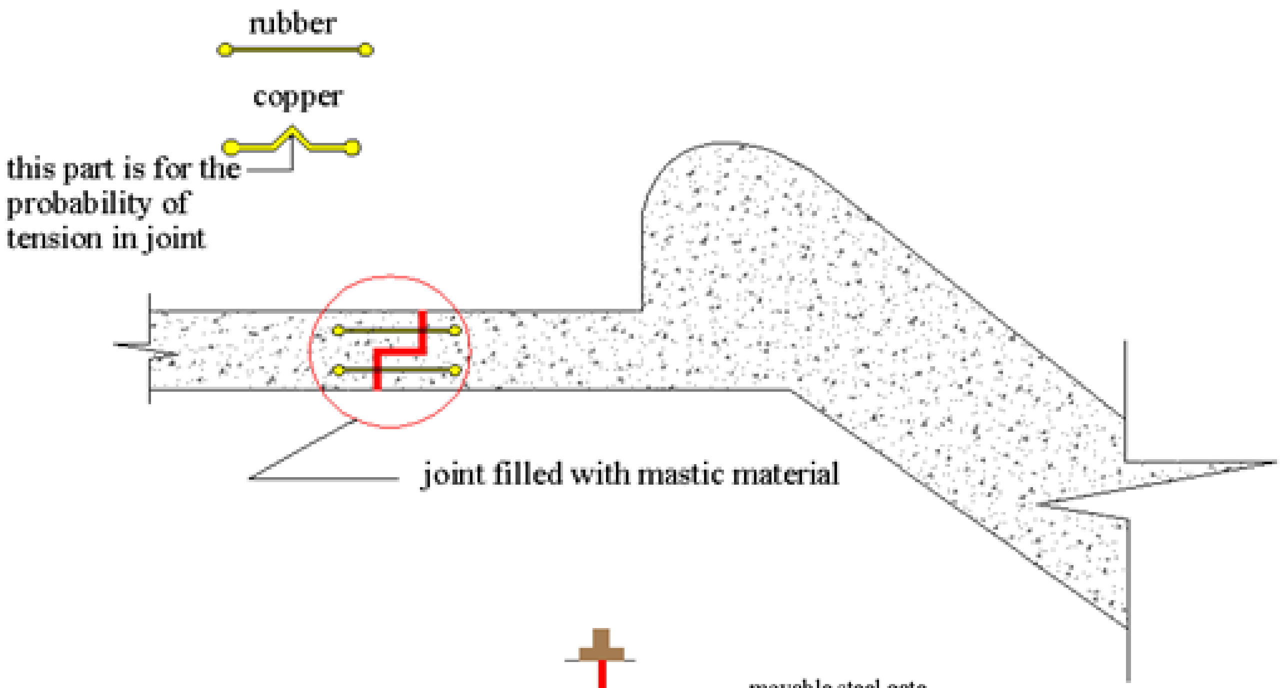
<u>Design of Weirs: -</u>

(1) To resist the existing up lift pressure due to max. head use: -

H (max. static head) = C .L (crest level) – R .L (reduced level of exit point)

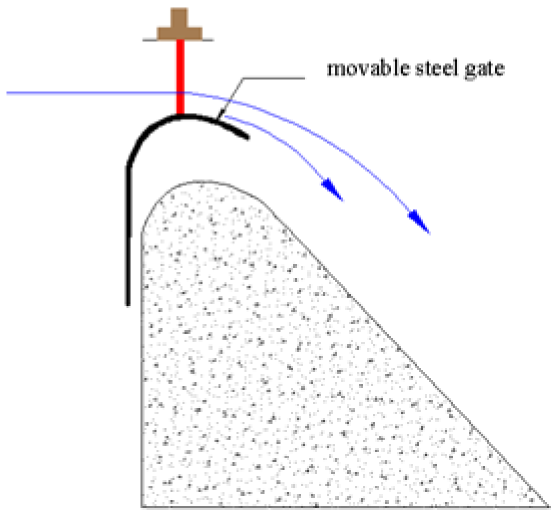
(2) The rest of the details follow the same lines as the regulators**.**

(3) For structural details (see the fig.)



Joint filled with mastic material (without this joint a crack in the conc. Floor may occur because of the large load of the inclined weir comparing with the thin flat floor at U/S).

(4) To take care of Q variation in canal, weirs with adjustable steel crest can be used which can be moved up and down to reduce or increase the head.



(5) Design of approaches (stone pitching …..)are the same as in regulator .

Example: -

A rectangles channel 6 m wide carrier 2800 liters per sec at a depth of 90cm. What height of broad crested rectangular weir must be installed to double the depth? Assume a weir co. efficient of 0.86

Solution : -

$Q = 1.70\ C_d\ L\ H^{3/2}$ ……. broad crested weir equation

$$Q = \frac{2800}{1000} = 2.8 m^3/sec; C_d = 0.86, L = 6m$$

Thus $\mathbf{2.8 = 1.70(0.86)\ (6)\ H^{3/2} =}$

$\Rightarrow \mathbf{H = 0.467\ m}$

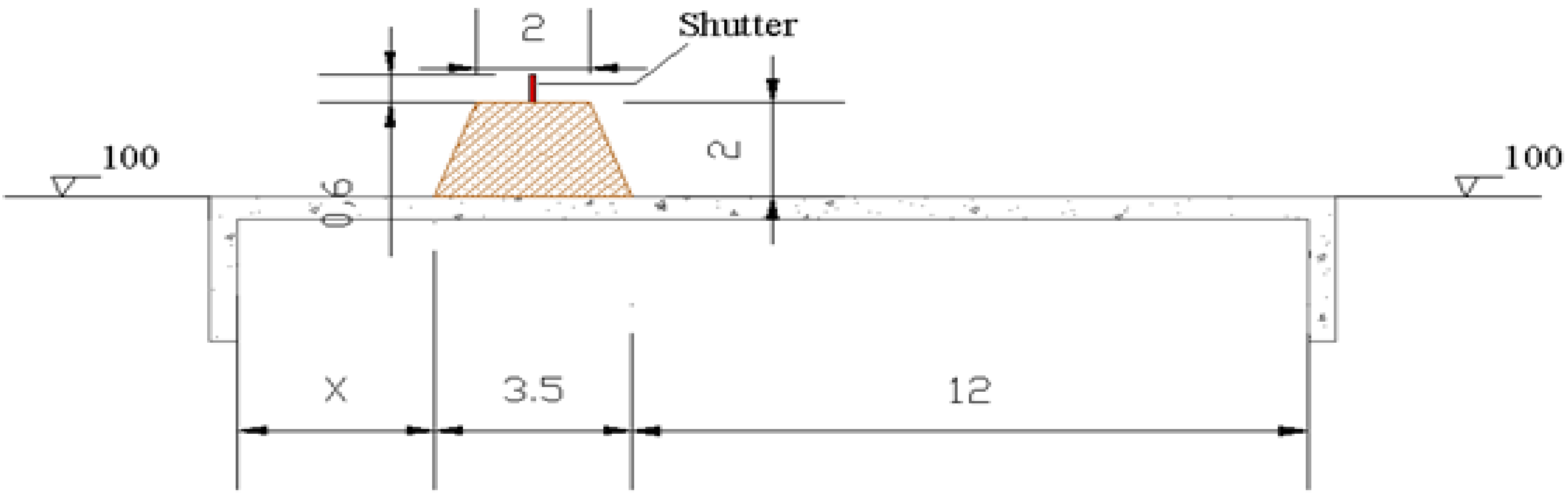
The depth of flow required to be developed in the channel = 2(0.90)

= 1.80 m

$\therefore$ Height of the broad crested weir P = (1.8 – 0.467) = 1.333 m

Example: -

Given a weir with the following particulars: -



C_w (Lanes coeff. of creep) = 7.23

Flood discharge	300 cumecs
Length of weir	40 m
Hight of weir P	2.0 m
Hight of shutter	0.6

Top Width of weir 2.0 m

Bottom width of weir 3.5 m

C_d (discharge coeff. for the weir) = 1.0

f (silt factor) 0.6

Calculate the floor length and thickness for the given cross section.

Solution: -

Total max static head = 2 +0.6 = 2.6m

$L_w = C_w H = 7.23\ (2.6) = 18.8m$

$Q = 1.7\ C_d\ L\ H^{3/2}$

$300 = 1.7\ (1)\ (40)\ H^{3/2} \rightarrow H = 2.68\ m$

$\therefore$ U /S Full Supply Level = 100+ 2.0 + 2.68 = 104.68m

$R = 1.35\ (\frac{q^2}{f})^{1/3} \qquad q = \frac{Q}{L} = \frac{300}{40} = 7.5 cumecs/m$

$R = 1.35\ (\frac{7.5^2}{0.6})^{1/3} = 6.124\ m$

U/S pile depth below U/S. F.S .L = 1.25R = 1.25(6.124) = 7.65 m

U/S pile depth below U/S. F.S .L = 1.25R = 1.25(6.124) = 7.65 m

∴ Level of bottom of U /S pile = 104.68 – 7.65 = 97.03 m

Provide a depth of 100 –97.03 = 2.97m says 3.0m for U/S cut off

Since the D/S full supply level is not known, and the exact calculation for the depth of D/S sheet pile can not be done Use a similar D/S cut off of 3 m depth below the weir floor.

Now total creep length provided except U /S floor L_w =

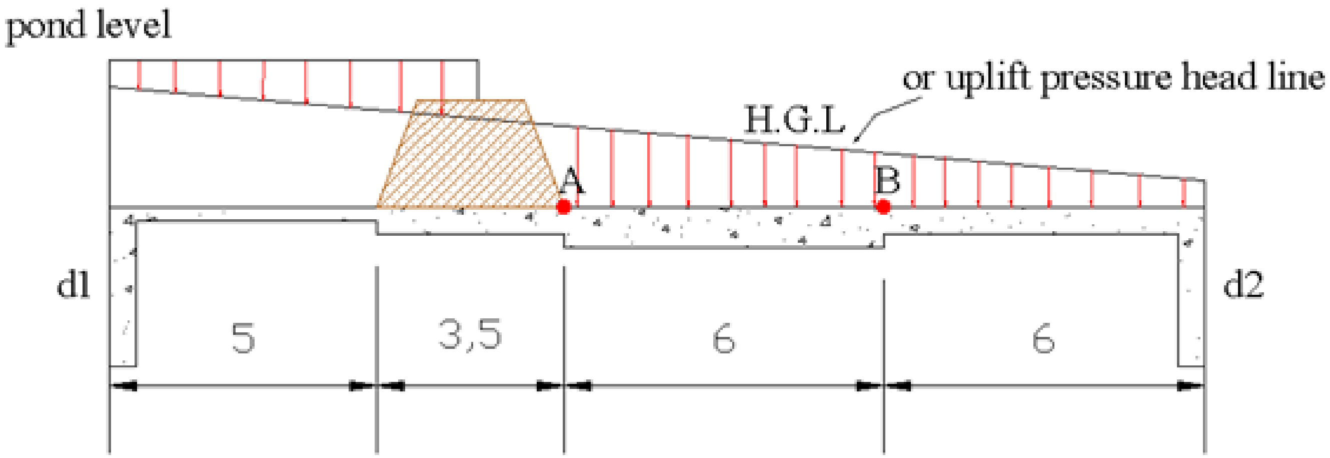
$$\left(\frac{12+3.5+x}{3}\right)+(3\times 2)+(3\times 2)$$

$$18.8=\frac{L_H}{3}+12$$

$\Rightarrow$ L_H = 20.4 m, x = 20.4- 15.5 = 4.9 m

Therefore, provided an U/S floor of 5.0 m Hence total horizontal floor length = 5 + 3.5 + 12 = 20.5 m

Floor thickness: -



The hydraulic grade line is now plotted as shown, the max **U.P** on the bottom of the floor for the D/S position is at the junction of weir with

(point A): $h_A = 2.6 - \frac{2.6}{18.8}(3 \times 2 + \frac{8.5}{3}) = 1.38\text{m}$

$$t_A = \frac{1.38}{2.4 - 1} = 0.98\text{m} \cong 1.00\text{m}$$

- Hence provide 1.0 m thickness for D/S floor from just near its

- Hence provide 1.0 m thickness for D/S floor from just near its junction with weir wall .
- The thickness required at half way D/S floor length

(Point B): $h_B = 2.6 - \frac{2.6}{18.8}(3\times 2 + \frac{14.5}{3}) = 1.09\text{m}$

$t_B = \frac{1.1}{2.4-1} = 0.786\text{m}$ **use 0.80 m**

Further, provide 0.80 m thickness below the weir and 0.40 m at the U/S side.

* For U/S and D/S protection work, use same detail as in head regulates

ملاحظة:

فى حاله وجود هدار أو مطفح وفوقه بوابات :

The discharge formula to be used in the design of a gated broad crested weir.

$$Q = 1.7\, C_d\, (L - K\, n\, H)\, H^{3/2}$$

where: -

Q = discharge in cumecs

H = total head in (meters) including velocity head.

n = No. of end contractions (Twice the number of gated bays).

L = clear waterway length in (meters).

K = Coeff. of end contraction, generally taken as 0.1 in ordinary calculation

Example:

The following data refer to a broad crested weir of $C_d = 0.85$.

Total number of vertical gates = 3

span of each gate = 5 m

Full supply water level = 60 m

crest level = 58 m

Compute the discharge passing over the weir; Neglect (h_a).

Solution:

$$Q = 1.7\, C_d\, (L - \overset{0.1}{k}\, n\, H)\, H^{3/2}$$

$H = 60 - 58 = 2$ m

$L_e = L - (0.1 \times \text{No. of end contractions} \times H)$

$L_e = L - (\underbrace{0.1 \times 2 \times 3 \times 2}_{1.2})$

$L_e = 5 \times 3 - 1.2$

$= 13.80$ m

$Q = 1.7\,(0.85)\,(13.8)\,(2)^{3/2}$

$= 56.4 \ m^3/sec.$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