HYDRAULIC DESIGN OF JUBLEE KHAL

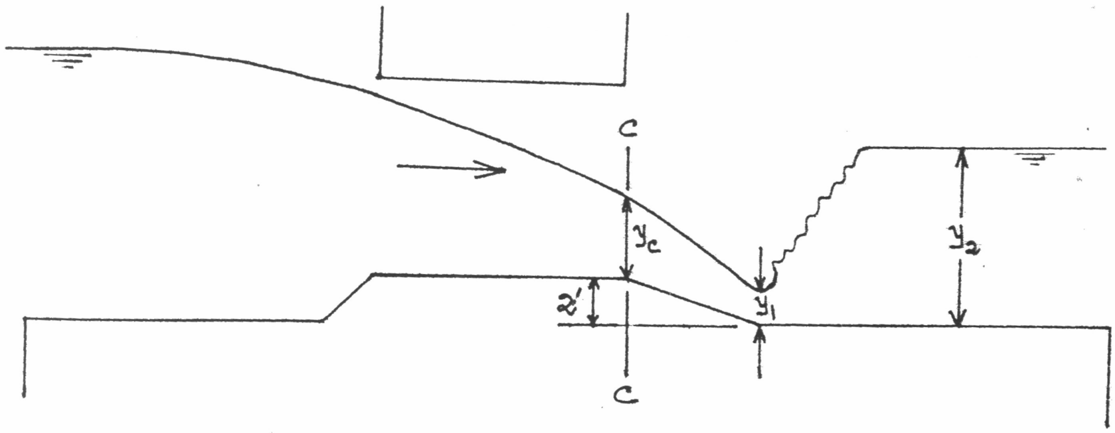
Discharge, Q = 1200 cfs.

No. of vents = 4

Size of vent = 5'x 6'

Unit discharge, $q = \dfrac{1200}{4\times5} = 60$ cfs/ft

Critical depth, $Yc = (q^2/g)^{1/3} = (60^2/32.2)^{1/3} = 4.82$ ft.



Applying Bernoullis Equation between the critical section c-c and sec. 1-1.

$$Y_c + \frac{V_c^2}{2g} + Z = Y_1 + \frac{V_1^2}{2g}$$

$b_1 = (4\times5') + (3\times1.25') + 2\times6\times\tan 10^o = 25.86$ ft.

$q_1 = 1200/25.86 = 46.40$ cfs/ft

At the critical state $\dfrac{V_c^2}{2g} = \dfrac{Y_c}{2}$ and $V_1 = q_1/y_1$

Assuming Z = 2 ft.

So eq. (1) becomes,

$$Y_c + \frac{Y_c}{2} + Z = Y1 + \frac{q_1^2}{2gy_1^2}$$

$$\text{or, } 4.82 + \frac{4.82}{2} + 2 = Y_1 + \frac{46.40^2}{2\times 32.2\times y_1^2}$$

$$\text{or, } 9.23 = y_1 + \frac{33.43}{y_1^2}$$

By trial and error, we get $y_1 = 2.18$ ft.

Since $Y_1 < Y_c$, the flow is supercritical.

$V_1 = q/y_1 = 46.40/2.18 = 21.28$ fps

$F_1 = V_1/(gy_1)^{0.5} = 21.28/(32.2 \times 2.18)^{0.5} = 2.50$ OK

$$\frac{Y_2}{Y_1} = \frac{1}{2}\left[(1+8F_1^2)^{0.5} - 1\right]$$ [Ref: Open Channel Hydraulics - Ven Te Chow]

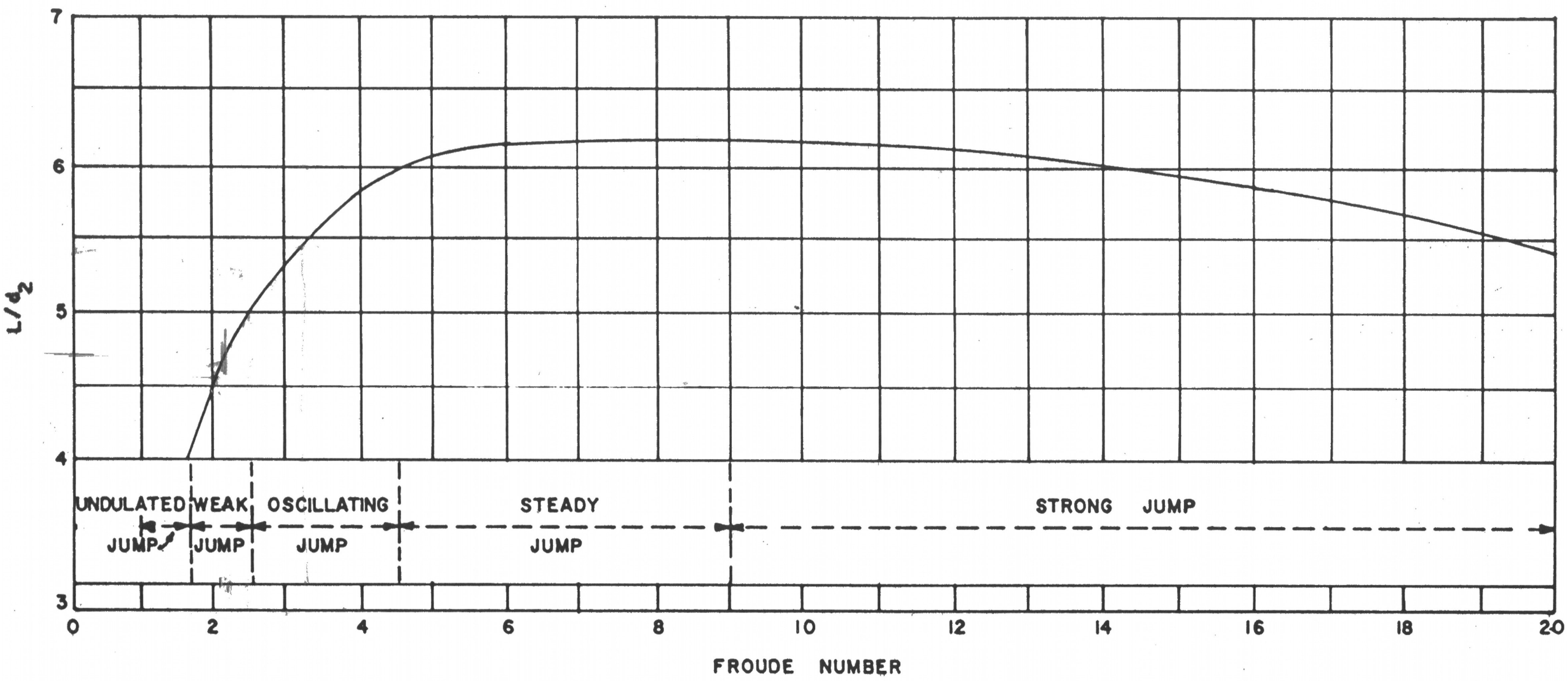
$$= \frac{1}{2}\left[(1+8\times 2.5^2)^{0.5} - 1\right]$$

$$= 3.07$$

$Y_2 = 3.07\ Y_1 = 3.07 \times 2.18 = 6.7$ ft

From curve L/Y_2 vs. F (Fig.23).

$L/Y_2 = 5.0$



LENGTH OF JUMP

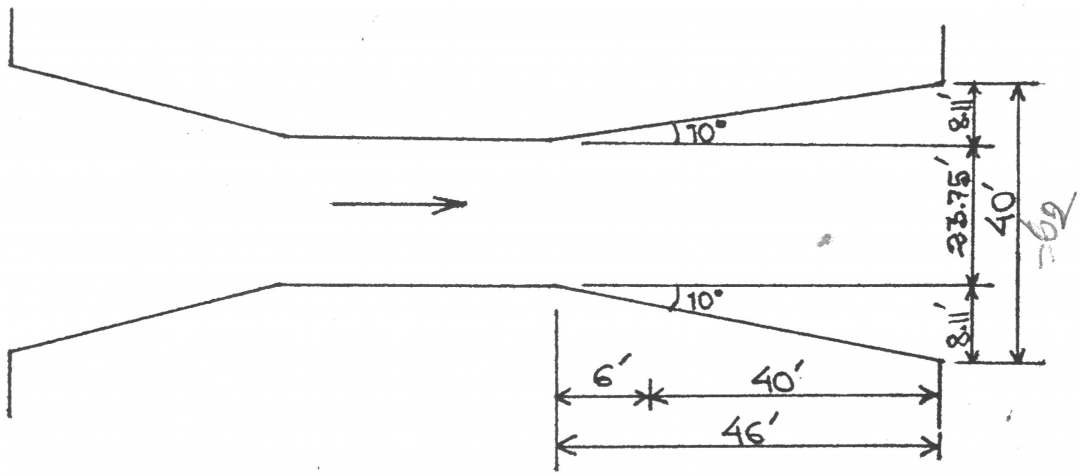
Figure - 23

$$L = 5.0\, Y_2 = 5.0 \times 6.7 = 33.5 \text{ ft.}$$

Make it 40 ft.

Scour Depth

Assuming : Angle of flaring of d/s wing wall = 10^{o}
Slope of d/s glacis = 1:3



Width of flow at the end of river side

$$b_2 = (5' \times 4) + (1.25' \times 3) + 2 \times 46' \times \tan 10^{o} = 40 \text{ ft.}$$

$$\text{Discharge at exit, } q_2 = \frac{1200}{40} = 30 \text{ cfs/ft}$$

Depth of scour is determined by Lacey's regime scour depth

$$R = 0.91\, (q_2^{2}/f)^{1/3}$$

where, silt factor, $f = 1.76\, (dmm)^{0.5}$

From grain size distribution, dmm = 0.029

$$\text{i.e. } f = 1.76\, (0.029)^{0.5} = 0.30$$

$$R = 0.91\, (q_2^{2}/f)^{1/3} = 0.91\, (30^{2}/0.3)^{1/3} = 13.12 \text{ ft.}$$

Scour depth at upstream = 1.25 R = 1.25 x 13.12 = 16.4 ft

Scour depth at downstream = 1.50 R = 1.50 x 13.12 = 19.7 ft

[Ref : Irrigation Engineering and Hydraulic Structures
-Santosh Kumar Garg]

4/5 water level

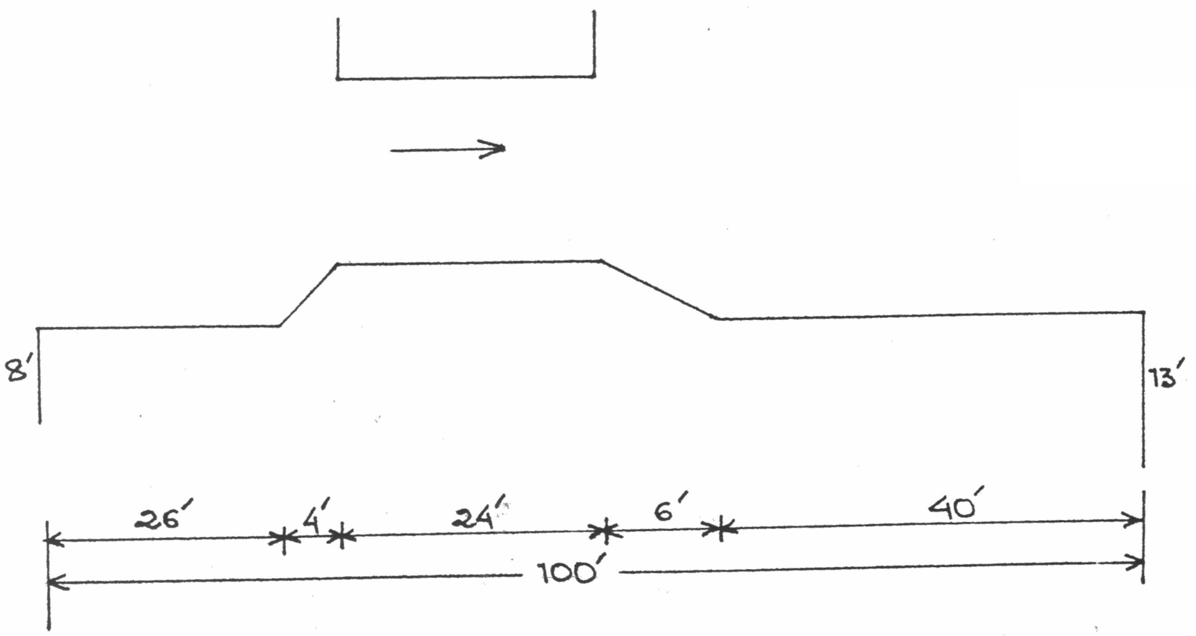
Depth of upstream cutoff = 16.4 - 8 = 8 ft.
Depth of downstream cutoff = 19.7 - 6.7 = 13 ft.

Floor length

Length of u/s glacis = 4 ft. [with 2 ft drop and 1:2 slope]
Lenght of barrel = 4+1+14+1+4 = 24 ft.
Length of d/s glacis = 6 ft [with 2 ft. drop and 1:3 slope]
Length of d/s apron = 40 ft.
Length of u/s apron = 40 x 2/3 = 26 ft.

Total floor, length, b = 26+4+24+6+40 = 100 ft.

Now we will check this floor length whether it is adquate with respect to exit gradient.



CHECK FOR EXIT GRADIENT

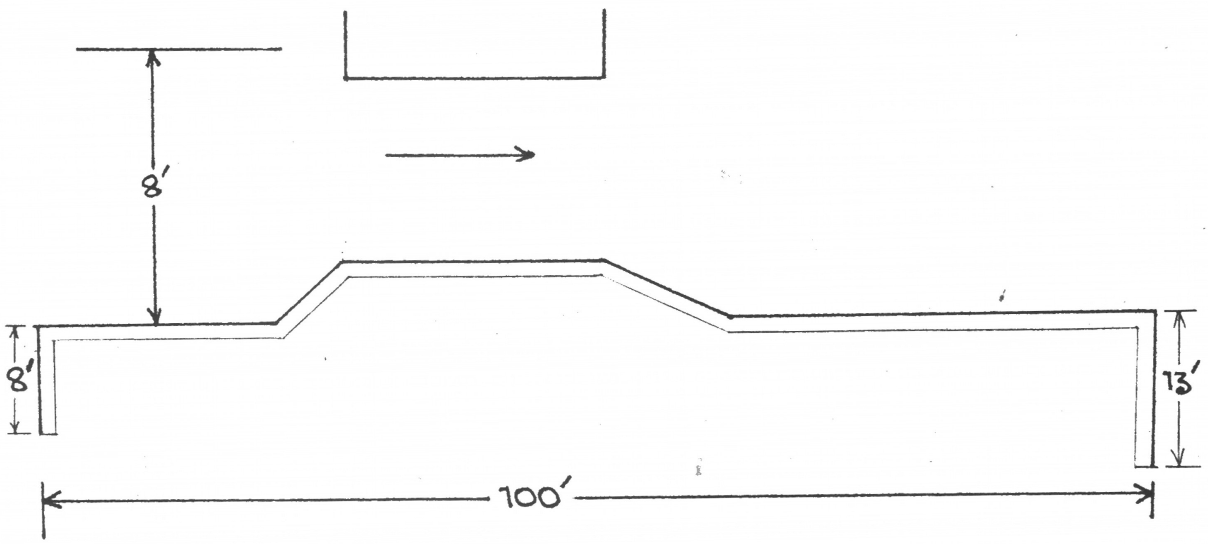
$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{L}}$$

where, H = maximum difference of water level between country side and river side

d = depth of cutoff wall

$$L = \frac{1+(1+a^2)^{0.5}}{2} \quad \text{where } a = \frac{b}{d}$$

b = total apron length



$H = 8'$ u/s cutoff depth, $d_1 = 8'$

$b = 100'$ d/s cutoff depth, $d_2 = 13'$

The exit gradient at the downstream end,

$$G_E = \frac{H}{d_2} \times \frac{1}{\pi\sqrt{L}}$$

$$a = \frac{b}{d_2} = \frac{100}{13} = 7.69$$

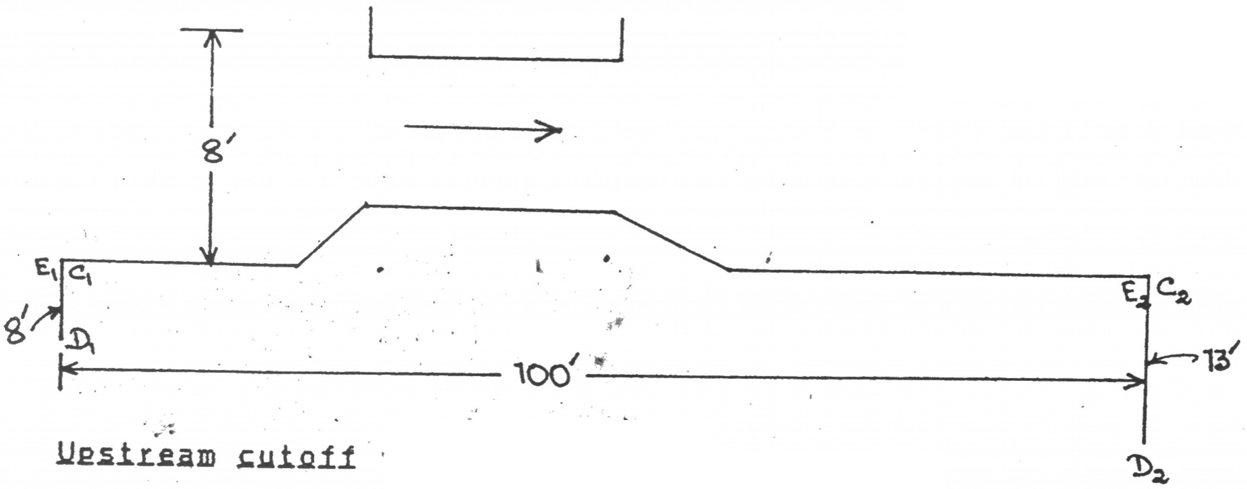
$$L = \frac{1 + (1 + 7.69^2)^{1/2}}{2} = 4.38$$

$$G_E = \frac{8}{13} \times \frac{1}{\pi \sqrt{4.38}}$$

$$= 0.093 < 0.143 \quad [1/7 = 0.143]$$

Hence O.K.

Uplift Pressure Calculation



Upstream cutoff

b = 100'

d = 8'

1/a = d/b = 8/100 = 0.08

From fig. 24

PE_1 = 100%

PE = 26% $\quad PC_1$ = 100-PE = 100-26 = 74%

PD = 17% $\quad PD_1$ = 100-PD = 100-17 = 83%

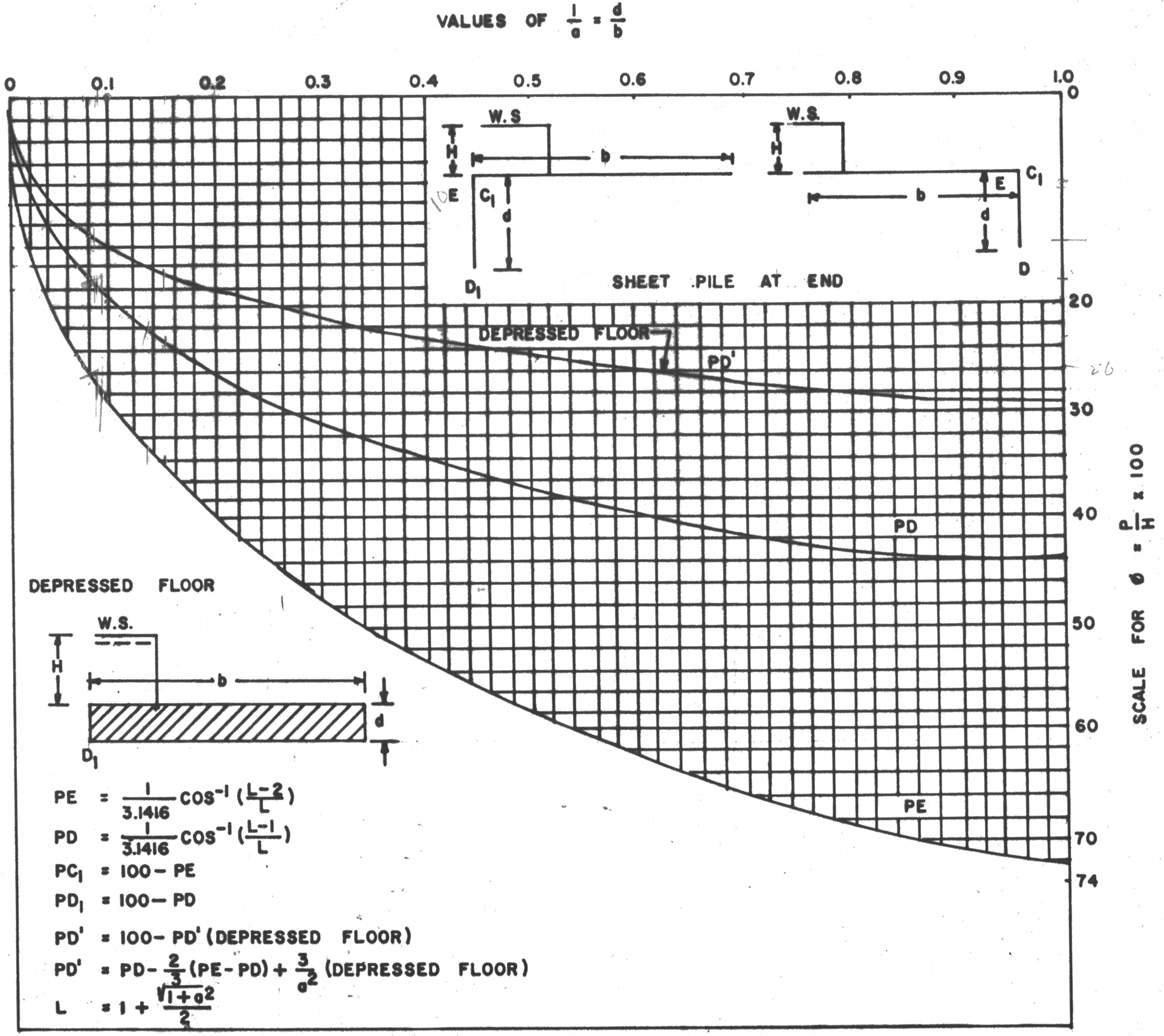


Figure – 24

Figure – 24

Corrections for PC_1

a) Effect of downstream cutoff on upstream cutoff:

$$\text{Correction} = 19\left[\frac{D}{b'}\right]^{1/2}\left[\frac{d+D}{b}\right]$$

Where, D = depth of downstream pile
d = depth of upstream pile
b'= distance between two piles
b = total floor length

Assumed 2 ft. thickness throughout the floor length.

D = 13 - 2 = 11 ft.
d = 8 - 2 = 6 ft.
b' = 100 - 2 = 98 ft.
b = 100 ft.

$$\text{Correction} = 19\left[\frac{11}{98}\right]^{1/2}\left[\frac{6+11}{100}\right] = 1.08\ \%\ \ (+ve)$$

b) Correction for depth :

$$\text{Correction} = \frac{PD_1 - PC_1}{d} \times \text{thickness}$$

$$= \frac{83 - 74}{8} \times 2$$

$$= 2.25\ \%\ (+ve)$$

PC_1 (corrected) = 74 % + 1.08 % + 2.25 %
= 77.33 % ≈ 77 %

<u>Downstream cutoff</u>

b = 100'
d = 13'

1/a = d/b = 13/100 = 0.13

From fig. 24

$PC_2 = 0\%$

$PE_2 = 33\%$

$PD_2 = 22\%$

Corrections for PE_2
----

a) Effect of upstream cutoff on downstream cutoff:

$$\text{Correction} = 19 \left[\frac{D}{b'}\right]^{1/2} \left[\frac{d+D}{b}\right]$$

where, $D = 8 - 2 = 6$ ft.
$d = 13 - 2 = 11$ ft.
$b' = 100 - 2 = 98$ ft.
$b = 100$ ft.

$$\text{Correction} = 19 \left[\frac{6}{98}\right]^{1/2} \left[\frac{11+6}{100}\right] = 0.80\ \%\ (\text{-ve})$$

b) Correction for depth:

$$\text{Correction} = \frac{PE_2 - PD_2}{d} \times \text{thickness}$$

$$= \frac{33 - 22}{13} \times 2$$

$$= 1.69\ \%\ (\text{-ve})$$

PE_2 (corrected) = 33 % - 0.80 % - 1.69 % = 30.51 % $\approx$ 31 %

Upstream pile

$PE_1 = 100\ \%$

$PD_1 = 83\ \%$

$PC_1 = 77\ \%$

Downstream pile

$PE_2 = 31\ \%$

$PD_2 = 22\ \%$

$PC_2 = 0\ \%$

WATER LEVEL FREQUENCY CURVE

RIVER : BARNAI

STATION : NALDAGA

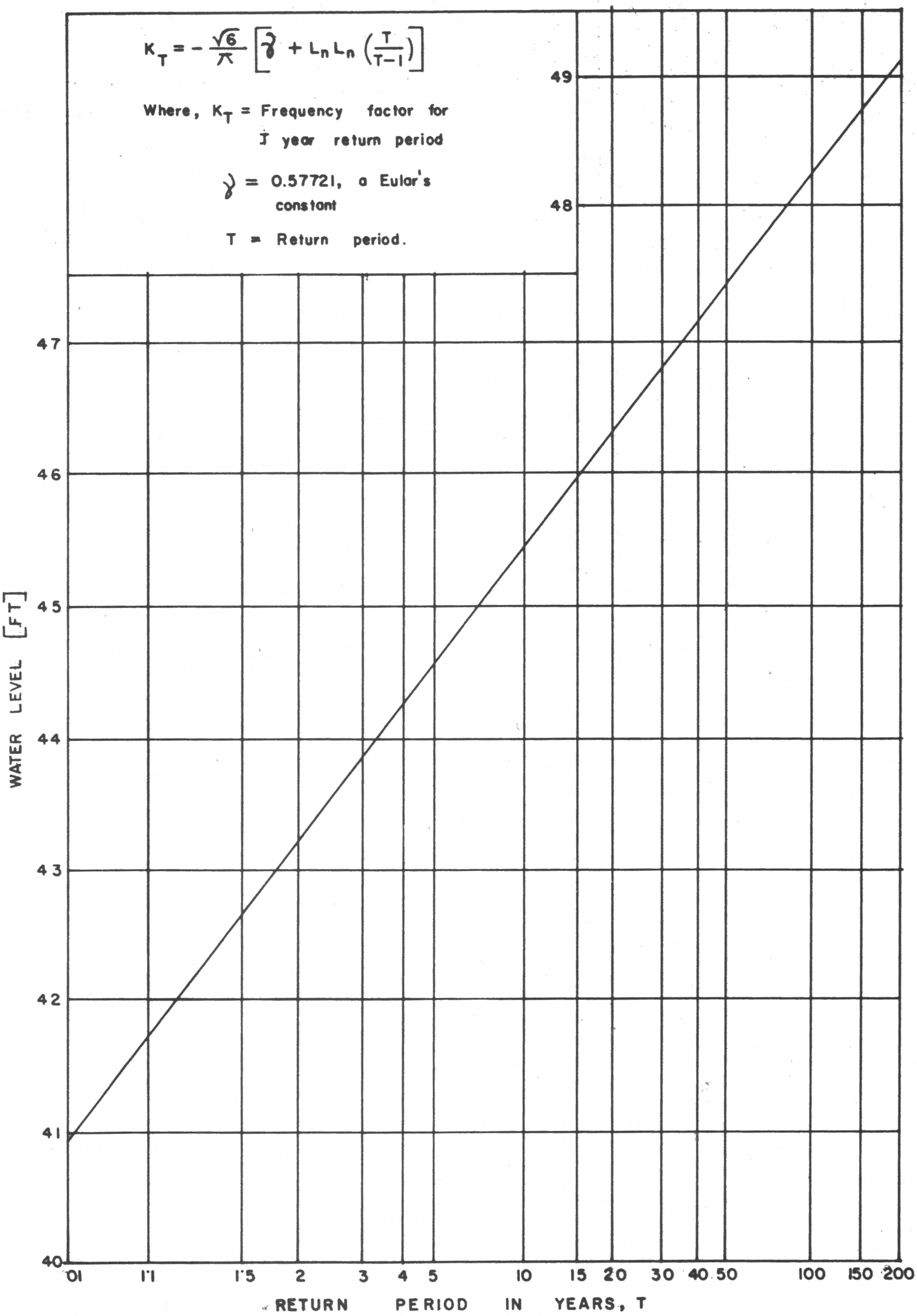


Figure 25

20 years flood (Fig.25) = + 46.30

C/s water level (July 31) = + 41.55

Head for designing country side floor thickness

= 46.30 - 41.55 = 4.75 ft. 5 ft.

$H = 5'$ u/s cutoff depth, $d_1 = 13'$

$b = 100'$ d/s cutoff depth, $d_2 = 8'$

The exit gradient at the downstream end,

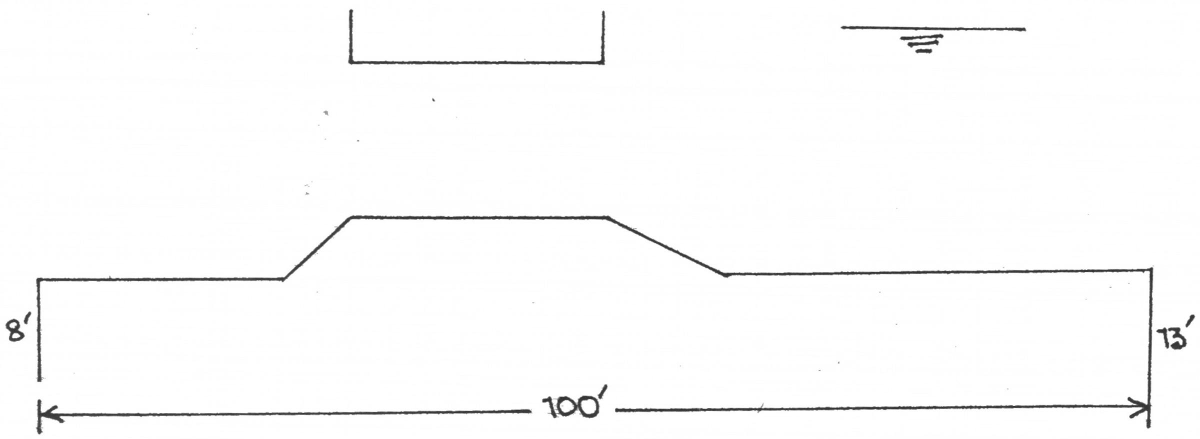
$$G_E = \frac{H}{d_2} \times \frac{1}{\pi \sqrt{L}}$$

$$a = \frac{b}{d_2} = \frac{100}{8} = 12.50$$

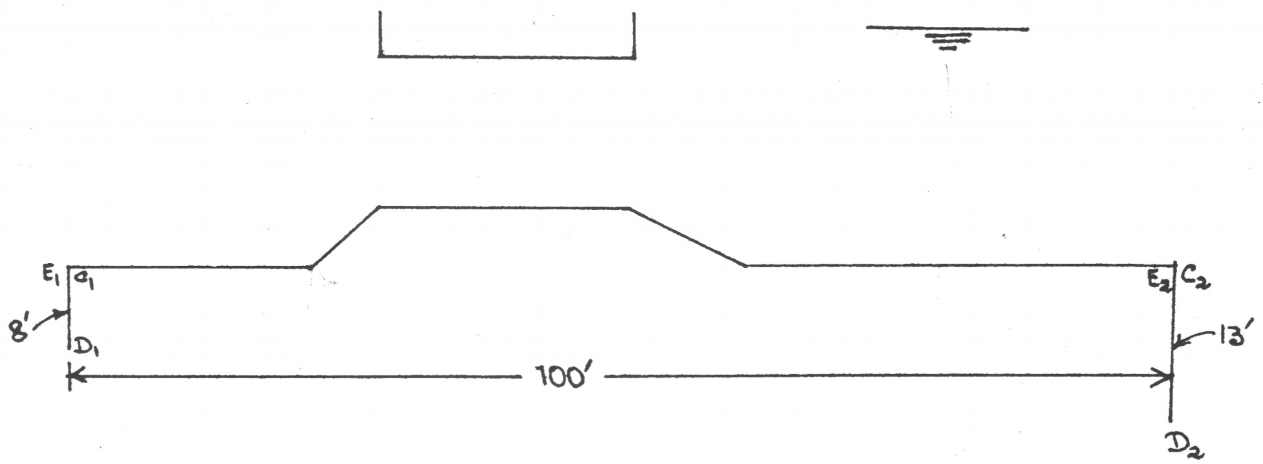
$$L = \frac{1 + (1 + 12.50^2)^{1/2}}{2} = 6.77$$

$$G_E = \frac{5}{8} \times \frac{1}{\pi \sqrt{6.77}}$$

$= 0.076 < 0.143$ [1/7 = 0.143]



Uplift Pressure Calculation



Upstream cutoff

b = 100'

d = 13'

1/a = d/b = 13/100 = 0.13

From fig. 24

PE_1 = 100%

PE = 33% PC_1 = 100-PE = 100-33 = 67%

PD = 22% PD_1 = 100-PD = 100-22 = 78%

Corrections for PC_1

a) Effect of downstream cutoff on upstream cutoff:

$$\text{Correction} = 19\left[\frac{D}{b'}\right]^{1/2}\left[\frac{d+D}{b}\right]$$

Where, D = depth of downstream pile
d = depth of upstream pile
b'= distance between two piles
b = total floor length

Assumed 2 ft. thickness throughout the floor length.

$D = 8 - 2 = 6$ ft.
$d = 13 - 2 = 11$ ft.
$b' = 100 - 2 = 98$ ft.
$b = 100$ ft.

$$\text{Correction} = 19 \left[\frac{6}{98}\right]^{1/2} \left[\frac{11+6}{100}\right] = 0.80\ \%\ (+ve)$$

b) Correction for depth :

$$\text{Correction} = \frac{PD_1 - PC_1}{d} \times \text{thickness}$$

$$= \frac{78 - 67}{13} \times 2$$

$$= 1.69\ \%\ (+ve)$$

PC_1 (corrected) $= 67\ \% + 0.80\ \% + 1.69\ \%$
$= 69.5\ \%$
$\approx 70\%$

__Downstream cutoff__

$b = 100'$
$d = 8'$

$1/a = d/b = 8/100 = 0.08$

From fig. 24

$PC_2 = 0\%$

$PE_2 = 26\%$

$PD_2 = 17\%$

Corrections for PE_2

a) Effect of upstream cutoff on downstream cutoff:

$$\text{Correction} = 19 \left[\frac{D}{b'}\right]^{1/2} \left[\frac{d+D}{b}\right]$$

where, $D = 13 - 2 = 11$ ft.
$d = 8 - 2 = 6$ ft.
$b' = 100 - 2 = 98$ ft.
$b = 100$ ft.

$$\text{Correction} = 19 \left[\frac{11}{98}\right]^{1/2} \left[\frac{6+11}{100}\right] = 1.08\ \%\ (\text{-ve})$$

b) Correction for depth:

$$\text{Correction} = \frac{PE_2 - PD_2}{d} \times \text{thickness}$$

$$= \frac{26 - 17}{8} \times 2$$

$$= 2.25\ \%\ (\text{-ve})$$

$$PE_2\ (\text{corrected}) = 26 - 1.08 - 2.25 = 22.67\ \% \approx 23\ \%$$

Upstream pile	Downstream pile
$PE_1 = 100\ \%$	$PE_2 = 23\ \%$
$PD_1 = 78\ \%$	$PD_2 = 17\ \%$
$PC_1 = 70\ \%$	$PC_2 = 0\ \%$

$$\text{Specific weight of concrete, } SWC = \frac{UWC - UWW}{UWW}$$

where, UWC = unit weight of concrete
UWW = unit weight of water
SWc = specific weight of concrete

$$SWc = \frac{150-62.4}{62.4} = 1.40$$

Thickness of the floor slab

$$\text{Point a} \quad t = \frac{5 \times 0.23}{1.40} = 0.82'$$

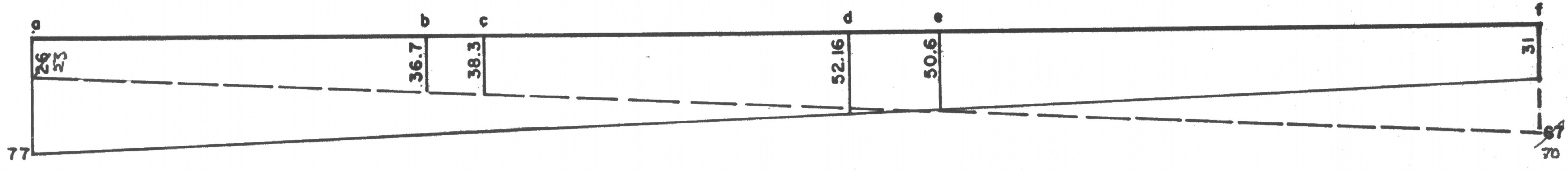
$$\text{" b} \quad t = \frac{5 \times 0.367}{1.40} = 1.31'$$

$$\text{" c} \quad t = \frac{5 \times 0.383}{1.40} = 1.37'$$

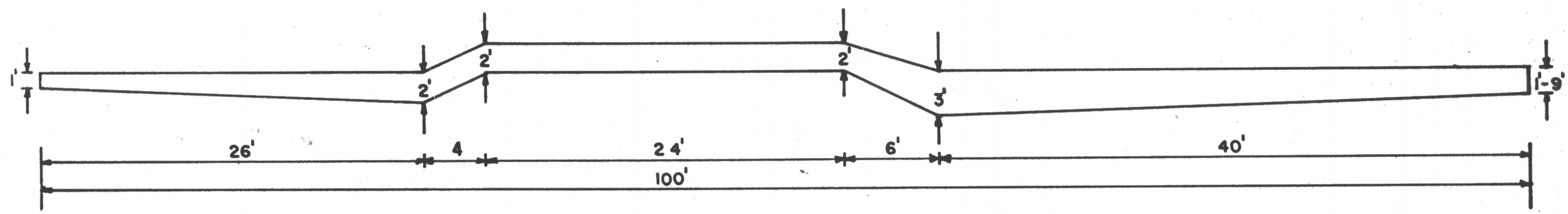
$$\text{" d} \quad t = \frac{8 \times 0.5216}{1.40} = 2.98'$$

$$\text{" e} \quad t = \frac{8 \times 0.506}{1.40} = 2.89'$$

$$\text{" f} \quad t = \frac{8 \times 0.31}{1.40} = 1.77'$$



PERCENTAGE OF PRESSURES



THICKNESS OF THE FLOOR SLAB

REFERENCES

1. Hydrologic & Hydraulic design procedure for drainage structure by Design Directorate (Water) Dhaka.

2. Flood routing through drainage structures, by M.F.A. Siddique & W.M. Emerson.

3. Estimate Flood Peaks for small drainage basins, by M.F.A. Siddiqui & W.M. Emerson.

4. Conditions of Flow, by M.F.A. Siddiqui & W.M. Emerson.

5. Design Report of Jublee Khal Regulator (4 vents), by A.N.M. Wahedul Huq.

CHAPTER VII

HYDRAULIC DESIGN OF DRAINAGE SLUICE (TIDAL) AT BAMNADANGA IN KHULNA

Design of tidal drainage sluice

Introduction

A drainage sluice is a structure through which excess water from an area within finite boundaries are to drain off. The number of standard 5'x6' vents is based on drainage requirements determined by catchment area, the assumed rainfall, and the tide lockage. In addition to catchment area consideration must be given to inflow from outside the area through spill channels. To design a tidal drainage sluice one must be familiar to the following terms.

Tide lockage

The tide lockage is determined by the range of tide and the elevation to which it is proposed to drain the country side of the embankment. This elevation has been taken usually one foot higher than the average ground level on the assumption that crops can withstand this amount of flooding for short periods. Tide lockage occurs from this elevation upto the elevation of high tide as the water outside the dykes is then higher than the water on the inside. No drainage takes place until the tide falls bellow the elevation of the water on the country side behind the dykes.

Discharge through sluice

The control for the discharge through the sluice is at the inlet when the tailwater elevation is lower than one half of the sum of the critical depth plus the height of the vent. This relationship is designated by the formula $\frac{Dc + D}{2}$. This means the discharge is constant and independent of the tailwater fluctuations until the tailwater raises above this elevation which is designated by the height $\frac{Dc + D}{2}$ above the horizontal flow line of the vent. As the tailwater continues to rise the tailwater assumes control of the discharge. The discharge then decreases rapidly and becomes zero as the tide level becomes at a height of average country side ground level plus one foot as assumed.

The amount of discharge possible for each culvert is to be determined through the use of the curves based on experiments and published by the U.S. Bureau of Public Roads. The relationship between tide lockage and the amount of drainage possible with respect to ground level and vent height during a tide cycle is shown by Fig. 1. For simplicity the tide curve is shown as a straight line (Fig. 2) from low tide to high tide and back down to low tide, over a tide cycle period of twelve hours. This makes the tide curve a triangle with the apex at the high tide, the ordinate being elevation and the abscissa a twelve hour period. Also plotted on the ordinate scale is the drainage discharge in cubic feet per second. The discharge is constant

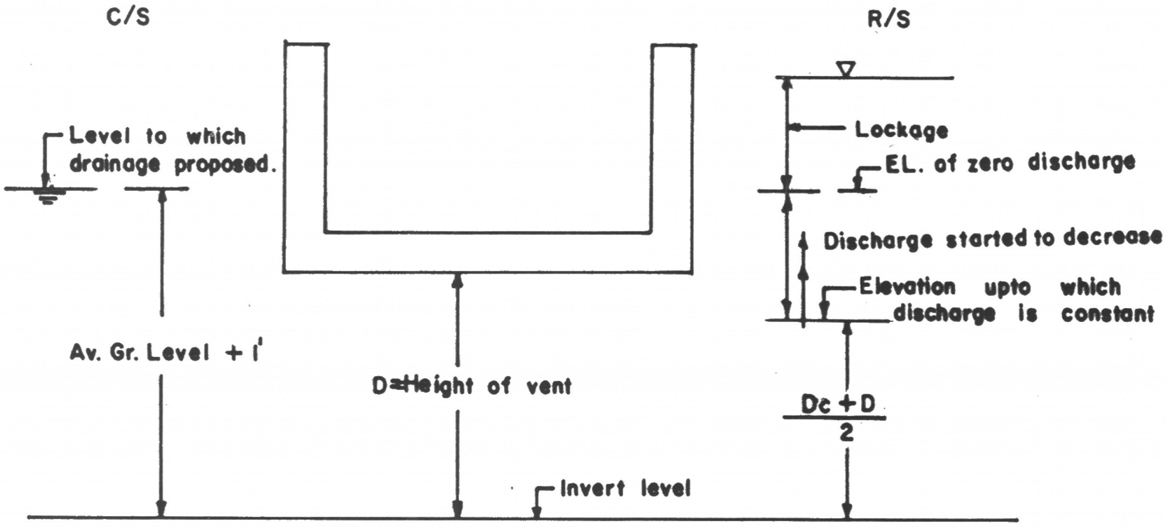
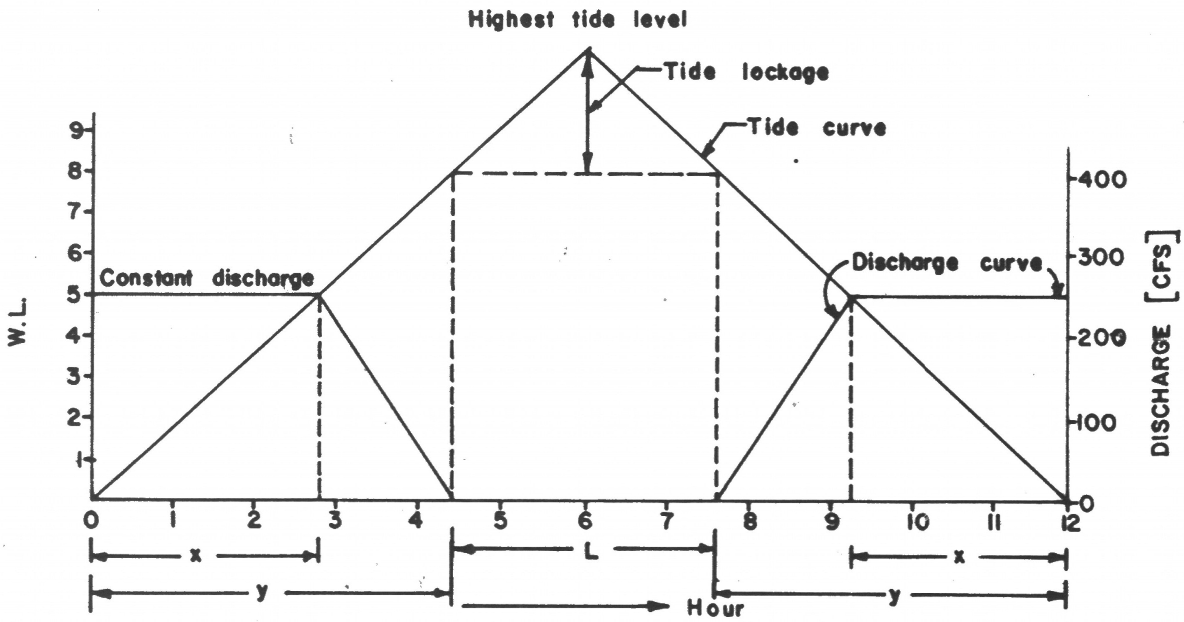


FIG. 1 Elevation of zero discharge, constant discharge and lockage



2x = Time through which discharge is constant.

2y = Total time through which discharge takes place.

L = Lockage period.

FIG. 2. TYPICAL TIME DISCHARGE CURVE

during the time the tide rises from its low point to elevation $\frac{D + Dc}{2}$. The length of this time is x. As the tide continues to rise the discharge is controlled by the tail water, the discharge diminishes very rapidly and becomes zero at y hours from the point of low tide, while the corresponding tide water level is equal to average ground level plus one feet. Above this level upto highest tide level is the lockage period. The drainage curve is duplicated in reverse order on the falling tide.

The area under the curve represents the total volume of drainage flow. Tide lockage is L hours for each tide cycle or approximately 2L hours/day.

<u>Data required to design a tidal drainage sluice</u>

1. A brief description of the project and the purpose it is to serve.

2. A topographic map of the entire drainage basin area.

3. Rainfall record within the basin.

4. Estimate of percentage of paddy & non paddy lands.

5. Tide records for atleast one year.

6. Profile and cross-section of the main drainage channel for atleast one mile upstream of the structure.

PROCEDURE

1) The geological location and size of basin is to be located from the topographic maps.

2) The geographical centre (centroid) of the basin area is to be located. The basin centroid can be found by vertically suspending a cardboard cutout of the basin shape successively from three or more points and finding the intersection of plumb lines from each point.

3) The location of the basin is to be find out on the isohyetal map of 4-month rainfall index of Bangladesh presented by IECO the general Consultant of BWDB, to get 4 month rainfall index of the area.

4) The 4-month rainfall index is to be multiplied by daily combined indices from Table 1 of frequency 10 yrs or 25 years. This gives the accumulated point rainfall in inches for storms of 1,2,3,4 & 5 days.

Table 1.

Storm Frequency for 10 yrs and 25 yrs.

DAYS	STORM FREQUENCIES	
	10-year	25-year
1	0.128	0.153
2	0.192	0.228
3	0.230	0.272
4	0.257	0.303
5	0.276	0.326

5) Equivalent uniform depth of rainfall

The equivalent uniform depth of rainfall is defined as the depth of water which results from spreading the total volume of basin rainfall uniformly over the total basin area. The IECO Master Plan states that a fairly good relationship exists between depth of rain and distance from the storm centre. Table 2 shows the relationship in tabular form. Basin centroid is assuemed as storm centre.

Table 2.

Rainfall Intensities with distance from storm centre.

Distance from storm Centre (mile)	5-day storm percentage of point rainfall
1/4	100
1	88.0
2	81.7
3	77.5
4	74.2
5	72.0
6	69.8

The procedure for determining the uniform equivalent depth of rainfall is as follows:

a) The geographical centre of the basin is to be located.

b) Concentric circles or isohyetals at even miles intervals around the centre is to be drawn.

c) The areas between the respective isohyetals within the basin area is planimetered.

d) The area is to be multiplied by the proper percentage from table 2.

e) A summation of the values from step (d) is to be made and divide by the total area.

f) The result is a percentage that when multiplied with point rainfall gives the equivalent uniform depth of rainfall.

6) The accumulative total of equivalent uniform depth of rainfall is to be separated into daily increments. The daily increments is then arranged in an order (arbitrary) giving the worst of flood condition. A unit time interval (3 hrs or 6 hrs) is to be chosen for breaking down the daily increments.

The smallest time interval to be used will depend on the length of main drainage course within the basin. The following intervals are recommended:

1. Less than 2 miles; one hour
2. 2 to 6 miles ; 3 hours
3. Over 6 miles ; 6 hours

7) Determination of rainfall losses:

To determine rainfall losses, the land is to be classified as percentage of area in Paddy land & Non-paddy land. The losses are assumed as follows:

a) Initial soil moisture loss:

Paddy land : No initial loss under all conditions. It is assumed the soil is saturated.

Non paddy land: 0.50 inch

b) Subsequent Soil-Moisture loss:

Paddy land and Non paddy land: under all conditions a constant rate of infiltration of 0.04"/hr or 1.00"/day is to be assumed.

c) Depression storage:

Paddy land: under all conditions the first 4" of rain falling on paddy land is assumed to go into storage.

Non paddy land: A maximum constant storage rate of 0.033 inch/hour provided the rainfall is available, until a total of 1.00 inch is stored to be assumed.

8) Determination of rainfall excess (Runoff distribution)

Paddy land: The losses of rainfall computed by step 7 is to be separated from rainfall time distribution of step (6) by assuming the entire basin area consist of paddy land.

Non paddy land: The same process is to be repeated assuming the entire basin area consists of Non-paddy land.

9) Weighted Basin Average:

The net runoff of paddy and non paddy land of step (8) is to be multiplied by respective land classification percentage to get weighted runoff. The combined weighted runoff of paddy and non-paddy land will give the basin weighted runoff.

10) Computation of number of vents:

Once the basin weighted runoff is known, the total volume of water to be discharged through the sluice can be computed as follows:

Since 1" of rainfall excess over an area of 1 sq mile would generate 27 cfs, total discharge for the basin would be

Q = 27 x basin weighted runoff x area in square miles.

From tide-discharge curve volume of water to be discharged by each vent can be computed. If this discharge is q/vent. Then the no. of ventage required will be Q/q.

DESIGN OF BAMONDANGA SLUICE

HYDROLOGIC DESIGN

DESIGN DATA AVAILABLE

1.	Basin Area in sq. mile	:	5 sq. miles
2.	Purpose	:	Drainage
3.	Average Ground Level	:	+ 3.50
4.	Highest water level (R/s)	:	+ 10.50
5.	Lowest water level (R/s)	:	- 6.50
6.	Monsoon lowest water level (R/s)	:	- 5.40
7.	Crest level of Embankment	:	+ 13.00
8.	Top width of Embankment	:	14'-0"
9.	C/s slope of Embankment	:	1:2
10.	R/s slope of Embankment	:	1:3

Non Tidal Sill level fixation for flushing sluice Minm Avg [illegible] 2' [illegible]

Tidal [illegible] LWL [illegible]

(LWL [illegible] value [illegible])

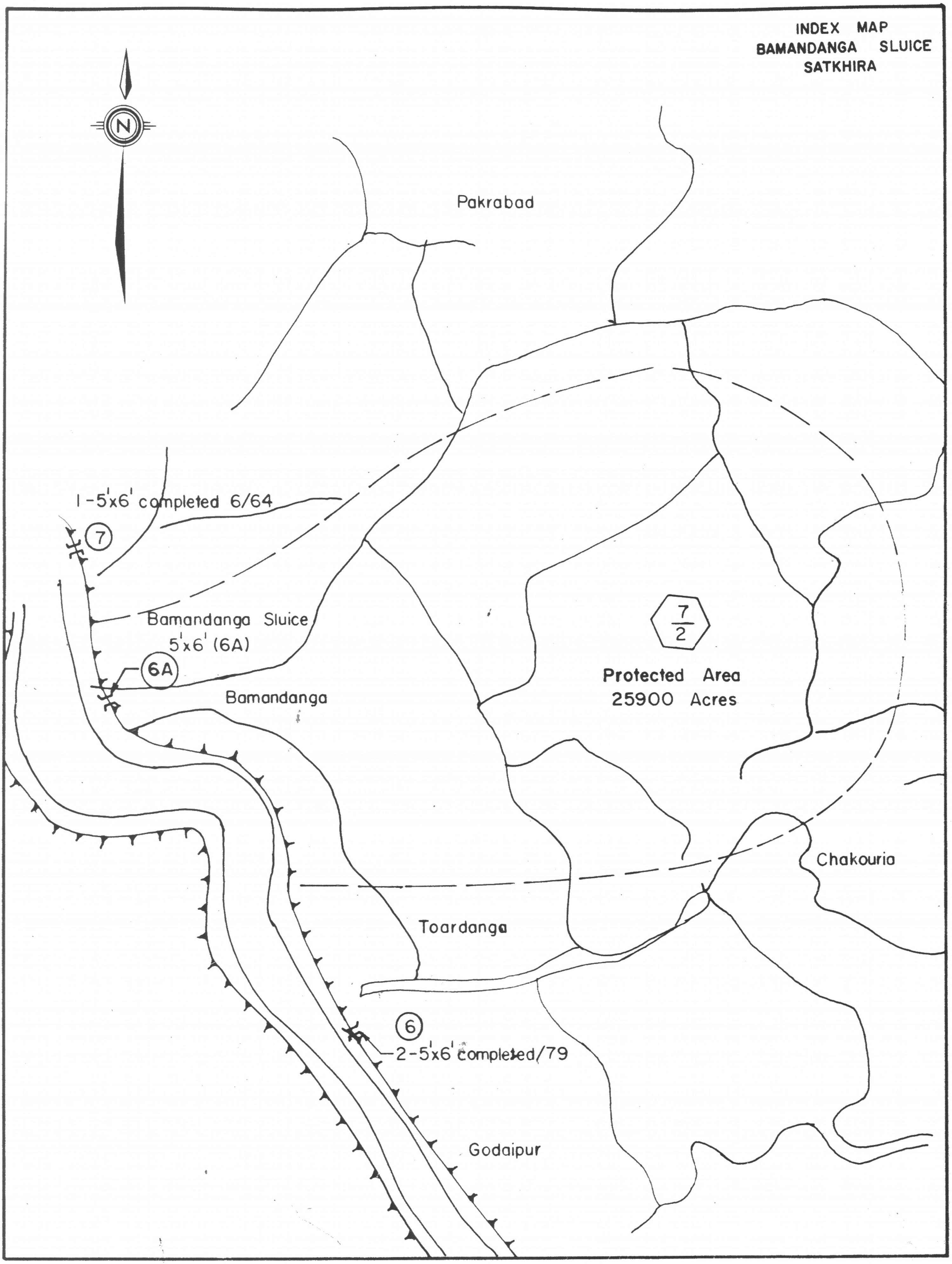


FIG. 3 INDEX MAP OF BAMANDANGA SLUICE

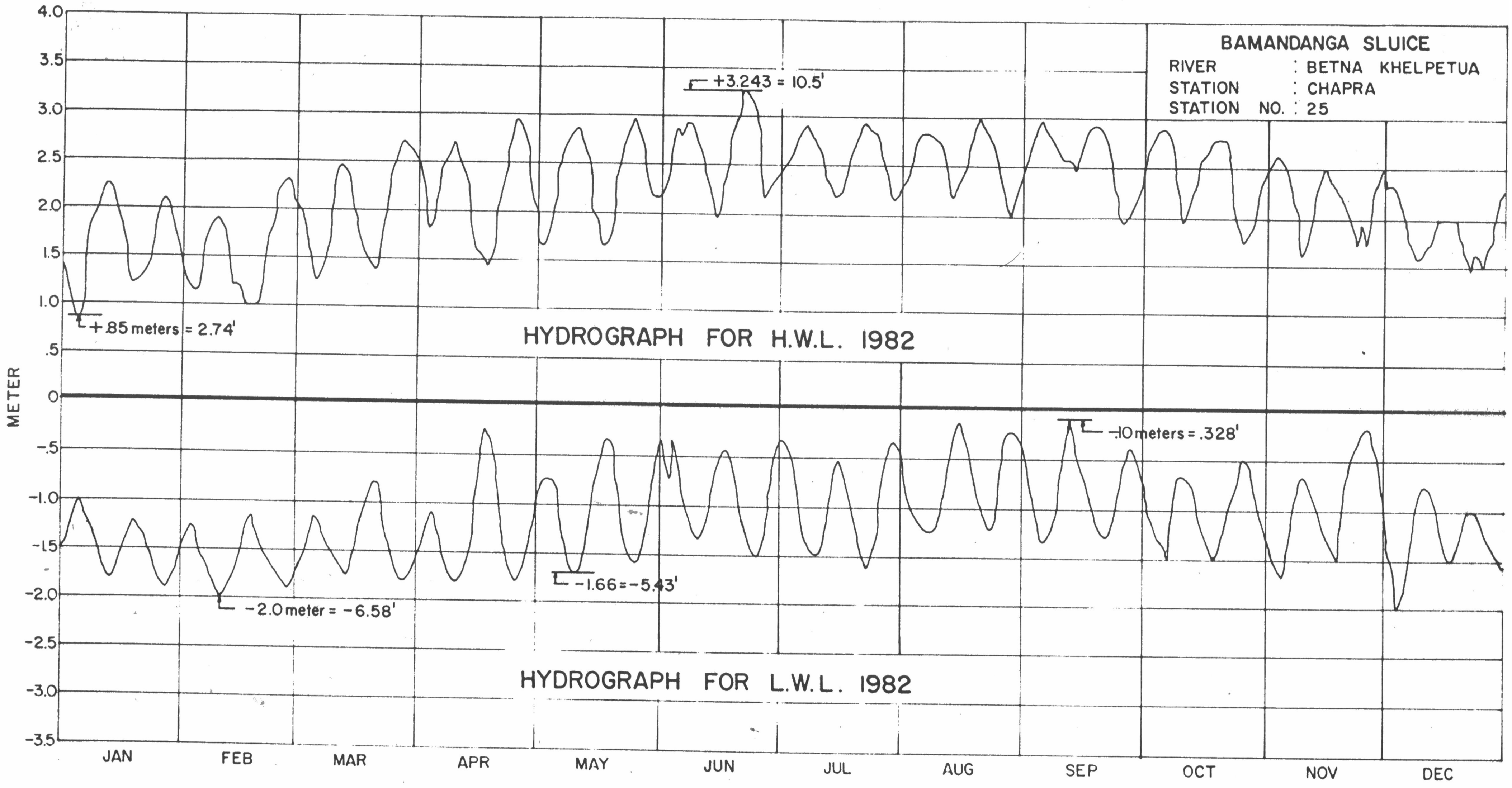
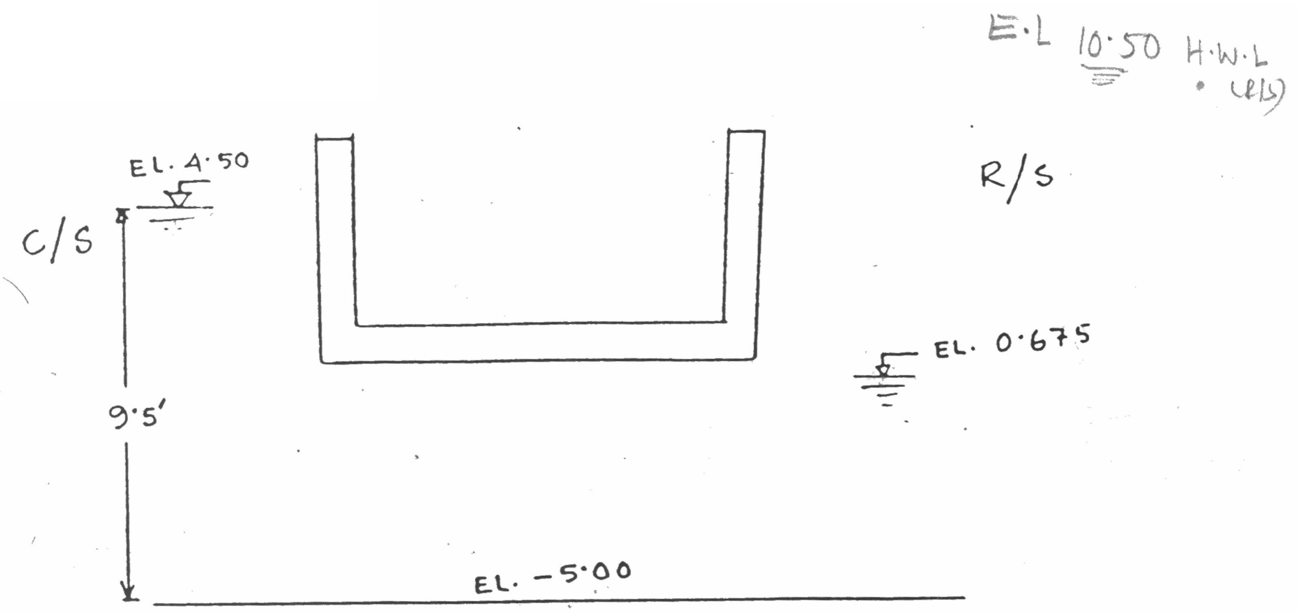


FIG. 4 HYDROGRAPH OF BETNA KHELPETUA RIVER AT CHAPRA



The highest W.L. on the C/s is assumed to be at EL.4.50 (Av.Ground level 3.50+1' = 4.50)

Considering the lowest tide level the invert level is fixed at EL. - 5.00.

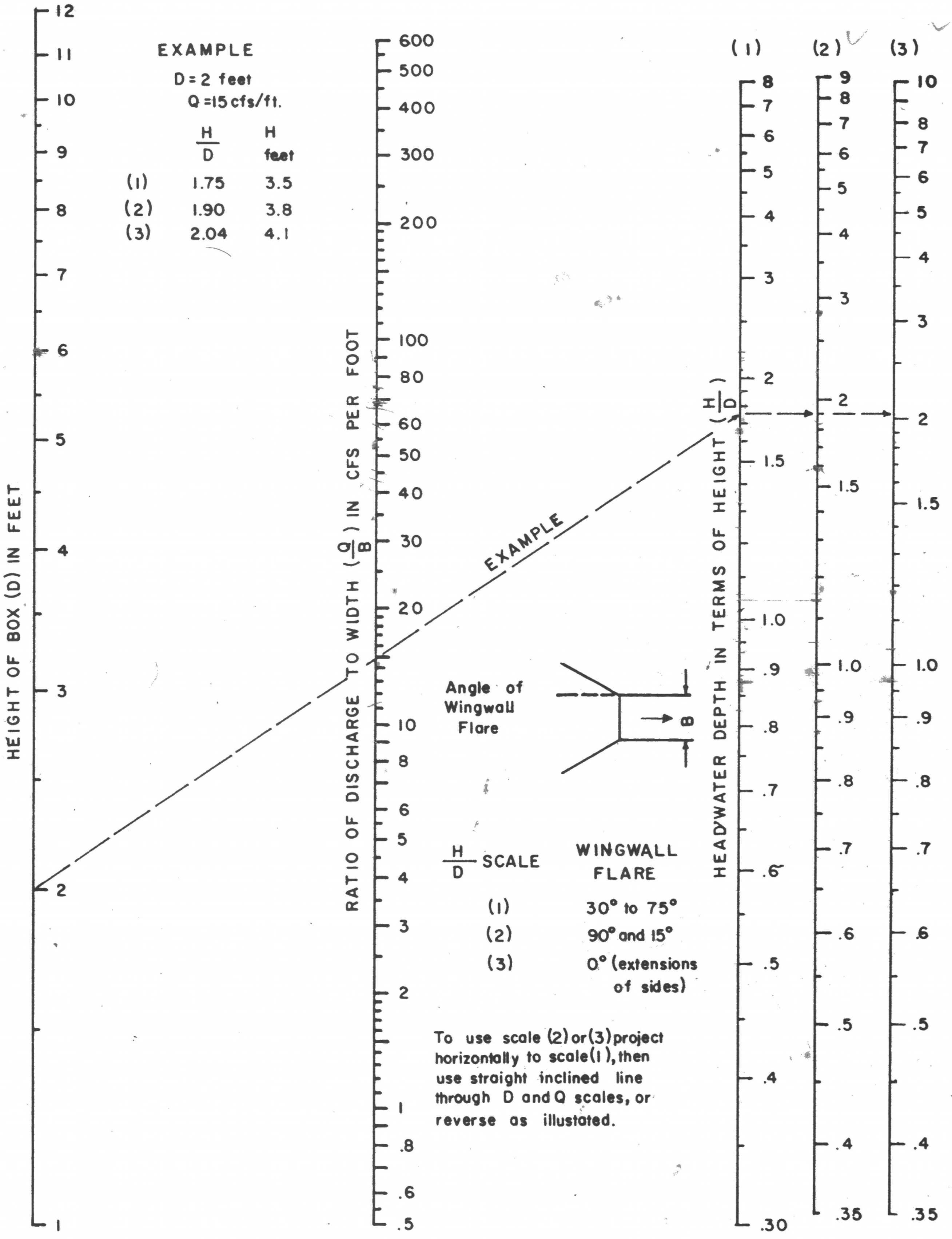
$$HW = 5.00 + 4.50 = 9.50'$$

From curve of U.S. Bureau of public roads for box culvert with entrance control (Fig. 5):

$$\frac{HW}{D} = \frac{9.50}{6} = 1.58$$

From Fig. 5 $\frac{Q}{B}$ = 70 cfs/ft width

or $Q = 70 \times B = 70 \times 5 = 350$ cfs/vent



Headwater depth for box culverts with entrance control. (U.S. Bureau of Public Roads)

FIG. 5 DESIGN MONOGRAPH - FLOW THROUGH CULVERTS

Critical depth may be computed from the formula

$$D_c = 0.315 \left(\frac{Q}{B}\right)^{2/3} \qquad \text{or} \qquad D_c = \left(\frac{q^2}{g}\right)^{1/3}$$

$$= 0.315 \left(\frac{350}{5}\right)^{2/3} \qquad\qquad = \left(\frac{70^2}{32.2}\right)^{1/3}$$

$$= 5.35 \text{ ft.} \qquad\qquad = 5.34 \text{ ft.}$$

$$= 5.35'$$

The control remains at the entrance until the following elevation reached by the tailwater.

$$\frac{D_c + D}{2} = \frac{5.35 + 6}{2} = 5.675'$$ above from horizontal flow line of the structure, which is at elevation of + 0.675. (-5)

The discharge at all stages of tailwater from -5.00 to + 0.675 remains constant at 350 cfs/vent.

The discharge will be controlled by tail water from Elevation +0.675 to elevation + 4.5, the discharge decreases in a straight line from 350 cfs to zero.

The total volume under the discharge curve from one tide cycle and average for 24 hour period is computed from the curve as follows:

Time of constant discharge by Similar Triangle LMN & LBC

$$\frac{6}{15.50} = \frac{x}{5.675}$$

$$\text{or} \qquad x = \frac{6 \times 5.675}{15.50} = 2.196 \text{ hour}$$

Time drainage reaches zero from change of tide to tide lockage, from similar triangles LMN & LQR

$$\frac{6}{15.5} = \frac{Y}{9.5}$$

$$\text{or} \qquad Y = \frac{9.5 \times 6}{15.5} = 3.67 \text{ hrs.}$$

Total volume under drainage curve for one tide cycle

$$V = 2\left\{(350 \times 2.196) + \frac{\cancel{2}(3.67 - 2.196) \times 350}{2}\right\} \times 3600$$

= 7391160 cft for 12 hours

= 169.67 = 170 Acre ft.

= 340 Acre ft. for 24 hours

= 170 cfs/vent for 24 hours

43560 cft = 1 Acre ft.

1 cfs = 2 Acre ft.

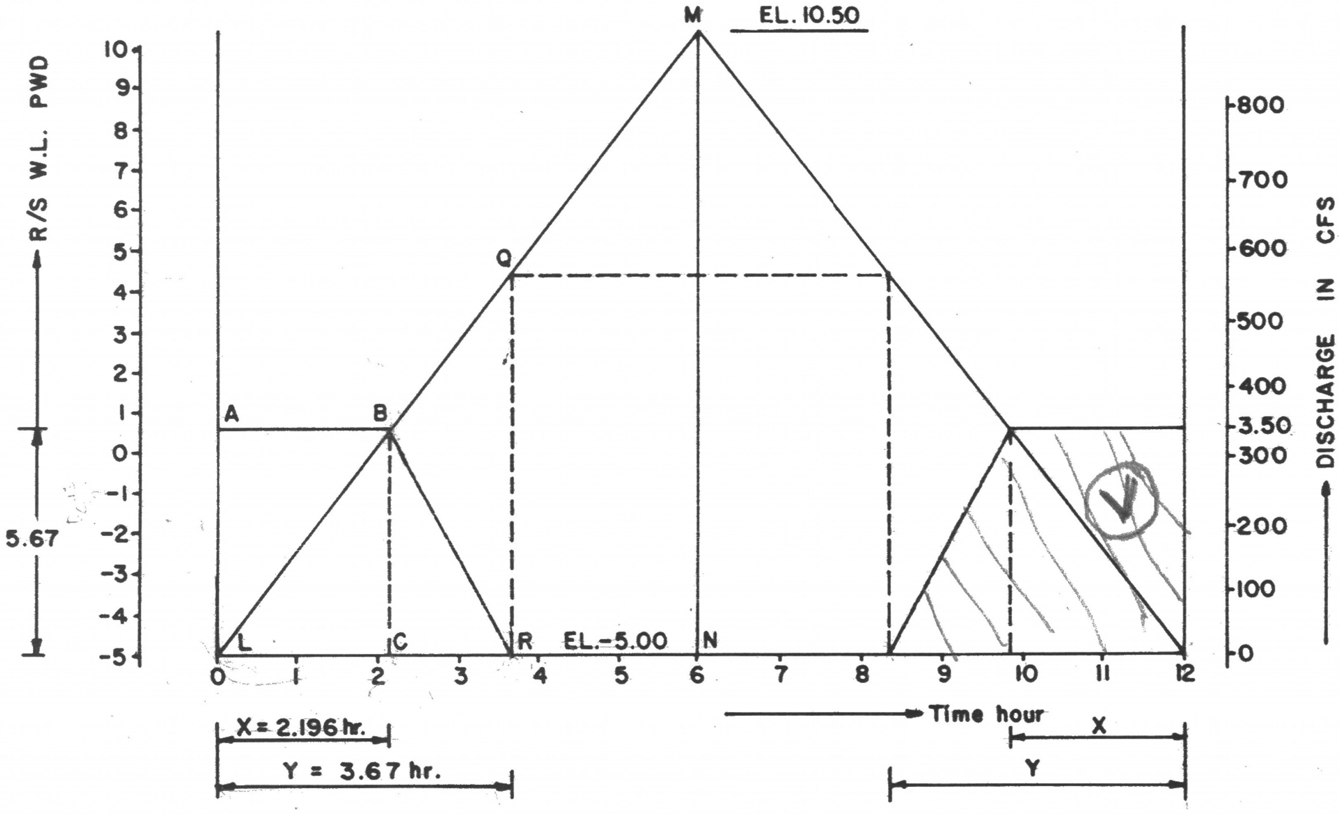


FIG. 6. 12 HOUR TIME DISCHARGE CURVE

$A = \left(\frac{1}{2} \times B \times h\right) \times 2$

Computation of Design runoff

From the isohyetal map the 4 (Fig. 7) month rainfall index for the project area is 50".

Rainfall indices of storm frequencies of 10 years and 25 years are shown in Table 1. The rainfall duration are taken for 5 days.

Point Rainfall

Point rainfall is the quantity of rain falling at a specific point, usually measured at a rain gaging station. We will define point rainfall as the product of the four month rainfall index and the combined rainfall indices. These rainfall volumes for the period from one to five days form the basis for our design storm.

Table 3. Storm percentage of point rainfall.

Days	Accumulative point rainfall (inch) = 4 month rainfall index rainfall indices
1	6.4"
2	9.6"
3	11.5"
4	12.9"
5	13.8"

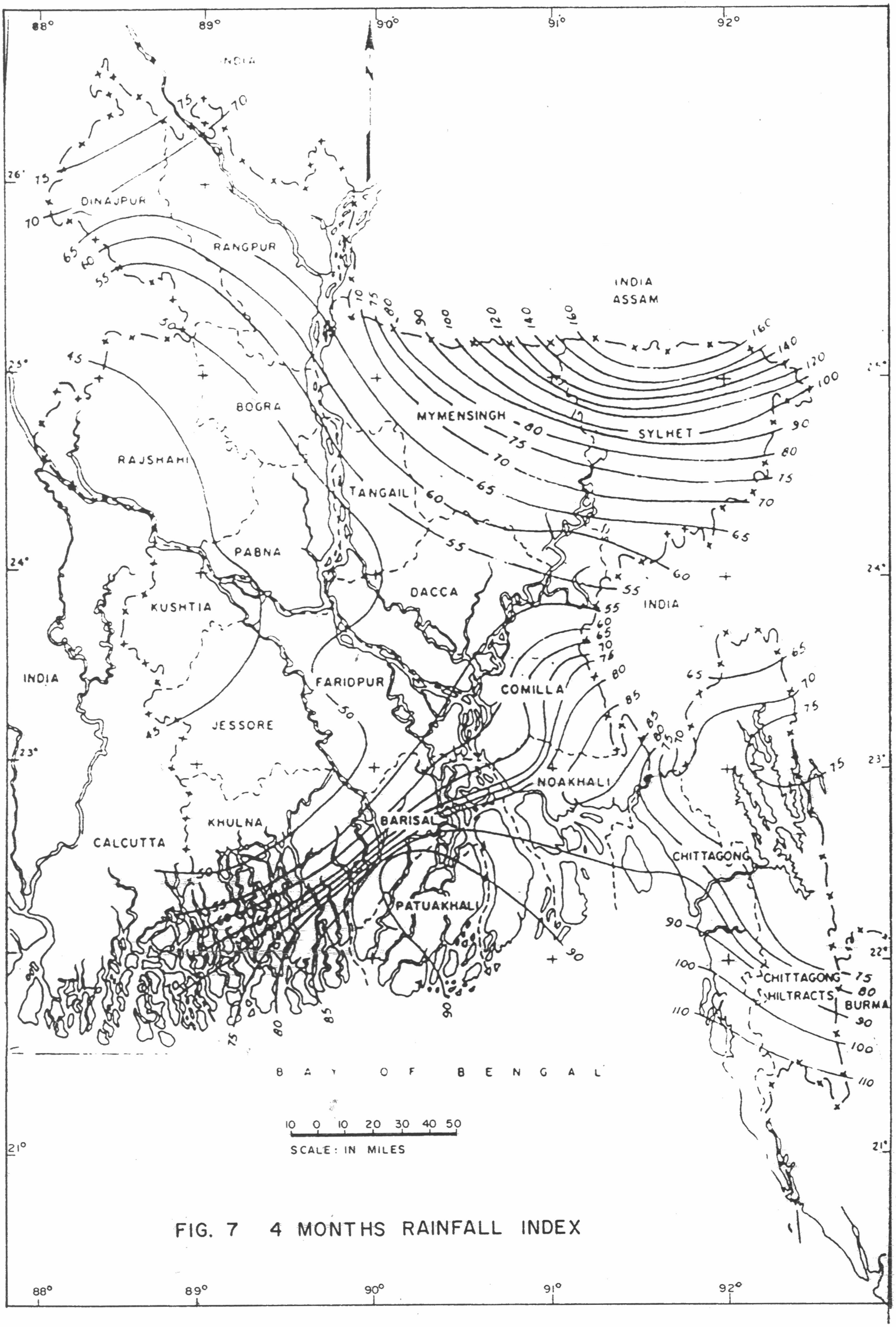


FIG. 7 4 MONTHS RAINFALL INDEX

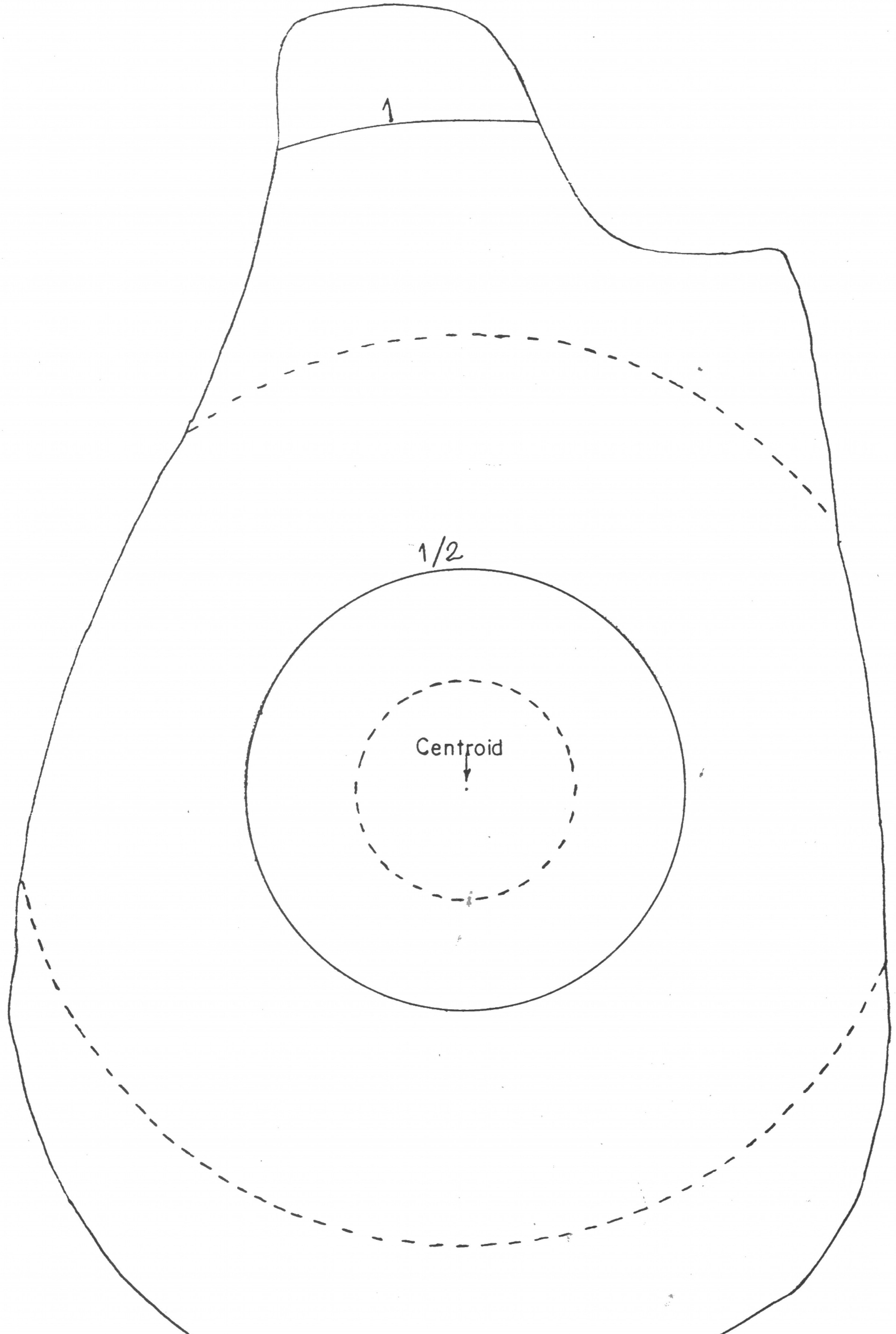


FIG. 8 Basin area showing centroid and isohyetals at even mile intervals

Area from isohyetals

Table 4. Area times storm percentage.

Average Distance From Storm Centre	Area Sq. miles	Area Times Percentage = Area x storm percentage
1/4	0.183	0.183
1	4.20	3.70
2	0.62	0.50
	5.00	4.39

The percentage of point rainfall that form the uniform equivalent depth of rainfall is

$$\frac{4.39}{5.00} \times 100 = 87.8\%$$

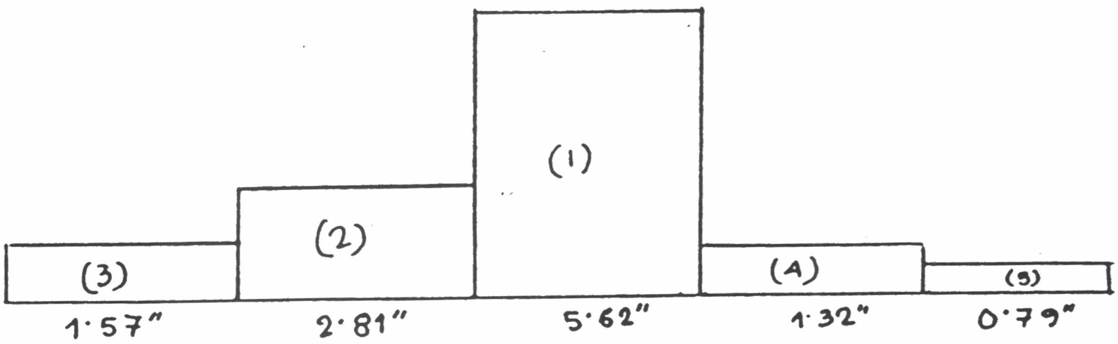
Equivalent uniform depth

The equivalent uniform depth is obtained by multiplying the point rainfall by 87.8%.

Table 5. Equivalent uniform depth.

Days	Equivalent uniform depth: Accumulative Total	Daily increments
1	5.62	5.62
2	8.43	2.81
3	10.00	1.57
4	11.32	1.32
5	12.11	0.79

The daily increments of rainfall can occur in any order, we will arbitarily arrange a sequence of 3,2,1,4,5. A graphical arrangments of this sequence looks like this:



<u>24 hour rainfall time distribution</u>

We shall use the unit storm interval of 6 hours. The rainfall time distribution of Satkhira is not available. The available rainfall time distribution closest to project area is Jessore having 4 month rainfall index 46". So an adjustment is the ratio of 50/46 is to be made to get the rainfall time distribution for the project area.

Table 6. Rainfall time distribution.

	Hours			
	6	12	18	24
Jessore maximum accumulative point rainfall	3.80	4.43	5.15	5.90
Project area maximum accumulative point rainfall	4.13	4.82	5.60	6.41
Project area maximum incremental point rainfall	4.13	0.69	0.78	0.81
Project area uniform equivalent depth	3.63	0.60	0.69	0.71

The increments of rainfall within 24 hour period can not be predicted. We will arrange arbitarily a sequence of 3,4,1,2. Thus during the maximum 24 hour rainfall period the 6 hour incremental rainfall is 0.69", 0.71", 3.62" & 0.60".

Design Storm

Table 7. 5-day design storm with 6-hour interval.

Day	6 hour increment
1	0.69 0.71 0.00 0.17
2	0.69 0.71 0.81 0.60
3	0.69 0.71 3.63 0.59
4	0.69 0.00 0.00 0.63
5	0.69 0.00 0.00 0.10

Rainfall Excess - Paddy Land

Table 8. rainfall excess for paddy land.

Days	Hour	Rainfall inches	Losses in inches			Available paddy storage	Net runoff
			Soil moisture		Depression Storage		
			Initial	Subsequent			
1	0-6	0.69	0	-0.25	-4.00	-3.56	0
	6-12	0.71		-0.25		-3.10	0
	12-18	0.00		-0.25		-3.35	0
	18-24	0.17		-0.25		-3.43	0
2	0-6	0.69		-0.25		-2.99	0
	6-12	0.71		-0.25		-2.53	0
	12-18	0.81		-0.25		-1.97	0
	18-24	0.60		-0.25		-1.62	0
3	0-6	0.69		-0.25		-1.18	0
	6-12	0.71		-0.25		-0.72	0
	12-18	3.63		-0.25		0	2.66
	18-24	0.59		-0.25		0	0.34
4	0-6	0.69		-0.25		0	0.44
	6-12	0.00		-0.25		-0.25	0
	12-18	0.00		-0.25		-0.50	0
	18-24	0.63		-0.25		-0.12	0
5	0-6	0.69		-0.25		0	0.32
	6-12	0.00		-0.25		-0.25	0
	12-18	0.00		-0.25		-0.50	0
	18-24	0.10		-0.25		-0.65	0

Rainfall Excess - Non paddy Land

Table 9. Rainfall excess for non-paddy land.

Days	Hour	Rainfall inches	Losses in inches			Available non paddy storage	Net runoff
			Soil moisture		Depression Storage		
			Initial	Subsequent			
1	0-6	0.69	-0.50	-	-0.19	0	0
	6-12	0.71		-0.25	-0.20	0	0.26
	12-18	0.00		-0.25	0	-0.25	0
	18-24	0.17		-0.25		-0.33	0
2	0-6	0.69		-0.25	-0.11	0	0
	6-12	0.71		-0.25	-0.20	0	0.26
	12-18	0.81		-0.25	-0.20	0	0.36
	18-24	0.60		-0.25	-0.10/1"	0	0.25
3	0-6	0.69		-0.25	0	0	0.44
	6-12	0.71		-0.25	0	0	0.46
	12-18	3.63		-0.25	0	0	3.38
	18-24	0.59		-0.25	0	0	0.34
4	0-6	0.69		-0.25	0	0	0.44
	6-12	0.00		-0.25	0	-0.25	0
	12-18	0.00		-0.25	0	-0.50	0
	18-24	0.63		-0.25	0	-0.12	0
5	0-6	0.69		-0.25	0	0	0.32
	6-12	0.00		-0.25	0	-0.25	0
	12-18	0.00		-0.25	0	-0.50	0
	18-24	0.10		-0.25	0	-0.65	0

Rainfall Excess - Weighted Basin Runoff

Assuming paddy land 50% & non paddy land 50%
(From Land classification)

Table 10. Rainfall excess - weighted basin runoff.

Days	Hour	Paddy Land		Non Paddy Land		Basin Weighted
		Net runoff	Weighted runoff	Net runoff	Weighted runoff	
1	0-6	0	0	0	0	0
	6-12	0	0	0.26	0.13	0.13
	12-18	0	0	0	0	0
	18-24	0	0	0	0	0
2	0-6	0	0	0	0	0
	6-12	0	0	0.26	0.13	0.13
	12-18	0	0	0.36	0.18	0.18
	18-24	0	0	0.25	0.125	0.125
3	0-6	0	0	0.44	0.22	0.22
	6-12	0	0	0.46	0.23	0.23
	12-18	2.66	1.33	3.38	1.69	3.02
	18-24	0.34	0.17	0.34	0.17	0.34
4	0-6	0.44	0.22	0.44	0.22	0.44
	6-12	0.00	0	0	0	0
	12-18	0.00	0	0	0	0
	18-24	0.00	0	0	0	0
5	0-6	0.32	0.16	0.32	0.16	0.32
	6-12	0	0	0	0	0
	12-18	0	0	0	0	0
	18-24	0	0	0	0	0
						5.13"

Total runoff = 5.13"

No. of vents required

Now from hydraulic computation, the basin weighted runoff is 5.13" for 5 days storm

So the runoff per day = 5.13/5 = 1.026" per day.

So the total discharge = 27 x 1.026 x 5

= 138.51 cfs.

Discharge/vent = 170 cfs. (Ref: page)

$$\text{No. of ventage required} = \frac{138.51}{170}$$

= 0.814 vents.

= 1 vent.

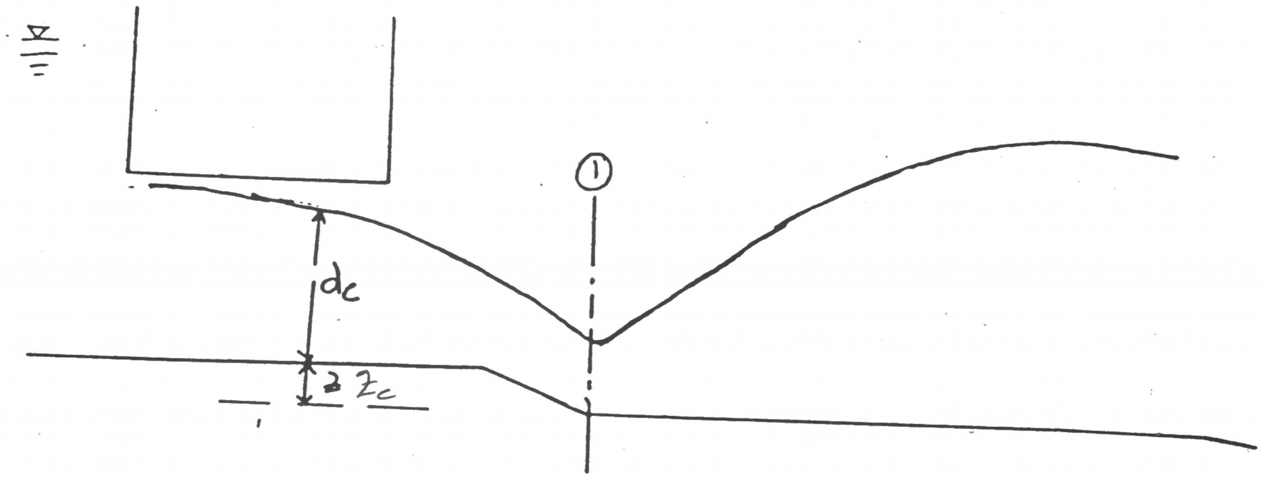
Hydraulic Design

Floor length by hydraulic jump

Q = 350 cfs. (From hydraulic analysis)

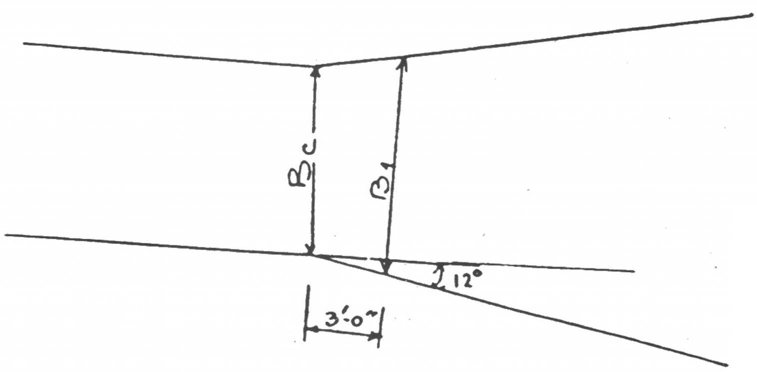
(This discharge is constant for 2.19 hours)

$$\text{discharge/ft width} = \frac{350}{5} = 70 \text{ cfs.}$$



$$d_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{70^2}{32.2}\right)^{1/3} = 5.33 \text{ ft.}$$

$$V_c = \frac{q}{dc} = \frac{70}{5.33} = 13.13 \text{ ft/sec.}$$



B_c = 5' - 0"

$B_1 = 5 + 2 \times 3 \tan 12^{o} = 6.27'$

Applying Bernoulli's Equation at sections (1) and (c)

$$d_c + \frac{V_c^2}{2g} + Z_c = d + \frac{V_1^2}{2g}$$

[At the critical state flow velocity head is equal to half the hydraulic depth.]

$$\text{or} \quad 1.5\, d_c + 1 = d_1 + \frac{V_1^2}{2g} \qquad \text{- - - - - - - - - - - - - (1)}$$

At point (1) the discharge/ft width q_1 is given by

$$q_1 = \frac{70 \times 5}{6.27} = 55.82 \text{ cfs/ft.}$$

From equation (1)

$$1.5 \times 5.33 + 1 = d_1 + \frac{55.82^2}{d_1^2 \times 2g} \qquad \left[V_1 = \frac{V_1}{d_1} \right]$$

$$\text{or} \quad 7.99 + 1 = d_1 + \frac{48.38}{d_1^2}$$

$$\text{or} \quad 8.99 = d_1 + \frac{43.38}{d_1^2}$$

By trial $d_1 = 2.80'$

$$\text{Now} \quad V_1 = \frac{q_1}{d_1} = \frac{55.82}{2.80} = 19.93 \text{ ft/sec.}$$

$$\text{Froude No.} \quad F = \frac{V_1}{\sqrt{gd_1}} = \frac{19.93}{\sqrt{32.2 \times 2.80}} = 2.09$$

Length of Jump

$$\text{We know} \quad \frac{d_2}{d_1} = \frac{1}{2}\left[(1 + 8F^2)^{1/2} - 1\right]$$

$$\text{or} \quad \frac{d_2}{d_1} = \frac{1}{2}\left[(1 + 8 \times 2.09^2)^{1/2} - 1\right]$$

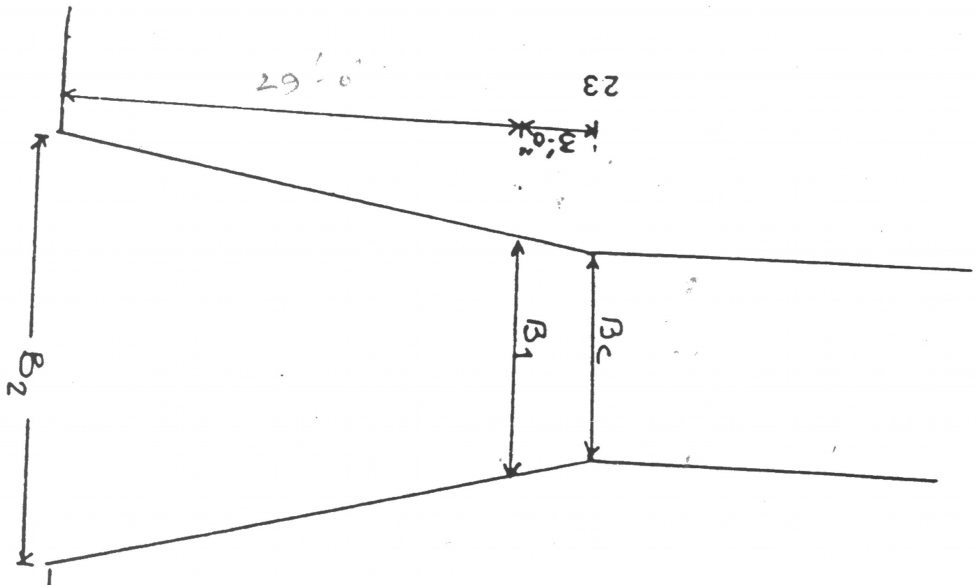
$$\text{or} \quad \frac{d_2}{d_1} = 2.49$$

$$\text{or} \quad d_2 = 2.49\, d_1 = 2.49 \times 2.80$$

$$\text{or} \quad d_2 = 6.94 \text{ ft.}$$

$$\begin{aligned} \text{Length of Jump} &= 6.9\,(d_2 - d_1) \\ &= 6.9\,(6.94 - 2.80) \\ &= 28.56 \\ &= 29\text{-}0" \end{aligned}$$

We may provide 29'+ 3' = 31'- 0" floor length considering hydraulic jump.



$$B_2 = B_c + 2 \times 31 \tan 12^o$$

$$= 5 + 13.17$$

$$= 18.17$$

Provide 20 - 0"

discharge/ft width at end

$$q_2 = \frac{350}{20} = 17.5 \text{ cfs/ft.}$$

<u>Scour depth with respect to hydraulic jump</u>

$$R = 0.91 \left(\frac{q_2^2}{f}\right)^{1/3}$$

f = silt factor
= 1.76 dm

where dm = Average diameter of particle in mm.

$$= 0.91 \left(\frac{17.5^2}{0.40}\right)^{1/3}$$

$$= 8.32'$$

<u>Design scour depth</u>

U/s = 1.25xR = 1.25x8.32 = 10.40 ft.

D/s = 1.5 xR = 1.5 x8.32 = 12.48 ft.

<u>Scour level</u>

U/s = U/s W.L. - 10.40

= 3.50 - 10.40 (Considering C/s water level at average ground level)

= -6,90

– 6.90

It means the cut off depth required = -5-(-~~7.93~~)

= 1.90' = 2'-0"

D/s scour level = D/s W.L. - 12.48

= 0.675 - 12.48

= -11.80

Depth of cutoff required = -6- (-11.80)

= 5.8' = 6'-0"

<u>Floor length and cutoff by exit gradient</u>

In calculating the floor length and depth of cutoff by exit gradient the water level on the R/s will be considered to be at highest tide level while the water level on the C/s to be at retention level to get the maximum head difference.

Now H.W.L. (R/S) : 10.50

Retention level (C/s) : 4.50

Head difference H = 10.50 - 4.50

= 6'- 0"

Considering the safe exit gradient 1/7. Taking maximum head difference 6'-0". Assuming depth of cutoff 6'-0" at C/s.

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

$$\text{or } \lambda = \left(\frac{H}{G_E \times d \times \pi}\right)^2$$

π = 3.14

$$\text{or } \lambda = \left(\frac{6 \times 7}{6 \times \pi}\right)^2$$

$$= 4.96$$

$$\text{again } \lambda = \frac{1 + (1+\alpha^2)^{1/2}}{2}$$

$$\text{or } 1 + (1 + \alpha^2)^{1/2} = 2\lambda$$

$$\text{or } 1 + \alpha^2 = (2\lambda - 1)^2$$

$$\text{or } \alpha^2 = (2\lambda - 1)^2 - 1$$

$$\alpha^2 = (2 \times 4.96 - 1)^2 - 1$$

$$\alpha = 8.88$$

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

$$\text{or } 8.88 = \frac{b}{6}$$

or b = 53'-0"

It is seen that if total floor length is 53'- 0" the depth of cutoff 6' will satisfy the existence of the structure from exit gradient consideration.

The total length required from hydraulic jump consideration.

Total floor length = U/s floor length + barrel + glacis + D/s floor.

For barrel and gate fixing = 24'-0"

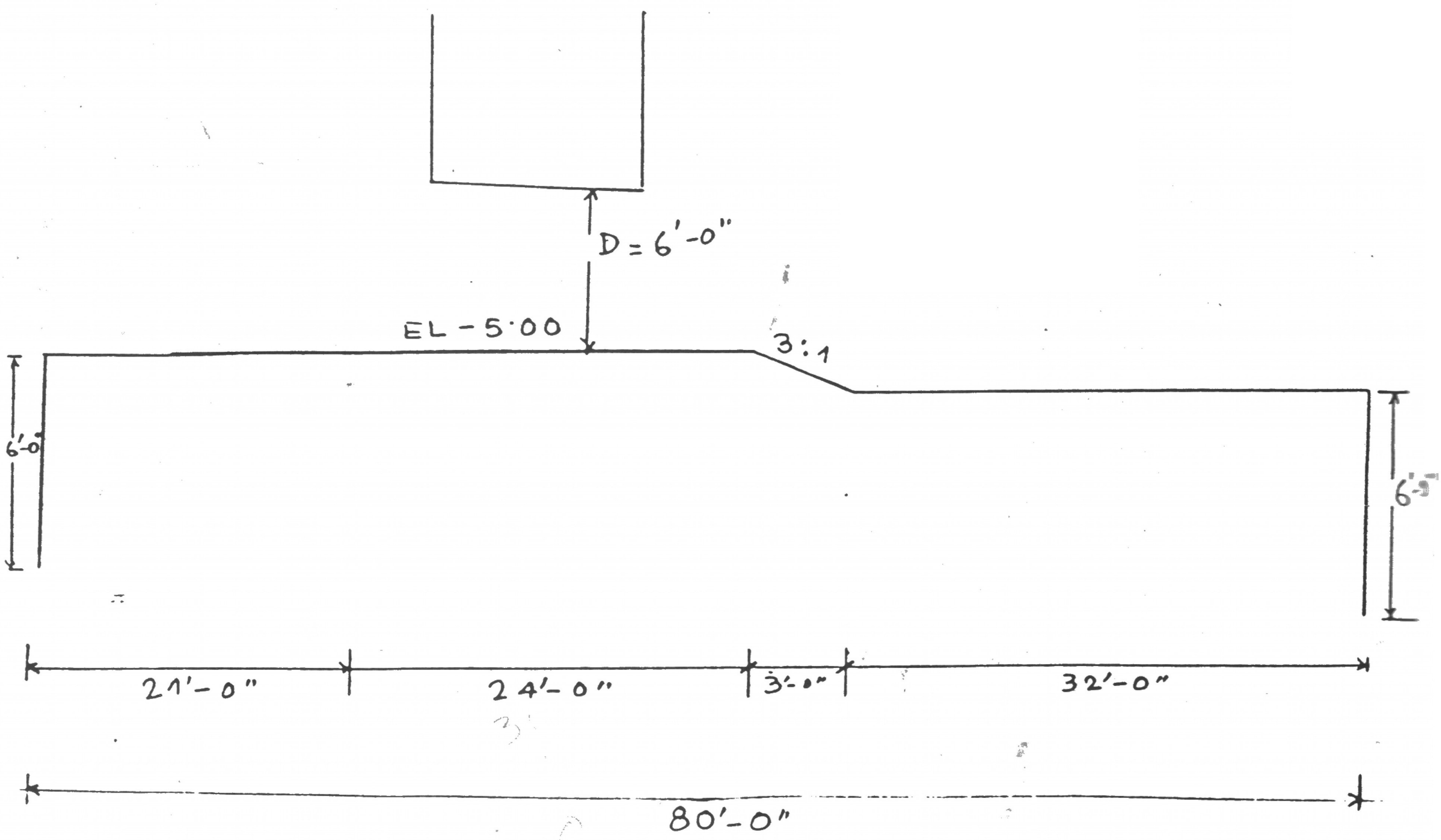
glacis = 3' - 0"

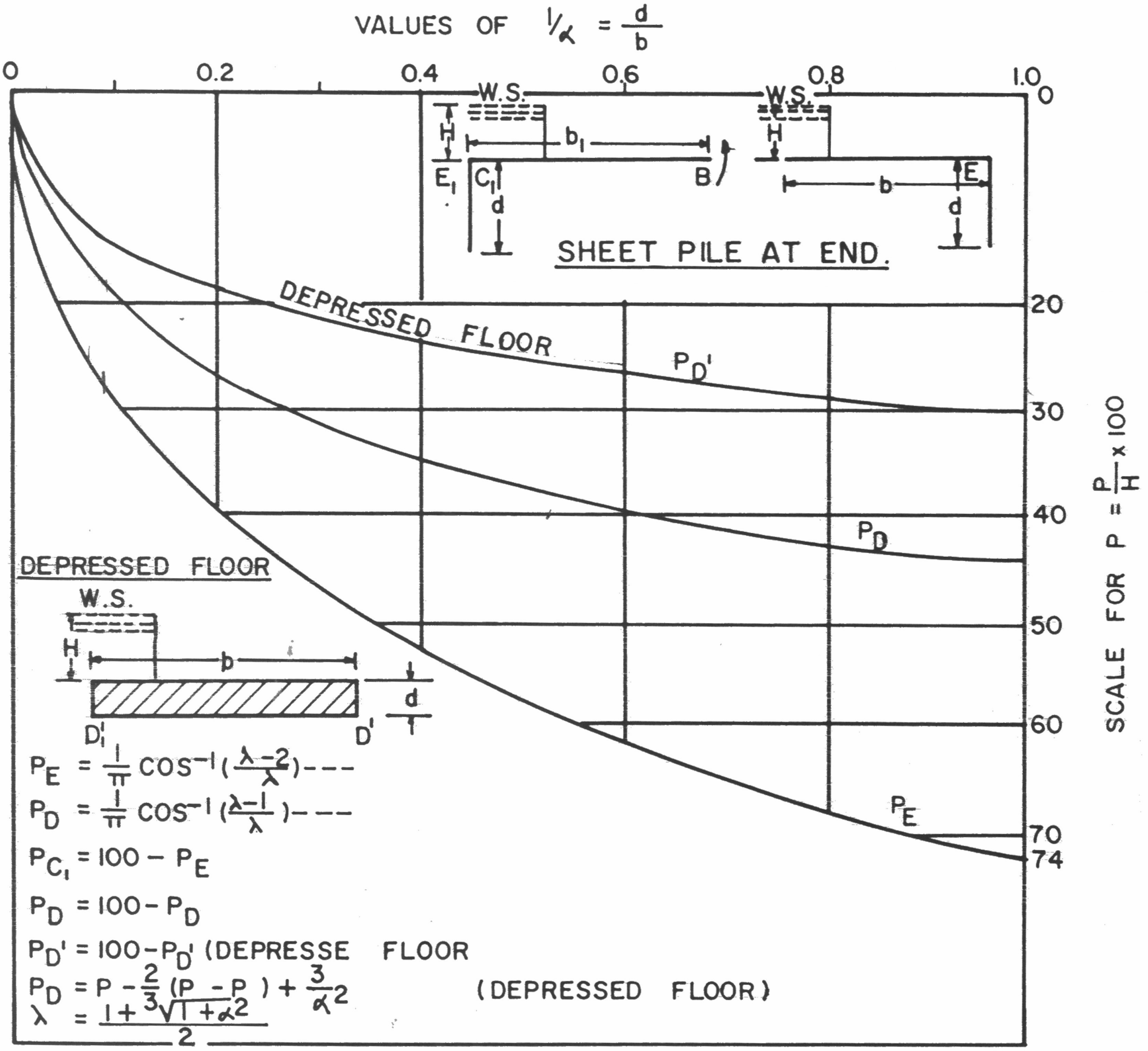
D/s floor = 29' - 0"

U/s floor = 20' - 0"

Total = 76'- 0"

Let us provide total floor length 80' - 0" with D/s floor 32' U/s floor 21', barrel 24' and glacis 3' and both U/s and D/s cutoff depth 6'




VALUES OF 1/α = d/b
0
0.2
0.4
0.6
0.8
1.0
W.S.
H
b1
E1
C1
B
d
E
b
SHEET PILE AT END.
DEPRESSED FLOOR
PD'
PD
PE
DEPRESSED FLOOR
D1'
D'
0
20
30
40
50
60
70
74
SCALE FOR P = P/H x 100
PE = 1/π cos⁻¹((λ−2)/λ) ---
PD = 1/π cos⁻¹((λ−1)/λ) ---
PC1 = 100 − PE
PD = 100 − PD
PD' = 100 − PD' (DEPRESSE FLOOR
PD = P − 2/3 (P − P) + 3/α2
(DEPRESSED FLOOR)
λ = (1 + 3√(1 + α2))/2

Floor thickness in relation to uplift pressure
(Gate closed, R/s controlling)

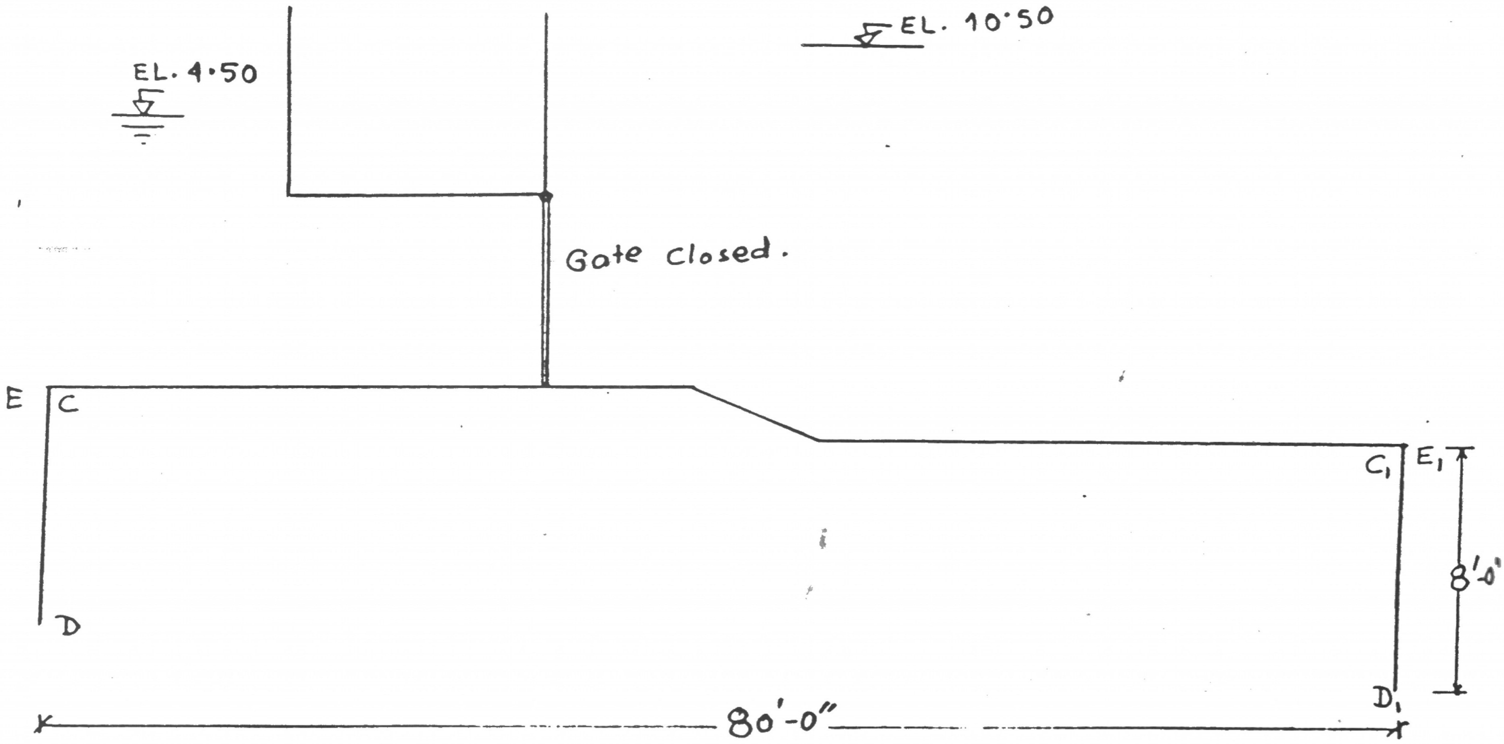
Now we have already fixed

Floor length b = 80' - 0"

R/s depth of cutoff d = 6' - 0"

C/s depth of cutoff d = 6' - 0"

Let us assume floor thickness at U/s and D/s end 1' - 6"



$$\frac{1}{\alpha} = \frac{d}{b} = \frac{6}{80} = 0.075$$

From Khosla's pressure chart. (Fig. 9)

P_D = 16% $\qquad$ P_{D1} = 100 - 16 = 84%

P_E = 26% $\qquad$ P_{C1} = 100 - 26 = 74%

Correction due to interference of cutoff

Let C be the correction to be applied as percentage of head.

b = distance between two piles

D = depth of cutoff the influence of which has to be determined on the neighbouring cutoff of depth d

d = depth of cutoff on which the effect of cutoff of depth D is sought to be determined.

b = total length of floor.

Correction due to interferece of C/s cutoff on to R/s cutoff

$$C = 19 \left(\frac{D}{b}\right)^{1/2} \times \frac{d + D}{b}$$

in our case D = d = 4.5

$$C = 19 \left(\frac{4.5}{78}\right)^{1/2} \times \frac{4.5+4.5}{80}$$

$$= 0.51\%$$

This correction is positive for points in the rear or back water and subtractive for points forward in the direction of flow.

Correction due to floor thickness (R/s)

Floor thickness at end is assumed 1' - 6".

If C_F is the correction for floor thickness

$$\text{Then } C_F = \frac{84 - 74}{6} \times 1.5 = 2.5\% \ (+ve)$$

$P_{c1 \text{ (Corrected)}} = 74 + 0.51 + 2.5 = 77\%$

Correction for floor thickness C/s

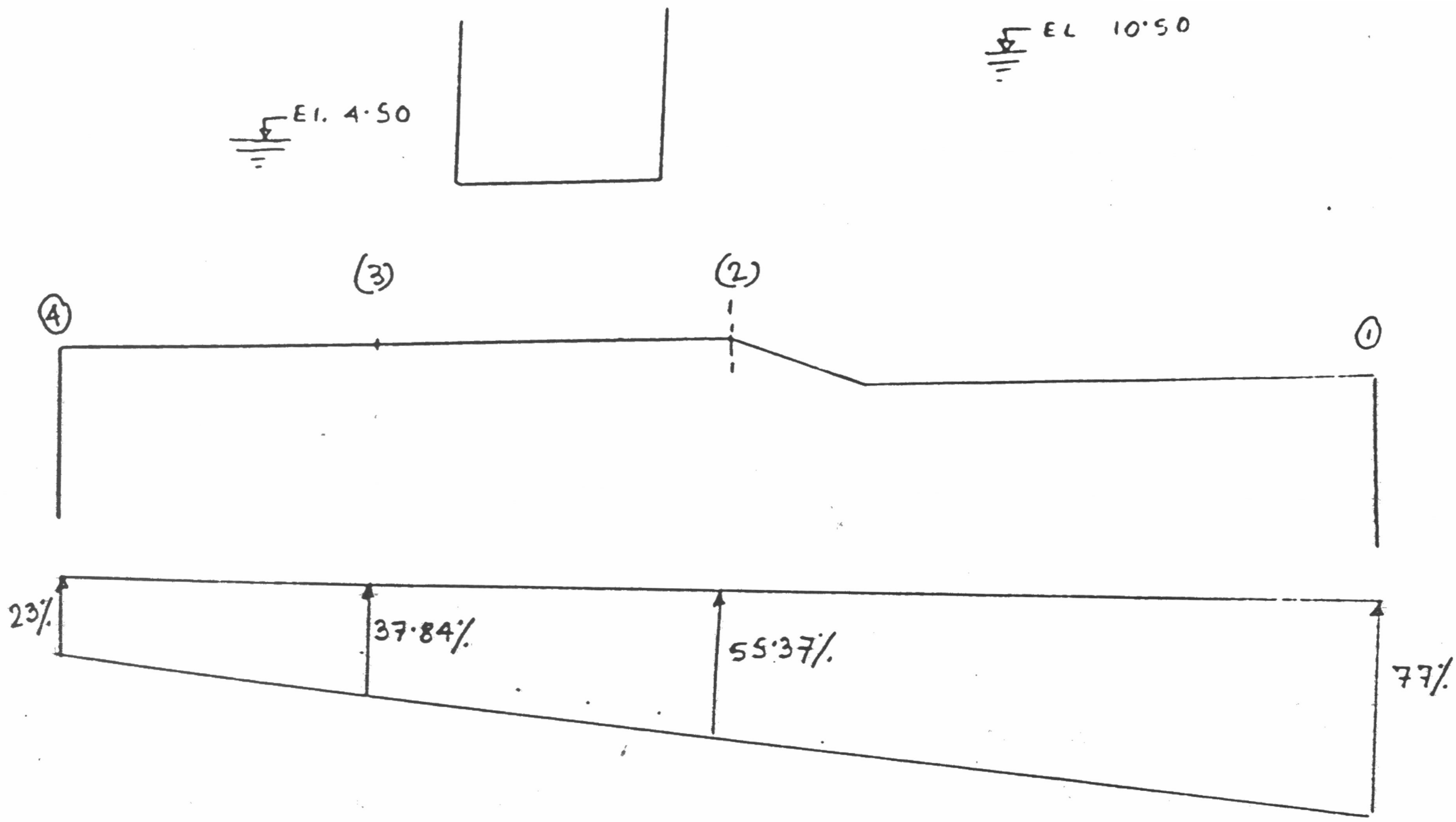
1) Correction due to interference of R/s cutoff on to C/s cutoff will be the same as before as the cutoff depth and floor length is same. But the the direction will be reverse. i.e. C = 0.51% (-ve).

2) Correction for floor thickness = $\frac{26-16}{6} \times 1.5 = 2.5\%$ (-ve)

$$P_{E \text{ (Corrected)}} = 26-0.51-2.5$$
$$= 23\%$$

The uplift pressure while the gate is open need not to calculate as the head difference is small as a result the uplift pressure will also be negligible. This happens as because while the gate is open during low tide the water level on the country side falls down discharging through the sluice as the tide level comes down resulting a small head difference.

Pressure as percentage of head difference



Pressure in ft. of water

$$P_1 = \frac{77}{100} \times 6 = 4.61 \text{ ft.}$$

$$P_2 = \frac{55.37}{100} \times 6 = 3.32 \text{ "}$$

$$P_3 = \frac{37.84}{100} \times 6 = 2.23 \text{ "}$$

$$P_4 = \frac{23}{100} \times 6 = 1.38 \text{ "}$$

Here the points (3) and (4) are critical. The uplift pressure of water at points (1) and (2) are is 4.61' and 3.32' respectively while the downward water pressure is 6' - 0". So theoretically no thickness is necessary. At points (3) and (4) the uplift water pressure due to head difference is 2.23' and 1.38' respectively which is to be balanced by equivalent weight of concrete and necessary thickness of concrete is to be provided.

<u>Thickness of concrete</u>

at point (3) $t_3 = \dfrac{2.23}{1.40} = 1.57$ ft.

at point (4) $t_4 = \dfrac{1.38}{1.40} = 1.78$ ft.

So our assumption of thickness at end is near to the calculated value and on safe side. We shall provide the thickness of concrete at end 1' - 6" and at glacis 2' - 0". At the barrel part the downward weight of barrel and soil will be more than sufficient to balance the uplift pressure. So at the barrel part a normal thickness of 2' - 0" may be provided.

Here the thickness is calculated only by considering the highest tide level at R/s and retention level on C/s. In the reverse case while the R/s water level becomes lowest i.e. at the lowest tide level the C/s water level will also be at very low level and the head difference will be very less, consequently the required thickness will also be very less. So the thickness as required by condition (1) is to be provided at both ends.

CHAPTER VIII

HYDRAULIC DESIGN OF FLUSHING REGULATOR

Flushing Regulator

Purpose:

Flushing Regulator is constructed for the following purposes:

a) Flushing of irrigation water

b) Prevention of early flood

c) Prevention of flood

The design of the flushing regulator is carried out through the following steps:

1) Ventage computation:

This will fix the required discharge capacity of the structure according to the irrigation water requirement and the size and no. of ventage of the structure is to be fixed accordingly.

2) Hydraulic Computation:

This will give the required dimensions of different parts of the structure. Such as the length of U/s and D/s apron, thickness of apron, depth of cutoff wall etc.

3) Structural Design:

After the completion of hydraulic analysis and hydraulic design, the structural design is to be done which will give the detail dimensions of various parts of the structure at any point, reinforcements etc.

1) Ventage Computation:

The number and sizes of ventage to be provided for the area to be irrigated depends on the crop water requirement. To find out the crop water requirement one must have clear idea of the following hydrological terms.

a) Evapotranspiration:

Evapotranspiration is the total evaporation, i.e. evaporation from all water, soil, vegetation and other surfaces plus transpiration. Climate is one of the most important factors determining the amount of water loss by evapotranspiration from the crop. A reference value of evapotranspiration (E_{To}) will be required later on which may be defined as "the rate of evapotranspiration from an extended surface of 8 to 15 cm tall green grass cover of uniform height, actively growing, completely shading the ground and not short of water".

b) Crop water requirement:

Crop water requirement (E_T) is defined as "the amount of water potentially required to meet the evapotranspiration needs of vegatative areas so that plant production is not limited from lack of water". This may also termed as crop evapotranspiration. E_T (crop) for a selected crop is given by

$$E_T = K_c \times E_{To}$$

Where K_c = Crop co-efficient

What is Kc?

Crop co-efficient K_c are presented to relate E_{To} to crop evapotranspiration or crop water requirement E_T(crop). The K_c value represents evapotranspiration of a crop grown under optimum conditions producing optimum yields.

c) Net irrigation requirement:

Net irrigation requirement for a given crop or cropping pattern is the deficit of soil water balance. The main variables composing the soil water balance to be considered include i) crop water requirements ii) contribution from precipitation iii) ground water and iv) carry over of soil water and where applicable v) in and outflow of water, either surface or sub-surface.

Net irrigation water may be expressed as

$$In = \underbrace{(Er\ (crop) + F + R)}_{\text{Losses}} - \underbrace{(Pe + Ge + N + W)}_{\text{gains}}$$

where

In = Net irrigation requirement

Er = crop water requirement or evapotranspiration(crop)

F = deep percolation

R = surface or sub-surface out flow

Pe = contribution to root zone by rainfall

Ge = contribution to root zone by ground water

N = surface or sub-surface inflow

W = change in soil water content in the root zone

For practical purposes net irrigation requirement is taken as crop water requirement i.e. In = Er (crop)

d) Effective rainfall:

Effective rainfall is only a portion of total rainfall. Part of rain may be lost by surface runoff, by deep percolation below root zone. In regions with heavy and high intensity rains only a portion can enter and be stored in the root zone and the effectiveness of rain is consequently low.

Value of E_T & K_C

The value of monthly crop water requirement were calculated by the radiation method for different zones of Bangladesh and tabulated in table (1) (Table attached). The value Kc were taken from Crop Water Requirement Irrigation & Drainage paper 24 by FAO and tabulated in Table 2.

e) Field irrigation requirement

Field irrigation requirement is the amount of water and timing of its application needed to compensate soil water deficits i.e. the amount of water to compensate the net irrigation requirement In. Irrigation is never 100% effective and allowance must be made for unavoidable or avoidable losses including deep percolation, surface runoff and other managerial and technical faults. Irrigation application efficiency Ea normally expressed in fraction or percentage of In is applied to get field irrigation requirement if.

$$If = \frac{In}{Ea}$$

$$\text{Field irrigation requirement} = \frac{\text{Net irrigation requirement}}{\text{Field application efficiency}}$$

f) <u>Project Diversion Requirement or Irrigation Supply requirement : Distribution efficiency</u>

To quantify the supply needed to meet irrigation requirement (irrigation supply requirement) at the field the efficiency of distribution system must be determined. Distribution efficiency, Ed, is determined as that portion of water released at the headworks and that received at the field inlet.

The main factors deciding distribution efficiency are method of water delivery (continuous, rotation, demand) size of project area and effectiveness of management organization. Distribution efficiency can be sub divided into conveyance efficiency, Ec, between headworks and the inlet to a block of fields, & field ditch efficiency, Eb between main canal and inlet of an individual field. Distribution efficiency is obtained from Ed = Ecx Eb.

$$E_c = \frac{\text{Field irrigation requirement}}{\text{Project diversion requirement}}$$

$$E_b = \frac{\text{Net irrigation requirement}}{\text{Field irrigation requirement}}$$

$$E_c \times E_b = \frac{\text{Field irrigation requirement}}{\text{Project Diversion Requirement}} \times \frac{\text{Net irr.requirement}}{\text{Field irr.requirement}}$$

$$E_d = \frac{\text{Net irrigation requirement}}{\text{Project diversion requirement}}$$

or

$$\text{Project diversion requirement} = \frac{\text{Net irrigation requirement}}{Ed}$$

Design discharge capacity for inlet structure

The design discharge capacity for an irrigation inlet structure is the irrigable area multiplied by the overall project duty (cfs/acres).

Now the overall project duty i.e. gross project diversion requirement per acre of irrigable area is required to be computed.

The necessary data taken for the bridge cum flushing regulator over SHAKARIA KHAL is taken as follows:

Project Area : 7000 Acres

Cultivable Area : 6000 Acres

Irrigable Area : 1800 Acres

Efficiencies and land preparation as follows:

Land preparation :	Paddy Land	- 7"
	Others	3"
Field application efficiency:	Paddy crops	- 50%
	Others	- 75%
Conveyance efficiency:	Paddy crops	- 80%
	Others	- 70%

In our case project duty and project diversion requirement have been calculated as follows:

1. The irrigation period for the paddy is between June to November

2. Crop water requirement/Net irrigation requirement (neglecting dependable rainfall due to short duration draught)

$$E_{T(crop)} = K_c \times E_{To}$$

$$E_T = \frac{1.25 \times 5.70}{30} = 0.243''/\text{day}$$

Ref: Table (2)

The crop co-efficient used above has been taken maximum during the growing period.

Table-1
POTENTIAL EVAPOTRANSPIRATION AND DEPENDABLE RAINFALL
IN BANGLADESH

	REGION	1	2	3	4	5
JAN	ETP	3.5 in	3.4 in	3.5 in	2.1 in	2.3 in
	DR	0.0 in	0.0 in	0.0 in	0.0 in	0.0 in
FEB	ETP	5.0	4.5	4.9	3.4	4.0
	DR	0.1	0.1	0.1	0.1	0.1
MAR	ETP	7.2	6.6	6.8	5.2	5.1
	DR	0.1	0.2	0.5	1.0	1.8
APR	ETP	8.0	7.2	5.9	5.8	5.4
	DR	0.4	1.2	2.8	5.7	1.8
MAY	ETP	6.8	6.3	6.2	5.9	5.6
	DR	5.0	3.9	6.4	13.5	6.0
JUNE	ETP	5.7	5.4	5.7	5.2	5.1
	DR	7.1	7.3	8.9	15.1	15.0
JULY	ETP	6.2	5.2	5.7	5.2	5.1
	DR	9.1	9.7	10.4	9.8	16.0
AUG.	ETP	5.4	4.9	5.6	5.0	4.8
	DR	8.7	7.7	10.4	9.9	15.2
SEP.	ETP	4.8	4.3	5.0	4.3	4.4
	DR	7.2	5.7	6.9	8.0	8.8
OCT.	ETP	4.4	4.1	4.6	3.7	3.9
	DR	3.2	2.6	2.8	3.7	5.1
NOV.	ETP	3.7	3.5	3.7	2.6	3.3
	DR	0.0	0.0	0.0	0.0	0.0
DEC.	ETP	3.4	3.1	3.4	2.3	3.1
	DR	0.0	0.0	0.0	0.0	0.0

Region 1 : Rangpur, Dinajpur, Bogra, Rajshahi, Pabna
Region 2 : Kushita, Jessore, Faridpur, Khulna
Region 3 : Tangail, Dacca, Comilla, Mymensingh, Jamalpur
Region 4 : Sylhet
Region 5 : Chittagong, Noakhali, Patuakhali, Barisal
.... Potential evapotranspiration(ETP)from green grass
.... Dependable Rainfall, (DR), 75% probability of being equalled or exceeded.

3. Field irrigation requirement

$$I_f = \frac{\text{Net irrigation requirement}}{\text{Field application efficiency}}$$

$$= \frac{0.243}{0.50} = 0.487"/\text{day}$$

4. Project diversion requirement = $\frac{\text{Field irrigation requirement}}{\text{Conveyance efficiency}}$

$$= \frac{0.487}{0.80} = 0.61"/\text{day}$$

Table 2 gives coefficients (KC) for multiplying ET_o to find the evapotranspiration of specific crops. The given KC values are for the maximum monthly water use during the peak stage of plant growth. The value of E_{To} was taken from Table (1).

Table-2
CROP EVAPOTRANSPIRATION COEFFICIENTS
AND EFFECTIVE ROOT ZONE DEPTH

Crop	Effective Root Depth	Coefficient (KC)
Flooded rice	-	1.25
Wheat & grains	3.0 ft.	1.1
Sugarcane	3.0	1.1
Bananas	4.0	1.0
Pulses	2.0	1.1
Vegetables	2.0	1.1
Vine crops, melons	2.5	1.0

* Crop evapotranspiration is calculated by multiplying KC by EPT given in Table 1. The KC values given are for the crop at maximum vegetative growth stage. Evapotranspiration before and after will be less.

Now 0.61" of water over one acre land gives a volume

of $\frac{0.61}{12} = 0.051$ acre ft.

We know 2 acre ft. = 1 cusec day

0.051 acre ft = $1/2 \times 0.051$

= 0.0255 cusec day

Now the area that can be irrigated by one cusec day

i.e. duty = $\frac{1}{0.0255}$ Acre = 39.21 Acres

Let us take duty = 40 acres/cusec

5. Design discharge for the irrigation inlet structure

$$= \frac{\text{Irrigable area}}{\text{Overall project duty}}$$

$$= \frac{1800}{40} = 4.5 \text{ cfs}$$

Considering cultivable area

$$\text{Q division} = \frac{6000/}{40} = 150 \text{ cfs}$$

i.e. for diversion requirement of 0.61"/day/acre we require to have the capacity of 0.0255 cusec-day. If we increase the capacity 1 cusec-day we can irrigate 1/0.0255 acres.

So 39.21 acres can be irrigated by 1 cusec-day our required irrigable area 6000 acres can be irrigated by $\frac{6000}{40} = 150$ cfs. with a factor of safety of 2.00 considering the further extension of the project the design capacity of the structure may be taken 300 cfs.

Computation for ventage requirement

The size and number of ventage may be calculated as follows:

Given

1) Design discharge = 300 cfs (Fom hydraulic analysis)

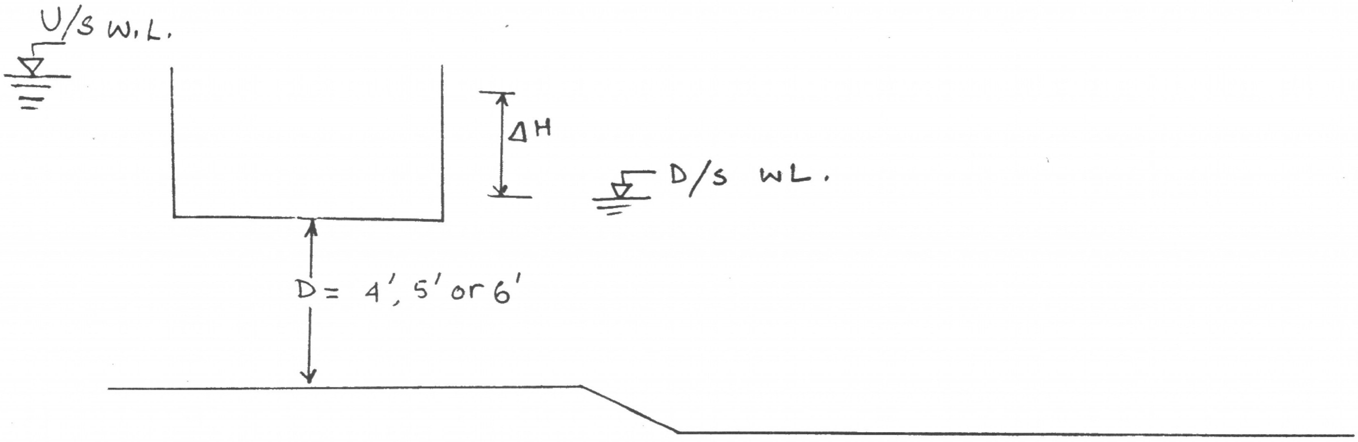
2) Bank level or Ground level : 41.00 PWD

3) The head difference between inlet & outlet under submerged condition shall be between 6" to 9".

4) When the head difference will be higher than 6", the country side water level and the design flow will be maintained by progressive closing of the water-way opening.

5) A number of curves may be drawn with head difference vs discharge for different vent sizes. From these curves required size and No. of ventage may be fixed as per required capacity.

submerged orifice flow



Box width = 3', 4', or 5'

For submerged orifice flow $V = C\,(2g\Delta H)^{1/2}$

$C = 0.82$

$V = 0.82 \times (2g\Delta H)^{1/2}$

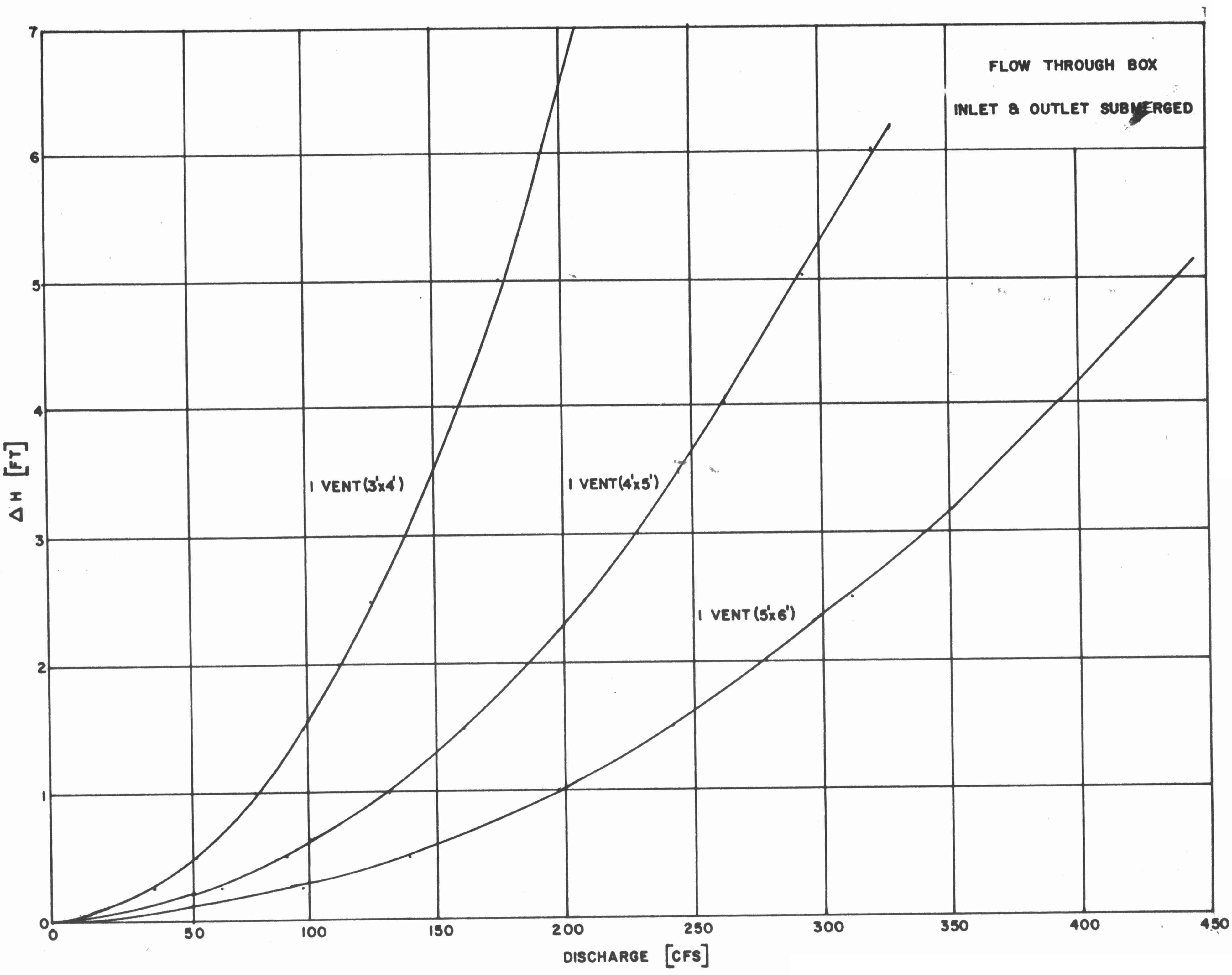
$= 6.58\,(\Delta H)^{1/2}$

$Q = A_{vent} \times v$

Submerged orifice flow : $V = C \ (2g\Delta H)^{1/2}$

$C = 0.82 \qquad Q = A \text{ vent} \times V$

H Ft.	V ft/sec	Discharge Q, cfs. 3 x 4	4 x 5	5 x 6	Remarks
0.25	3.29	39.50	65.80	98.70	
0.50	4.65	55.80	93.00	139.50	
0.75	5.69	68.28	113.80	171.00	
1.00	6.58	79.0	131.60	197.40	
1.50	8.06	96.72	161.20	241.80	From the curves it is seen that a 2 vent structure of 5x6 size may serve the purpose giving Q = 171x2 = 342 cfs which is near & greater than 300 cfs.
2.00	9.30	11.16	186.0	279.0	
2.50	10.40	12.50	208.0	312.0	
3.00	11.40	137.0	228.0	352.0	
4.00	13.16	158.0	263.20	394.8	
5.00	14.71	176.52	294.20	441.30	
6.00	16.12	193.44	322.40	483.6	
7.00	17.41	208.92	384.20	522.30	
8.00	18.61	223.32	372.20	558.30	


FLOW THROUGH BOX
INLET & OUTLET SUBMERGED
1 VENT (3'x4')
1 VENT (4'x5')
1 VENT (5'x6')
ΔH [FT]
DISCHARGE [CFS]
0
1
2
3
4
5
6
7
50
100
150
200
250
300
350
400
450