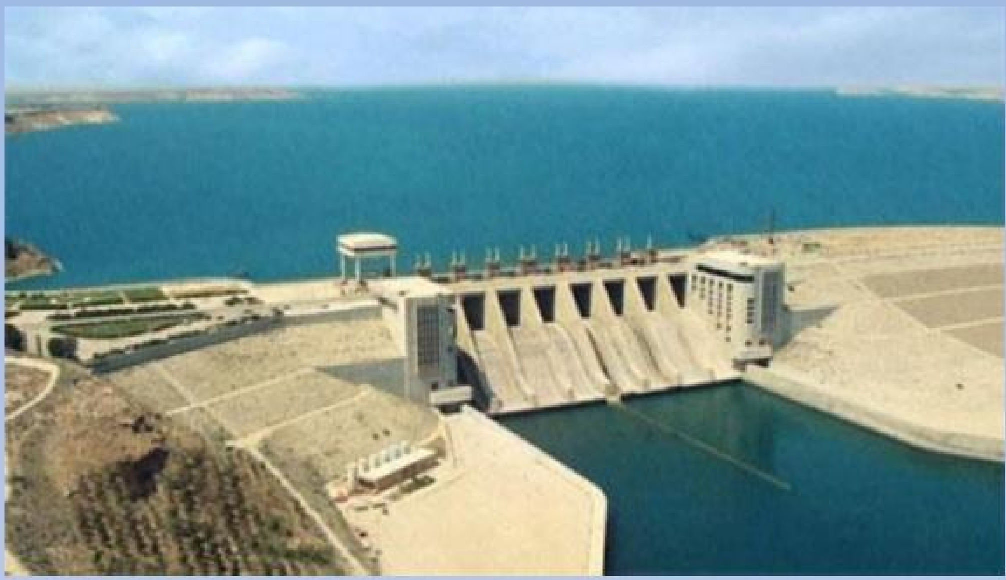

Design Of Hydraulic Structures

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Contents

Chapter 1 The Type Of Hydraulic Structure And Its Design Procedure
1.1 Introduction
1.2 Types of Hydraulic Structures
1.3 Steps for design of Hydraulic Structures
1.4 Site conditions
1.5 Structures on gypsum soil

Chapter 2 Seepage under hydraulic structures
2.1 Introduction
2.2 Causes of failure of hydraulic structures founded on previous foundation
2.3 Bligh's Creep Theory
2.4 LANE'S WEIGHTED CREEP THEORY
2.5 KHOSLA'S THEORY
Problems

Chapter 3 Control Structures
3.1 Introduction
3.2 Regulators
Problem

Chapter 4 Energy Dissipated Structures
4.1 Introduction
4.2 Hydraulic Jump
4.3 Vertical drop
4.4 Chutes
4.5 Stilling basin

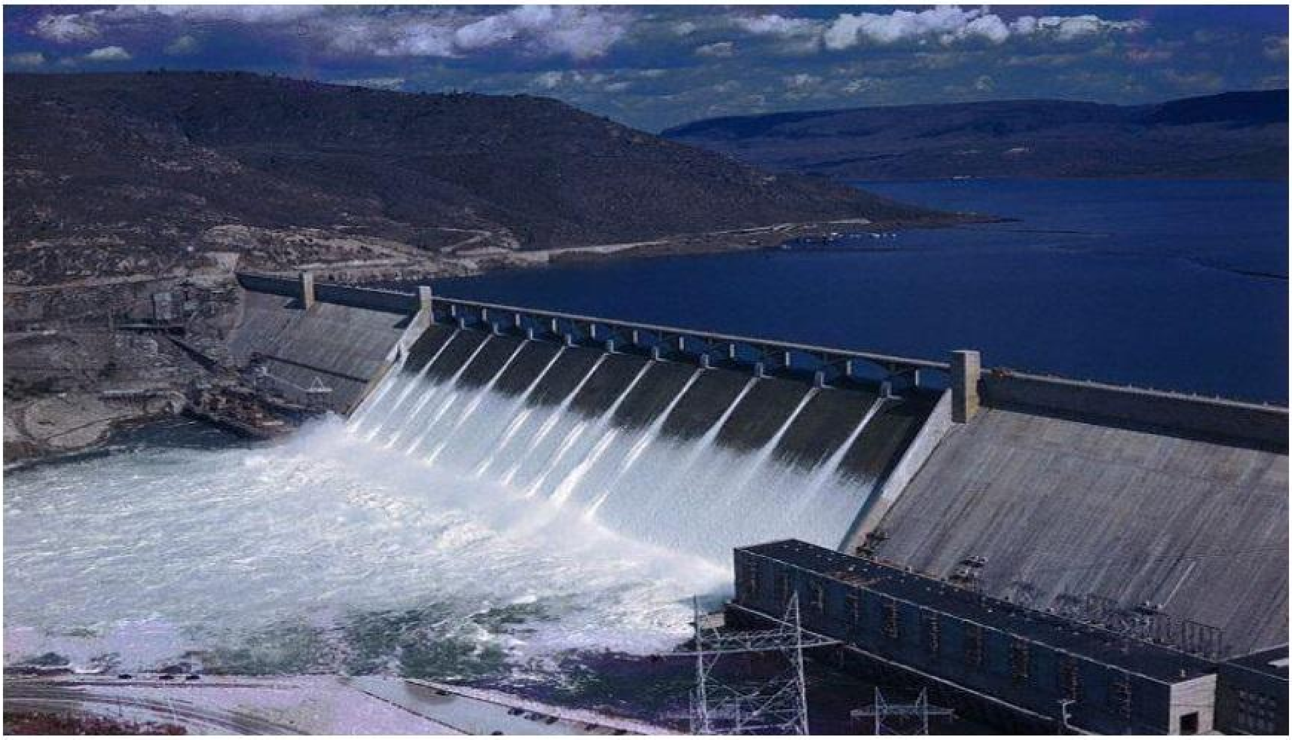
Chapter 5 Protection of approaches for horizontal floor
5.1 Introduction
5.2 Method of Protection
5.3 Factors affected the selecting rip-rap

Chapter 6 Conveyance Structures
6.1 Introduction
6.2 Culverts
6.3 Aqueduct Structures
6.4 Inverted Siphon
Problems

Chapter 7 Design of gates
7.1 Types of gates
7.2 Principle for design of lifting gate
Problems

Chapter One

The Type Of Hydraulic Structures And Its Design Procedure



Chapter 1 The Type Of Hydraulic Structure And Its Design Procedure

1.1 Introduction

A **hydraulic structure** is a structure submerged or partially submerged in any body of water, which disrupts the natural flow of water. They can be used to divert, disrupt or completely stop the flow. An example of a hydraulic structure would be a dam, which slows the normal flow rate of the river in order to power turbines. A hydraulic structure can be built in rivers, a sea, or any body of water where there is a need for a change in the natural flow of water.

Hydraulic structures may also be used to measure the flow of water. When used to measure the flow of water, **hydraulic structures** are defined as a class of specially shaped, static devices over or through which water is directed in such a way that under free-flow conditions at a specified location (point of measurement) a known level to flow relationship exists. **Hydraulic structures** of this type can generally be divided into two categories: flumes and weirs.

1.2 Types of Hydraulic Structures

Hydraulic structures can be classified according to the purpose of its used into the following types :

No.	Type Of Structure	Examples
1	Storage	reservoirs & tanks.
2	Conveyance	pipe lines, tunnels, siphons, Aqueducts, culverts, …etc.
3	Energy dissipation	stilling basins, drop structures

4	Flow measurement	weirs, venture, flumes, nozzles.
5	Control Structures	barrages, regulators.
6	Sediment &Chemical Control Structure	sedimentation tanks and basins silt traps, mixing basins
7	Navigation Locks	
8	Pumping stations	
9	Power stations	

Notes :-

For any hydraulic structures to design we must study the following:

1. Hydrologic studies.
2. Hydraulic studies.
3. Structural studies.

1.3 Steps for design of Hydraulic Structures

1. Prepare information for design:-
   a. The precise function of design.
   b. Discharge (Max. & Min.) Use 1.2Q max. discharge & 0.7Q for min. discharge.
   c. Head lose.
   d. U/S & D/S canal.
2. Determine the best location of the structure.
3. The shape of approach and the other components of the structure.
4. The requirements of water-way.
5. Protection against scouring.
6. The best method of dissipation energy

7. Forces acting on various parts of the structure, Hydraulic forces (hydrostatic
8. pressure, dynamic forces) & other forces, live loads, dead loads, earth pressure.

1.4 Site conditions

In design of any structure, site condition have be taken into accounts-:

1. Soil properties.
2. Ground water.
3. Soil strength parameter.
4. Permissible bearing pressure.
5. Permeability.
6. Mineral content (especially sulphates) to both soil & ground water.

1.5 Structures on gypsum soil

Regardless of the mode occurrence, the effect of saturation of the pore space with datively fresh water is that gypsum as taken into solution.Permeability is increased with consequent increase in seepage rate, soil strength is reduced, cavities are formed in the soil structure & foundation failure & piping may occur.Where site investigation shows significant gypsum concentration in the underlying soil strata, every efforts should be made to relocate structures to more favorable locations.Channels U/S & D/S of the structure should lined and particular attention paid to joints to ensure that water tights is maintained.

Chapter Two

Seepage under hydraulic structure

Chapter 2 Seepage under hydraulic structures

2.1 Introduction

The hydraulic Structures such as barrages, regulators, culverts, etc…, may either founded on un impervious solid rock foundation or a pervious foundation. It is subjected to seepage of water beneath the structure in addition to all other forces to which will be subjected. When founded on pervious rock foundation, the water seeping below the body of the hydraulic Structures in dangers the stability of the structures & may cause its failure.

2.2 Causes of failure of hydraulic structures founded on previous foundation

2.2.1 Failure by piping or undermining

Water starts seeping under the base of hyd. Structure. It starts from U/S side and tries to exit at the D/S' end of the impervious floor. At the point of the exit, the exit gradient may become more than the critical gradient, in which cause, the water starts dislodging the soil particles & carrying it away with it causing formulation a hole in the subsoil. So, formed resulting in the failure of the structure. Piping can prevented by the following methods:-

A)<u>By providing sufficiently long impervious floor:-</u>

This long length will reduce the exit velocity & exit gradient. As the water has to travel along distance beneath the floor, its head will sufficiently lost before it exits & its velocity will be such that it's cannot wash away any soil or sand particles.

B)<u>By providing piles at both U/S and D/S ends:-</u>

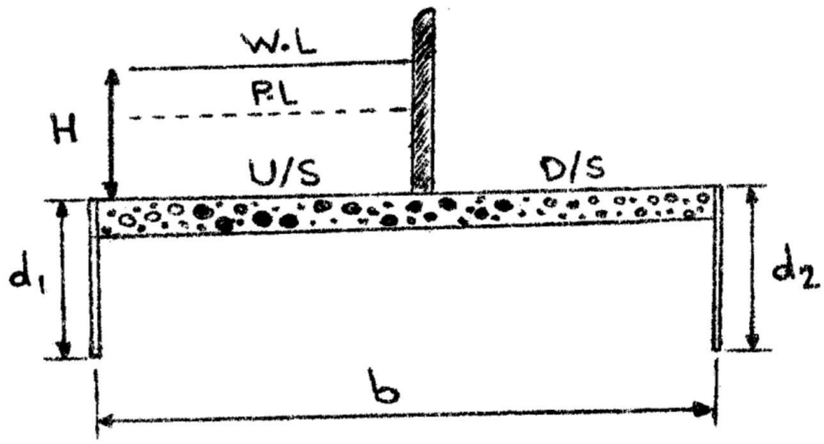
This measure also results is increasing the path of the travel of seepage water & hence it decreases its exit velocity & exit gradient

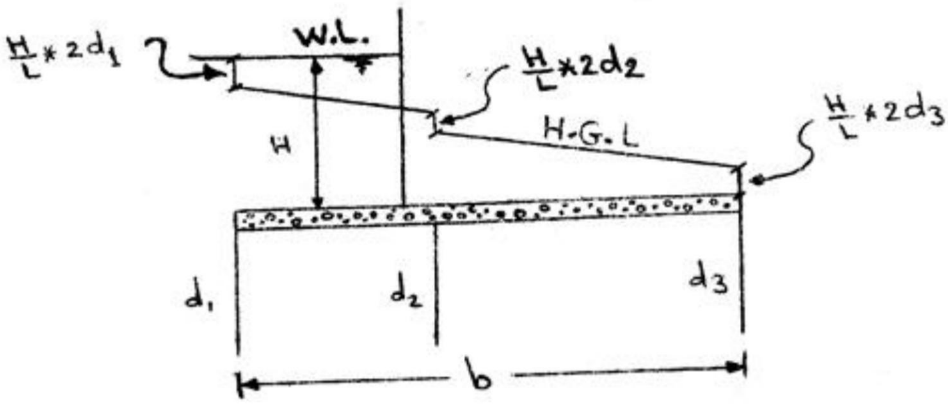
2.2.2 Failure by direct uplift

The water seeping below the structure exerts on uplift pressure on the floor of the structure if this pressure is not counter balanced by the weight of concrete or masonry floor. The structure will fail by a rupture of a part of the floor.

2.3 Bligh's Creep Theory

According to Bligh’s theory, water creeps along the bottom contour of the structure. The length of the path of water is called the length of creep and the loss of head is proportional to the length. If HL is the total head loss between upstream and downstream and L is the length, then the loss of head per unit of creep length (i.e. H L /L) is called the hydraulic gradient. Bligh’s theory makes no discrimination between horizontal and vertical creeps.





H=(U/S)water level – (D/S)water level

$L=2d_1+b+2d_2$

Hydraulic Gradient $= \frac{H}{L}$

<u>Safety against piping or undermining</u>:-

L=CH ; C: is a constant for the soil (Bligh’s coefficient)

Table 2.1

Values of Bligh's coefficient's

Type of soil	C	Safe hyd. Gradient should be less than
Fine sand, alluvial soil	15	$\frac{1}{15}$
Course grained sand	12	$\frac{1}{12}$
Sand with boulders & gravel & for loam soil	5-9	$\frac{1}{5}-\frac{1}{9}$
Light sand	8	$\frac{1}{8}$

2.3.1 Safety against uplift pressure

The ordinate of hydraulic grade line above the bottom of the floor represent the residual uplift water head at each point. Say for example; if at any point the ordinate of H.G.L above the bottom of the floor is 1m, then 1m head of water will acts as uplift at the point.

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

Uplift pressure= $w.h'$
w: the density of water.
Downward pressure = $(w.G)t$
G: specific gravity of the floor material. h'

For equilibrium:

$$w * h' = w * G * t \Rightarrow h' = G * t \Rightarrow h' - t = G * t - t$$

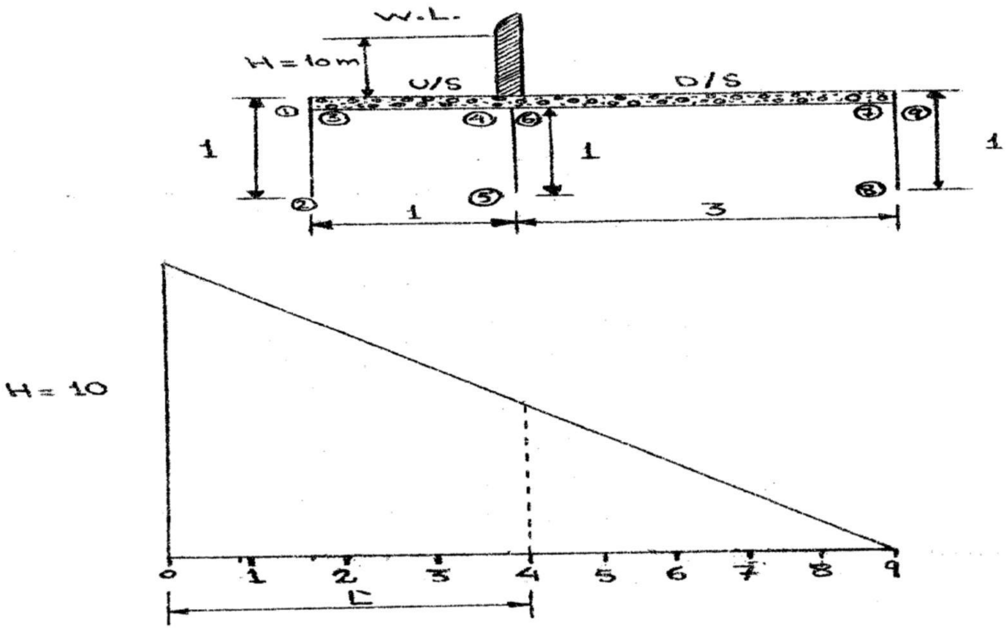
$$t = \frac{h' - t}{G - 1} \Rightarrow t = \frac{h}{G - 1}$$

Notes

For concrete t = (2/3)h

Example 2.1

Find the uplift pressure at key point (4)



Solution

$$UP = H - \frac{H}{L} * L'$$

$$(U.P)4 = 10 - \frac{10}{10} * 3 = 7m$$

$$t = \frac{2}{3} * 7 = \frac{14}{3} m$$

2.4 LANE'S WEIGHTED CREEP THEORY

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

$$L = \frac{1}{3}b + d \qquad ; \qquad L = C_l H$$

Table 2.2

VALUES OF LANE'S COENFFICIENTS (C_l)

Type of soil	C_l	Safe Lane's Hyd. Grade should be less than
Very fine sand or silt	8.5	$\frac{1}{8.5}$
Fine sand	7.0	$\frac{1}{7.0}$
Course sand	5.0	$\frac{1}{5.0}$
Gravel & sand	3.5-3.0	$\frac{1}{3.5} - \frac{1}{3.0}$
Boulders gravel and sand	3.0-2.5	$\frac{1}{3.0} - \frac{1}{2.5}$
Clay soil	3.0-1.6	$\frac{1}{3.0} - \frac{1}{1.6}$

2.4.1 Critical Exit Gradient

Factor of safety (F) = $\frac{\Gamma b}{\Gamma_w * G_e} = \frac{(\Gamma_s - l)(l-n)}{l * G_e} = \frac{(2.65-l)(l-0.4)}{l*G_e}$

$\therefore F = \frac{1}{G_e}$

The exit gradient can be taken as safe if a factor of safety as below:

$F = 4$ for the rock.

$F = 5$ for the small rock.

$F = 6$ for the gravel.

- In Iraq, for the alluvial soil, take $F = 8 - 10$.
- We can use the following eq. to calculate the exit gradient (GE):

$GE = \frac{KH}{d}$ $\quad ; where \quad k = 0.45/\sqrt{1+m}$

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

b: horizontal length of the flow.

d: depth of the downstream cut-off.

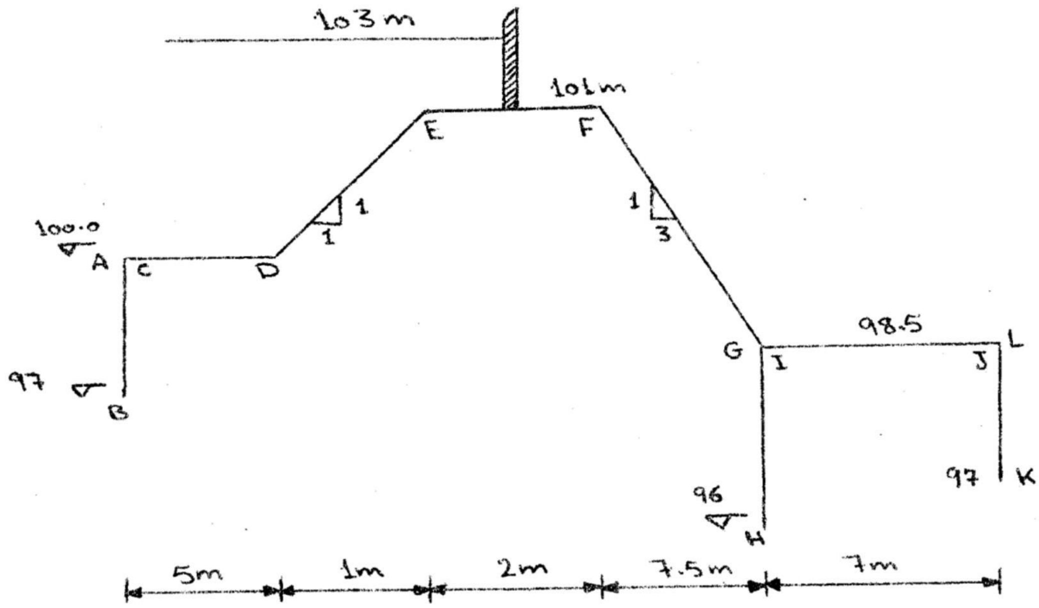
$H = U/S\ water\ level - D/S\ water\ level$ (level of the exit point).

***Note*:**

the exit gradient in the U/S side of the structure don't cause any piping or scouring.

Example 2.2

A hydraulic structure as shown in fig below, determine whether the structure is safe against piping & design the thickness of floor, the soil is light sand?



solution

1) By Bligh's theory:-

$L = 3 + 3 + 5 + 1.414 + 2 + 7.9 + 2.5 + 2.5 + 7.0 + 1.5 + 1.5 = 37.314\ m$

Seepage head=$103 - 98.5 = 4.5\ m$

Hydraulic gradient = $\frac{H}{L} = \frac{4.5}{37.314} = \frac{1}{8.3} < \frac{1}{c}$

Hence, it is safe against piping because $\frac{1}{8.3} < \frac{1}{8}$.

The residual head at point, B, C, D, …,etc, are obtained by multiplying L_B, L_C, L_D,…etc.

Distance	Length (m)	Point at which req. head	Residual head (m)
L_A	37.314	A	4.5
L_B	34.314	B	4.13
L_C	31.314	C	3.77
L_D	26.314	D	3.16
L_E	24.9	E	3.0
L_F	22.9	F	2.76
L_G	15.0	G	1.79
L_H	12.5	H	1.51
L_I	10.0	I	1.2
L_J	3.0	J	0.357
L_K	1.5	K	0.18
L_L	0.0	L	0.0

- On the U/S side where water is standing, no thickness is required, but a min. 0.5m should be provided to make the floor impervious.
- On the D/S side; the thickness of the floor required is calculated (floor may be made stepped).

$$t_G = \frac{2}{3} * 1.79 = 1.2\ m$$

$$t_I = \frac{2}{3} * 1.2 = 0.8\ m$$

$$t_J = \frac{2}{3} * 0.357 = 0.3\ m$$

2) By Lane's theory:-

$$L = 3 + 3 + \frac{5}{3} + 1 + \frac{2}{3} + \frac{7.5}{3} + 2.5 + 2.5 + \frac{7}{3} + 1.5 + 1.5 = 22.2\ m$$

$$H = 4.5\ m \Rightarrow \frac{H}{L} = \frac{4.5}{22.2} = \frac{1}{4.9}$$

Distance	Length (m)	Point	Residual head (m)
L_A	22.2	A	4.5
L_B	19.2	B	3.9
L_C	16.2	C	3.3
L_D	14.53	D	2.95
L_E	13.53	E	2.74
L_F	12.87	F	2.61
L_G	10.4	G	2.1
L_H	7.87	H	1.6
L_I	5.4	I	1.1
L_J	3.0	J	0.62
L_K	1.5	K	0.31
L_L	0.0	L	0.0

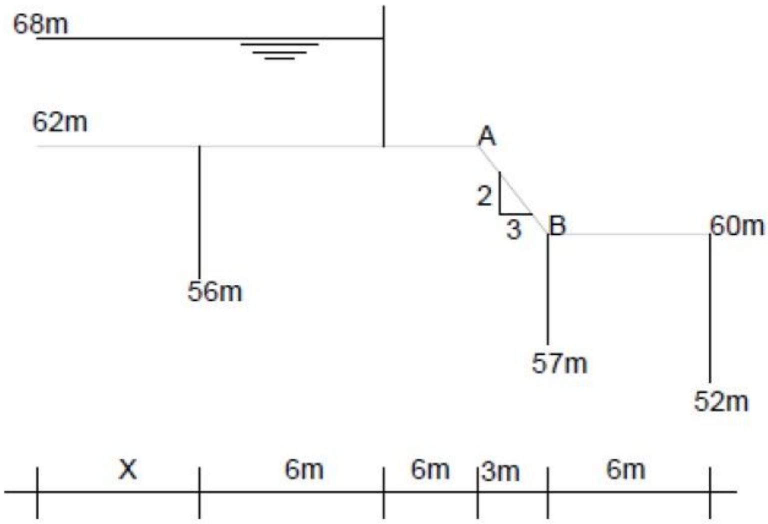
$$t_G = \frac{2}{3} * 2.1 = 1.4\ m$$

$$t_I = \frac{2}{3} * 1.1 = 0.75\ m$$

$$t_J = \frac{2}{3} * 0.62 = 0.4\ \text{m}$$

Example 2.3

Find the minimum length of the floor (×) for the show fig. If the thickness of the floor at A and B equal 3m and 1.8m respectively , c=6 using lanes method.



Solution

$$H = 68 - 60 = 8m$$

$$\text{L} = \frac{\times}{3} + \frac{18}{3} + \frac{3}{3} + 2 * 6 + 2 * 3 + 2 * 8 = 41 + 0.33 \times$$

$$L@A = 16 + 0.33 \times$$

$$L@B = 23 + 0.333 \times$$

$$u.p@A = 8 - \frac{8}{41{+}0.333\times}(16 + 0.333 \times) = 8 - \frac{128{+}2.66\times}{41\_0.33\times}$$

$$u.p@B = 8 - \frac{8}{41{+}0.333\times}(23 + 0.333 \times) = 8 - \frac{184{+}2.66\times}{41\_0.33\times}$$

$$3 = \frac{2}{3}\left(8 - \frac{128{-}2.66\times}{41{+}0.33\times}\right)$$

$$\times = 10.34\ m$$

$$1.8 = \frac{2}{3}\ (8 - \frac{184 - 2.668 \times}{41 + 0.333 \times})$$

$$\times = 37.04m$$

$L \geq c * H$ $\qquad 41 + 0.333 \times = 48 \qquad \times = 21$

Use $\times = 37.04m$

$$1.8 = \frac{2}{3}\left(8 - \frac{184 - 2.668 \times}{41 + 0.333 \times}\right)$$

$\times = 37.04m$

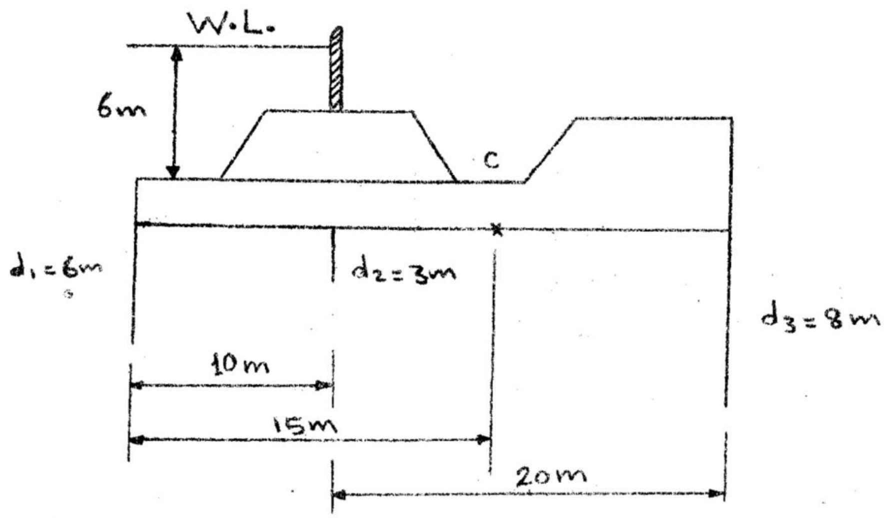
$L \geq c * H$ $41 + 0.333 \times = 48$ $\times = 21$

Use $\times = 37.04m$

Example 2.4

the following figure:

1) Compute the hydraulic gradient.
2) The thickness of floor at point C.



Solution

$$1) L = 6 + 6 + \frac{10}{3} + 3 + 3 + \frac{20}{3} + 8 + 8 = 44\ m$$

$$H = 6\ m$$

$$\frac{H}{L} = \frac{6}{44} = \frac{1}{7.3}$$

$$2) L' = 12 + \frac{10}{3} + 6 + \frac{5}{3} = 23\ m$$

$$h = 6 - \frac{6}{44} * 23 = 2.864\ m \quad ; t = \frac{2}{3} * 2.864 = 1.9\ m$$

2.5 KHOSLA'S THEORY

After studying a lot of dam failure constructed based on Bligh's theory, Khosla came out with the following;

Following are some of the main points from Khosla's Theory

- From observation of Siphons designed on Bligh's theory, by actual measurement of pressure, with the help of pipes inserted in the floor of two of the siphons.
- Does not show any relationship with pressure calculated on Bligh's theory. This led to the following provisional conclusions.
- Outer faces of end sheet piles were much more effective than the inner ones and the horizontal length of the floor.
- Intermediated piles of smaller length were ineffective except for local redistribution of pressure.
- Undermining of floor started from tail end.
- It was absolutely essential to have a reasonably deep vertical cut off at the downstream end to prevent undermining.
- Khosla and his associates took into account the flow pattern below the impermeable base of hydraulic structure to calculate uplift pressure and exit gradient.
- Starting with a simple case of horizontal flow with negligibly small thickness. (various cases were analyzed mathematically.)
- Seeping water below a hydraulic structure does not follow the bottom profile of the impervious floor as stated by Bligh but each particle traces its path along a series of streamlines.

Khosla's method considered a most complicated and accurate method to determine the uplift pressure under floor of hydraulic structure , but its results need to correct. The following formulas are used to determine the percent of uplift pressure at key point:

$$\%\Phi_E = 0.318 \cos^{-1}\left(\frac{n_1 - n_2 - 2}{n_1 + n_2}\right)$$

$$\%\Phi_D = 0.318 \cos^{-1}\left(\frac{n_1 - n_2}{n_1 + n_2}\right)$$

$$\%\Phi_C = 0.318 \cos^{-1}\left(\frac{n_1 - n_2 + 2}{n_1 + n_2}\right)$$

$$n_1 = \sqrt{1 + \left(\frac{b_1}{d}\right)^2} \qquad ; \qquad n_2 = \sqrt{1 + \left(\frac{b_2}{d}\right)^2}$$

But these values are not corrected and we must do the following correction:

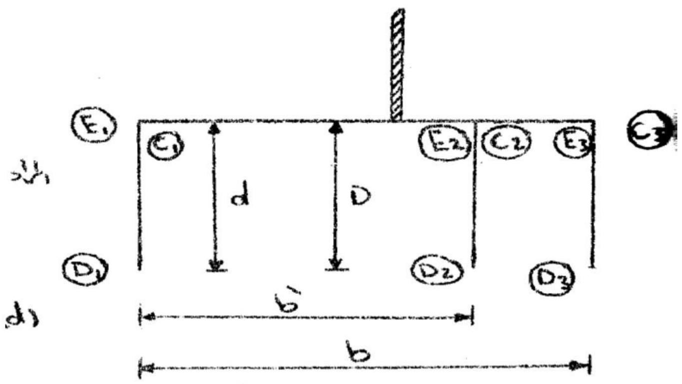
a) The correction for the mutual interference of piles.

b) Correction for the thickness of floor.

c) Correction for the slope of the floor.

A)The correction for the mutual interference of piles.

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



b': distance between 2 piles.

d: depth of pile.

D: depth of pile influence which has to be determine on the neighboring pile of depth (d).

B) Correction of floor thickness

U/S:-

$$C = (\frac{\Phi D_1 - \Phi C_1}{d_1})t$$

ΦD_1: uplift pressure at D_1 ; ΦC_1: uplift pressure at C_1

d_1: depth of sheet pile ; t: thickness of floor at U/S

Note: the correction is positive for point C always (For U/S & D/S).

D/S:

$$C = (\frac{\Phi E_2 - \Phi D_2}{d_2})t$$

d_2: depth of sheet pile at D/S.

Note

The correction is negative for point E always (For U/S & D/S)

C)Correction for slope

A correction is applied for slopping floor and take (positive for the down slope) & (Negative for the up slope)

Table 2.3

Slope	1:1	2:1	3:1	4:1	5:1	6:1	7:1	8:1
Correction $* \frac{b_s}{b}$	11.2	6.5	4.5	3.3	2.8	2.5	2.3	2.0

Where ;

b_s: horizontal length of slope.

b: Length of the floor between 2 piles.

2.5.1 Depth of cutoff

$$D/S = (1.25 - 1.5)R \quad , \qquad U/S = (1 - 1.25)R$$

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

$$f = \ silt\ factor = 1.76\,\sqrt{Dmm}$$

D: Diameter of particle of soil.

$$q = \frac{Q}{B}(m^3/sec/m)$$

2.5.1 Exit gradient

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

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} \qquad , \qquad \alpha = \frac{b}{d}$$

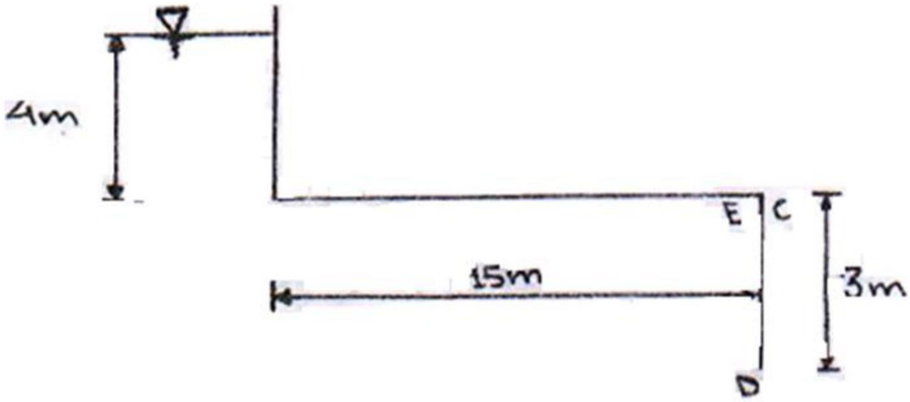
d= depth of cut-off in D/S; b: length of floor (horizontal).

Table 3.1

Type of soil	Shingle	Course sand	Fine sand
Safe exit gradient (GE)	$\frac{1}{4} - \frac{1}{5}$	$\frac{1}{5} - \frac{1}{6}$	$\frac{1}{6} - \frac{1}{7}$

Example 2.5

length of horizontal floor in alluvial soil 15m and 3m deep vertical sheet pile is attached at its downstream and the head of water is 4.0m. Find the thickness of the floor (using Khosla's theory)? Is the structure safe against the exit gradient?($F = 8, G_s = 2.45\ ton/m^3$)



Solution

$$\%\Phi E = 0.318 \cos^{-1}\left(\frac{n_1 - n_2 - 2}{n_1 - n_2}\right)$$

$$n_1 = \sqrt{1 + \left(\frac{15}{3}\right)^2} = 5.099 \quad ; \quad n_2 = 1$$

$$\therefore \%\Phi E = 0.39$$

u.p) $E = 0.39 * 4 = 15.6\ m$

$$t = \frac{1.56}{2.45 - 1} = 1.075 \simeq 1.1$$

$$\text{G.e} = \frac{\text{KH}}{\text{d}} \quad ; \quad \text{m} = \sqrt{1 + \left(\frac{15}{3}\right)^2} = 5.095$$

$$\text{K} = \frac{0.45}{\sqrt{1+\text{m}}} \Longrightarrow \text{K} = 0.1822 \Longrightarrow \text{Ge} = \frac{4 * 0.1822}{3} = 0.245$$

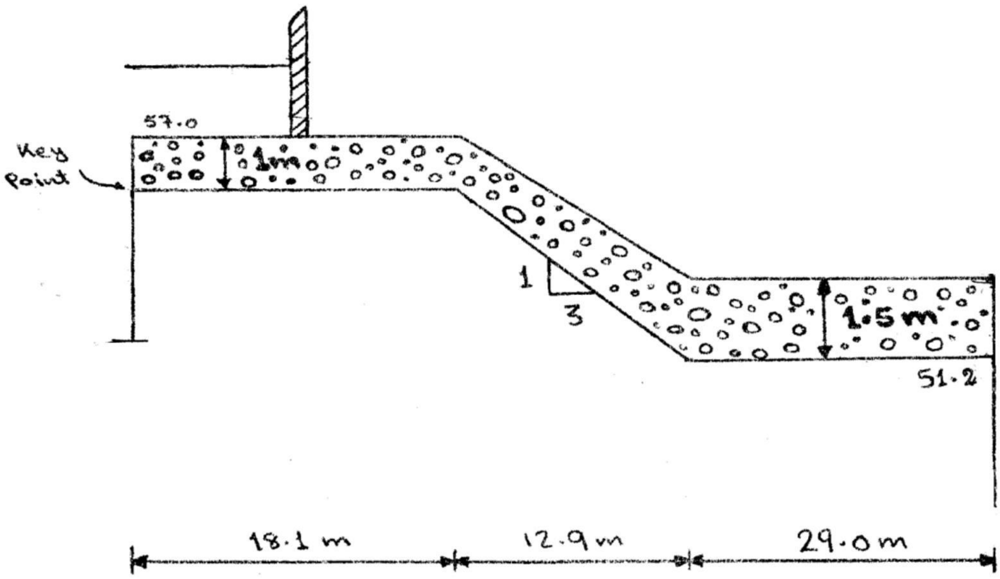
$F = \frac{1}{Ge} = \frac{1}{0.245} = 4.116 < 8$ ∴unsafe

Example 2.6

Given: $Q = 1980m^3/sec, water\ way = 85m\ U/S$ High $flood\ level = 63.2$, D/S High flood level=61.7m pond level=60.6m, $safe\ exit\ gradient = 1/6$, silt factor (f)=1.

Calculate:-

1) The depth of cut-off and elevation.
2) Check the floor length.
3) The percentage of pressure of key points.
4) Plot the hydraulic gradient for pond level U/S & no water D/S.



Solution

$$Q = \frac{1980m^3}{sec}\ , water\ way = 85m$$

$$q = \frac{1980}{85} = 23.3m^3/s.m$$

Depth of scour (R)=$1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35\left(\frac{(23.3)^2}{1}\right)^{\frac{1}{3}} = 11m$

$$D/S = 1.5R = 1.5 * 11 = 16.5m$$

$$U/S = 1.25R = 1.25 * 11 = 13.75m$$

D/S:- $Max.w.l. = 61.7m$

Bottom of sheet pile$= 61.7 - 16.5 = 45.2m$

U/S:- $63.2 - 13.75 = 49.45m \simeq 49.5m$

Floor length & Exit gradient:-

$Safe\ exit\ gradient = 1/6$

Max. static head @ pond level (all gates closed) No water @ D/S

$Ge = \frac{H}{d} * \frac{1}{\pi\sqrt{\lambda}}$ $\quad ,H = 60.6 - 52.7 = 7.9m \quad d = 52.7 - 45.2 = 7.5m$

$1/6 = \frac{7.9}{7.5} * \frac{1}{\pi\sqrt{\lambda}} \Longrightarrow \lambda = 4.107 \ , \ \lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \Longrightarrow \alpha = 7.14$

$b = \alpha . ds = 7.14 * 7.5 = 53.75m$

Uplift pressure:-

<u>U/S sheet pile (pile No. 1)</u>

$b = 60m \ ; \ d = 57 - 49.5 = 7.5m \quad , \quad \frac{1}{\alpha} = \frac{d}{b} = \frac{7.5}{60} = 0.125$

From the khosla curve:-

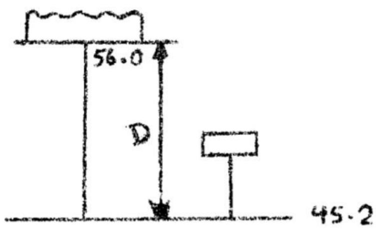
Find $\Phi E = 32\% \ ; \ \Phi c_1 = 100\% - 22\% - \ \Phi E = 100\% - 32\% = 68\%$

$\Phi D_1 = 100\% - \Phi D = 100\% - 22\% = 78\% \ ; \ \Phi E_1 = 100\%$

Correction for $ØC_1$:-

a) correction due to effect of sheet pile:

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



$$D = 56 - 45.2 = 10.8$$

$$d = 56 - 49.5 = 6.5$$

Correction $= 19\sqrt{\frac{10.8}{60}}\left(\frac{6.5+10.8}{60}\right) = 2.35\%$

since the point c_1 in the rear direction of flow,

hence its $+ve = 2.35\%$

b)correction for thickness of floor:

$$C = \left(\frac{\phi D_1 - \phi C_1}{d_1}\right)t_1 = \left(\frac{78\% - 68\%}{7.5}\right)*1 = 1.33\%$$

Since the pressure @ c_1,more than the pressure @ C_1^- have $+ve = +1.33\%$

c)correction for slope=0:

$$\phi C_1 corrected = 68\% + 2.35 + 1.33\% = 71.68\%$$

$(U.P)C_1 = 0.7168 * 7.9 = 5.66272 \Rightarrow 7.9 * 0.2958 = 2.33m$ Drop in H.G.L

Correction for $ØE_2$:

<u>(D/S)pile (pile No.2)</u>

$d = 52.7 - 45.2 = 7.5m$; $b = 60m$

$\frac{1}{\alpha} = \frac{d}{b} = \frac{7.5}{60} = 0.125$; from the khosla curve

$\phi C_2 = 0$, $\phi D_2 = \phi D = 22\%$, $\phi E = \phi E_2 = 32\%$

a) correction due to effect of pile:-

$D = 51.2 - 49.5 = 1.7m$; $d = 51.2 - 45.2 = 6m$

$b^- = b = 60m$

Correction $= 19\sqrt{\frac{1.7}{60}}\left(\frac{6+1.7}{60}\right) = 0.42\%$ $(-ve)$

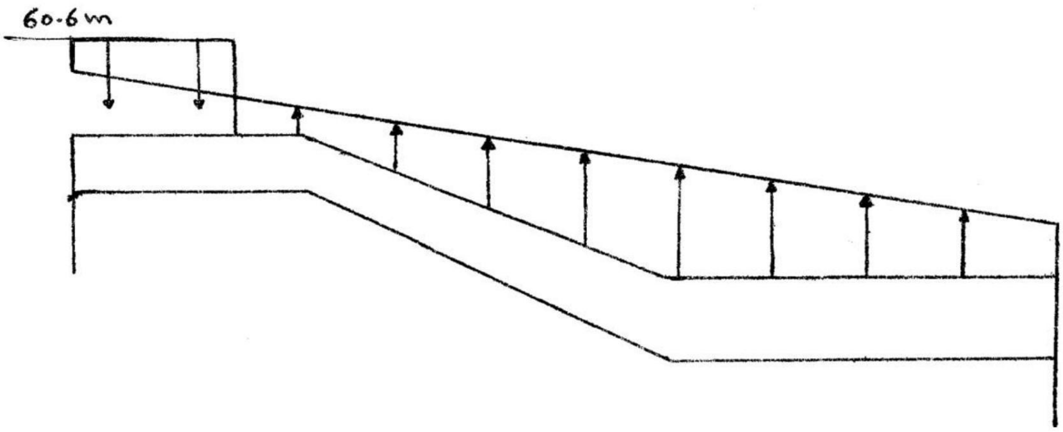
b) correction due to the thickness of floor:- $(t = 1.5m)$

$C = \left(\frac{\phi E_2 - \phi D_2}{d_2}\right)t_2$; $d_2 = 52.7 - 45.2 = 7.5m$

$$C = \left(\frac{32\% - 22\%}{7.5}\right) * 1.5 = 2\% \quad (-ve)$$

c) correction for slope of floor=0 :-

ϕE_2 corrected $= 32\% - 2\% - 0.42\% = 29.58\%$



$7.9 * 29.58\% = 2.33 =$ Drop in H.G.L

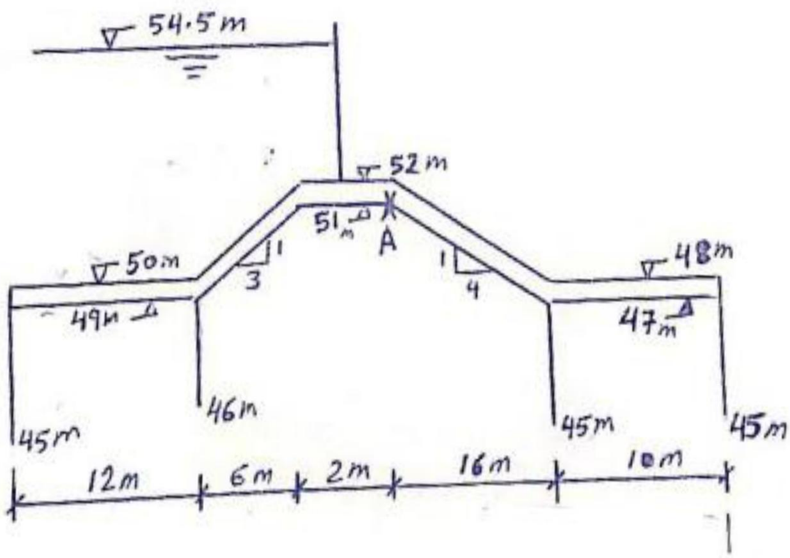
$100 - 71.68 = 28.32 * 7.9 = 2.23m$

Or:

$0.7168 * 7.9 = 5.66m \Rightarrow 7.9 - 5.66 = 2.23m$

Example 2.7

Check the exit gradient and thickness of floor at point "A" of the hydraulic structure as shown in fig. below by using khoslas theory and take (F=5)



Solution

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

H=54.5-48=6.5m,

d=48-45=3m

$$\lambda = \frac{\mathbf{1+\sqrt{1+\alpha 2}}}{\mathbf{2}} = \frac{\mathbf{1+\sqrt{1+\left(\frac{46}{3}\right)^2}}}{\mathbf{2}} = 8.183$$

$$Ge = \frac{6.5}{3 * 3.14 * \sqrt{8.183}} = 0.241$$

$F=\frac{1}{Ge} = 4.15 < 5 \ \ not\ ok$

Sheet pile no.2

$n_1=\sqrt{1+\left(\frac{12}{4}\right)^2} = 3.162, \quad n_2=\sqrt{1+\left(\frac{34}{4}\right)^2} = 8.558$

$D2=0.318\cos^{-1}\left(\frac{3.162-8.558}{3.162+8.558}\right) * 100\% = 65.17\%$

$C2=0.318\cos^{-1}\left(\frac{3.162-8.558+2}{3.162+8.558}\right) * 100\% = 59.3\%$

a)$\%c=19\sqrt{4/24} * \left(\frac{3+4}{46}\right)\% = 1.18\%$

b)$\%c=\frac{65.17\%-59.3\%}{4} = 1.47\%$

c)%c=(6/24)*4.5%=1.125%

C2correction=59.3%+1.18%+1.47%-1.125%=60.825%

Sheet pile no.3

$n_1=\sqrt{1+\left(\frac{36}{3}\right)^2} = 12.042, \quad n_2=\sqrt{1+\left(\frac{10}{3}\right)^2} = 3.48$

$D3=0.318\cos^{-1}\left(\frac{12.042-3.48}{12.042+3.48}\right) * 100\% = 31.37\%$ Φ

E3=$0.318\cos^{-1}\left(\frac{12.042-3.48-2}{12.042+3.48}\right) * 100\% = 36.04\%$ Φ

a-%c=$19\sqrt{1/24} * \left(\frac{1+2}{46}\right)\% = 0.253\%$

b-%c=$\frac{36.04\%-31.37\%}{3} * 1 = 1.567\%$

c-%c=(16/24)*3.3%=2.2%

E3correction=36.07-0.253%-1.567%+2.2%=36.45%

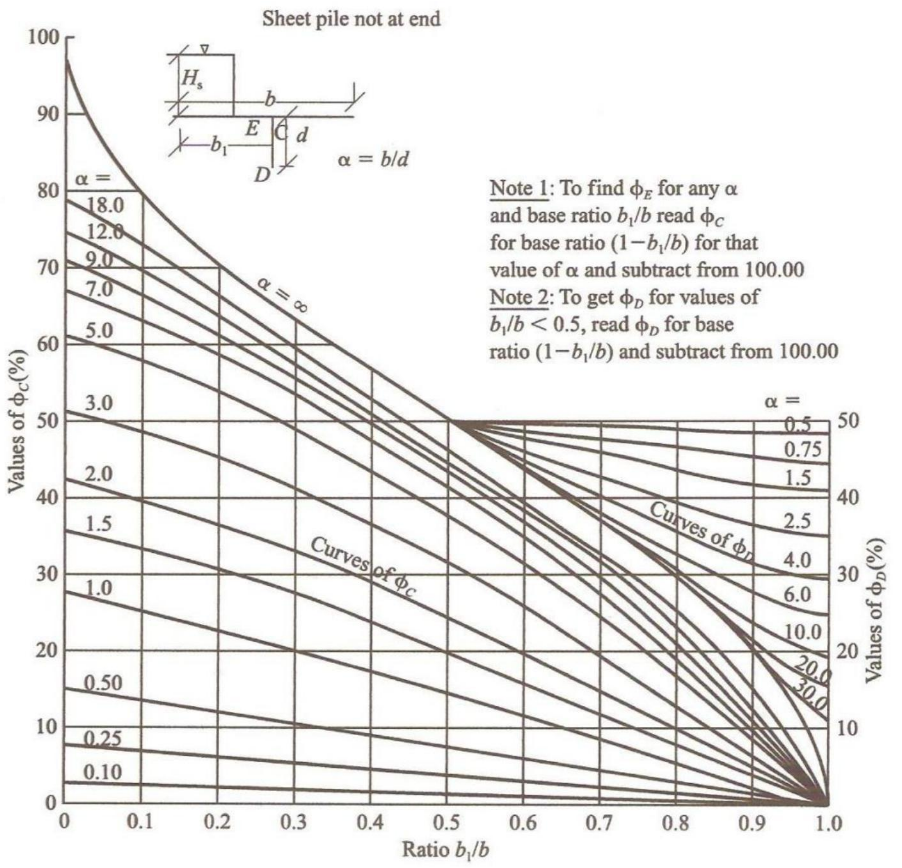
$\frac{3.954-2.37}{24} = \frac{Y}{16}$ by interpolation

Y=1.056m

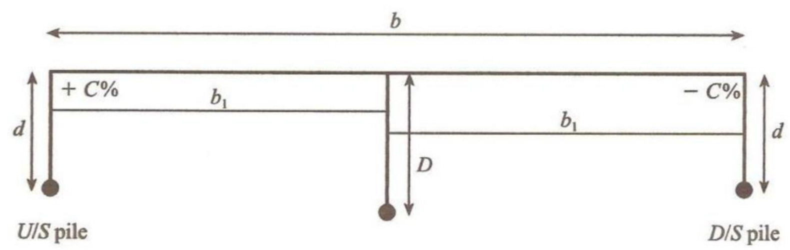
Uplift pressure at A=1.056+2.37=3.426

thickness at A=2/3 h=2.28>1 not ok.

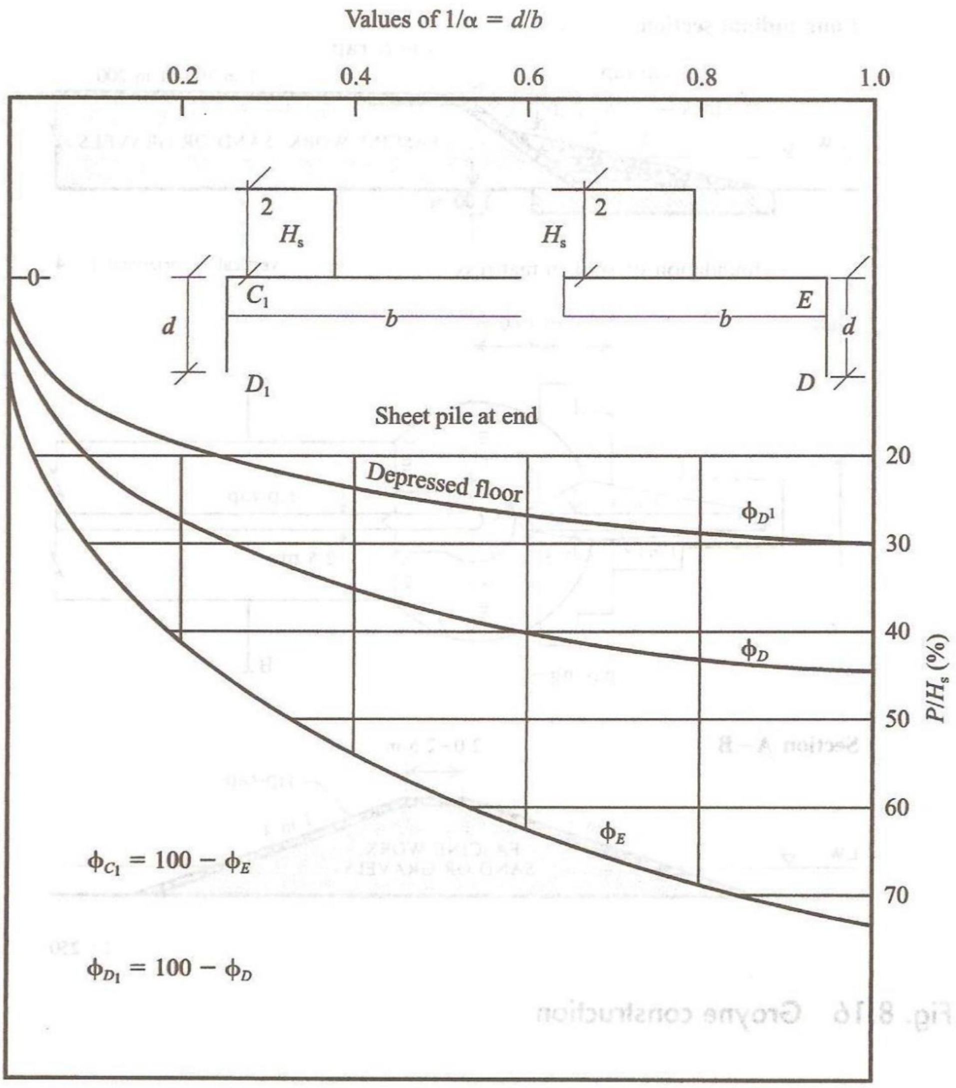
2.5.3 Khosla's Curves



Khosla's pressure chart with intermediate pile



Mutual interference of piles

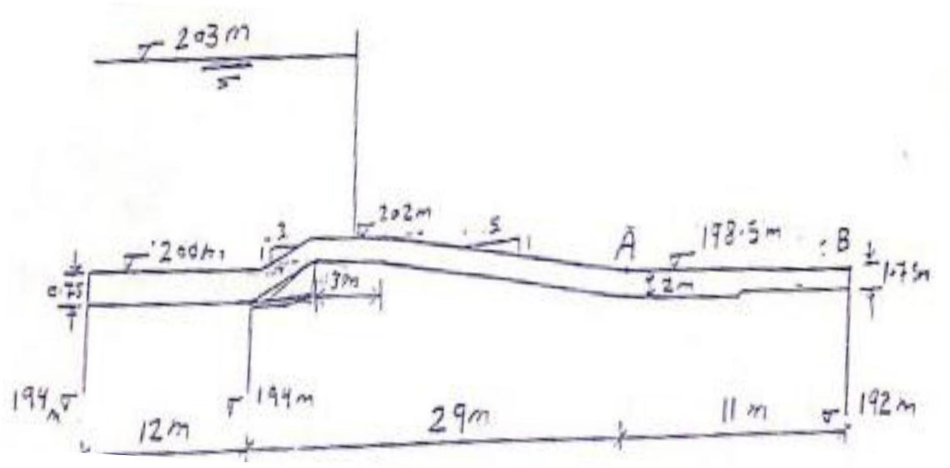


Khosla's pressure chart with end pile

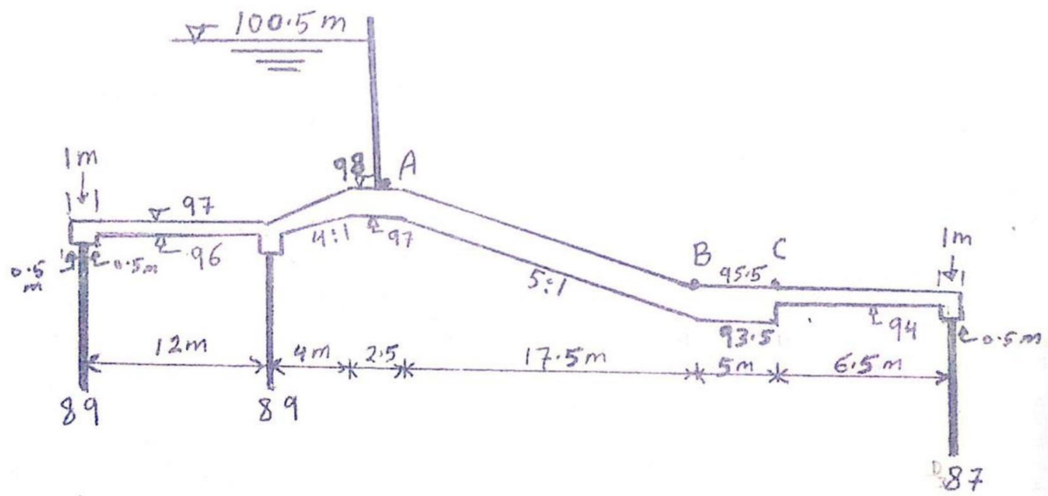
Chapter 2 Seepage under hydraulic structures

Problems

2.1. The figure of the section of a hydraulic structure.Determine whether the section provided is safe against uplift pressure at point A and B the founded of structure is fine sand with permissible exit gradient 1/6.



2.2. Figure shows the sketch of barrage . The various dimensions and reduced levels are shown in the figure itself . Find out the uplift pressure at important key points (put the final results in a table), also check the thickness at points A , B & C.



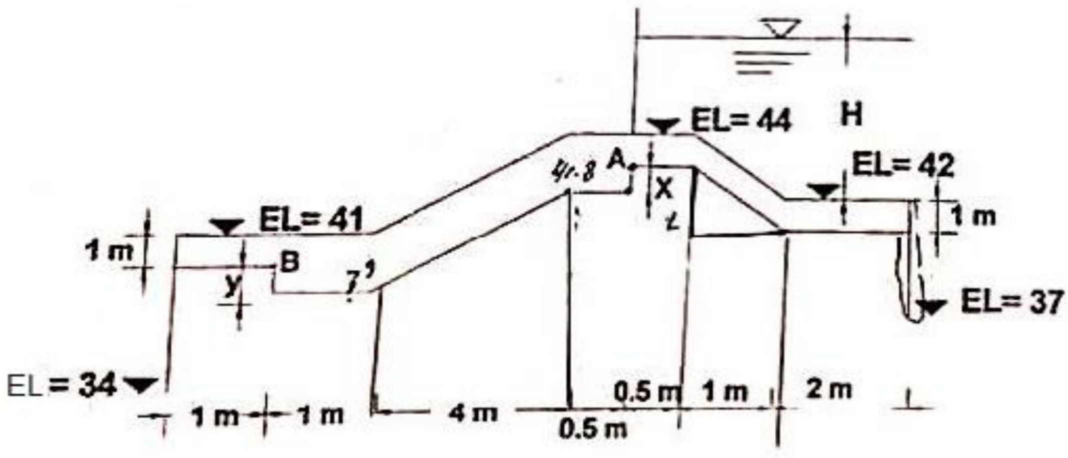
2.3. A three reach of a floor with total horizontal length 30m, if the first reach at a straight line with a horizontal length 8m and the second reach at slope 1:4 with horizontal length 12m, and the last reach is a straight line, 2m deep vertical cut off is attached at its upstream end and other at downstream end. The head of water 3m when closed the gate and 2m when opened. Find the thickness of the floor of the structure using Khosla theory if the specific density of floor is 2.24.

2.4. Find the uplift pressure and also check the exit gradient of a horizontal floor 30m length is attached with 4m of cut off in upstream and downstream ends, the head on the structure is 3.5m, the thickness of floor is 1.5m, solve the problem in the south of Iraqi soil using Lane method. (Put the final results in a table).

2.5. For the following hydraulic structure lies in Iraqi soil as shown below; find:

a) the maximum values of X&Y to checking the seepage only, if the H equal to 3m and ($t_A = 1.1t_B$)

b) The maximum value of H if the X=1.2m and y=1m by Bligh method.



Chapter Three

Control Structures

Chapter 3 Control Structures

3.1 Introduction

The water which enters into the main canal from the river has to be divided into different Branches and Distributaries , in accordance with the relative urgency of demand on different channels. This process of distribution is called 'Regulation'. To distribute water effectively , the discharge has to be adjusted to any value. This aim is achieved by means of regulators such as ;Barrages and Regulator.

3.2 Regulators

Cross Regulator:-

Is a structure constructed across any canal (a main canal , a branch canal). Its regulators the quantity of water supplied and also the level of water on the U/S & D/S side.

Head Regulator:-

Is a structure constructed at the head of an off-take channel, following are the functions of head regulator:-

a) For regulating the quantity of water passing into the canal.

b) For controlling the amount of silt interning the cannel.

c) For shutting out the river floods.

3.2.1 Hydraulic design of regulators

1)Design of approach channel . Fix the water level just U/S of the gate of regulator.

2)Calculate the water way width of the gated section consider the gate fully open (max.discharge). Use the eq. :-

$$Q = Cd.S.yt\sqrt{2gH}$$

OR $Q = Cd.A\sqrt{2gH}$ (A=S.yt)

Where:- Q: Design discharge ; A: Area of water way

S: Net width of regulator opening (width of gate) yt: Tail water depth above the sill level

Cd: Discharge constant = 0.82 (for square opening)

= 0.92 (for round opening)

$$H = h + ha$$
$$h = \left(U/_S\right)W.L - \left(D/_S\right)W.L$$

$$ha = \frac{v^2}{2g}$$ (velocity head)

3)Discharge-head relationship, there are two cases:-

a)Free flow:-

w: high opening

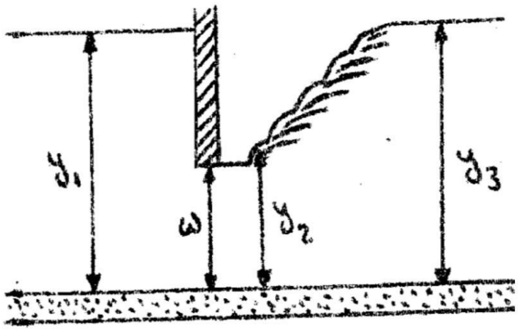
-Free flow condition under

Vertical gate occurs when

The tail water (yt) is sufficient

Low to allow hyd. jump to

occur down stream of the gate.

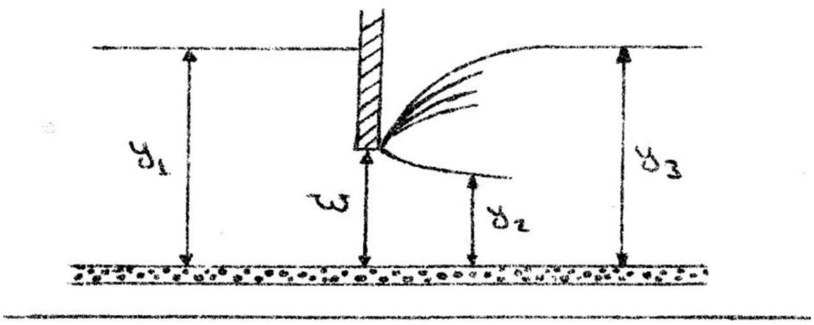


-The discharge given by:-

$$Q = Cd.S.W.\sqrt{2gH} \quad ; \quad Cd = \frac{0.61}{\left(1 + 0.61\frac{W}{y_1}\right)^{\frac{1}{2}}}$$

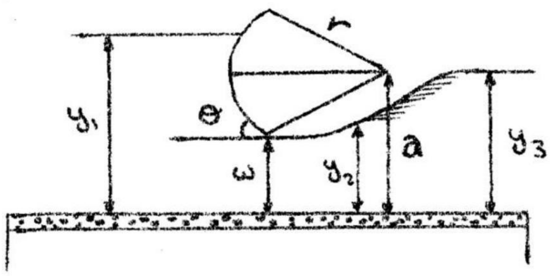
b)Submerged flow

When the tail water (yt) rises , the jump and flow is submerged and this effect the discharge coefficient (d) , easy method adopted is to use the curve which is plotted from energy and momentum



Radial gate:-

a) Free flow



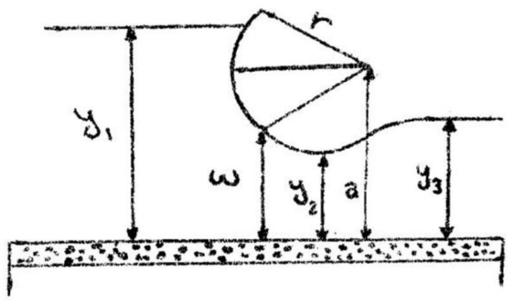
$$\sin\theta = (a-w)r$$

$$Cd = \frac{Cc}{\left(1 + Cc\frac{w}{y_1}\right)^{\frac{1}{2}}}$$

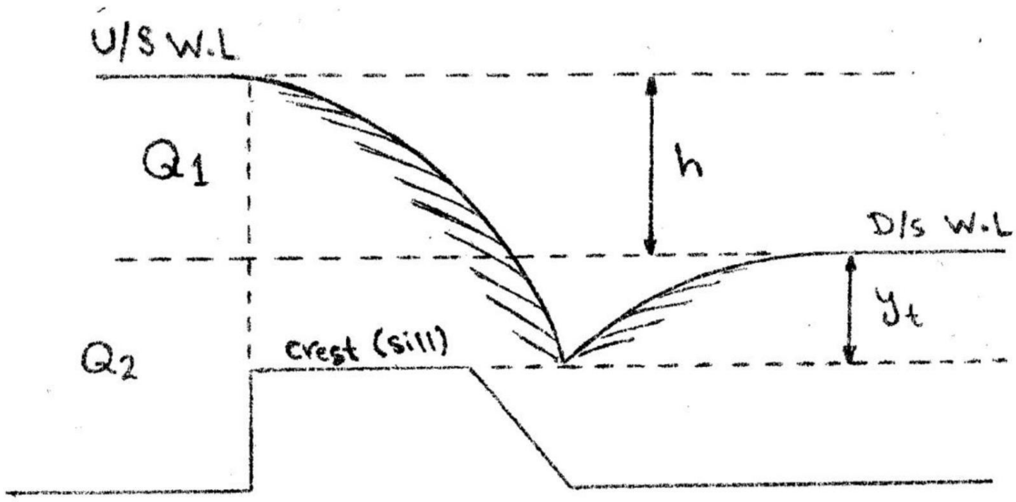
$$Cc = 1 - 0.75\left(\frac{\theta}{90^o}\right) + 0.36\left(\frac{\theta}{90^o}\right)^2$$

b) Submerged flow

when submerged flow occurs , calculate y_1 , w , Cd , use the curve developed by solution of energy & momentum eq. (9-11 Curve)



Drowned Weir:-



$$Q_1 = \frac{2}{3}C_1.\sqrt{2g}.S\left[(h+hv)^{\frac{3}{2}} - hv^{\frac{3}{2}}\right]$$

$$Q_2 = C_2.S.yt.\sqrt{2g(h+hv)}$$

$$Q = Q_1 + Q_2$$

yt: depth of water head above the crest.

hv: head due to velocity (velocity head $hv = \frac{v^2}{2g}$

$$h = \left(U/_S\right)W.L - \left(D/_S\right)W.L$$

$C_1 = 0.577$

$C_2 = 0.8$

$$Q_1 = \frac{2}{3} C_1 . \sqrt{2g} . S \left[(h + hv)^{\frac{3}{2}} - hv^{\frac{3}{2}} \right]$$

$$Q_2 = C_2 . S . yt . \sqrt{2g(h + hv)}$$

$$Q = Q_1 + Q_2$$

yt: depth of water head above the crest.

hv: head due to velocity (velocity head $hv = \frac{v^2}{2g}$

$$h = \left(U/_S\right)W.L - \left(D/_S\right)W.L$$

$C_1 = 0.577$

$C_2 = 0.8$

Example 3.1

Given head regulator of following data , discharge = $35 \frac{m^3}{sec}$ U/S water level = 29.53 m U/S bead level =26.35 m , D/S water level =29.28 m, D/S bead level =26.35 m , length from U/S of floor = 20 m , length from D/S = 60m :-

1- Find the clear water way of the regulator if cd = 0.92
2- If the flow under the gate is submersed flow and the water way is 6m ; find the height of the gate cd =0.92m
3- If we used sill under the gate weir crest level 0.6m above the bed level of the cannel find the clear water way (s) of the regulator using downed way neglect velocity head [$c_1 = 0.577$, $c_2 = 0.8$]

Solution

1-

$Q = 35 = cd * s * y_t * \sqrt{4gH}$

$y_t = 29.28 - 26.35 = 2.93\ m$

$\text{h} = 29.53 - 29.28 = 0.25\ m$

$s = 5.86\ m$

2-

$35 = 0.92 * 6 * w\ \sqrt{19.6 * 0.25}$

$w = 2.86\ m$

3-

$Q + Q_1 + Q_2$

$$Q_1 = \frac{2}{3}\ c_1\sqrt{2g}\ s\left[H^{\frac{3}{2}} - h_a{}^{\frac{3}{2}}\right]$$

$Q = c_2 * s * y_t * \sqrt{2gH}$

$y_t = 29.28 - 26.35 - 0.6$

$$35 = \frac{2}{3} * (0.577)(s) * \sqrt{196} * h^{\frac{3}{2}} + (0.8)(s)(2.33)\left(\sqrt{19.6 * 0.25}\right)$$

$s = 8\ m$

Example 3.2

Aboard crested is constructed across channel if which the discharge is (30cumces) the cross section Area of the channel is ($35m^2$) if the crest of the weir is 2m below u\s water level . find the height of crest . take the discharge coff. = 0.97

Solution

$Q = cd * A * \sqrt{2 * 9.81 * H}$

$30 = 0.97 * 35 * \sqrt{2 * 9.81 * H}$ $H = 0.04m$

$v\frac{30}{35} = 0.86\ m\backslash sec$ $ha = \frac{(0.86)^2}{19.6} = 0.038m$

$H = h + ha$ $h = 0.04 - 0.038 = 0.002m$

$yt = 2 - h$ $yt = 1.99m$

$\mathrm{Qt} = \mathrm{Q1} + Q2$

$Q1 = \frac{2}{3}(0.577)\sqrt{19.6}\ \ (S)(0.002)^{\frac{3}{2}}$

$Q1 = 0.000152 * S$

$Q2 = 0.8 * S * 1.99\sqrt{19.6 * 0.04}$ $\mathrm{S} = 21.58\mathrm{m}$

$A = S * height\ of\ crest$

$Height\ of\ crest = 1.63m$

Example 3.3

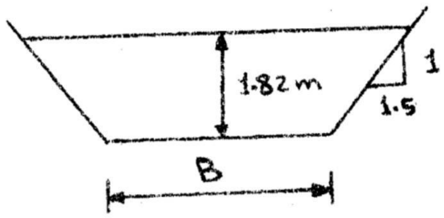
Design a main canal head Regulator to draw water from river through approach channel . Data regarding the river and main canal are as follows :-

River Data
Low discharge Q = 46m^3/sec
Water level in the rive r = 42.7m
Bed level in the river = 40.88m
Depth of flow in the river = 1.82m
Flood discharge Q = 536m^3/sec
Depth of flow = 5.1m
Flood level = 44m

Main Canal Data
Peak discharge =10m^3/sec
Water level in main canal = 42.6m
bed level = 40.38m
depth of water = 2.22m
length (approach channel) =100m
slope = 8.7 cm/km

Solution

1)Design of approach channel



Assume B = 5m

A = (5+1.5*1.82)*1.82=14.068 m^3

P = 11.562m

R = 1.217m

Consider the approach bed level in a canal equal to the bed level of river.

$$Q=\frac{1}{0.015}*14.068(1.217)^{2/3}*(0.000087)^{1/2}$$
$$Q=9.97\approx 10\ m^3/\sec \Rightarrow B=5m$$

2)Water level at the end of approach channel (Near gate)

$$42.7-\frac{0.087}{1000}*100=42.69m$$

Bed level = 42.69 – 1.82 = 40.87m

3)Water way (S)

$$Q=Cd*S*yt*\sqrt{2gh}$$
$$yt=42.6-40.87=1.73m$$
$$h=U/S\,W.L-D/S\,W.L=42.69-42.6=0.09m$$
$$ha=\frac{V^2}{2g}=\frac{(0.71)^2}{2*9.81}=0.026$$
$$H=h+\frac{V^2}{2g}=0.116m$$
$$10=(0.82)S(1.73)(2*9.81*0.116)^{1/2} \qquad \left(V=\frac{Q}{A}\right)$$
$$S=4.8m$$

Use 2 opening [use 2 gates which width of each gate = 2.4m]

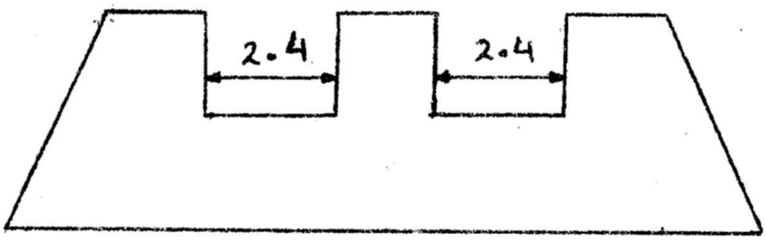
4)Discharge head Relation ship:

y_1 = head water depth = 44 – 40.87 = 3.13 m (in river)

y_2 = Tail water depth = 42.6 – 40.87 = 1.37 (in canal)

$$q=\frac{10}{4.8}=2.088\ m^3/\sec.m$$

$$\frac{q^2}{y_1^3}=\frac{(2.088)^2}{(3.13)^3}=0.14$$
$$\frac{yt}{y_1}=\frac{1.73}{3.13}=0.55$$



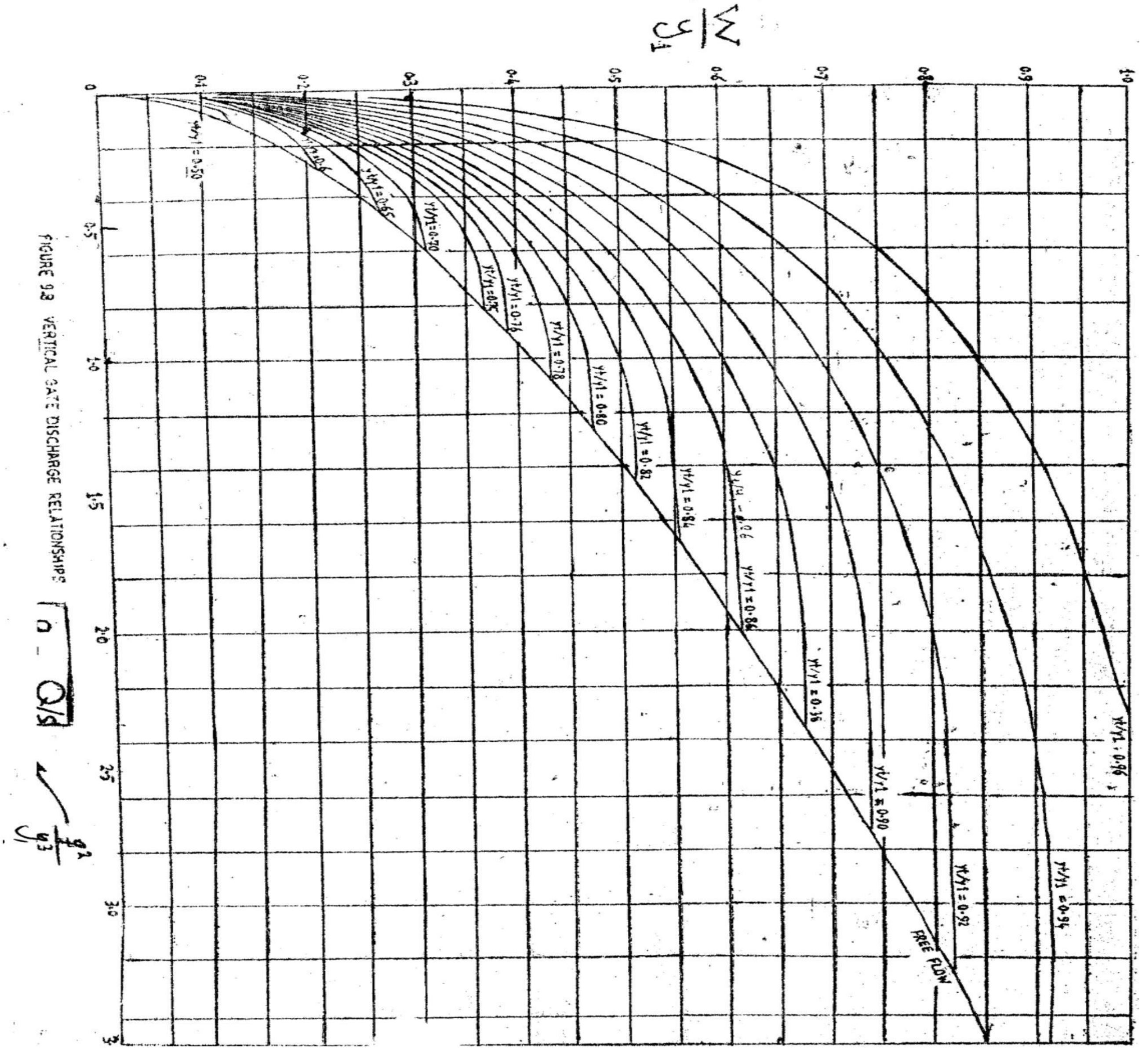
$$Cd = \frac{0.61}{\left(1+0.61\frac{w}{y_1}\right)^{1/2}} = \frac{0.61}{(1+0.61*0.15)^{1/2}} = 0.584$$

$$y_2 = Cc * w = 0.61*0.47 = 0.287$$

$$\frac{y_3}{y_2} = 0.5\left[\left(1+8Fr_2^{\,2}\right)^{1/2} - 1\right]$$

$$Fr_2^{\,2} = \frac{v^2}{gy_2} = \left(\frac{q^2}{gy_2^{\,3}}\right)^{1/2} = \left[\frac{(2.088)^2}{9.81*(0.287)^3}\right]^{1/2} = 4.32$$

$$\therefore y_3 = 1.62m$$



Chapter 3 Control Structures

Example 3.4

Four Opening of ahead regulator 3m for each opening. If the vertical opening of the gate is 1.0m and the water flow between the upper and lower gates , The head on the regulator is 0.5m . If the U/S W.L rises by 0.2m , How much the upper gate must be lowered to maintain the discharge unaltered ($cd = 0.62$)?

Solution

$$Q = Cd * S * W * \sqrt{2gy_1}$$

$$= 0.62 * (4 * 3) * 1 * \sqrt{2 * 9.8 * 0.5} = 23.29 \frac{m^3}{\sec}$$

$$Q = 23.29 = 0.6 * 12 * W * \sqrt{2 * 9.8 * 0.7} \rightarrow (0.5 + 0.2)$$

$$W = 0.845m$$

The gate must be lowered by $\Rightarrow 1 - 0.845 = 0.155m$

Example 3.5

Design suitable head regulator for the following data:

Full supply discharge of the canal =180 m^3/sec

D/S max. full supply level of canal = 60.2 m

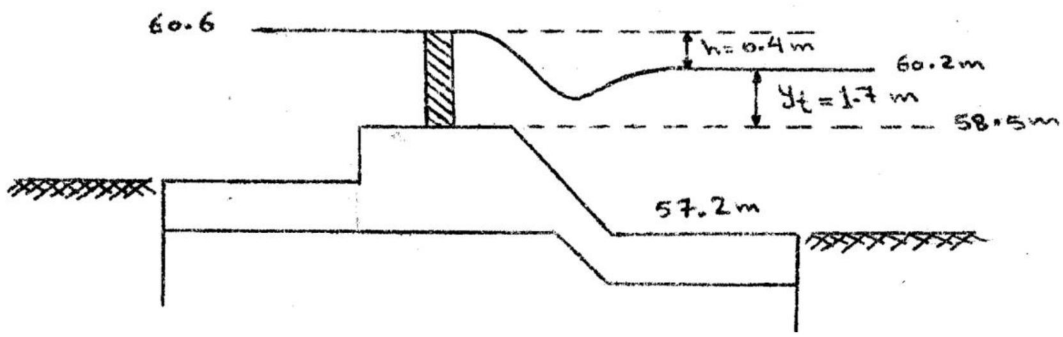
U/S max. supply level of canal = 63.2 m

Bed level of the canal = 57.2 m

Pond level = 60.6 m

Crest level of regulator = 58.5 m

Neglect the velocity head , Determine the water way width assume cd1=0.577,cd2=0.8



Solution

$$Q = \frac{2}{3} . C_1 . \sqrt{2g} . S\left[(h + hv)^{3/2} - hv \right] + C_2 . S . y . \sqrt{2g(h + hv)}$$

$$Q = \frac{2}{3} . C_1 . \sqrt{2g} . S . (h)^{3/2} + C_2 . S . yt . \sqrt{2gh}$$

$$180 = \frac{2}{3} * 0.577 * \sqrt{2g} * S * (0.4)^{3/2} + 0.8 * S * (1.7) \sqrt{2 * 9.81 * 0.4}$$

$$S = \frac{180}{4.32} = 42.5m$$

Chapter 3 Control Structures

Use S = 45m

-Full supply discharge passing down during high flood when U/S W.L 63.2(high flood).

-Water shall pass over the regulator and gated opening provided between the still level and pond level shall be a gusted by partial opening.

Let the gate opening = w

$Q = Cd.A\sqrt{2g}$;Cd = 0.62

h = 63.2 – 60.2 = 3m

$$180 = 0.62 * (42.5 * w) * \sqrt{2 * 9.81 * 3} \Rightarrow w = 0.89m$$

Pond level = 60.2m

h = 60.6 – 60.2 = 0.4m

$$180 = 0.62 * (42.5 * w) * \sqrt{2 * 9.81 * 0.4} \Rightarrow w = 2.44m$$

Use w = 2.44m

Chapter 3 | Control Structures

Problems

3.1. A cross regulator, with the following data

Find the water way width of regulator.

Branch canal	Main canal
Max.discharge=$6m^3/sec$	Low discharge=$13m^3/sec$
Water level=19.8m	Water level=20m
Bed level=17.8m	Bed level=18.1m
Depth of flow=2m	Depth of flow=1.5m
Length of approach canal=80m	Flood discharge=$22m^3/sec$
Slope=10cm/km	

3.2. A distributary head regulator for a rectangular canal which take off water from main canal with the following data:

Main canal	Distributary canal
Q=$22m^3/sec$	Max.discharge=$3m^3/sec$
Bed level=49.51m	Bed level=50.6m
Water level=51.9m	Water level=51.8m
Depth of flow=2.39m	Depth of water=1.2m
Flood level=52.14m	Bed width=2m
Bed width=5m	

If the silt factor is 1,the horizontal length of floor=13.2m,U/S and D/S ends with 1.2m depth of cut off.

a) Find the clear water way of distributary regulator (cd=0.82)

b) Check the depth of D/S cut off.

Chapter 3 | Control Structures

3.3A cross regulator is to be constructed on a two opening with clear width=2.5m to each open with the following data:

U/S and D/S width =10m , downstream side slope 1:1 ,longitudinal slope 10.5cm/km , n=0.023,U/S and D/S bed level=32m,downstream water level=36m,cd=0.92,for fully opening conditions.

Calculate the upstream water level of the regulator.

Neglect velocity of approach>

Chapter Four

Energy Dissipated Structures

Chapter 4 Energy dissipated structures

4.1 Introduction

The flow velocity at the toe of a high-head spillway is usually high and may cause serious scour and erosion of the downstream channel if and proper precautions are not taken. For this purpose, energy dissipators are provided to dissipate sufficient amount of energy before water enters the downstream channel. The energy dissipators structures such as; Stilling Basin Drop structures.

4.2 Hydraulic Jump

A hydraulic jump is formed whenever flow changes from supercritical to subcritical flow. In this transition from the supercritical to subcritical flow, water surface rises abruptly, surface rollers are formed, intense mixing occurs, air is entrained, and usually a large amount of energy is dissipated. By utilizing these characteristics, a hydraulic jump maybe used to dissipate energy, to mix chemicals, or to act as an aeration devise.

Types of Jump:-

The jump can be classified according to Froud No. :-

1-Fr =1.0 ⇒ CRITICAL ⇒ No. Jump

2- Fr = 1.0 – 1.7 ⇒ UNDULAR JUMP

3- Fr = 1.7 – 2.5 ⇒ WEAK JUMP

4- Fr = 2.5 – 4.5 ⇒ OSCILLATING JUMP

(Loss of energy in this jump equal (17%-45%)

5- Fr = 4.5 – 9.0 ⇒ STEADY JUMP

(Loss of energy in this jump equal (45%-70%)

6- Fr ≥ 9.0 ⇒ STRONG JUMP

(Loss of energy in this jump equal (85% (Min.))

Chapter 4 | Energy dissipated structures

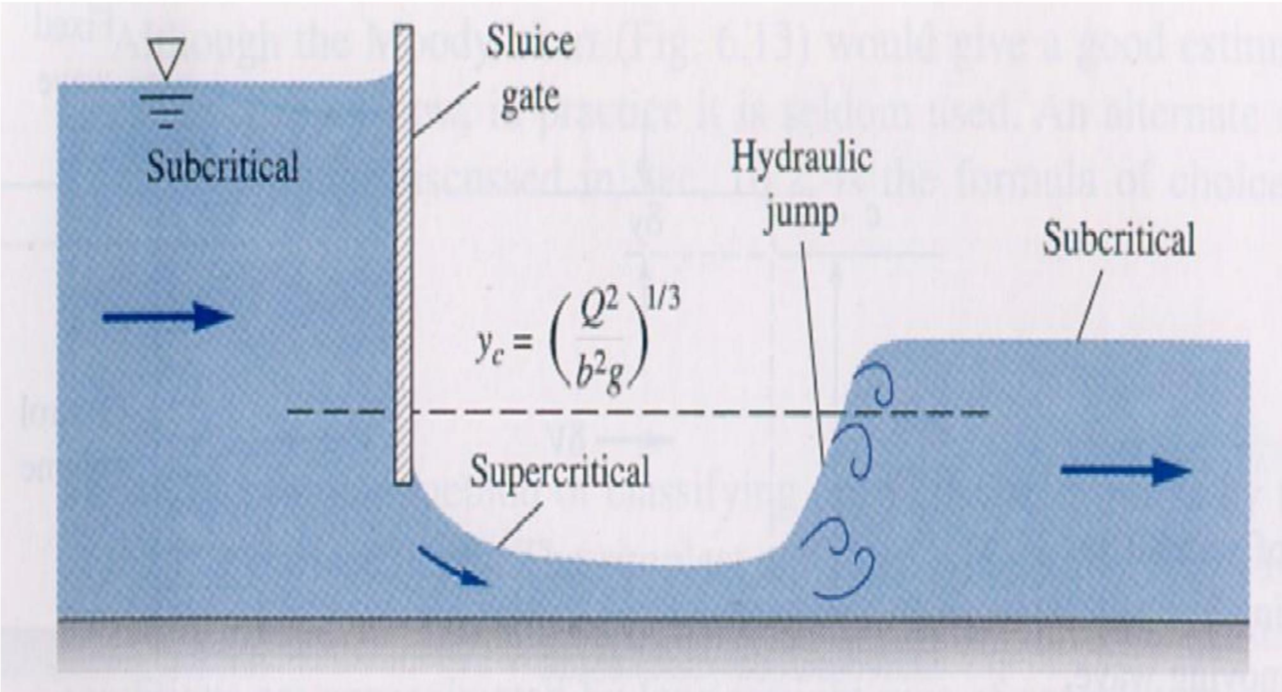


Fig.4.1 Hydraulic Jump

Chapter 4 | Energy dissipated structures

Hydraulic Jump with modification:-

Board crested weir or sharp crested weir can be used as a control for the location of hydraulic jump , if the D/S depth in **the canal (yt)** is larger than the sequent depth (conjugate) (y_2) , it is required to control the jump by drop in the basin floor.

When the D/S depth (yt) is smaller then the sequent depth (y_2) , the jump can control by rise .

Efficiency of Hydraulic Jump(η):-

$$\eta = \frac{E_2}{E_1} = \frac{E_1 - \Delta E}{E_1}$$

$$\eta = \frac{\left(1+8Fr_1^{\,2}\right)^{3/2} - 4Fr_1^{\,2} + 1}{8Fr_1^{\,2}\left(2+Fr_1^{\,2}\right)}$$

$$\mathrm{E}_1 = y_1 + \frac{v_1^{\,2}}{2g}$$ (specific energy before jump)

$$\mathrm{E}_2 = y_2 + \frac{v_2^{\,2}}{2g}$$ (specific energy after jump)

Example 4.1

If a sluice gate is opened , water is sues with a velocity of $6\frac{m}{\sec}$ and depth of 0.5m at the Venacontracta. Determine whether a hydraulic jump (H.j) will form or not. If so calculate the energy dissipated?

Solution

$$yc = \sqrt[3]{\frac{q^2}{g}}$$

$$q = V * y \Rightarrow q = 6 * 0.5 = 3 \, \frac{m^3}{\sec} \Big/ m$$

$yc = 0.972m \succ y_1$ (the jump will form)

$$\frac{y2}{y1} = 0.5\left[\sqrt{1+8Fr_1^2} - 1\right] \Rightarrow y_2 = 1.675m$$

$$\Delta E = 0.492 \frac{Kg.m}{Kg}$$

Example 4.2

Along rectangular channel is 3m wide & carries water at the rate of 13m^3/sec. At a certain point there is an approach change in the slope from 1:60 to 1:600 , determine: a)whether the hyd. Jump will form?

Solution

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

$$13 = \frac{1}{0.015}(3y_1)(\frac{3y_1}{3+2y_1})^{2/3}(\frac{1}{60})^{1/2}$$

$$y_1 = 0.79\ m$$

$$13 = \frac{1}{0.015}(3y_2)(\frac{3y_1}{3+2y_2})^{2/3}(\frac{1}{600})^{1/2}$$

$$y_2 = 1.82m$$

$$y_c = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{\left(13/3\right)^2}{9.81}} = 1.24\ m$$

$$y_1 < y_c < y_2 \quad \textbf{The jump accure}$$

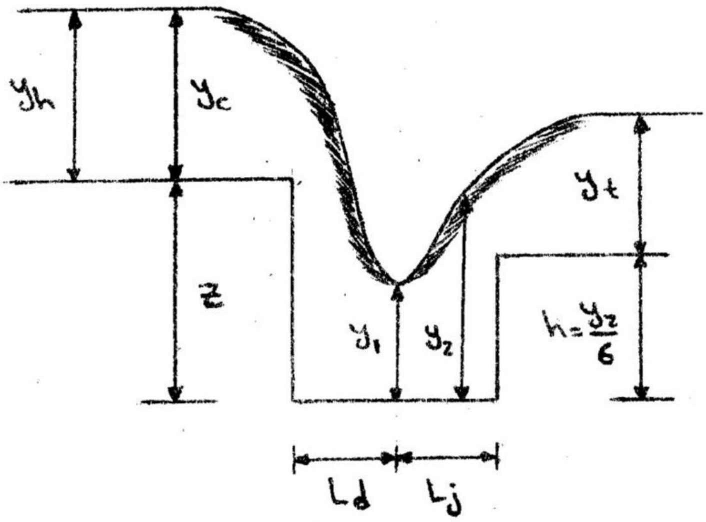
$$A = 3 * 0.79 = 2.37$$

$$\frac{y_2}{y_1} = 0.5\ [\sqrt{1+8\,(f_{r1})^2} - 1]$$

$$f_{r1} = \frac{5.48}{\sqrt{9.81 * 0.79}} = 1.976$$

H.W/Find The location of the jump if formed ,use manning coe. = 0.015

4.3 Vertical Drop



Drop number (D)= $\left(\frac{y_c}{z}\right)^3$

$L_d = 4.3\ ZD^{0.27}$

$L_j = 6.9(y_2 - y_1)$

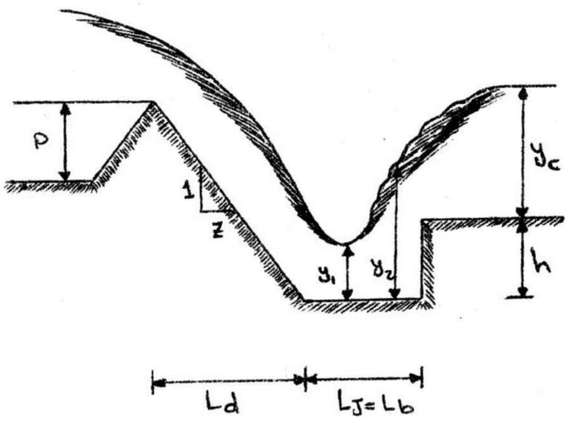
$y_1 = 0.54\ ZD^{0.425}$

$y_2 = 1.66\ ZD^{0.27}$

Note

when natural slope are steep , use
drop (Z)=1m depth in unlined canal & use 2m drop depth in lined canal .

4.4 Chutes



$$\frac{y_2}{y_1} = 0.5\left[\left(1 + 8Fr_1^{\,2}\right)^{1/2} - 1\right]$$

$$Fr^2 = \frac{q^2}{gy_1^{\,3}}$$

$$He = \Delta E = \frac{(y_1 - y_2)^3}{4y_1y_2}$$

$hj = y_2 - y_1$; Lj = length of the jump

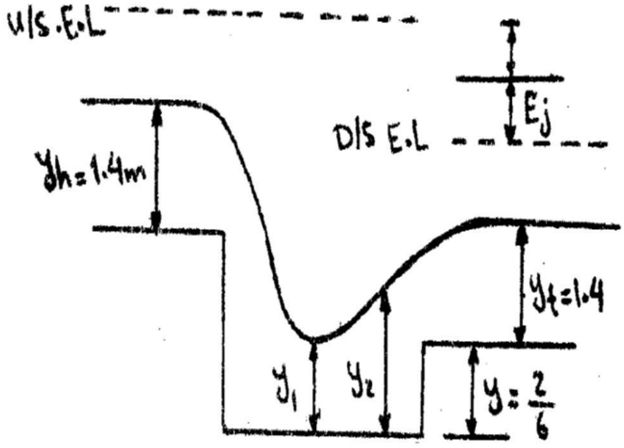
Lb = length of basin

Example 4.3

Design a vertical drop structure in a line canal carrying a normal discharge of $5m^3/sec$ to lower the water level from U/S to D/S by 2m , the canal sections in the reaches U/S & D/S of structure are the same & as given below:

1- Normal discharge Q = $5m^3/sec$
2- Depth of flow (y) = 1.4m
3- Bed width = 2.4m
4- Longitudinal slope = 17.7 cm/km
5- Side slope = 1.5:1
6- Manning(n) = 0.01

U/S & D/S depth of flow are the same



Solution

U/S W.L – *D/S* W.L = 2m

U/S Energy line – *D/S* Energy line = 2m

U/S B.L – *D/S* B.L = 2m

Drop depth (z) =2m

Area = By + zy^2 =(2.4*1.4 + 1.5*1.4^2) = 6.3m^2

$$V = \frac{Q}{A} = \frac{5}{6.3} = 0.794\,{}^{m}\!/\!_{sec}$$

$$E = y + \frac{V^2}{2g} = 1.4 + \frac{(0.792)^2}{2*9.81} = 1.432m$$

$$yc = \frac{2}{3}E = \frac{2}{3}*1.432 = 0.96m$$

$$yc = \left(\frac{q^2}{g}\right)^{1/3} = 0.96 \Rightarrow q = 2.937\ m^3/sec.m$$

$$B = \frac{Q}{q} = \frac{5}{2.937} = 1.7m$$

Drop No. = D = $\left(\frac{yc}{z}\right)^3 = \left(\frac{0.96}{2}\right)^3 = 0.11$

$Ld = 4.3\ ZD^{0.27} = 4.73m$

$y_1 = 0.54\ ZD^{0.425} = 0.41m$

$Y_2 = 1.66\ ZD^{0.27} = 1.826m$

$Lj = 6.9(y_1 - y_2) = 9.7m$

$Ld + Lj = 4.73 + 9.7 = 14.43m$

4.5 STILLING BASINS

In stilling basin , kinetic energy is converted to turbulent and ultimately to heat.

4.5.1 Stilling Basins Appurtenanous:

1)Chute Blocks:

It used to reduce the incoming get & lift apportion of it from the floor producing shorter length of jump & is used to stabilized the jump & improve it.

2)**Sill:**

It is provides at the end of the stilling basin , it is function:

a)To reduce further the length of the jump.

b)To control the scour.

3)**Baffle Pier**:

It is used to stabilized the jump and increase turbulence by assisting in dissipation of energy.

4.5.1 Types of Stilling Basin:-

1)S. A. F Basin(saint Authory falls)

It is used for small structure and for $Fr = 1.7 - 17$

The stilling basin side wall may be parallel in case of rectangular or may be diverge in case of trapezoidal stilling basin.

TABIE(5)

Fr_1	1.7 — 5.5	5.5 — 11	11 — 17
LB / y_2	$4.5/Fr_1^{0.76}$	$4.5/Fr_1^{0.76}$	$4.5/Fr_1^{0.76}$
y_t / y_2	$1.1 - Fr_1^2/120$	0.85	$1.1 - Fr_1^2/800$

Chapter 4 Energy dissipated structures

2)U.S.B.R stilling Basin

a)U.S.B.R stilling Basin No.I :-

Is a basin created by a jump occurring on flat flood with out chute blocks or sill , such a basin is not practical because it is expensive.

Length and its lock of control:-

Length of basin = length of jump $(Lb = Lj)$

b)U.S.B.R stilling Basin No.II :-

it is used when the incoming velocity exist $(15 {}^{m}/_{sec})$ and for high spill way or for high head and large structure (Fr > 4.5)

1)the basin contains chute blocks @ the ${}^{U}/_{S}$ end and sill near the ${}^{D}/_{S}$ end and no baffle.

2)The length of the basin is related to the Fr No. as given below in table(6):

TABLE(6)

Fr	**4**	**6**	**8**	**≥10**
Length of basin	**3.6**y_2	**4**y_2	**4.2**y_2	**4.3**y_2

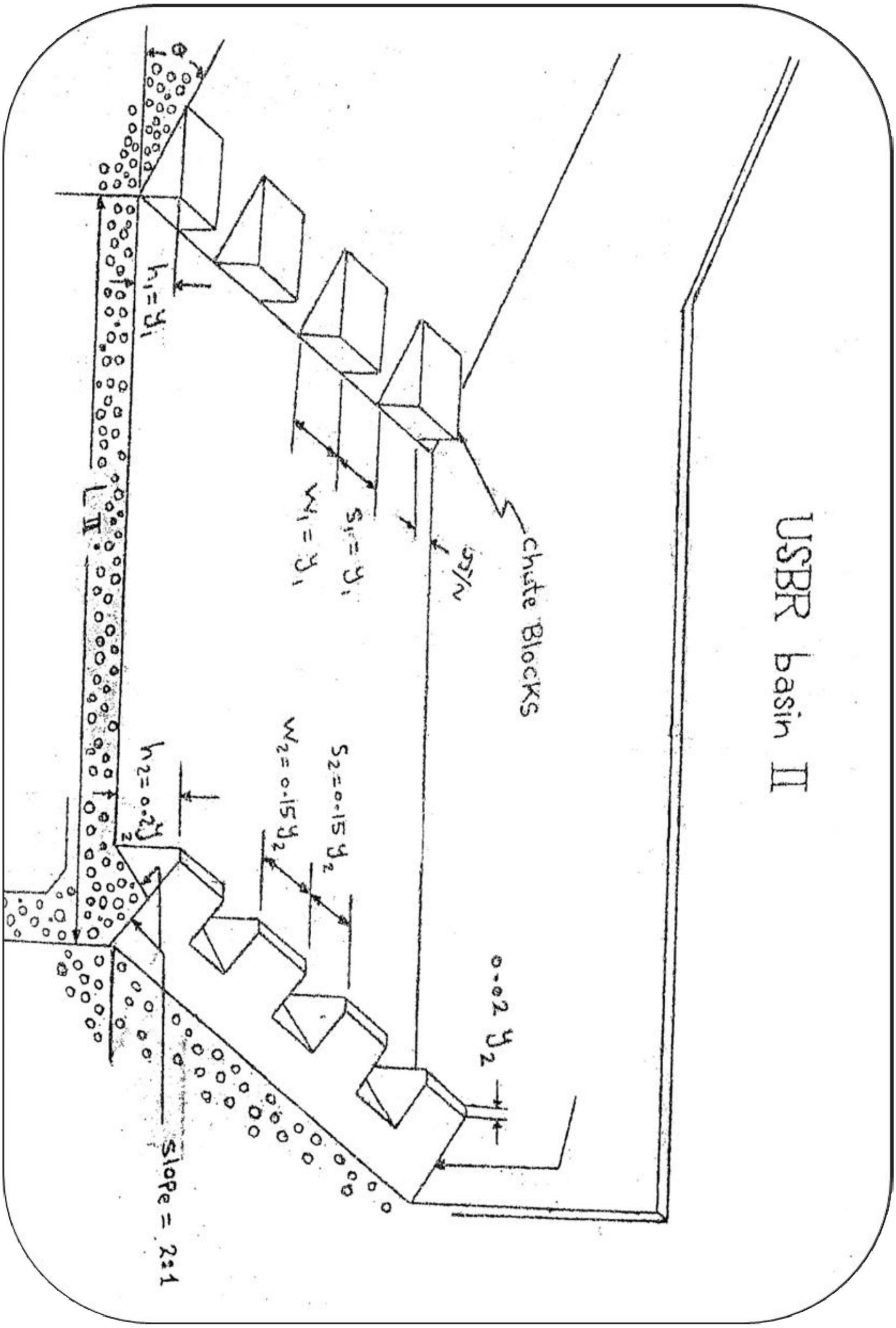
c) U.S.B.R stilling Basin No.III:-

The basin is used when the in coming velocity (not exceed $15 {}^{m}/_{sec}$)and(Fr > 4.5)but for small structure,it the same as a basin NoII, but with addition of impact blocks(baffle).

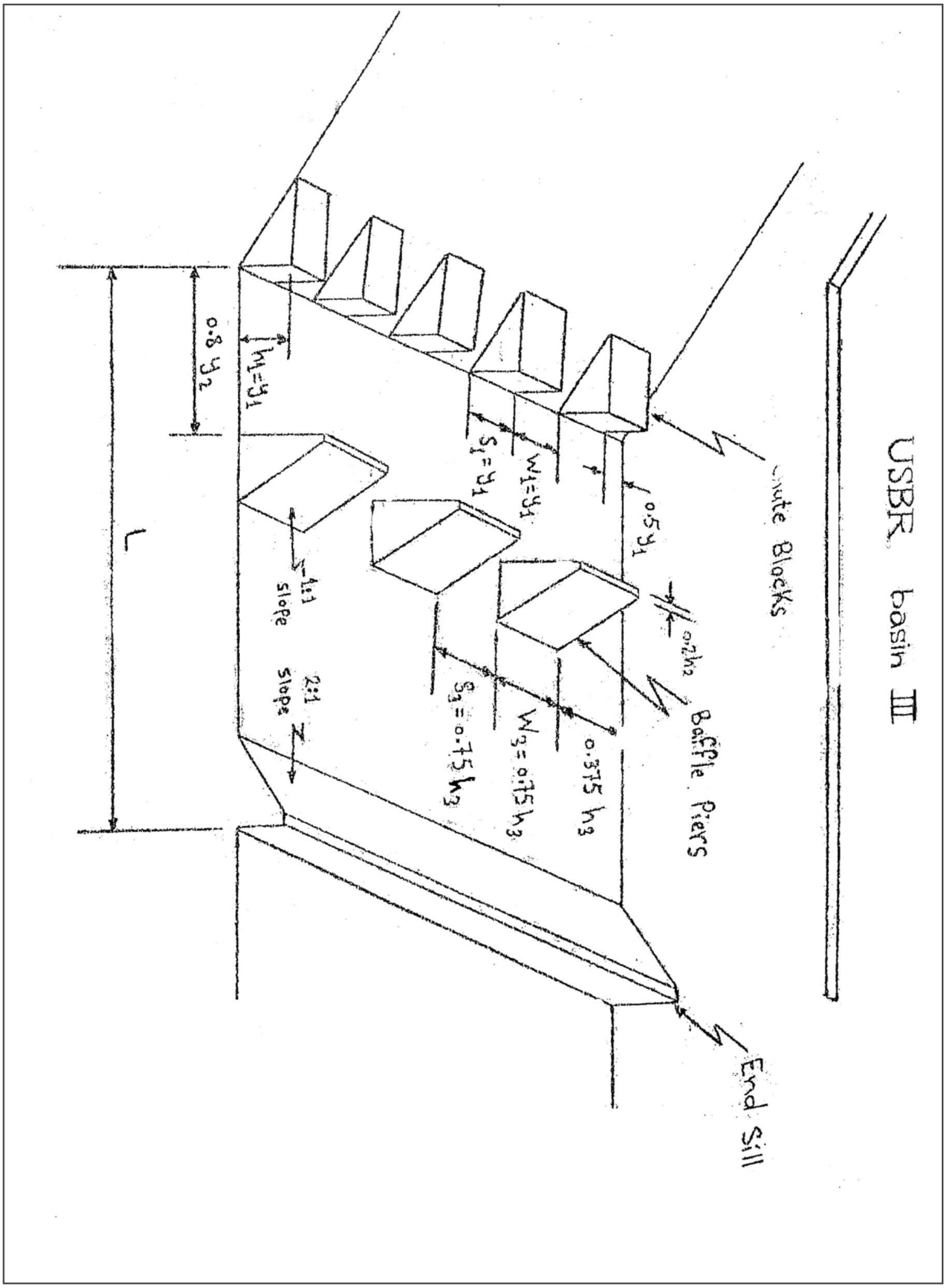
d) U.S.B.R stilling Basin No.IV:-

It is suitable for use with Fr (2.5 – 4.5) which usually occure in canal structure and divergent dams and it use in rectangular cross section only.

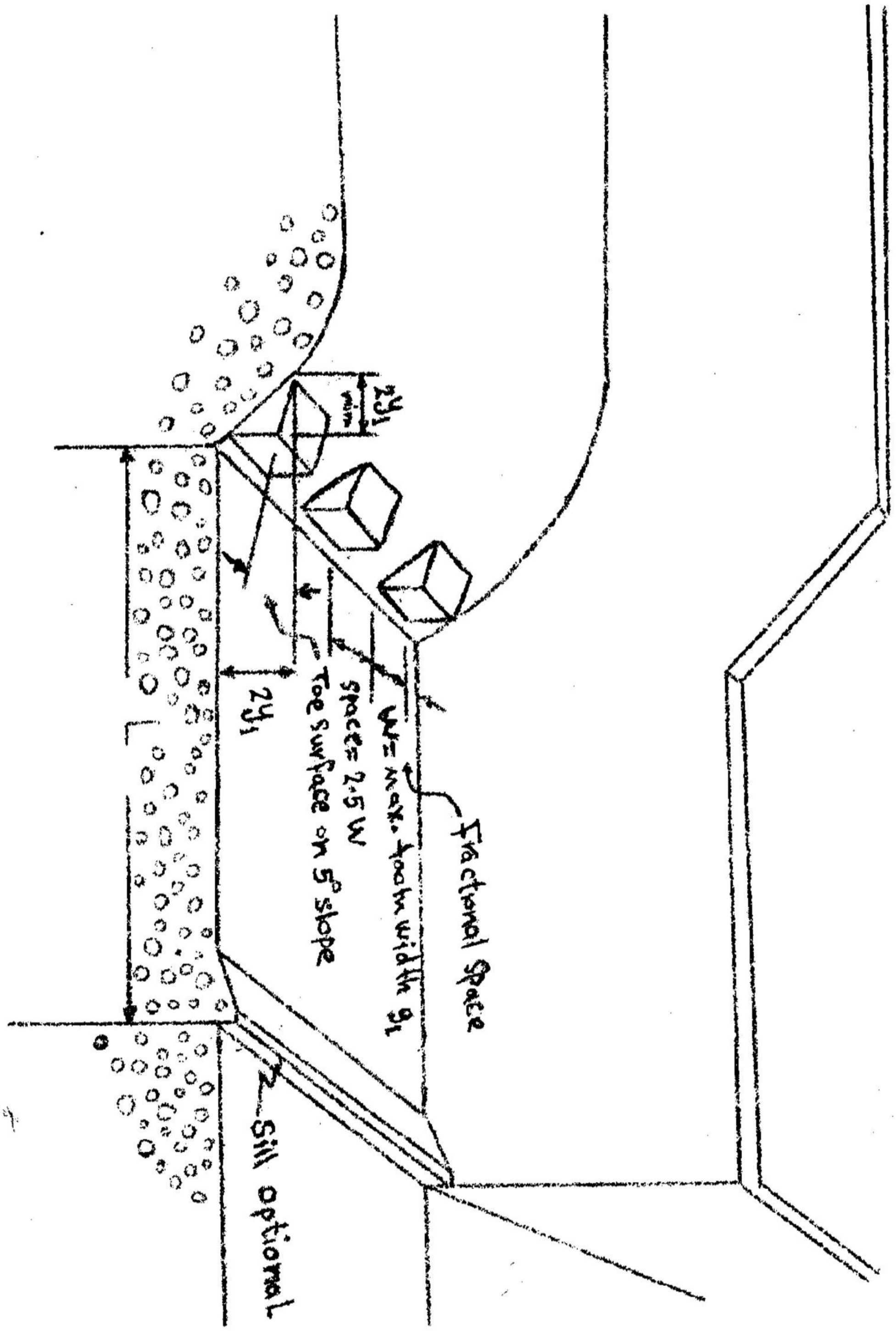
Chapter 4 Energy dissipated structures



Chapter 4 | Energy dissipated structures



Chapter 4 | Energy dissipated structures



USBR basin IV

Chapter 4 | Energy dissipated structures

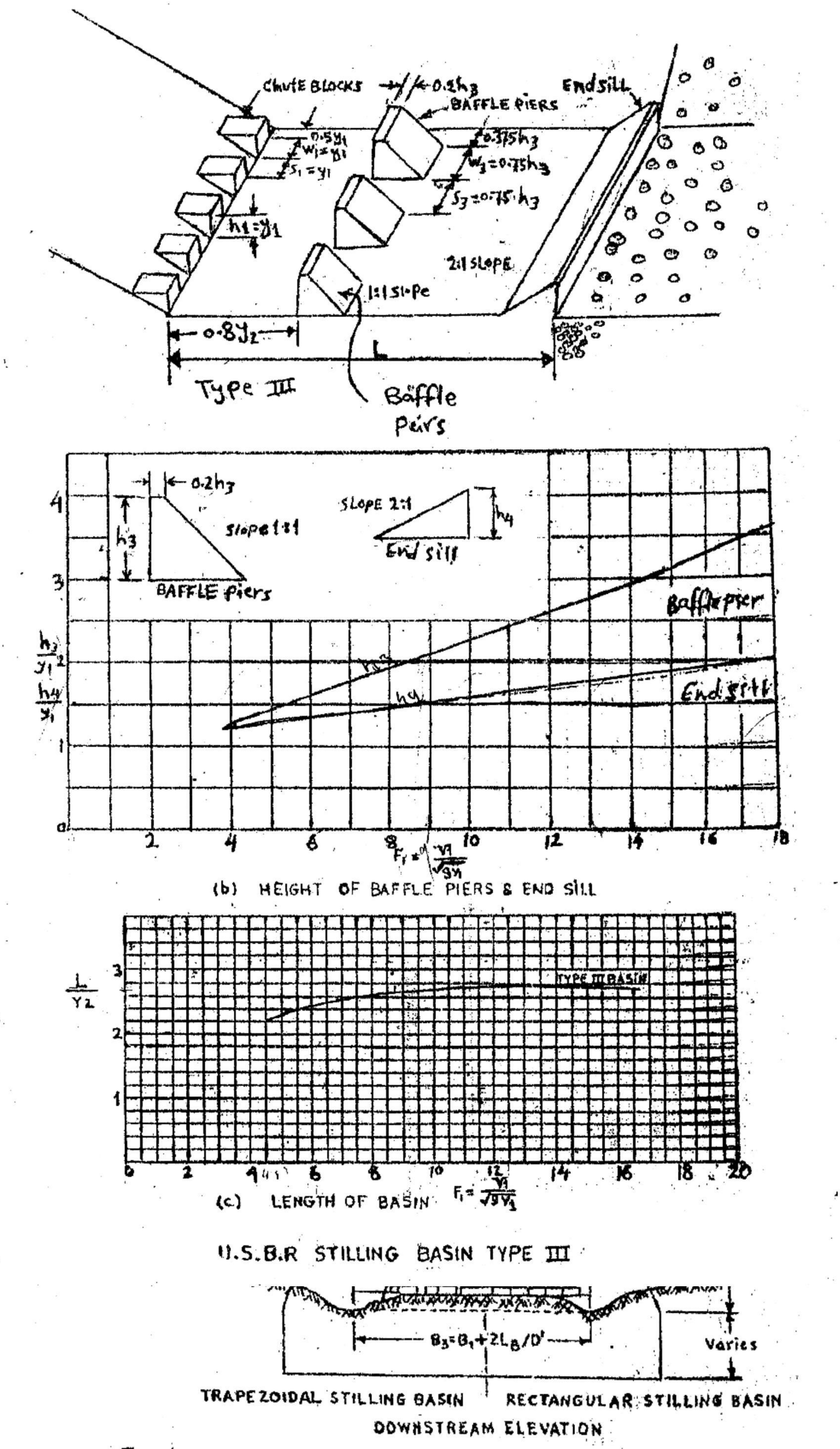
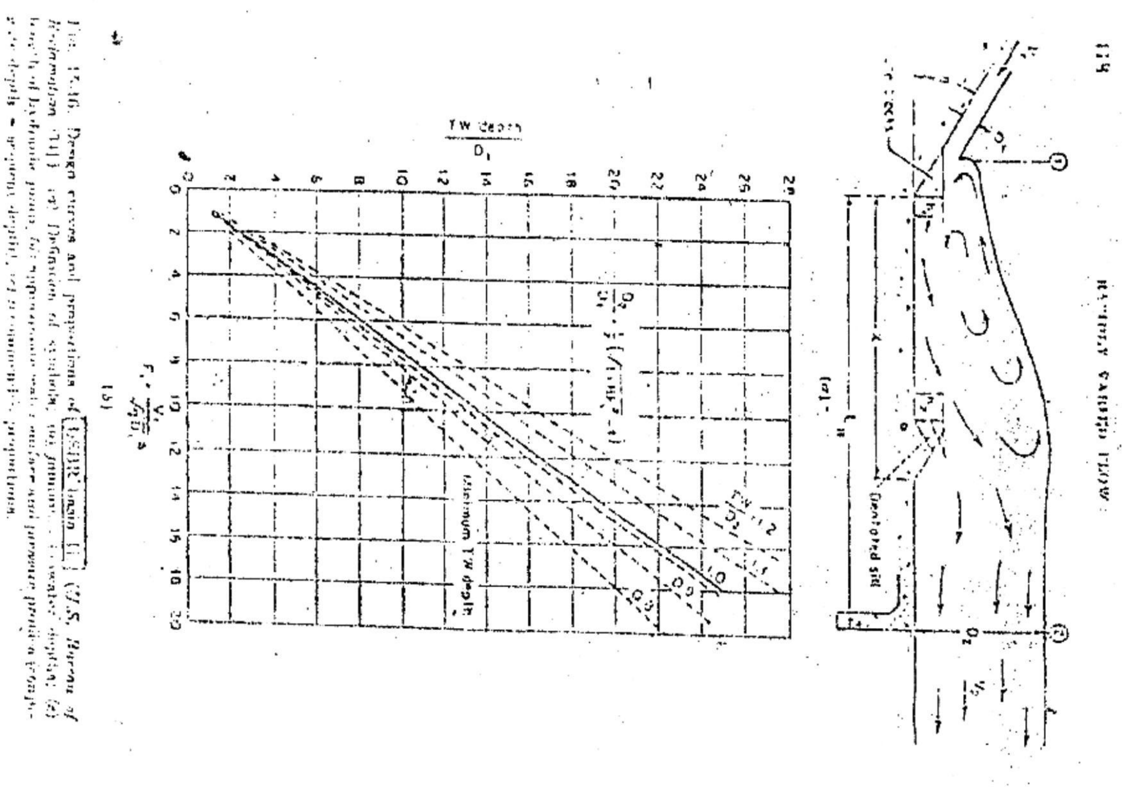
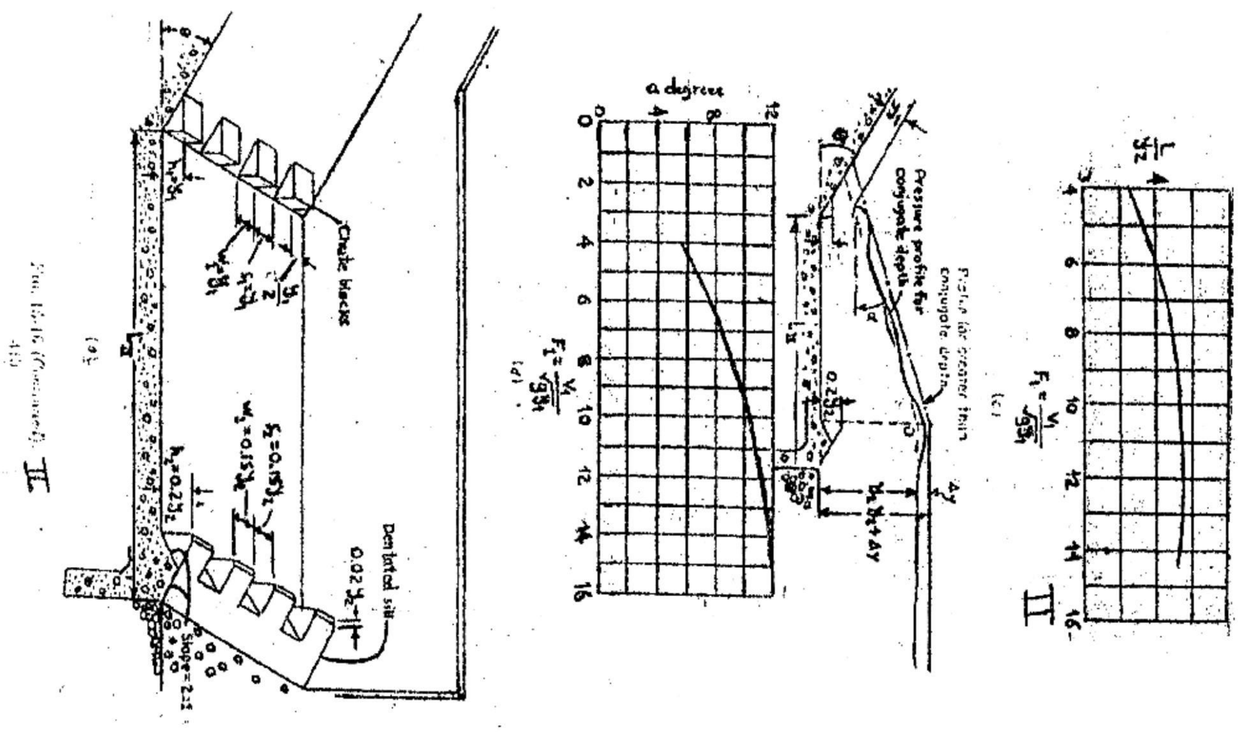


FIG. 15-15. Proportions of the SAF basin. (*U.S. Soil Conservation Service* [64].)


Dentated sill
Maximum TW depth
TW depth / D_1
(a)
(b)


Chute blocks
Dentated sill
Pressure profile for conjugate depth
a degrees
(c)
(d)
II

Chapter 4 Energy dissipated structures

Example 4.4
five kilometer length of a trapezoidal line canal at discharge (20 cumecs) and side slope (1:1) , the bottom width (5m) . regulator is concentrated at (tow km) with the gate , when opened the gate 0.6m the (u/s) water level (2m) .Determine the hydraulics jump will from or not and find U.S.B.R NO.

Solution

$$\frac{Q^2}{g} = \frac{400}{9.81} = 40.7$$

$$40.7 = \frac{(5yc + yc^2)^3}{2yc + 5} \qquad yc = 1.109m$$

$y1 < yc$ *the jump is accurs*

$A1 = 5(0.6) + (0.6)^2 = 3.36\ m^2$

$$v = \frac{20}{3.36} = 5.9\ m\backslash sec$$

$yh1 = \frac{A}{T} = \frac{3.36}{5.3} = 0.62$ $\quad Fr1 = \frac{5.9}{\sqrt{0.62 * 9.81}} = 2.37 < 4.5$ $\quad v1 = 5.9 < 15m\backslash sec$

Use U.S.B.R stilling Basin No.4

Example 4.5
Design U.S.B.R stilling basin for an overall spillway for a given data :-
Design discharge =2204m^3/sec Length of spill way=200m
Crest level=342.5 m B.L of river =320 m
Tail water in the river =325.68 m Discharge coff.=2.22
Q=2.22*L*$H^{\frac{3}{2}}$

Solution

Q=2.22*L*$H^{\frac{3}{2}}$

2204=2.22*200*$H^{\frac{3}{2}}$ $\Longrightarrow$ H=2.91m

U / S .w.L = 342.5+2.91=345.5m

B.L=320m

Specific energy(U / S)(E_1) =345.5-320=25.5m

$$E_1 = 25.5 = y_1 + \frac{V_1^2}{2g}$$

$$25.5 = y_1 + \left[\frac{2204}{200}\right]^2 * \frac{1}{2 * 9.81 * y_1^2}$$

By trail $\Longrightarrow y_1 = 0.5$m

$$V_1 = \frac{q}{y_1} = \frac{Q}{y_1 L} = \frac{2204}{0.5 * 200} = 22.04\ ; Fr_1 = \frac{V_1}{\sqrt{g y_1}} = \frac{22.04}{\sqrt{9.81 * 0.5}} = 9.95$$

Use stilling basin No. II

$$\frac{y_2}{y_1} = \frac{1}{2}\left[\sqrt{1 + 8Fr_1^2} - 1\right] \qquad y_2 = 5.419$$

$h_1 = y_1 = 0.5m \Longrightarrow S_1 = 0.5m$ $\qquad h_2 = 0.2 * 5.41 = 1.082$

$\mathrm{Fr} > 10$ $\qquad S_2 = 0.15 * 5.41 = 0.8115$

Length of Basin $\left(\frac{LB}{y_2} = 4.3\right) \Longrightarrow LB = 4.3 * 5.41 = 23.3m$ (from table)

Chapter 4 | Energy dissipated structures

Problems

4.1. A discharge of $20m^3$/sec flows in trapezoidal channel having a bottom width of 5m & side slope 1:1 , determine:

b)the height of the jump?

a)The depth after the jump when the initial depth is 0.6m?

c)the horse power lost in the jump?

4.2.A hydraulic jump is formed in 5m wide at a short distance of gate. If the flow depths just upstream and downstream of the gate is 1.5m respectively, and the outlet discharge is 130 m^3/sec ,determine :

a)The horse power lost in the jump.

b)Design and sketch a suitable U.S.B.R stilling basin of the jump.

4.3. If the sequent depth equal to1.7m, and the energy dissipated =0.5 $\frac{kg.m}{kg}$ of a hydraulic jump in a rectangular canal, what's the length of the jump?

4.4. A long rectangular open channel 3m wide carries a discharge of $15m^2$/sec. The channel slope is 0.004 and Manning's coefficient is 0.01. At a certain point in the channel where the flow reaches normal depth:

a)determine the state of flow.

b)If a hydraulic jump takes place at this depth, what is the sequent depth at the jump.

c)Estimate the energy head loss through the jump.

Chapter Five

Protection Of Approaches For Horizontal Floor

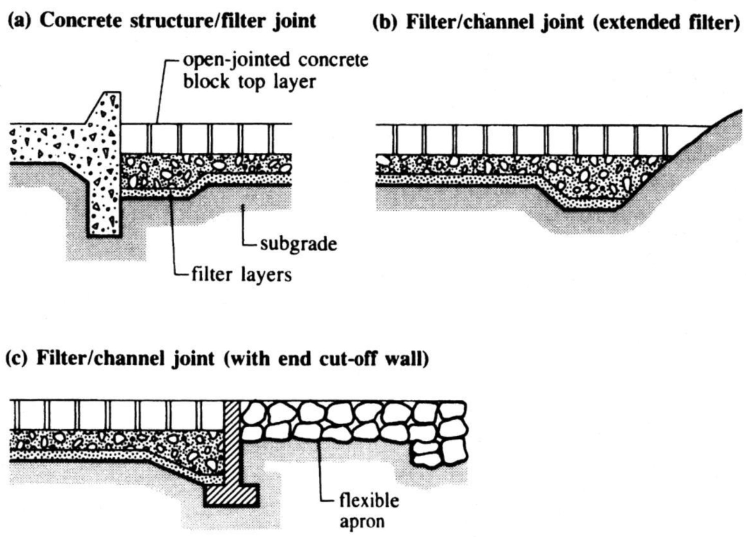
Chapter 5 Protection of approaches for horizontal floor

5.1 Introduction

The need to evaluate the various procedures being used to design rock rip rap has been indicated by the diverse results that may be obtained depending on the procedure used and assumptions concerning hydraulic and geomorphic conditions at a site. Failure at a site is usually attributed to excessive hydraulic forces acting on the bank and causing displacement of the stones that comprise the riprap (fig. 1). However, other factors, such as improper gradation or placement of stone, inadequate assessment of probable morphologic changes, or failure of the original bank material may contribute to the riprap failure.

Protection work are required on the *U/S* as well as on *D/S* in order to avoid:

1) Scouring from flowing water due to turbulence
2) Scouring due to seepage pressure (exit gradient)



5.2 Methods of protection

1) Stone pitching or rip –rap is required concrete block
2) To protect the channel material form wash out inverted

   Graded filter is the best solution.

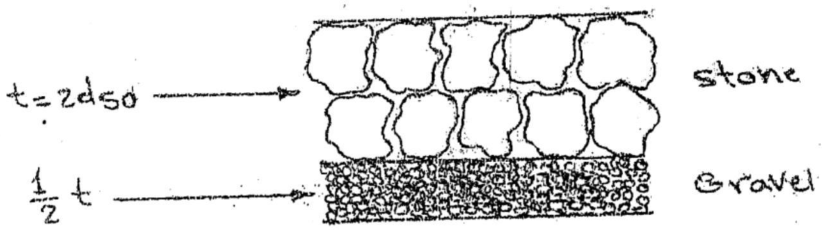
Fig 5.1 Rip Rap

5.3 Factors affect the selecting of rip –rap

1) Velocity
2) Flow direction
3) Turbulence
4) Waves

$d_{50} = \dfrac{V^2}{24}$

VA $=4.915 d^{1/2}$

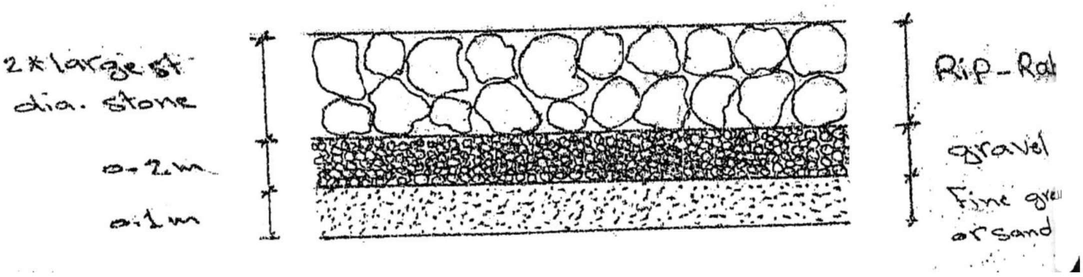


VA= Average velocity of flow (m/sec)

d 50 =main size diameter of stone

Apron=2.25 Dt/1.5D=1.5t

<u>Filterconstruction:-</u>



<u>D/ s</u> **<u>protection</u>:-**

a)filer:-

just at the end of concrete floor an inverted filter (1.5-2)D long is provided , where D depth of scour below original canal bed.

Total scour depth = (1.25 – 1.5)R

R: is lacey normal scour depth.

D=(XR-Y) ; x=1.25 -1.5

y: depth of water.

b)launching apron:-

is usually provided for a length of 1.15 D and at slope 2:1 .

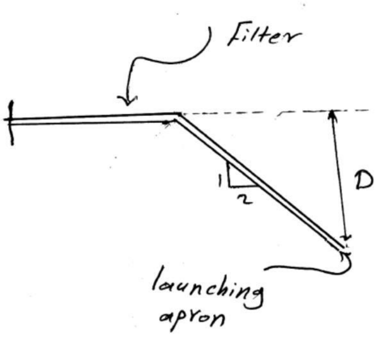
the volume of stone per meter length will be

$\sqrt{5}$ Dt=2.25Dt m^3/m length.

Where t is the thickness of the floor .

If the unbalanced apron laid in length of 1.5D , the thickness of unbalanced

Apron=2.25 Dt/1.5D=1.5t



U/S protection:-

Block protection laid over packed stone of length D

D=(XR –y) ; X=1.25 – 1.5

D=(1.25R-y)

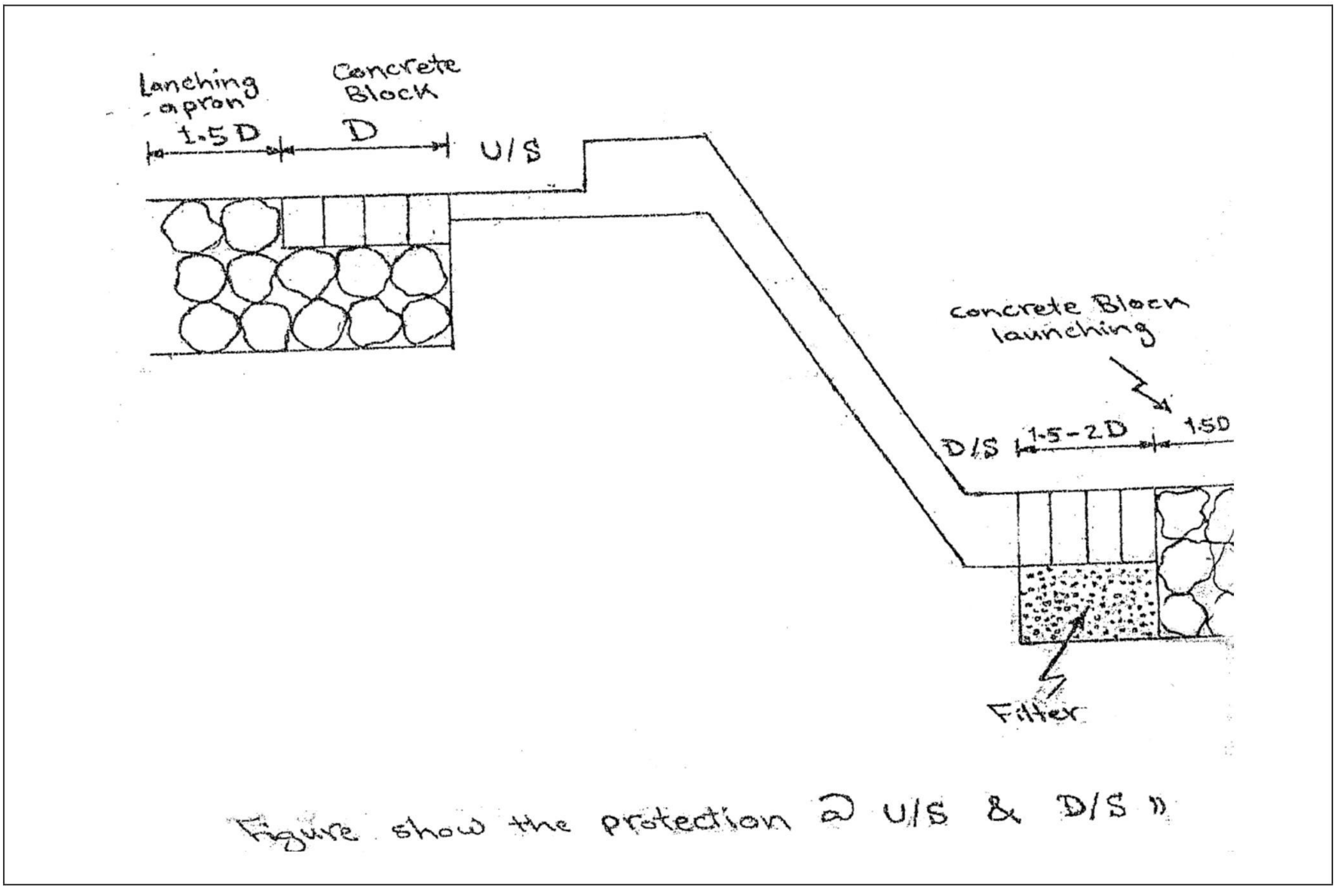


Fig 5.1 Protection at Upstream and Downstream

Chapter 5 Protection of approaches for horizontal floor

Example 5.1
Design protection for *U/s* and *D/s* of the regulator for the following data :-
q= 4 $m^3/sec.m$; silt factor(f)=1.0
U/S W.L= 60.6 m ; B.L = 57m
D/S W.L = 60.2 m ; B.L.(*D/S*) = 57.2 m
Crest level =58.5m

Solution

For *D/S* protection:-

$$R = 1.35 \left(\frac{q^2}{f}\right)^{1/3} = 1.35 \left(\frac{4^2}{1}\right)^{1/3} = 3.4\text{m}$$

$$D = XR - y \qquad ; X = 1.25 - 1.5 \Longrightarrow take\ 1.5$$

$$; y = 60.2 - 57.2 = 3m$$

$$D = 1.5 * 3.4 - 3 = 2.1m \Longrightarrow 2 * 2.1 = 4.2m$$

$$Length = 1.5D - 2D \ \ (use\ 2D)$$

$$2 * 2.1 = 4.2$$

$$(2{:}1)\ launching\ apron = 1.5D = 1.5 * 2.1 = 3.15$$

$$Thickness = \frac{Volme}{Area} = 1.5t$$

$$Use \quad t = 0.5\ m$$

Thickness of filter $= 1.5 * 0.5 = 0.75m$

Length of apron $= \frac{3.15}{0.75} = 4.2\ m$

<u>Block protection:-</u>

$$2D = 2 * 2.1 = 4.2\ m^3$$

$L = \frac{4.2}{0.75} = 5.6m$, use 6 m

Use block 1*1*0.4 m

Chapter 5 Protection of approaches for horizontal floor

U/S protection:-

$D = 1.25\ R - y$

$D = 1.25(3.4)\_(60.6\ -\ 57)\ =\ 0.65\ m$

Launching apron $= 1.5D\ \ =\ 1.5(0.65)\ = 0.975m\ \approx\ 1m$

Assume thickness $= 0.5m$

Thickness of launching apron $= 1.5 * 0.5\ \ = 0.75m$

Length= $\frac{1}{0.75}\ = 1.3m$

Block $= \frac{D}{0.75} = \frac{0.65}{0.75}\ = 0.86\ use\ 0.7m$

$2Row\ * 0.5 * 0.5t$

Chapter Six

Conveyance Structures

Chapter 6 Conveyance Structures

6.1 Introduction

A canal conveying water from the head works has to run for large distances and has to maintain the water levels appropriately, as designed along its length. It has to run through terrains which generally would have a different slope small than the canal . The surrounding areas would invariably have its own drainage system ranging from small streams to large rivers . The canal has to carry the water across these water bodies as well as across artificial obstacles like railway line or roads .

The main structures of a canal system for conveyance of canal flow and control of water levels are as follows .

1. Pipe conduits, culverts and inverted siphons to carry flow under railways and highways .

2. Aqueducts, syphon aqueducts, super-passage, canal siphon or level crossings across natural drainage courses or other depressions.

3. Transitions at changes in cross sections.

In this chapter we'll deal with Culverts , Aqueducts and Inverted siphons.

6.2 Culverts

A **culvert** is a structure that allows water to flow under a road, railroad, trail, or similar obstruction from one side to the other side. Typically embedded so as to be surrounded by soil, a culvert may be made from a pipe, reinforced concrete or other material.

6.2.1 Design of pipe and box culverts

A)Pipe Culvert

Types of pipes :-

1.Concrete : Plain and reinforce concrete are suitable for low pressure application

2.Steel: Are formed form rolled to shape and welded it ,Its suitable for medium to high pressure applications .

3.Asbests: Asbestos cement pipe ,it is suitable for low medium and high pressure.

4.Plastic: It is suitable for medium and high pressure.

5.Pre –stressed concrete: it is suitable for high pressure.

Hydraulic calculation for pipes or (closed conducts);-

If the flow in the pipe is full flow , manning eq. can be used to calculated the friction loss.

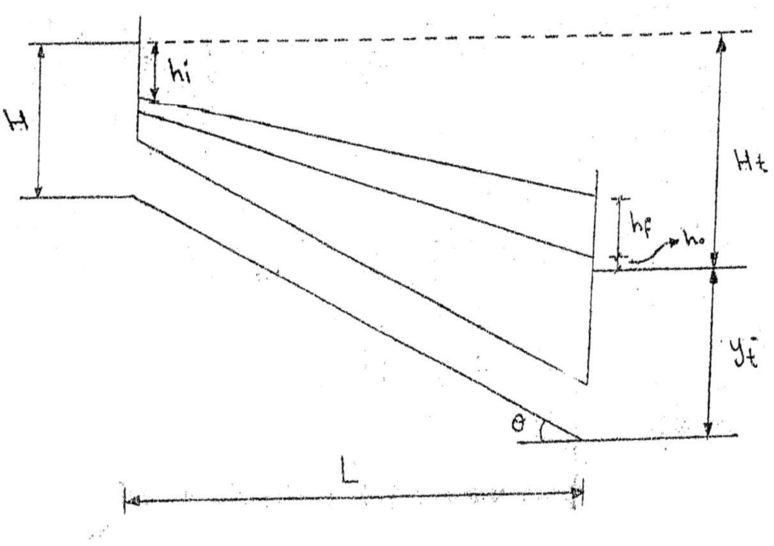
Friction loss in the pipe:-

$V = \frac{1}{n} * R^{2/3} * S^{1/2}$; $hf = S.L$

$\therefore hf = \frac{n^2 * V^2 * L}{R^{4/3}} * \frac{2g}{2g}$; $hf = \frac{2g * n^2 * L}{R^{4/3}} * \frac{V^2}{2g}$

$R = \frac{D}{4}$ where D = diameter of the pipe.

$$hf = \frac{19.62 * n^2 * L}{(D/4)^{4/3}} * \frac{V^2}{2g} = \frac{19.62 * n^2 * L * 6.35 V^2}{(D/4)^{4/3} * 2g}$$



$$hf = \frac{124.6 * n^2 L * V^2}{D^{4/3} * 2g} = \frac{\lambda L V^2}{2g}$$

$h_i = K_i \frac{V_1^2}{2g}$; $h_o = K_2 \frac{V^2}{2g}$; $h_f = \lambda L \frac{V^2}{2g}$

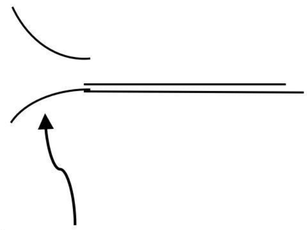
h_i= intenace loss ; h_o=outlet loss ; h_f= friction loss.

K_1= for square edge inlet =0.5

for round edge inlet =0.1

for curvature inlet if $\frac{r}{D}$=0. $\Longrightarrow$ K_1 =0.1

$\frac{r}{D}$=0.05 $\Longrightarrow$ K_1=0.25



For grooved or socket end pipe = 0.15 R

Value of K_2 =1

When out let discharge free ; it is measured to the center of the pipe:-

yt = $\frac{D}{2}$

Losses in bent $\Delta H_3 = K_3 * \frac{V^2}{2g}$

K_3it can be calculated as shown in the table (6.1)

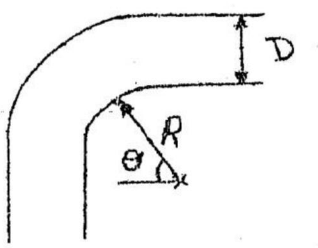


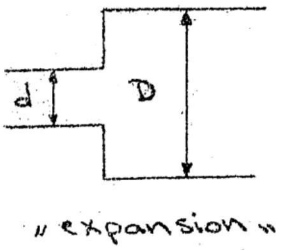
Table 6.1

R/D	90°	45°	22.5°
≥ 5	0.6	0.45	0.3
4.3	0.65	0.5	0.35
2	0.75	0.55	0.4
1	1	0.75	0.5
Elbow	0.7	---	----

T	1.8	---	----

Losses of sudden expansion or contraction:-

$$\Delta h_4 = K_4 * \frac{V^2}{2g}$$



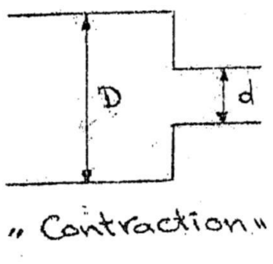


Table 6.2

D/d	1.5	2	2.5	5
Cont.	0.2	0.35	0.4	0.5
Exp.	0.35	0.6	0.75	1.0

Losses in fitting:-

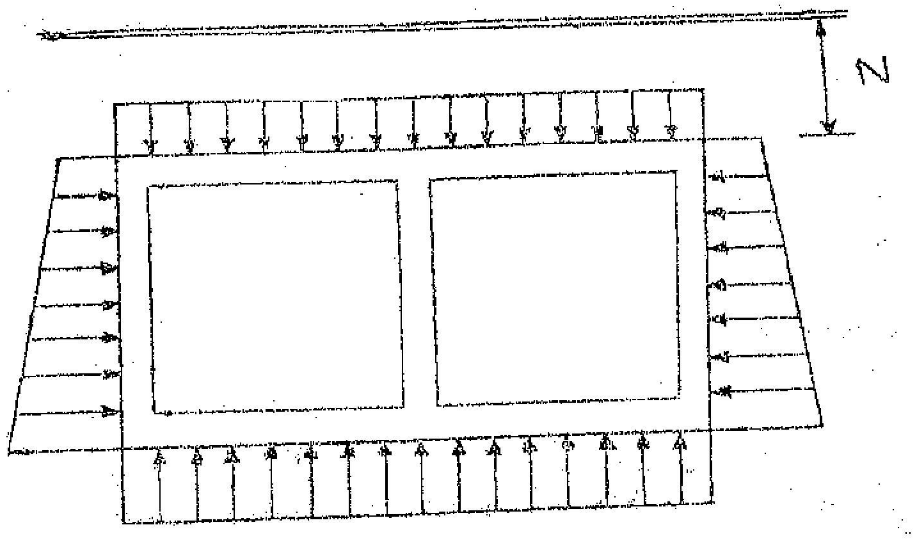
$$\Delta h_5 = K_5 * \frac{V^2}{2g}$$

Table 6.3

Cate value	**Butter fly value**	**Venture meter**	**Trash-rack**	**Check value**
0.1	**0.15**	**0.1**	**0.75**	**2.5**

$$H = \sum K * \frac{V^2}{2g}$$

B) Box Culvert



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

$$h_f = \frac{2gn^2L}{R^{4/3}}$$

K_1 :in let loss L :length of culvert.

K_2 :exit loss R : Hydraulic radius.

n :manning modulus. A : cross section of culvert.

For box culvert ⟹ $K_1 = 0.5$ (for square edge)

Full culvert

K_1=0.16

K_2=1

-Submerged inlet :when the inlet becomes submerge, the discharge can be calculated by sluice.

$$Q = Cd * B * D * (2gH)^{1/2}$$

$$Cd = 0.4 + 0.05 * \frac{H}{D}.$$

Where:

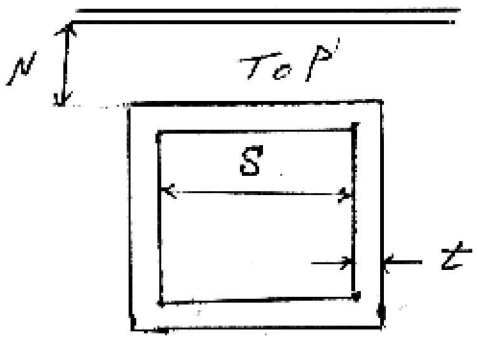
$$1.2 < \frac{H}{D} < 4$$

Structure Design of box – culvert:-

Procedure for structural desgin:-

1) Aasume wall thickness $\Longrightarrow$ t=S/12

S=clear width

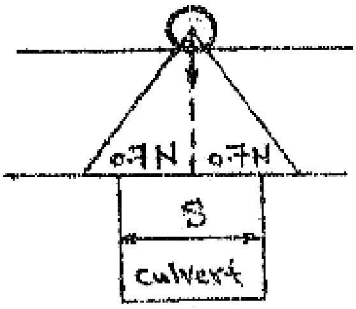


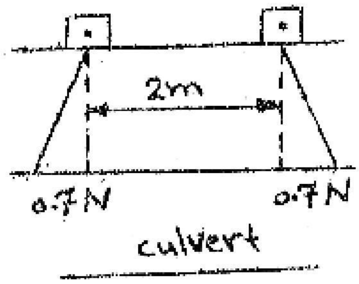
2) Determine the top level ,road level and the depth of earth level.

Loading (live load , impact load and dead load)

The impact factor can be determine as the following equation:-

$$\mathrm{I} = \frac{16}{40 + \mathrm{S}}$$





L.L =width *length

Width=0.7N+0.7N length=0.7N+0.7N+2

$$1.4N(1.4N + 2)$$

$\therefore The \frac{L.L}{Area} = \frac{0.8W(1+I)}{1.4N(1.4N+2)}$

Example 6.1
design R.C Box culvert of square dim.(3.5*3.5)m depth of asphalt =0.1m ,depth of earth filling =1m,road level =10m , γ of soil =1.6 T/m^3, γ concrete = 2.4 T/m^3, γ asphalt = 2300 Kg/m^3, fs = 1400 Kg/cm^3, fc = 60 Kg/cm^3,
Use H(20) loading ,ka =1/3 ,invert level of culvert = 5.1 m.

Solution

$$t = \frac{S}{12} = \frac{3.5}{12} = 0.3m$$

find top level of culvert

$10 - (0.1 + 1.0) = 8.9\ m$

Invert level of culvert = $5.1m$

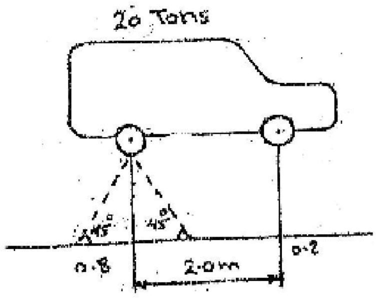
Loading :-

Weight of asphalt $= 0.1 * 2300 = 230\ Kg/m^3$

Wight of slab $= 0.3 * 2400 = 720\ Kg/m^3$

Earth filling $= 1.0 * 1600 = 1600\ Kg/m^3$

Live load $= 0.8 * 20 = 16\ Tons$



$$I = \frac{16}{40 + S} = \frac{16}{40 + 3.5} = 0.37$$

$1.37 * 16 = 21.92\ Tons$

Uniform distributed load $= \frac{21920}{(2+1.42*1.1)1.42*1.1}$

$$= 3950\ Kg/m^2$$

$$L.L = 3950\ Kg/m^2$$

Total load on the slap(top) $= 720 + 1600 + 3950 + 230$

$$= 6500\ kg/m^2$$

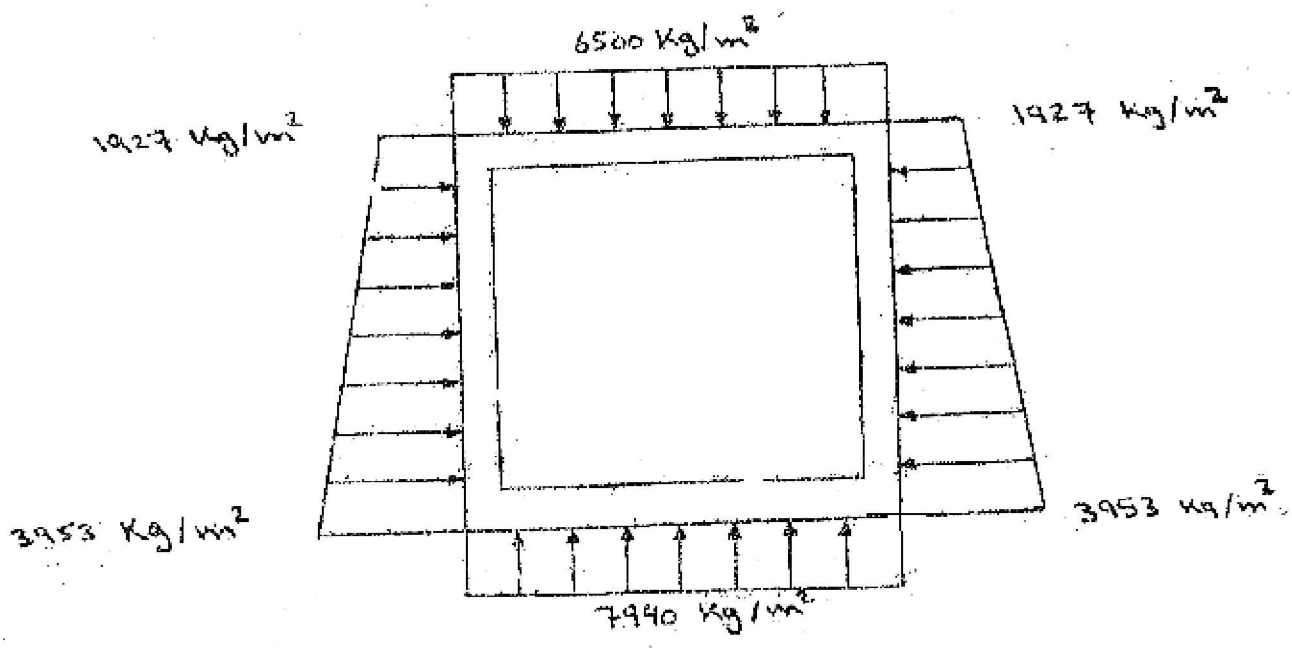
Weight of 2 vertical walls $= 2 * 0.3 * 3.8 * 2400$

$$= 5472\ \ Kg/m^2$$

$$\frac{6500*3.8+5472}{3.8} = 7940\ \mathrm{Kg/m^2}$$

$3950 + 1600 + 230 = 5780 \Longrightarrow 5780 * 1/3 = 1927$

$$\mathrm{P2}\ = \frac{1}{3} * (5780 + 1600 * 3.8) = 3953\ \ \mathrm{Kg/m^2}$$



Example 6.2
Design a box culvert given data :-
$Q = 43.8\, m^3/sec$,$L = 50m$,$K_{inle} = 0.3$, $K_{outlet} = 0.6$
$n = 0.015$,γ_s=1.6T/m^3,$ka = 1/3$ $\gamma_c = 2.4$, T/m^3, $Drop(H) = 0.4m$,
the inlet length for culvert $\leq$3m, thickness = $40cm$,depth of earth filling = $4m$,use H(20) & φ $20mm$,$f_s = 1400$ Kg/cm^2 $f_c = 60\ Kg/cm^2$

Solution

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

$$R = \frac{D}{4}$$

$$0.4 = (0.3 + 0.6 + \frac{2 * 9.81 * (0.015)^2 * 50}{(D/4)^{4/3}}) * \frac{(43.8)^2}{D^4} * \frac{1}{2 * 9.81}$$

From trail & error:-

D=4m ,we cannot use a box culvert because,

The max. height of culvert equal to 3.0m,

$\therefore$use rectangular box:-

$A = D^2 = 16 \quad \Longrightarrow \frac{A}{h} = \frac{16}{3} = 5.33m \& use\ t = 0.4m$

$-wt.of\ slab = 0.4 * 2.4 = 0.96\,T/m^2$

$-Earth\ of\ filling = 4 * 1.6 = 6.4\,T/m^2$

$-wt.of\ asphlt = 0.1 * 2.3 = 0.23\,T/m^2$

$-live\ load = 0.8 * 20 = 16\,Tons$

$$I = \frac{16}{40+S} = \frac{16}{40+5.33} = 0.35 \quad , \quad 1.35 * 16 = 21.6$$

Uniform dist. Load $= \frac{21.6}{(2+1.42*4.1)*1.42*4.1} = 0.474\, T/m^3$

Uniform dist. Load on top pf culvert $= \; 0.96 + 6.4 + 0.23 + 0.474 \; = \; 8.064 \; T/m^2$

Wt. of two vertical sides $= \; 2 * 0.4 * 3.4 * 2.4 \; = 6.528\, T/m$

Uniform dist. Load on bottom of culvert

$$= \frac{8.064 * 5.7 + 6.598}{5.7} = 9.209\, T/m^2$$

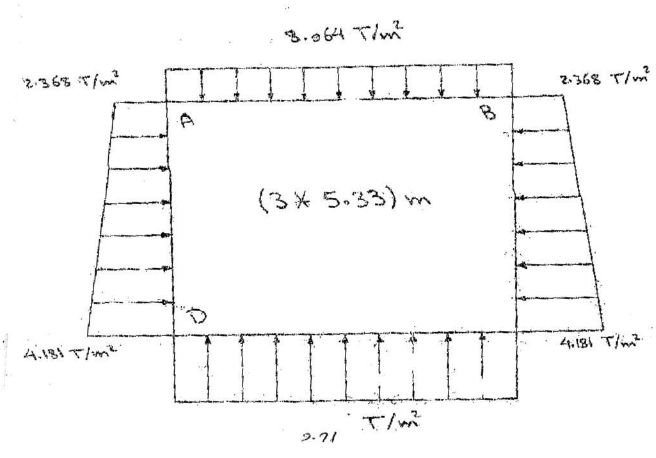
Uniform dist. Load on top side

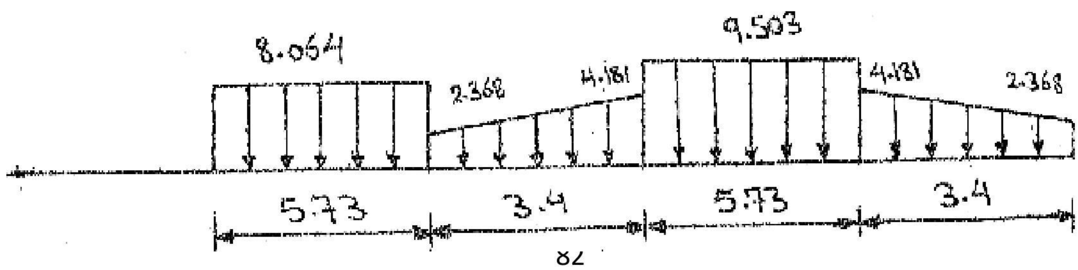
$$\frac{6.4 + 0.23 + 0.474}{3} = 2.368\, T/m^2$$

$Uniform\ dist. Load\ on\ bottom\ side \; = 1/3 * [1.6 * 3.4 + (6.4 + 0.23 + 0.474)] = 4.181\ T/m^2$

Distribution factor 1 $= (1/5.73)/[(1/5.73) + (1/3.4)] = \; 0.37$

Distribution factor 2 $= (1/3.4)/[(1/5.73) + (1/3.4)] = 0.67$





D.F	0.36	0.37	0.37	0.63	0.63	0.37	0.37	0.63
F.E.M	2.98	-22.06	22.06	-2.98	3.33	-26	26	-3.33
Bal.	12.02	7.06	-7.06	-12.02	14.28	8.38	-8.38	-14.28
c.o.m	-7.14	-3.53	3.53	7.14	-6.01	-4.19	4.19	6.01
Bal.	6.72	3.94	-3.94	-6.72	6.42	3.77	-3.77	-6.42
c.o.m	-3.21	-1.94	1.97	3.21	-3.36	-1.88	1.88	3.36
Bal.	3.26	1.9	-1.9	-3.26	3.3	1.93	-1.93	-3.3
c.o.m	-1.65	-0.95	0.95	1.65	-1.63	-0.96	0.96	1.63
Bal.	1.64	0.96	-0.96	-1.64	1.46	0.86	-0.86	-1.46
c.o.m	-0.73	-0.48	0.48	0.73	-0.82	-0.43	0.43	0.82
Bal.	0.76	0.45	-0.45	-0.76	0.79	0.46	-0.46	-0.79
c.o.m	-0.4	-0.22	0.22	0.4	-0.38	-0.23	0.23	0.38
Bal.	0.4	0.23	-0.23	-1.4	0.4	0.23	-0.23	-0.4
T.M.	14.65	-14.67	14.67	-14.65	17.78	-18.06	18.06	-17.78

$$M\ center = \frac{WL^2}{8} - \frac{WL^2}{12} = \frac{WL^2}{8} - M$$

$$= \frac{9.503 * (5.73)^2}{8} - 18.06 = 20.94\ T.M \leftarrow control$$

$$d = h - cover = 300 - 50 = 250\ mm = 25\ cm$$

$$F_s = 1400\ kg/cm^2$$

$$F_c = 60\ kg/cm^2$$

$$a = 0.855$$

$$AS(main) = \frac{M}{Fs.d.a}$$

$$M_1 = 20.94\ T.m \qquad As = \frac{20.94*10^5}{1400*25*0.855} = 69.94cm^2 \approx 70cm^2$$

$$S = \frac{Asb}{As} * 100 = \frac{804}{70} = 11.5\ cm$$

$\therefore$ $use\ 32\emptyset\ @\ 11.5\ cm\ \ c/c$

$M_2 = 14.65\ T.m \Longrightarrow As = 49\ \text{cm}^2$

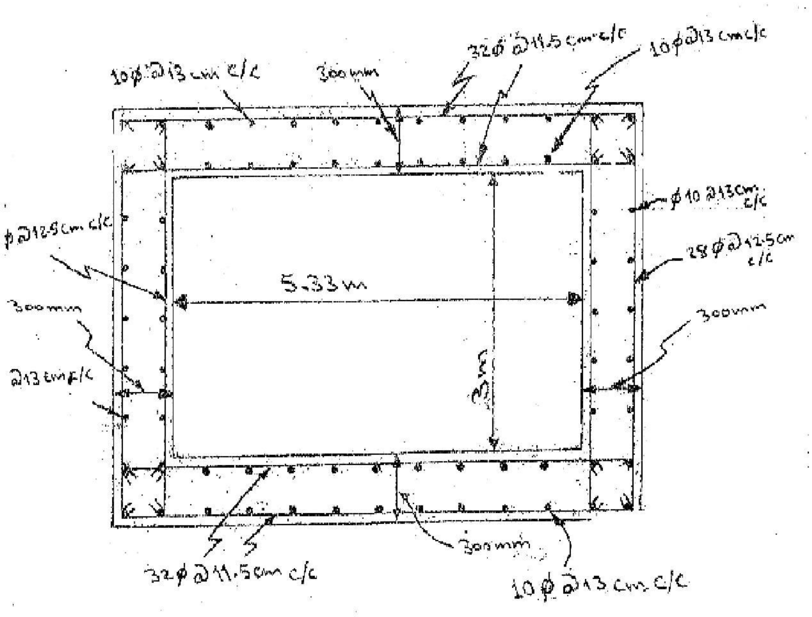
$$S = \frac{616}{49} = 13.56\ cm \approx 12.5cm$$

$\therefore$ $use\ 28\ \emptyset\ @\ 12.5\ cm\ c/c$

$As(min.) = 0.002 * b * h = 6cm^2.$

$$S = \frac{7.81}{6} = 13.1\ cm \approx 13cm$$

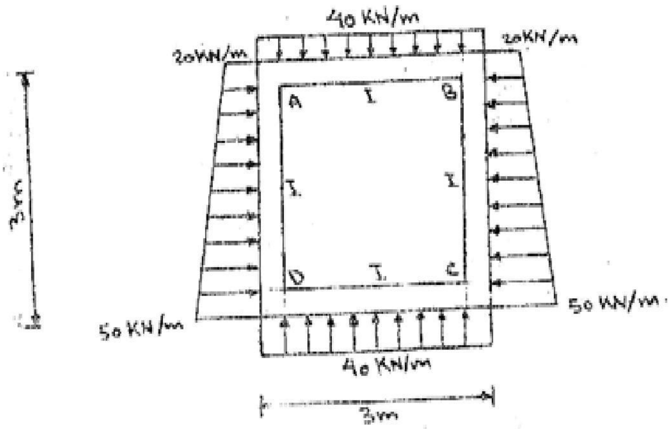
$use\ 10\emptyset\ @\ 13cm\ \ c/c$



Example 6.3

draw the (B.M.D) of the empty culvert shown in the fig. use the moment distribution method.

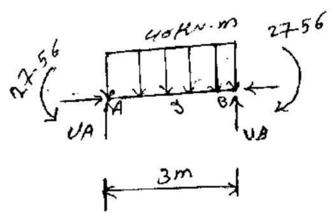
Solution



L=3m

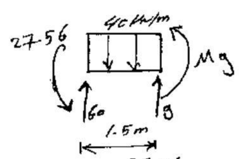
$$K_{AB} = K_{BA} = K_{BC} = K_{CB} = K_{CD} = K_{DC} = K_{DA} = K_{AD} = \frac{4EI}{L}$$

$\therefore$D.F $_{AB} = \frac{K_{AB}}{K_{AB}+K_{AD}} = 0.5$

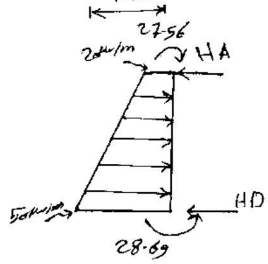


DF $_{AB}$= DF $_{BA}$= DF $_{BC}$=...............=0.5

$|M_{F\,AB}| = |M_{F\,BA}| = |M_{F\,CD}| = |M_{F\,DC}| = \frac{WL^2}{12} = \frac{40*3^2}{12} = 30KN.m$



$|M_{F\,AD}| = |M_{F\,BC}| = \frac{20*3^2}{12} + \frac{30*3^2}{3} = 24KN.m$

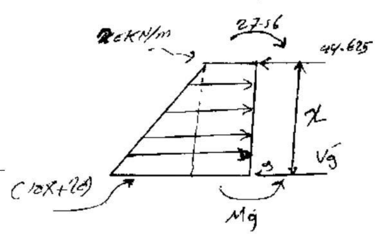


$|M_{F\,DA}| = |M_{F\,CB}| = \frac{20*3^2}{12} + \frac{30*3^2}{20} = 28.5KN.m$

$$From \sum MB = 0$$

$$VA = 60\ kN$$

$$From \sum M\ about\ g = 0$$



$$Mg \ = 17.4375 \ kN \ .m$$

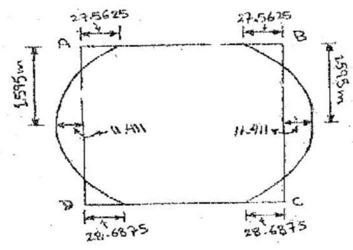
$$From \sum M \ about \ D = 0$$

$$HA = 44.625 \ kn$$

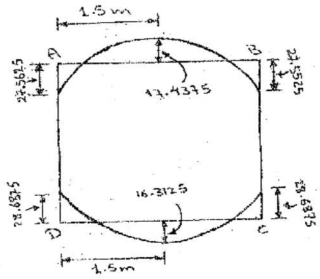
The point of zero shear is the point of max.

$$\left(\frac{20 + 10x + 20}{2}\right)x - \ 44.625 \ = 0$$

$$X = \frac{-4 \pm \sqrt{16 + 4 * 1 * 8.925}}{2} = 1.595 \ m$$



(BMD of AD & BC)



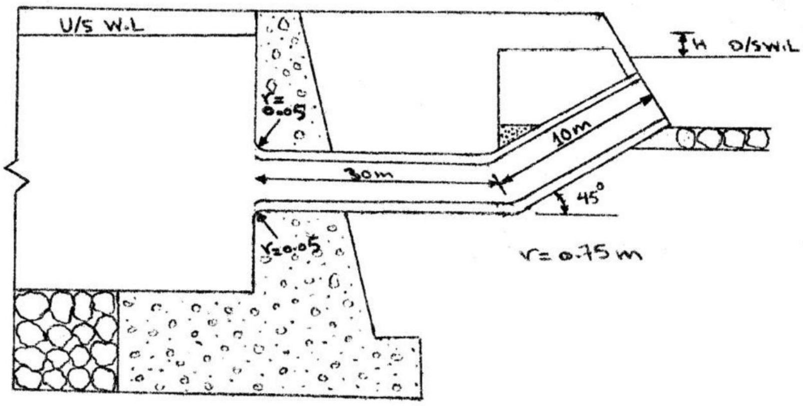
BMD of AB & CD

From $\sum$M about g^{-} =0

$$M \ g^{-} \ \ = 11.411 \ kN \ .m$$

Chapter 6 Conveyance Structures

Example 6.4
A precast concrete pipes culvert is to be constructed under road as sketched below. Find the head drop(H) that takes place with a discharge of 0.5 cumecs and a pipe diameter of 0.75m.



Solution

For inlet $k_1 = 0.25$ (approx.) because $r/D = \frac{0.05}{0.75} = 0.066$

For outlet $k_2 = 1.0$

For bend $R/D = \frac{0.75}{0.75} = 1.0 \quad \theta = 45 \quad \Rightarrow \quad K = 0.75$

For friction $k_f = \frac{124 n^2 L}{D^{4/3}} = \frac{124 * (0.015) * 40}{(0.75)^{4/3}} = 1.23$

$\sum K = 0.25 + 1.0 + 1.23 + 0.75 = 3.24$

$H = \sum K \frac{V^2}{2g} \quad , \quad H = 3.24 * \frac{(0.5)^2}{2 * 9.81} * \frac{1}{\pi^2 (\frac{0.75^2}{4})^2} = 0.21$

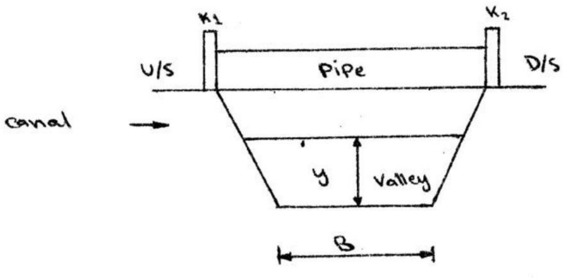
$\therefore H = 0.21m$

Chapter 6 Conveyance Structures

6.3 Aqueduct Structures

An aqueduct is provided when water is to carried over the canal or valley. It contain inlet pipe or flume acting as abridge & out let pipe.

-if the span is large it may be economic to provide support piers.



Types of aqueducts:-

A)Pipes:-

1- The span (L) , discharge(Q) , & over head loss (ΔH) are defined along U/S & D/S (with canal velocity).

2-Choose inlet & outlet & fixed the coefficient k_1 & k_2.

3-Calculate the pipe diameter by using the following eq.

$$\Delta H = \left[k_1 + k_2 + \frac{124.6.n^2.L}{D^{4/3}} \right] * \frac{V^2}{2g}$$

4-Safe span :-

$$L = 0.91 \left[\frac{f_s D}{30 + \frac{D}{t}} \right]^{1/2}$$

Where:-

L: safe span (m)

f_s: steel stress for mild steel = $9300\ kn/m2$

D: pipe diameter (m)

t: thickness of the pipe (m)

-Unit weight of steel (γ) = 77 kn/m^3

Example 6.4
Design pipe aqueduct on a lined branch canal with 1.5 : 1 side slope to cross an open drain 6.0m deep , 4.0m bed width , 2 : 1 side slope the discharge of aqueduct is 1.8m^3/sec , the overall head loss not exceed 0.21m , the canal bed width & depth of flow U/S & D/S are 1.5m & 0.9m respectively & the slope is 25.1cm/km , steel stress 93000kn/m^3 , k_1 = 0.5 , k_2 = 1.0 , n = 0.01 , L = 28m , t = 9.5mm

Solution

$$\Delta H = \left[\frac{124.6 * (0.01)^2 * 28}{D^{4/3}}\right] * \frac{(1.8)^2 * 16}{\pi^2 * D^4 * 2 * 9.81}$$

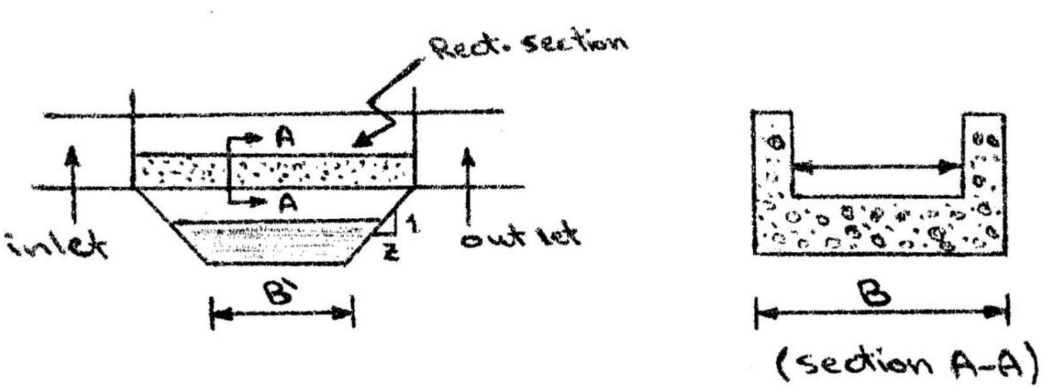
$$0.21 = \left[0.5 + 1.0 + \frac{124.6 * (0.01)^2 * 28}{D^{4/3}}\right] * \frac{(1.8)^2 * 16}{\pi^2 * D^4 * 2 * 9.81}$$

$$D = 1.2m \quad , \quad t = 9.5mm$$

$$L = 0.91 * \left[\frac{93000 * 1.2}{30 + \frac{1.2 * 1000}{9.5}}\right]^{1/2} = 24.31m < 28m \quad \therefore \text{we need piles}$$

B)Flumes:-

The most economic section is rectangular which is half square.



The head loss is the difference between U/S & D/S.

It is the total of inlet-outlet transition. The friction loss in the rectangular channel section. The dimension of rectangular section can be estimated by assuming the bed width (B) as the same as U/S & D/S of canal.

-The velocity in the rectangular section (V) can be calculated by :-

$$\Delta H = \left[(k_1 + k_2) \frac{V^2 - V_1^2}{2g} + \frac{V^2 n^2 L}{\left(B/4\right)^{4/3}} \right]$$

Where :-

V: velocity of flow in the flume.

V_1: velocity of flow in the canal.

B: width of flume = width of canal.

We can find S (slope) and use it @ manning eq. to find (y) add (20%y) free board.

Design of flume:-

We will discuss the hydraulic and structural design of flume.

a)Hydraulic design:

design steps:

1.assume the width of the flume is equal to the width of canal.

2.calculate the velocity in canal by using Mannings equation,

$V_{canal}=\frac{1}{n}\times R^{2/3}\times S^{1/2}$; $V_{canal}=V_1$

3.calculate velocity in flume(V)

$$\Delta H=\left[(k_1+k_2)\frac{V^2-V_1^2}{2g}+\frac{V^2n^2L}{\left(B/4\right)^{4/3}}\right]$$

4.find the side slope of flume;

$$S=\frac{\Delta H}{L}$$

5.find the depth of flume(y) by using Mannings equation and add 20% of y as free board.

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

b)Structural design:-

design steps:

1)calculate the thickness of flumes walls(t), where $t=\frac{L}{12}$.

2)calculate the load on the base of flume as follow:-

W(kg/m^2)=Self weight of slab+Weight of water

3)assume the slab simply supported and find bending moment:

$$M=\frac{WL^2}{8}$$

Chapter 6 Conveyance Structures

4)determine the area of steel(usually using working stress method) as follow:-

$$As = \frac{M}{fs.a.d} \qquad d = \sqrt{\frac{M}{Q.b}}$$

Where;

As=area of steel

M=maximum bending moment

fs=tensile strength of steel

a=0.862

Q=9.84

b=the width of slab strip=1m

5)design wall on sides as inverted beam.

Example 6.5
Design an open aqueduct to pass a discharge of canal with depth of flow 1.2m bed width 2m. side slope 1.5:1 longitudinal slope 10cm : 1km , n=0.015 , headloss=0.18, the length of aqueduct 25m take k_1=0.5 , k_2=1 .

Solution

Assume the width of flume = 2m (same as canal width)

$$V_1=\frac{1}{n}\times R^{2/3}\times S^{1/2}$$

$$V_1=\left(\frac{1}{0.015}\right)\times\left(\frac{1.2\times 2}{2+2\times 1.2}\right)^{\frac{2}{3}}\times\left(\frac{10}{100000}\right)^{\frac{1}{2}}$$

V_1=0.445 m/ses

$$0.18=\left[(0.5+1)\frac{V^2-0.445^2}{19.6}+\frac{V^2*0.015^2*25}{\left(2/4\right)^{4/3}}\right]$$

V=2.15m/sec

$$S=\frac{\Delta H}{L}=0.18/25=7.2\times 10^{-3}$$

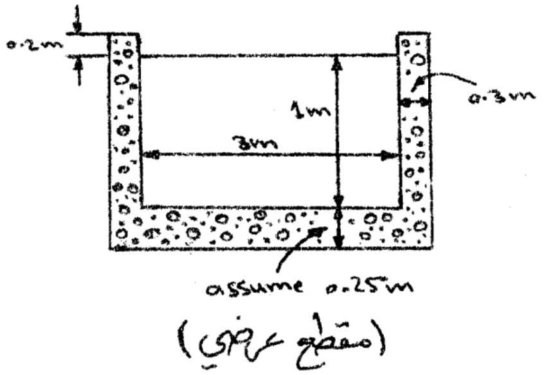
$$2.15=\left(\frac{1}{0.013}\right)\times\left(\frac{y\times 2}{2+2\times y}\right)^{\frac{2}{3}}\times\left(7.2\times 10^{-3}\right)^{\frac{1}{2}}$$

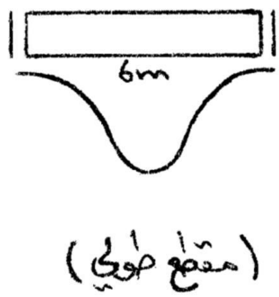
By trial and error y=0.23m

Total depth =1.2×0.23=0.276m

Example 6.6

Design an aqueduct for 3m wide canal having a water way of 6m , the depth of the water in the aqueduct is 1m , f_s = 1400 kg/cm , f_c = 55 kg/cm^2





Solution

Y = 1.0m , F.B = 0.2m ⇒ total depth = 1.2m

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

Assume t =0.25m

2400*0.25 = 600 kg/m^2 (Dead weight of slab)

Weight of water = 1000*1.2 = 1200 kg/m^2

W = 1200 + 600 = 1800 kg/m^2

Moment = $\frac{WL^2}{8}$ (simple beam)

L = 3 + 0.3 = 3.3m

$M=\frac{1800*3.3^2}{8}=2450.25$ kg.m

$$d = \sqrt{\frac{M}{Q.b}} = \sqrt{\frac{2450.25*100}{9.84*100}} = 15.8cm$$

$$t = d + cover = 15.8 + 8 = 23.8\ cm < 25\ cm \quad O.K$$

$$As = \frac{M}{fs.a.d} \quad , \quad \mathrm{d = 25 - 8 = 17cm}$$

$$As = \frac{2450*100}{1400*0.862*17} = 11.94cm^2$$

$Use\ 16Ø\ @\ 16cm\ c/c$

<u>at the end section:-</u>

$$M = \frac{\gamma.h^2}{2}\left(h/_3 + t/_2\right)$$

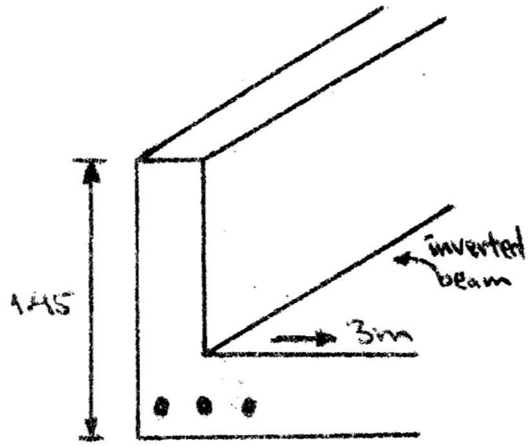
$$= \frac{1}{2}*1000*(1.2)^2\left(\frac{1.2}{3}+\frac{0.25}{2}\right) = 378kg.m$$

$$As = \frac{378*1000}{1400*0.862*17} = 1.84cm^2$$

<u>Distribution steel:</u>

$$\frac{0.2}{100}*25*100 = 5cm^2$$

$12\ Ø\ @\ 200cm\ c/c$



<u>Design of beam:</u>

Load per ran of the beam $L = 6.6m\ c/c$

$1000 * 1.2 * (3/2) = 1800\ kg/m$

$Dead\ load\ of\ beam = 0.3 * 1.45 * 2400 = 1044\ kg/m$

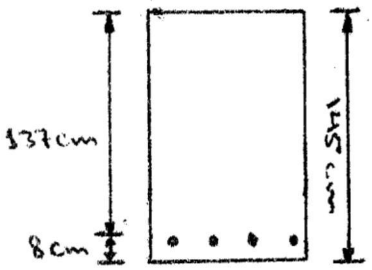
$Dead\ load\ of\ slab = 2400 * 3 * 0.25 * (1/2) = 900\ kg/\mathrm{m}$

Chapter 6 Conveyance Structures

$$W = 1800 + 900 + 1044 = 3744 kg/m$$

$$\therefore M = \frac{W.L^2}{8} = \frac{3744 * 6.6^2}{8} = 20386.08 kg.m$$

$$As = \frac{20386.08 * 100}{1400 * 0.862 * 137} = 12.33 cm^2$$



<u>Check of shear:-</u>

$$Maz. Shearing\ force = (3744 * 6.6/2) = 12355 kg$$

$$Shear\ stress = (S/a.b.d) = (12355/0.862 * 30 * 137)$$

$$Allowable\ shear\ stress = 5\, Kg/cm^2$$

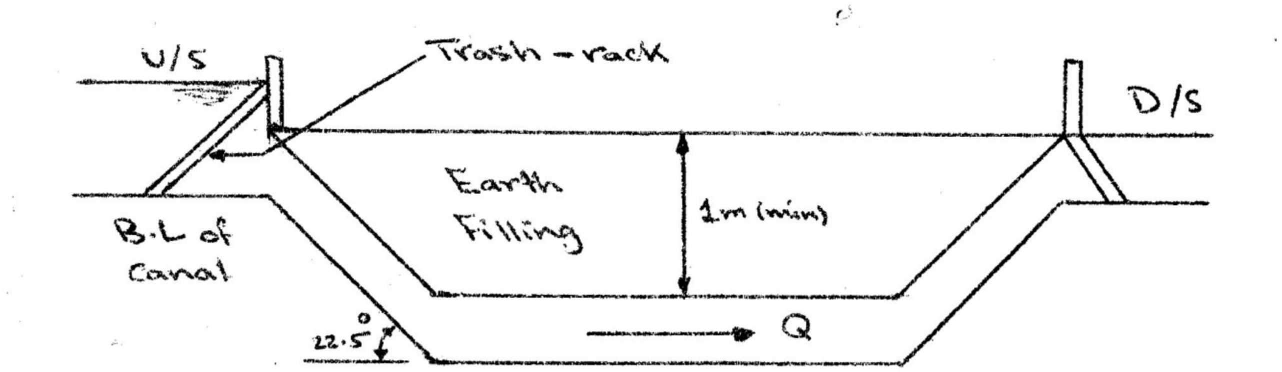
6.4 Inverted Siphon

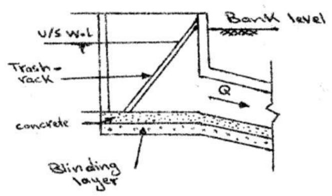
Inverted siphon are used to convey canal water by gravity under road rail. A siphon is a closed conduct designed to run pressure, if:-

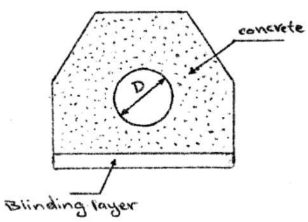
$Q > 2.5\, m^3/sec$ (Use square section)

$Q < 2.5\, m^3/sec$ (Use square round section)

-The total head loss:-







1- @ inlet & out let.

2- Head loss in the two screens.

3- Head loss due to friction.

4- Head loss in 2 elbows.

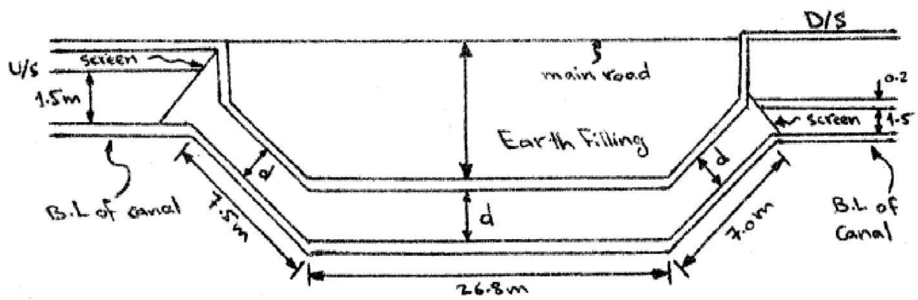
Chapter 6 Conveyance Structures

Example 6.7

Design an inverted siphon required to pass canal discharge of 4.0 m^3/sec under main road with $0.2m$ head loss. The velocity in the canal is $0.82\ m/sec$ & the depth of water $1.5m$. Safety screen are provided of entry and exit. The inverted siphon has 22.5° elbows of each end. The site dimension are shown as below

$n = 0.013\ , K_1 = 0.2\ ,, K_2 = 0.3\ ,, K_{secreens} = 0.2\ ,, K_{elbows} = 0.05$

$$\Delta H = 0.2m = U/S\ (W.L) - D/S\ (W.L)$$



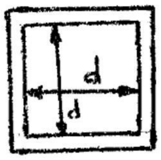
longitudinal section of siphon (L-section)

Solution

$Total\ head\ loss = 0.2m$

For discharging more than 2.5 m^3/sec , use box culvert

$Q = 4\, m^3/sec$, hence use box culvert



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

Velocity in the canal=0.82 $\mathrm{m^3/sec}$

$$\frac{V^2}{2g} = \frac{0.82^2}{2 * 9.81} = 0.034m$$

Discharge in the barrel $Q = 4\, m^3/sec$.

Assume velocity in the barrel = V_1

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

Velocity head$=\frac{V_1^2}{2g}=\frac{16}{2gd^4}$

Length of the barrel$=7.5+26.8+7=41.3m$

1)Friction losses of the barrel$=\frac{2gn^2L}{R^{4/3}}*\frac{V_1^2}{2g}$

$$=\frac{(0.013)^2*L*(4)^{1.33}}{(d)^{4/3}}*V_1^2$$

$=\frac{(0.013)^2*41.3*(4)^{4/3}}{(d)^{4/3}}*\frac{16}{d^4}=\frac{0.709}{d^{5.33}}$ -----(1)

2)Entry & Exit loss:

$$=K_1\left(\frac{V_1^2}{2g}-\frac{V^2}{2g}\right)+K_2\left(\frac{V_1^2}{2g}-\frac{V^2}{2g}\right)$$

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

3)Head loss in two elbows:

$$=2*0.05*\frac{V_1^2}{2g}=2*0.05*\frac{16}{2*9.81*d^4}$$

$=\frac{0.0816}{d^4}$ -----(3)

4)Head loss in two screens

$$=(0.2+0.2)\frac{V^2}{2g}=0.4*\frac{0.82^2}{2*9.81}$$

$=0.0316$ -------------(4)

Total loss$=1+2+3+4=0.2m$

$$0.2=\frac{0.709}{d^{5.334}}+\left(\frac{0.4077}{d^4}+0.017\right)+\frac{0.0816}{d^4}+0.0136$$

$$0.2=\frac{0.709}{d^{5.334}}+\frac{0.4893}{d^4}-0.0034$$

By trail $\Rightarrow$ $d=1.45$m , $\therefore$use dimension of Box $=1.5*1.5\ m$

Chapter 6 Conveyance Structures

Problems

6.1 Design a box culvert with inside dimension (3m*3m) for the following cases:- the box culvert has to carry D.L of $10000\,{}^{N}\!/_{m^2}$, the density of earth is $16000\,{}^{N}\!/_{m^2}$, coefficient of earth pressure $k_a = 1/3$, use t = 30cm , $\phi = 20$

1- L.L & D.L acting on the top & no water pressure from inside.

2- L.L & D.L acting on the top & water pressure acting inside.

3- L.L not acting on the sides of the culvert & water acting inside.

4-L.L & D.L acting on the top and half of water inside the culvert.

6.2A concrete pipe culvert is to be constructed under road to carry a max. Discharge 9m^3/sec. if the length of pipe culvert 10m @ slope 1/100 , find the diameter of the pipe & draw the culvert between the change of discharge & the diameter.

6.3 At km 4+100 from the project there is a crossing road with canal, design a suitable hydraulic structure for this case with the following data:

Q=9.97m^3/sec, u.s.w.l=7.45m, d.s.w.l.=7.35m ,u.s.b.l=4.8m ,d.s.b.l=4.8m, u.s.bed width=5.10m , d.s.bed width=5.10m , average ground level=11m,road width=8m ,and the length=26m.

Assume any missing data.

6.4 Calculate a discharge of a box culvert with three opening (2×2)m, (3×3)m, (2×2)m the length of each piece=2.5m, if the total head loss of the structure =0.35m and the length =33m, Ki=0.3, Ko=1.0, n=0.015 depth of earth filling 3.5m , how many pieces are you need to pass the water under the road.(Sketch the structure).

6.5 Design a flume aqueduct with write the steps of solution to a cross an open drain 7m deep , 3m bed width , 2:1 side slope , the discharge of canal 2.5m^3/sec. with side slope 1:1.5 and depth of flow 1m , the head loss not exceed 0.2m , the slope of the canal 25cm/km , the water way of the aqueduct 30m , Manning's coefficient for canal 0.015and for aqueduct 0.013.Assume any suitable data if you need.

Chapter 6 Conveyance Structures

6.6. Design an aqueduct at (D , 0.75D) where D is the diameter of aqueduct to pass half of flow from lined canal with 1.5:1 side slope to cross an open drain 5m deep , and free board 40cm with slope 25 cm/km , the canal section is the same at upstream and downstream with difference water level 0.3m , the length of the aqueduct 25m , k_1=0.5 , k_2=1,steel stress =93000kN/m^2 , assume any value to solve the problem

6.7 Design a twin opening of an inverted siphon (8+27+6)m required to pass canal discharge of 3m^3/sec under road with

0.18m head loss .The velocity in the canal is 0.78m/sec and the depth water in the canal is 1.4m , safety screens is provided of entry and exit . The inverted siphon has 22.5^o elbows of each end if n=0.013 , k_i=0.2 , k_o=0.3 , k_{screen}=0.2 and k_{elbow}=0.05.

(sketch the structure)

6.8. An inverted siphon at a twin opening was used to pass a discharge of canal under the main road with 0.2m head loss. The inner dimensions of this siphon are (1.5×1.5)m , safety screen was used at entry and exit . Two elbows of 22.5^o were used in this siphon.Find the discharge of canal , using the following site dimentions:

Manning (n) of siphon=0.13 , cross section area of canal = 6.8m^2

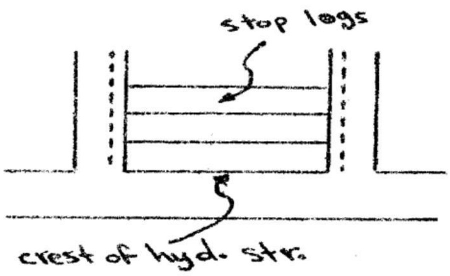
, k_{entry}=0.2 , k_{exit}=0.3 , k_{elbow}=0.05.Length of siphon = (40+70+4)m. (Sketch the structure).

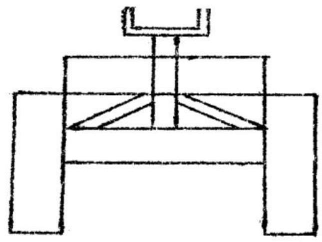
Chapter 7 Design Of Gates

7.1Types of gates:-

1- Stop logs:

2- Sliding gates or vertical lifting gate.





The design of vertical gate would involved the design of the following:

1- Gate leaf 2-Groove embedment 3- Heisting equipment's.

7.2 Principle for design of lifting gate:-

1- The gate will act as slab support on two walls.

2- The total load was not uniform distributed but it various from top to bottom

3- The total load transmitted to a stiffener member is equal to the area of loading may equal to the area of loading member

The loading may be equally divided into N number (N part), each stiffener is located such that it carry a total load equal to the calculated

$$A = \frac{1}{2}\gamma W h^2$$

$a = \frac{1}{2} y n^2$

Procedure for design:-

1- Find the water depth.

- Front side water level $(F.s.W.L)$

- Bed level $(B.L)$ under gate.

2- Height of gate= $water\ depth + 0.1 * water\ depth$

$$L = S * 1.1$$

3- Draw pressure diagram.

Pressure at any point= $\gamma W * depth\ of\ water$

4- Assume n division.

5- Find the location of stiffener from principle each.

6- Bending moment calculation.

Total load stiffener (beam) equal (w)

$$W = \gamma W * a * L$$

- Stiffener is simply supported.
- The stiffener it can be angle, I beam or channel.

$$B.M = \frac{W.L}{8}$$

7- Find Z (The section modulus= $\frac{M}{f_s}$, $Z = \frac{I}{c} = \frac{bd^2}{6}$

∴ The plate as continuous slab$\left(\frac{W.L^2}{10}\right)$

8- Calculate the thickness of plate. Consider a unit of 1m width of slab spacing vertically & supported on the stiffener.

Find the max. Bending moment

$M = fs * Z$

$M_{max.} = f_s * \frac{bd^2}{6}$ *then* Find t

Example 7.1
Design vertical gate with the following information:

$F.S.W.L. = 35.25m$

$Crest\ level = 32.75m$

$Width\ of\ gate = 3m$

$f.s = 20\ 000\ psi$

Solution

$Water\ depth = 35.25 - 32.75 = 2.5m$

$2.5 * 0.1 = 0.25m$

$\therefore\ Height\ of\ gate = 2.5 + 0.25$

$= 2.75m$

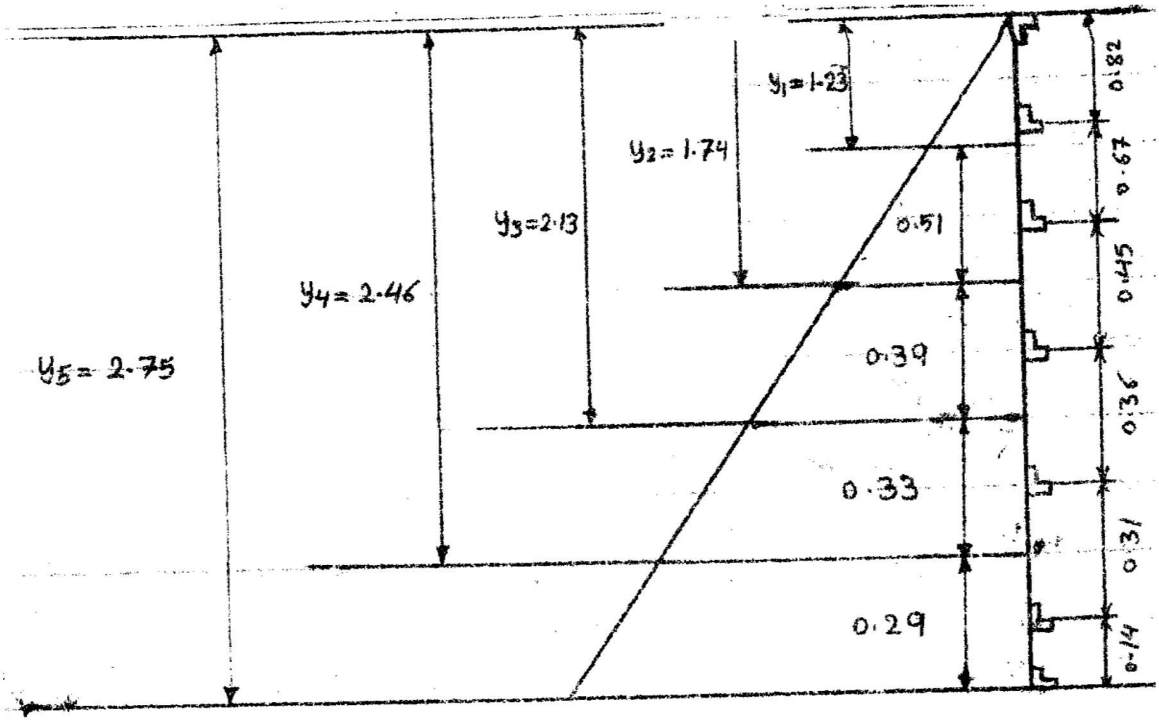
Assume the number of division (n) = 5

$$a = \frac{y^2}{2n} = \frac{(2.75)^2}{2*5} = 0.756m^2$$

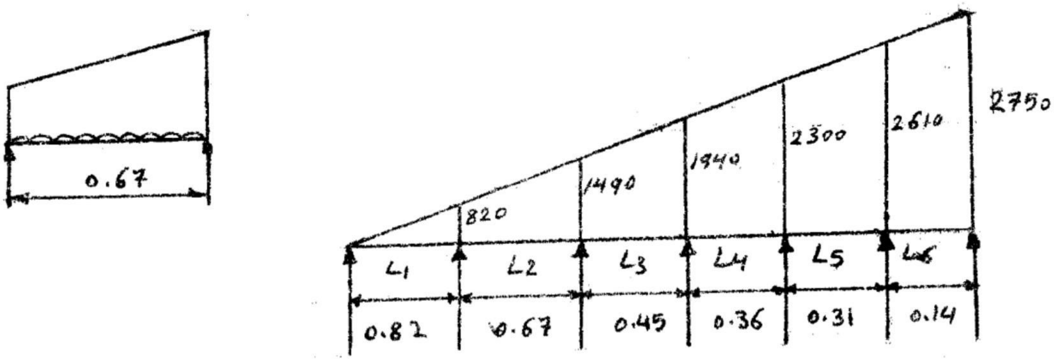
$$a_1 = \frac{y_1^2}{2} = 0.756 \quad \Rightarrow \quad y_1 = 1.23m$$

Area	Formula	Value of y	Division Height
0.756	½ y_1^2	y_1 = 1.23	1.23
1.512	½ y_2^2	y_2 = 1.47	0.51
2.268	½ y_3^2	y_3 = 2.13	0.39
3.024	½ y_4^2	y_4 = 2.46	0.33
3.78	½ y_5^2	y_5 = 2.75	00.29
			$\sum = 2.75$

Chapter 7 | Design Of Gates



Calculation of plate thickness:



Consider a unit of 1.0m

Width of slab spacing

Vertical & supporting on the stiffeners.

Loads on span L_1 , L_6 are about half strip

L_2 , L_3 , L_4 , L_5 are equal & about on fifth (1/5) of the total area.

Chapter 7 Design Of Gates

Assume an average uniform load on each strip , the total load (w) on each of them is the same.

The biggest moment will be the one with largest span.

Note: For accurate solution use moment distribution for determining (B.M.).

$$w = 0.67\left(\frac{820+1490}{2}\right) = 775kg$$

$$M = \frac{wl}{10} = \frac{775*0.67}{10} = 52kg.m = 4580 in.Ib$$

B.M(in.Ib) = B.M(T.M) * 88000

Use fs = 20000psi

Section modulus = $z = \frac{M}{fs}$ ....(1)

$z = \frac{bd^2}{6} = \frac{bt^2}{6} = \frac{40t^2}{6}$ ....(2) b = 1m = 40in

$$M = 4580 = \frac{20000*40*t^2}{6} \qquad (M = FS.Z)$$

$T = 0.185 = \frac{3}{16} \Rightarrow$ use (3/16)in

Design of stiffeners: (Beam Design)

Each stiffener is simply supported beam

$M = \frac{WL}{8}$ w: (total load)

$Area\ of\ one\ strip = 0.756m^2$

$W = \gamma w * a * L$

W=0.756 * 1000 * 3

W=2268 kg=2.268

$$M = \frac{WL}{8} = \frac{2.268 * 3}{8} = 0.851 T.m = 0.851 * 88000$$

$$= 74888 \, Ib.inch$$

$$Z = \frac{M}{fs}$$

$$Z = \frac{74888}{20000} = 3.744 \, inch^3$$

The stiffener can be angle , I beam

Problems

7.1. Design a sliding gate wing type of beams (W8×40) which have an elastic section modulus equal to 582cm^3. If the height of gate 5m , span 3.5m and fs = 10000 T/m^2?

7.2. Calculate the elastic modulus section for the stiffener of regular sliding gates , if the structure have five opening with water way 17.5m , the height of the gates is 5m with fs=10000 ton/m^2, using four division.

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