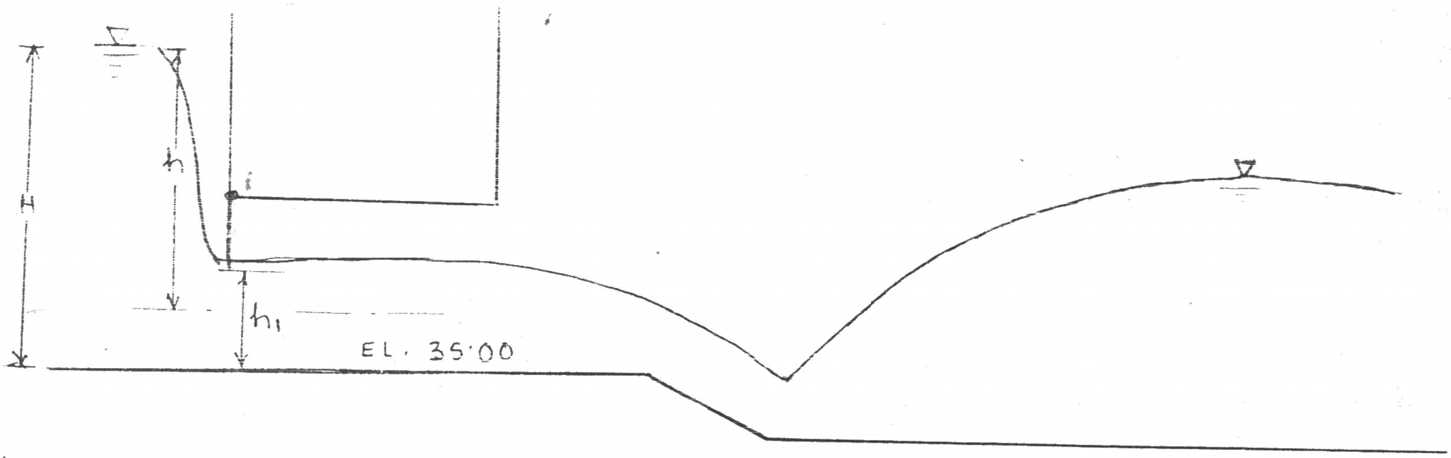
Computation of ventage opening with respect to water level elevation of R/s

Though, our required design discharge is 300 cfs and this may be achieved by a head difference of 0.6 to 0.75 with full opening of the gates. But in cases when the head difference is more, the discharge is to be maintained by progressive closing and opening of the water way as the head difference between R/s and C/s rises or lowers respectively.

So it is required to calculate the gate opening at different W.L. elevation of R/s so that the discharge passed through the vents remains at 300 cfs.



Now

$$Q = Cb\,h_1 \times \sqrt{2g(H-h1/2)}$$

or $300 = 0.60 \times 10\ h1 \sqrt{64.4\ (H-h1/2)}$ [C=0.60, & b = 5$'$ x 2=10$'$ - 0" i.e. width of both vents]

or $h1\sqrt{64.4\ (H-h1/2)} = 50.00$

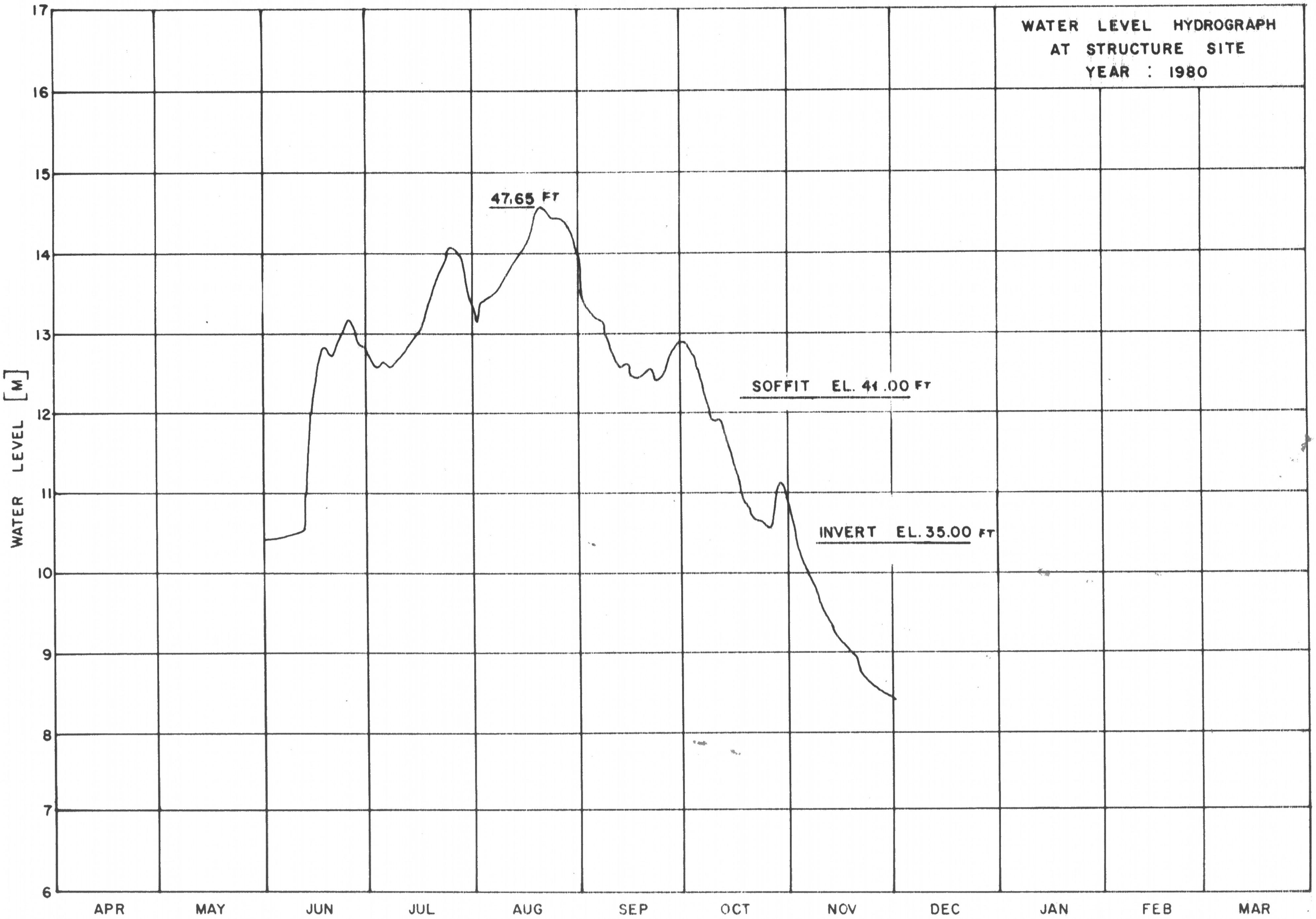
or $h1\sqrt{(H-h_1/2)} = 6.23$

Equating for various values of H, respective values of h_1 can be computed. A curve may be drawn as river side water level elevation vs gate opening so that one can open the gate as required at different elevation of R/s W.L.

Q = 300 cfs. Invert level : 35.00

W.L. Elevation R/s	H [ft]	h1 [ft]
42	7	2.75
44	9	2.25
46	11	2.00
48	13	1.80
50	15	1.60

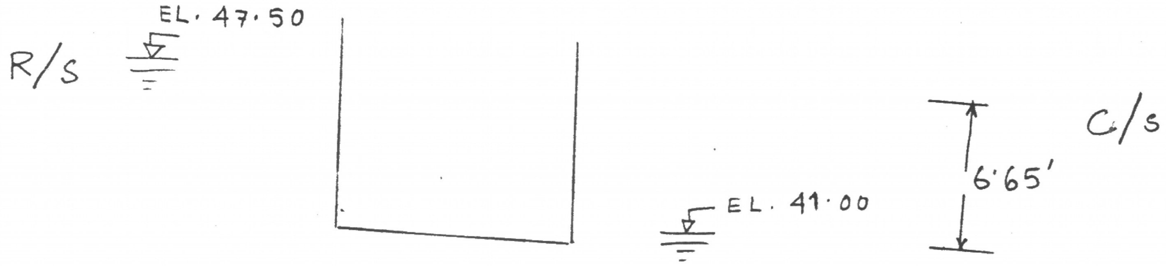
Note: The highest W.L. Elevation on the R/s is 50.00 PWD. (information collected from local people)

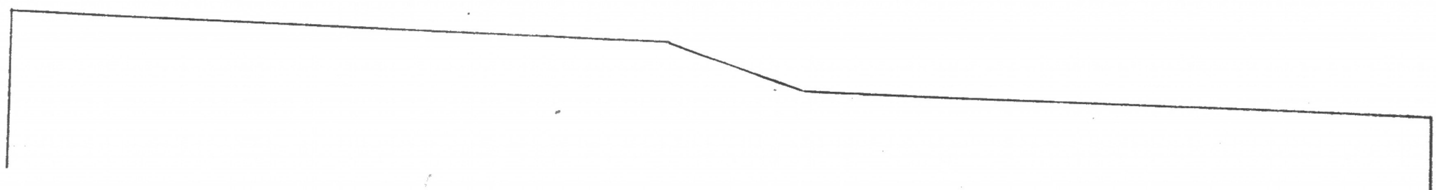

WATER LEVEL HYDROGRAPH
AT STRUCTURE SITE
YEAR : 1980
WATER LEVEL [M]
17
16
15
14
13
12
11
10
9
8
7
6
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC
JAN
FEB
MAR
47.65 FT
SOFFIT EL. 41.00 FT
INVERT EL. 35.00 FT

Floor length by hydraulic jump

Discharge: Though the required design discharge is 300 cfs. But for floor length computation the discharge to be considered at the time of maximum head difference with the gate full opening. This may happen due to ignorance of local people. The maximum W.L. on the R/s is 47.65 PWD. At this time we may consider the W.L. on the country side at 41.00 PWD the bank level of the canal, then the head difference is 6.65.

Here the maximum W.L. of the river has been taken from hydrological year book(H.Graph attached).



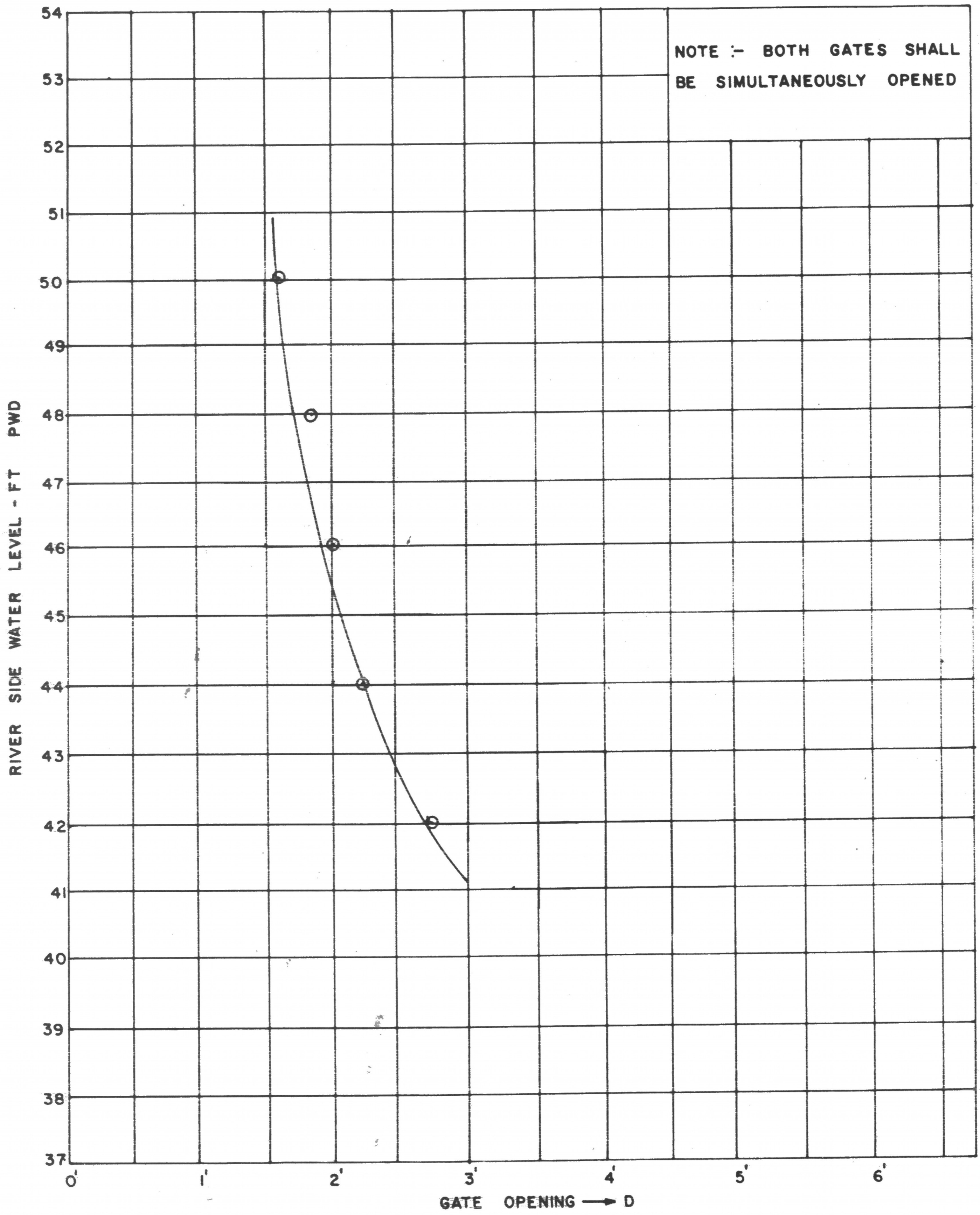


$$V = C\sqrt{2g\Delta H} = 0.08\sqrt{2g \times 6.65}$$

$$= 16.56 \text{ ft/sec.}$$

$$Q = AV = 2 \times 5 \times 6 \times 16.56 = 993.60 \text{ cfs.}$$

$$\text{discharge/ft width } q = \frac{993.6}{10} = 99.36 \approx 100 \text{ cfs/ft.}$$


NOTE :- BOTH GATES SHALL BE SIMULTANEOUSLY OPENED
RIVER SIDE WATER LEVEL - FT PWD
54
53
52
51
50
49
48
47
46
45
44
43
42
41
40
39
38
37
0'
1'
2'
3'
4'
5'
6'
GATE OPENING ⟶ D

Hydraulic Design

Data Given

1. Average bed level of canal : 35.70
2. Bed width of canal : 55'-0"
3. Bank level or Ground level : 41.00
4. Highest flood level : 50.00 PWD from public information(ever experienced
5. Highest flood level : 14.528 M PWD.(From hydrograph)
6. Safe exit gradient : 1/7
7. Silt factor f = 0.40 (in absence of soil report)

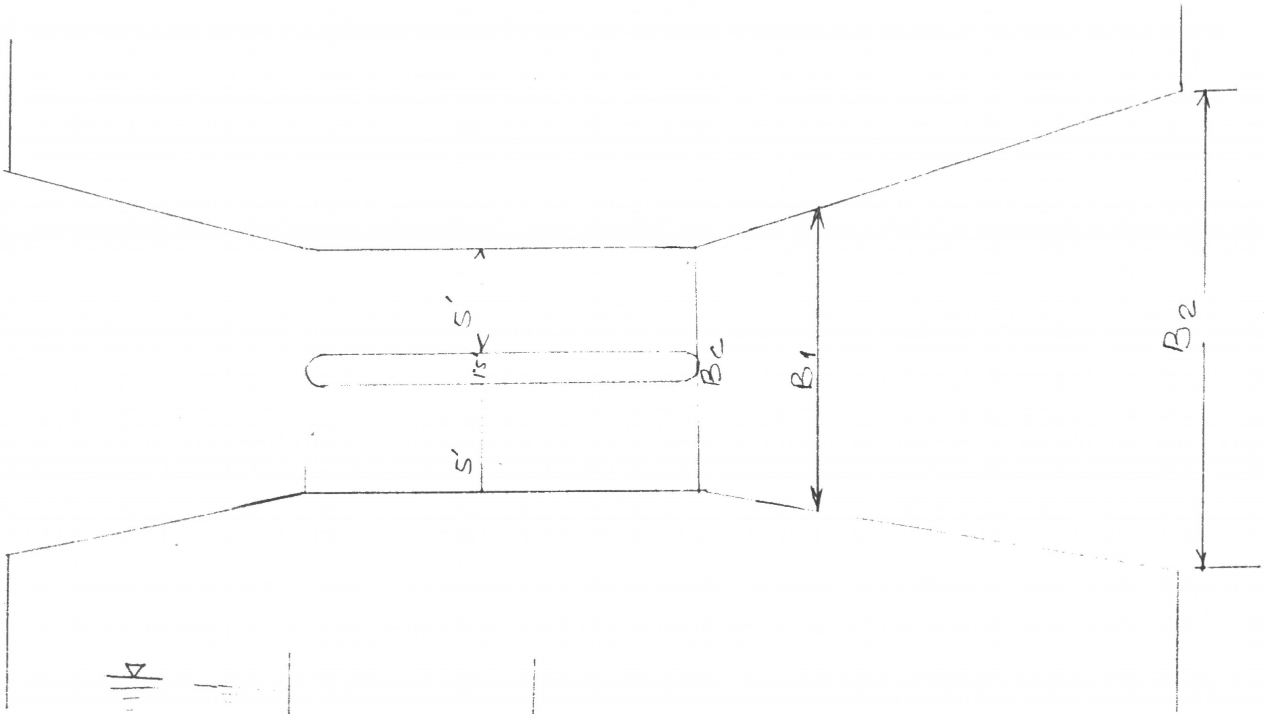
   $f = 1.76\sqrt{dm}$

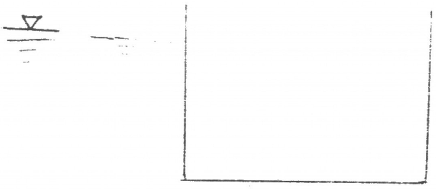
   where dm = average particle size in mm.

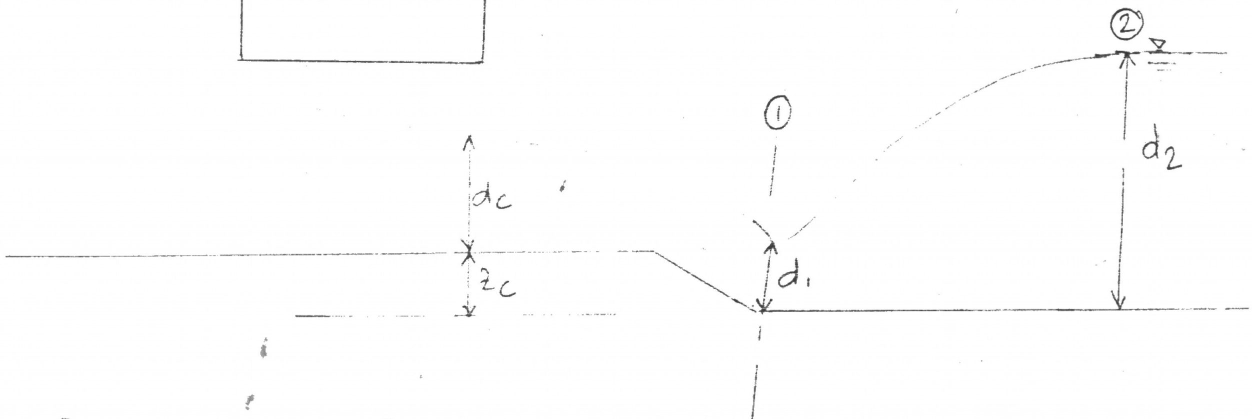
8. Crest width of Embankment : 14'-0"
9. Side slope C/s : 1:2

   R/s : 1:3

10. Proposed Crest Elevation : 53.00 PWD.







$$dc = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{100^2}{32.2}\right)^{1/3} = 6.77 \text{ ft}$$

$$Vc = \frac{q}{dc} = \frac{100}{6.77} = 14.77 \text{ ft/sec.}$$

$$\begin{aligned} BC &= 5\times2 &&= 10.00' \\ Pier &= 1.5\times1 &&= \underline{1.5'} \\ & && \quad 11.50 \end{aligned}$$

$$\begin{aligned} B_1 &= 11.50 + 2 \times 3 \text{ ton } \phi \\ &= 11.50 + 2 \times 3 \text{ ton } 12^{\circ} = 12.77 \text{ ft.} \end{aligned}$$

Applying B.E. at pt (1) & (c)

$$d_c + \frac{V_c^2}{2g} + Z_c = d_1 + \frac{V_1^2}{2g} \quad \ldots\ldots(1)$$

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

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

$$q_1 = \frac{100\times10}{12.77} = 78.30 \text{ cfs}$$

From equation (1) at pt (1)

$$1.5\, d_c + Z_c = d_1 + \frac{v_1^2}{2g} \qquad q_1 = v_1 d_1$$

$$1.5 \times 6.77 + 1 = d_1 + \frac{q_1^2}{d_1^2 \times 2g} \qquad \text{or } v_1^2 = \frac{q1^2}{d1^2}$$

$$\text{or} \quad 11.15 = d_1 + \frac{95.20}{d_1^2}$$

By trial $d_1 = 3.50$ ft.

$$\text{Now} \quad V_1 = \frac{q_1}{d_1} = \frac{78.30}{3.50} = 22.37 \text{ ft/sec}$$

$$\text{Froude No. } F = \frac{V_1}{\sqrt{gd_1}} = \frac{22.37}{\sqrt{32.2\times3.50}} = 2.10$$

Length of jump

We know $$\frac{d_2}{d_1} = \frac{1}{2}\left[(1 + 8F^2)^{0.5} - 1\right]$$

or $$\frac{d_2}{3.50} = \frac{1}{2}\left[(1 + 8 \times 2.10^2)^{0.50} - 1\right]$$

or $$\frac{d_2}{3.50} = 2.51$$

or $$d_2 = 2.51 \times 3.50 = 8.79 \text{ ft.}$$

$$\text{Length of jump} = 6.9(d_2 - d_1)$$

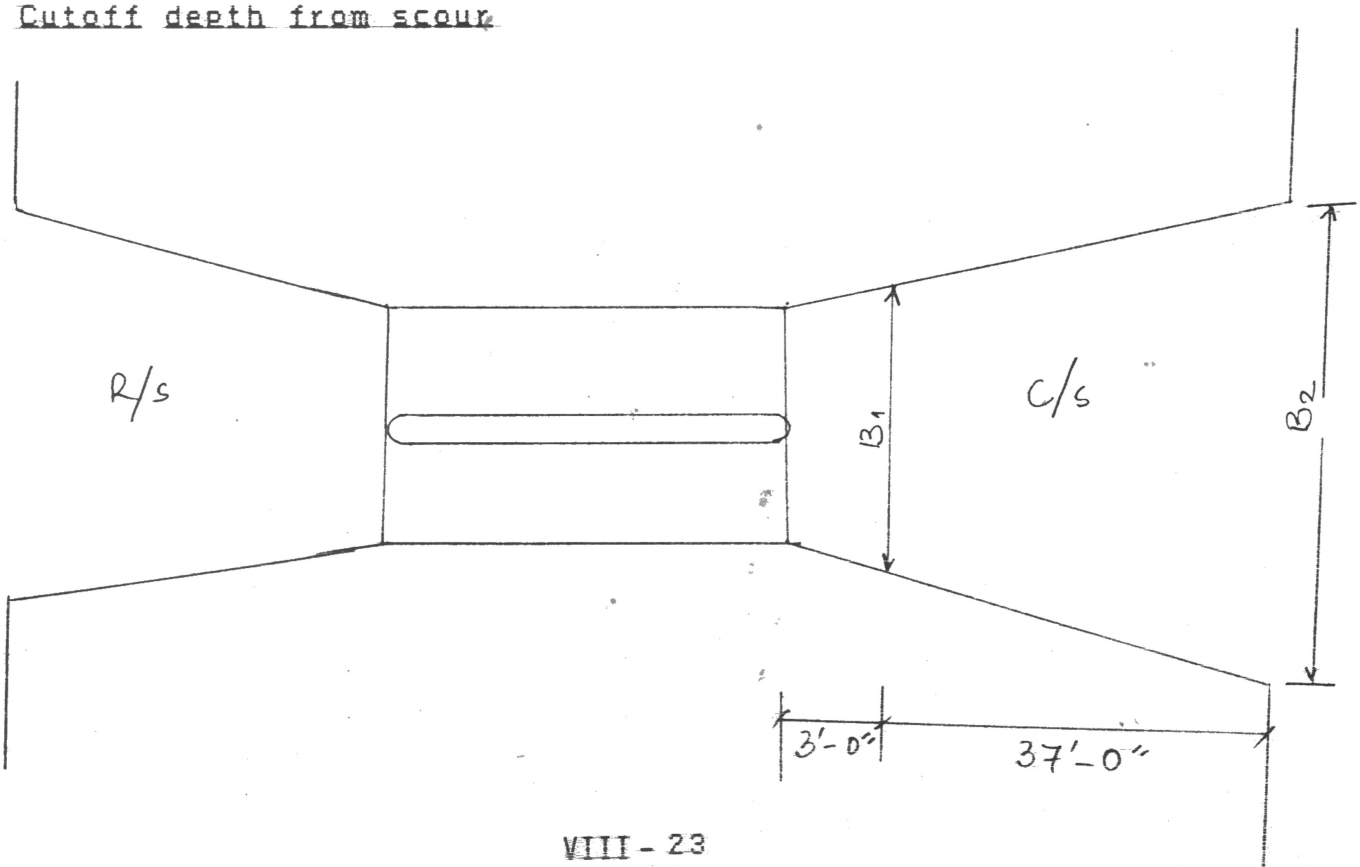
$$= 6.9(8.79 - 3.50)$$

$$= 36.50 \text{ ft.}$$

$$= 37 \text{ ft.}$$

We may provide d/s floor length 37 + 3 = 40 - 0"

Cutoff depth from scour



$$B_2 = B_c + 2 \times 40 \text{ ton } 12^o$$

$$= 11.50' + 17.00$$

$$= 28.50 \text{ ft.}$$

Let us fix B_2 = 30-0"

$$q_2 = \frac{100 \times 10}{30} = 33.33 \text{ cfs/ft. width at point (2)}$$

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

$$= 0.91 \left(\frac{q_2^2}{0.4}\right)^{1/3}$$

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$$= 0.91 \left(\frac{33.33^2}{0.4}\right)^{1/3}$$

.= 12.8 - 0"

Design scour depth

U/s = 1.25 R = 1.25 x 12.8 = 16.00

D/s = 1.50 x R = 1.50 x 12.8 = 19.2

Scour level U/s = U/s W.L. - 16.00

= 47.65 - 16.00

= 31.65 = 32

U/s depth of cutoff required = 35 - 30

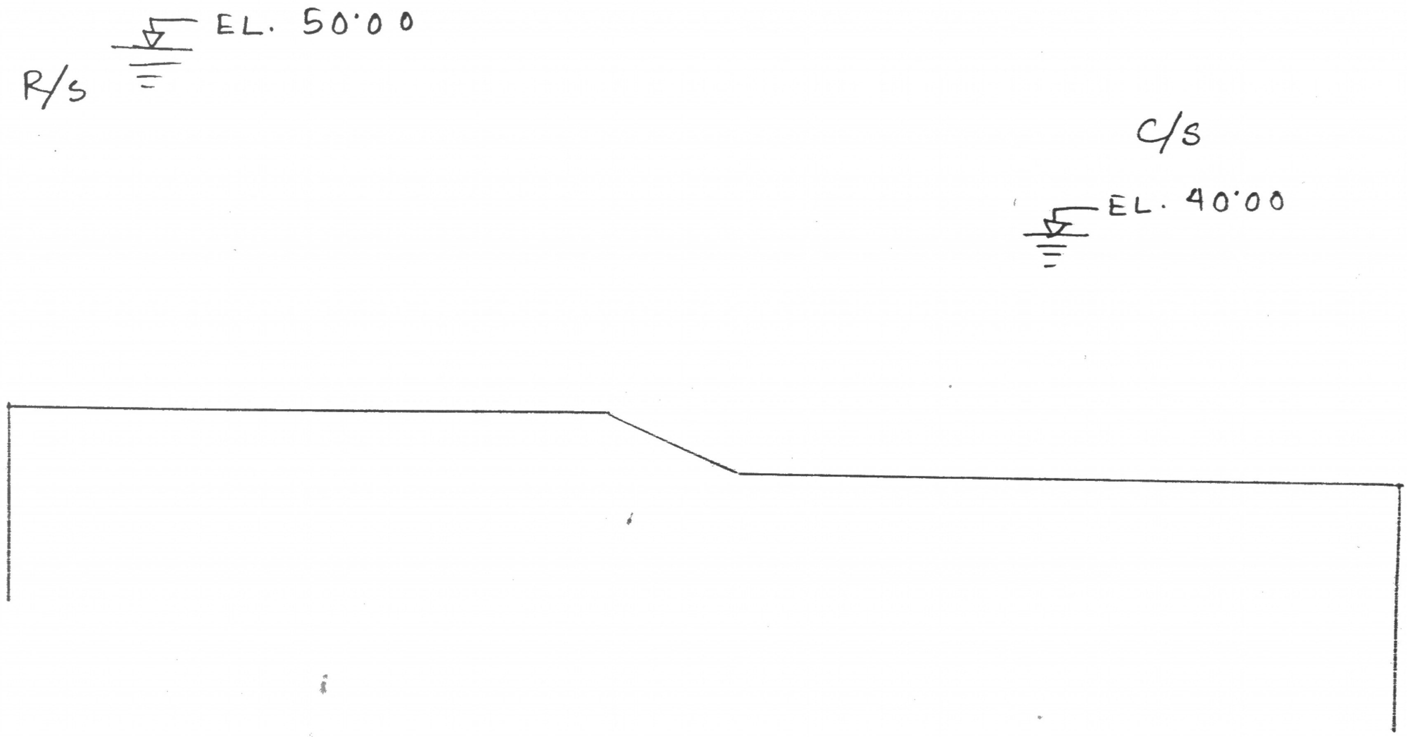
= 5 ft.

D/s depth of cutoff reqd. = 19.20 - 8.79

= 10.41

Use 12'-0"

Floor length & cutoff depth by exit Gradient



Consideration of head difference

In calculating the floor length and depth of cut off by exit gradient it is better to assume that the gate is closed. R/s W.L. is at its maximum elevation & C/s W.L. is at or near to bank level. We shall consider C/s W.L. is at 1' below the bank level i.e. at El. 40.00 and R/s W.L. is at maximum elevation 50.00. (information collected from local people, ever experienced)

In this case $H = 10$ ft.

Taking depth of cutoff d/s = 12 -0"

& $G_{E.} = 1/7$

$$G_{E.} = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}} \quad \text{or} \quad \lambda = \left(\frac{H}{G_{E.}\, d\, \pi}\right)^2$$

$$= \left(\frac{10 \times 7}{12 \times 3.14}\right)^2$$

$$= 3.45$$

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

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

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

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

$$= \sqrt{(2 \times 3.45 - 1)^2 - 1}$$

$$= 5.82$$

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

$$5.82 = \frac{b}{12}$$

$$b = 12 \times 5.82 = 70\text{-}0"$$

On the other hand during winter season while the water level on the river side remains at low stage the gate remains closed and the irrigation is to be done by the stored water on the canal. During this period we may consider the water level on the country side at El. 41.00 while that on the river side at floor level of El. 35.00 resulting a head difference of 6 - 0".

Considering the depth of cutoff 5 on R/s

and G.E = 1/7

$$G_{E.} = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$$

$$\text{or} \quad \lambda = \left(\frac{H}{G_{E.}\, d\,\pi}\right)^2$$

$$= \left(\frac{5 \times 7}{5 \times 3.14}\right)^2$$

$$= 4.96$$

$$\text{again} \quad \alpha = \sqrt{(2\lambda - 1)^2 - 1}$$

$$= \sqrt{(2 \times 4.96 - 1)^2 - 1}$$

$$= 8.86$$

$$\text{Now} \quad 8.86 = \frac{b}{d}$$

$$\text{or} \quad 8.86 = \frac{b}{5}$$

$$\text{or} \quad b = 44.3'$$

The length required by the structure is given by

b = D/s floor length + U/s floor length + Glacis + barrel & operating devices

= 37 + 24 (2/3rd of D/s floor) + 3 + 26

= 90 - 0"

We may fix the floor lenght 90 , U/s cutoff depth 5 and d/s cutoff depth 12 -0".

<u>Floor Thickness in relation to uplift pressure</u>

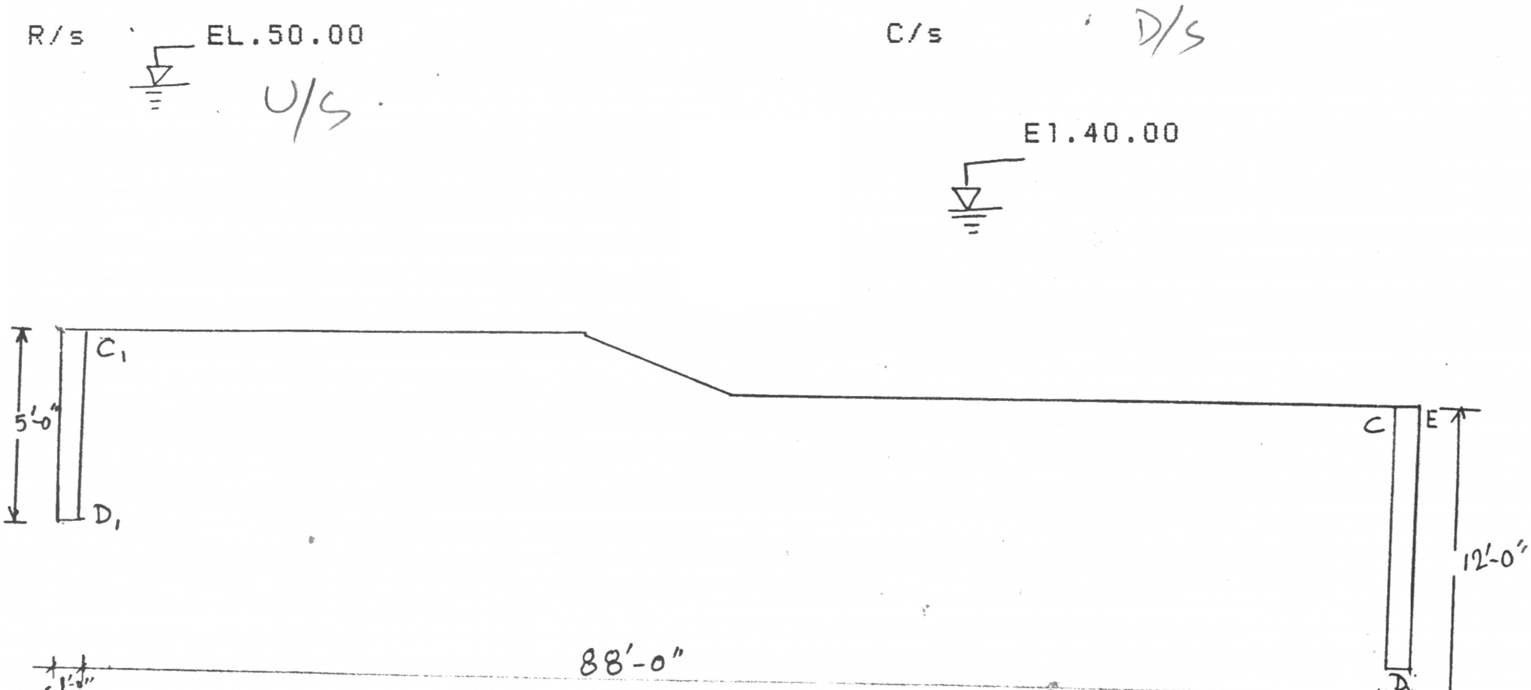
Now we have already fixed

Floor length b = 90 - 0"

depth of cutoff D/s, d = 12 - 0"

depth of cutoff U/s = 5 -0"

Let us assume floor thickness at U/s & D/s end 2 -0"



b' = 88 - 0"

b = 90 - 0"

d = 12 - 0"

D = 5 -0"

$$1/\alpha = \frac{d}{b} = \frac{5}{90} = 0.055$$

From Khosla's Pressure chart

$P_D = 13\%$ $\quad P_{D_1} = 100 - 13 = 87\%$

$P_E = 21\%$ $\quad P_{C_1} = 100 - 21 = 79\%$

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

$$\text{Then} \quad c = 19\sqrt{\frac{D}{b'}} \times \left[\frac{d + D}{b}\right]$$

Effect of u/s cutoff on to the d/s cutoff

In this case D = 3 ft. d = 10 ft.

$$c = 19\sqrt{\frac{3}{88}} \times \left[\frac{10+3}{90}\right]$$

$$= 0.50\% \quad (\text{-ive})$$

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

Effect D/s cutoff on to the u/s cutoff
In this case D = 10 ft.   d = 3 ft.

$$C_1 = 19 \sqrt{\frac{D}{b}} \times \frac{d+D}{b}$$

$$= 19 \sqrt{\frac{10}{88}} \times \frac{3+10}{90}$$

$$= 0.92\% \quad (+ive)$$

Correction due to floor thickness (R/s)

Floor thickness at end is assumed 2'-0".

If C_F is the correction for floor thickness

$$\text{Then } C_F = \frac{87 - 79}{5} \times 2 = 3.2\% \ (+ve)$$

$$P_{C_1} \text{(corrected)} = 79 + 3.2 + 0.92 = 83\%$$

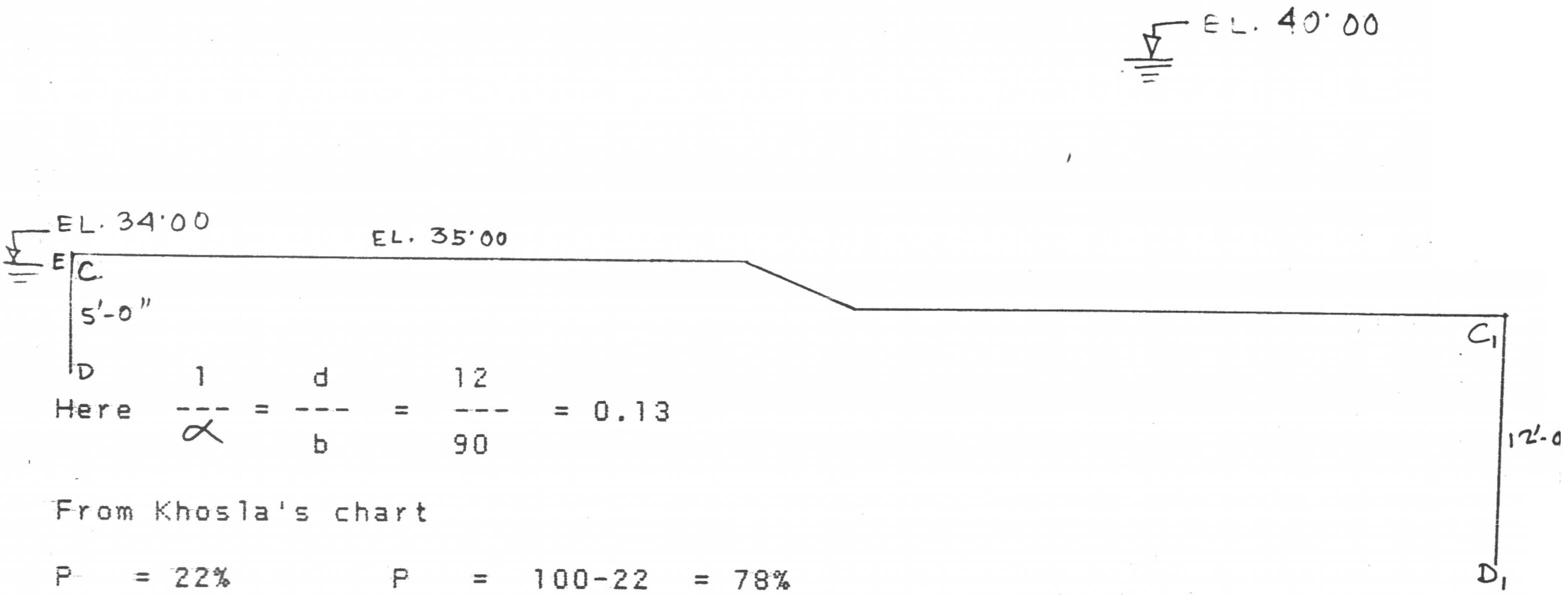
Correction for floor thickness C/s

i) Correction due to interference of cutoff will be due to the interference of R/s cutoff on to the C/s cutoff. Its direction is negative i.e. C = 0.50 (- ve)

ii) Correction for floor thickness $= \frac{21 - 13}{12} \times 2 = 1.33\% \ (- ve)$

$$P_E \text{ (Corrected)} = 21 - 1.33 - 0.50 = 19.17\%$$

Again while the water stored on the canal closing the gate as we assumed before, water level on the canal at El. 40.00 and R/s W.L. at El. 34.00, we may get the uplift pressure as follows:



Here $\frac{1}{\alpha} = \frac{d}{b} = \frac{12}{90} = 0.13$

From Khosla's chart

$P_D = 22\%$ $\quad\quad P_{D_1} = 100-22 = 78\%$

$P_E = 34\%$ $\quad\quad P_{C_1} = 100-34 = 66\%$

<u>Correction due to thickness of floor</u>

If C_F is the correction due to floor thickness then

$$C_{F_{(c/s)}} = \frac{78-66}{12} \times 2 = 2\%$$

$$C_{F_{(R/s)}} = \frac{34-22}{5} \times 2 = 4.8\%$$

Correction due to interference of pile

If C is the correction due to interference of file.

$$C_{(c/s)} = 19\sqrt{\frac{3}{88}} \times \frac{3+10}{90}$$

$$= 0.50\% \text{ (+ive)}$$

$$C_{(R/s)} = 19\sqrt{\frac{10}{88}} \times \frac{3+10}{90}$$

$$= 0.92\% \text{ (-ive)}$$

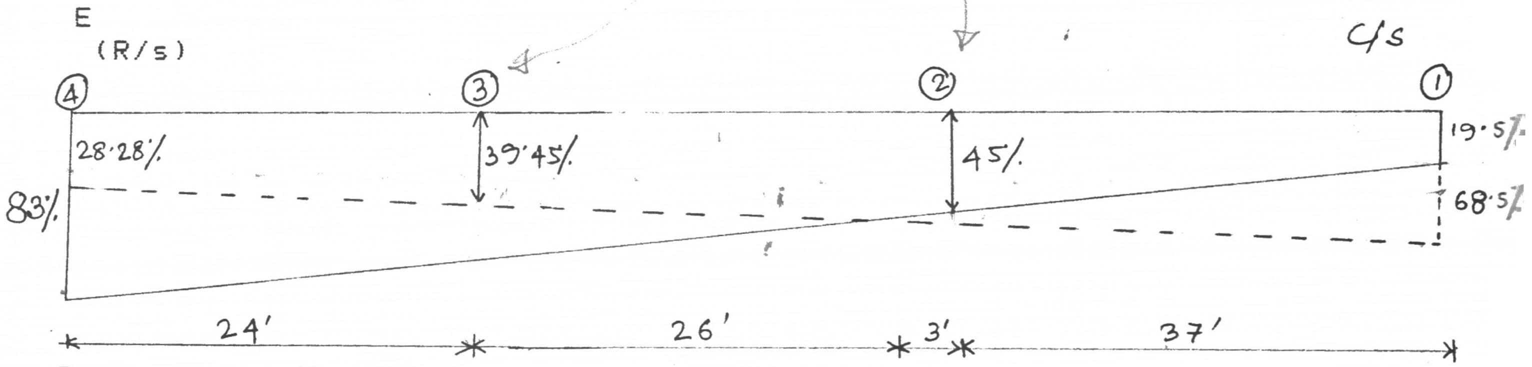
$\frac{64}{90} = \frac{x}{37}$

Correction for floor thickness

$$P_{C_{1}(c/s)} = 66+2+0.50 = 68.50\%$$

$\frac{40.22}{50} = \frac{x}{21}$

$$P_{E(R/s)} = 34-4.8-0.92 = 28.28\%$$



90

Pressure in ft of water

$$P_{1} = \frac{19.17}{100} \times 10 = 1.92 \text{ ft. of water}$$

$$P_{2} = \frac{45}{100} \times 10 = 4.5 \text{ " " "}$$

$$P_{3} = \frac{39.45}{100} \times 6 = 2.36 \text{ " " "}$$

$$P_4 = \frac{28.89}{100} \times 6 = 1.75 \text{ ft} \quad " \quad "$$

Here the critical uplift pressure will be at point (3) & (4) while C/s W.L. controlling. Because at point (1) & (2) the uplift pressure is less than the downward vertical pressure of water & the part betn pt. (2) & (3) there exist barrel, top slab, soil etc. which will counter balance the upward pressure exerted by head difference. On the other hand while R/s W.L. controlling the critical point is at point (1) & (2).

Required Thickness of floor

The uplift pressure exerted by difference of head is to be counter balanced by equivalent wt. of concrete i.e. sufficient thickness of floor is to be provided.

Submerged unit wt. of concrete $= \dfrac{\gamma_{conc} - \gamma_{water}}{\gamma_{water}}$

i.e. specific weight of concrete $= \dfrac{150 - 62.40}{62.40}$

$= 1.40$

So the required thickness of concrete to be provided at

Pt. (4) i.e. at end $t_4 = \dfrac{1.75}{1.40} = 1.25'$

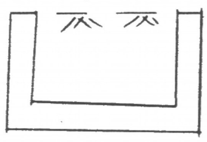
Pt. (3) $t_3 = \dfrac{2.36}{1.40} = 1.70'$

$$\text{Pt. (2) } t_2 = \frac{4.5}{1.40} = 3.21'$$

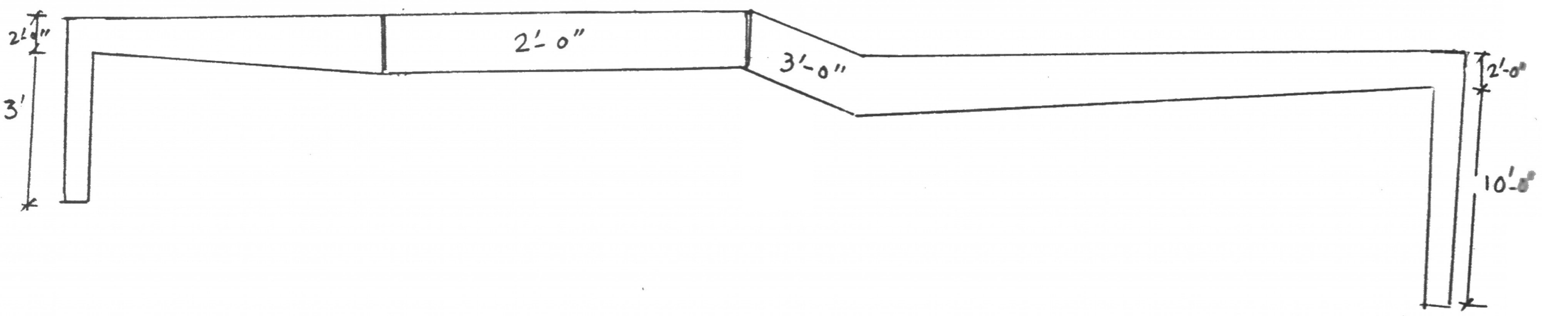
$$\text{Pt. (1) } t = \frac{1.92}{1.40} = 1.37'$$

So thickness of floor at end 2-0" at glacies 3'-0' on both sides and at barrel part 2-0" may be provided.

R/s



C/s



PREFACE

This publication includes

a. methodologies and details of computations for the design of a non tidal drainage regulator

b. methodologies and computations for hydrologic and hydraulic design of a non tidal and a tidal regulator and

c. hydrologic and hydraulic design of a flushing regulator

The purpose of these volumes are to train engineers of Bangladesh Water Development Board on design of small scale water control structures.

These publications have been produced with financial assistance from Canadian International Development Agency for the Small Scale Water Control Structures (SSWCS) Project.

The SSWCS Project has a training component which includes following categories of training :

1. Field workshops for Junior level field officers and contractors on construction management and O&M.

2. Dhaka Workshops for mid level engineers on planning and design aspects of small scale projects.

3. Advanced applied programs for mid level engineers at the Asian Institute of Technology, Thailand to provide in-depth training on water resource development.

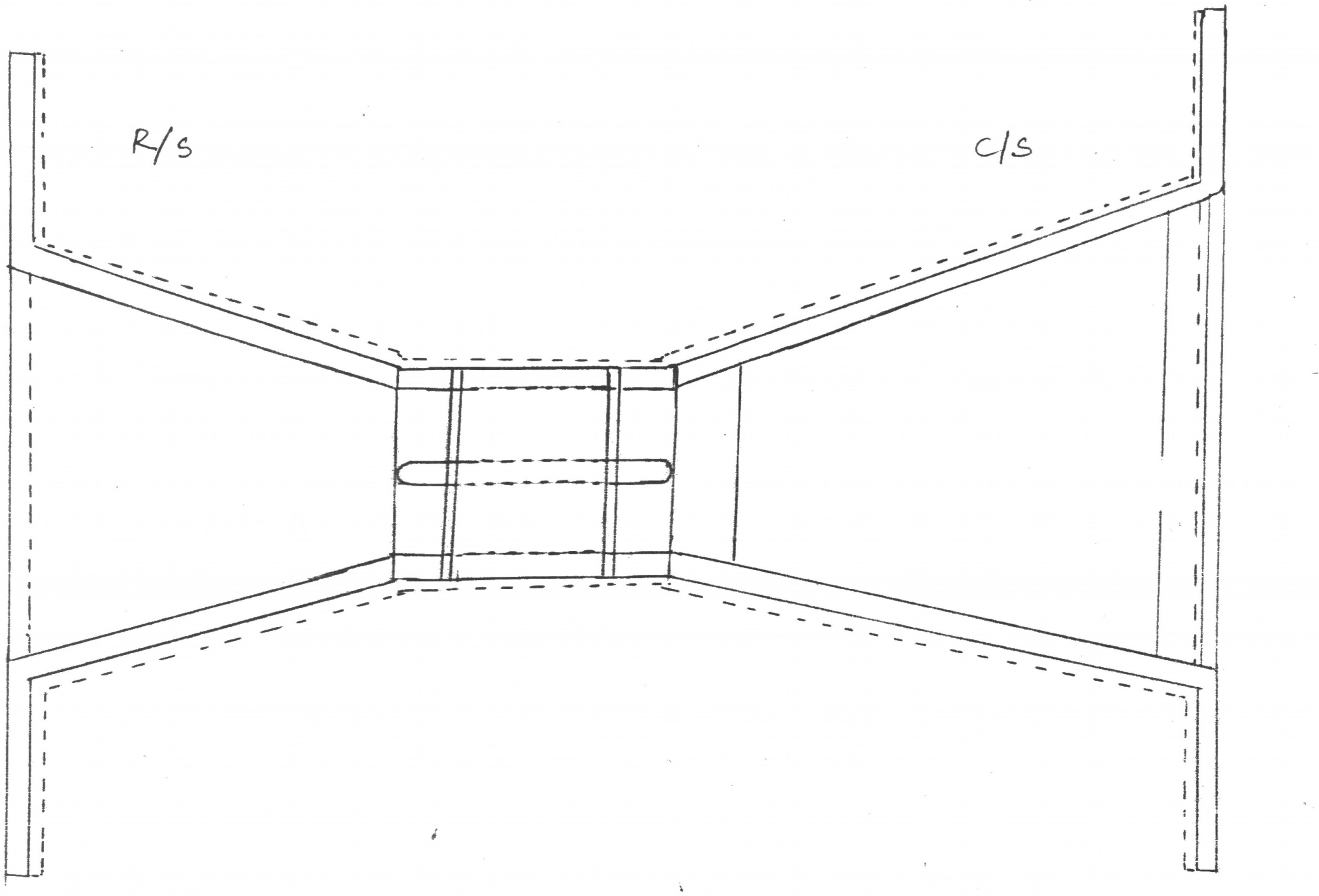
4. Brief Technical Missions by senior officials to Canada and South East Asian countries to visit similar programs.

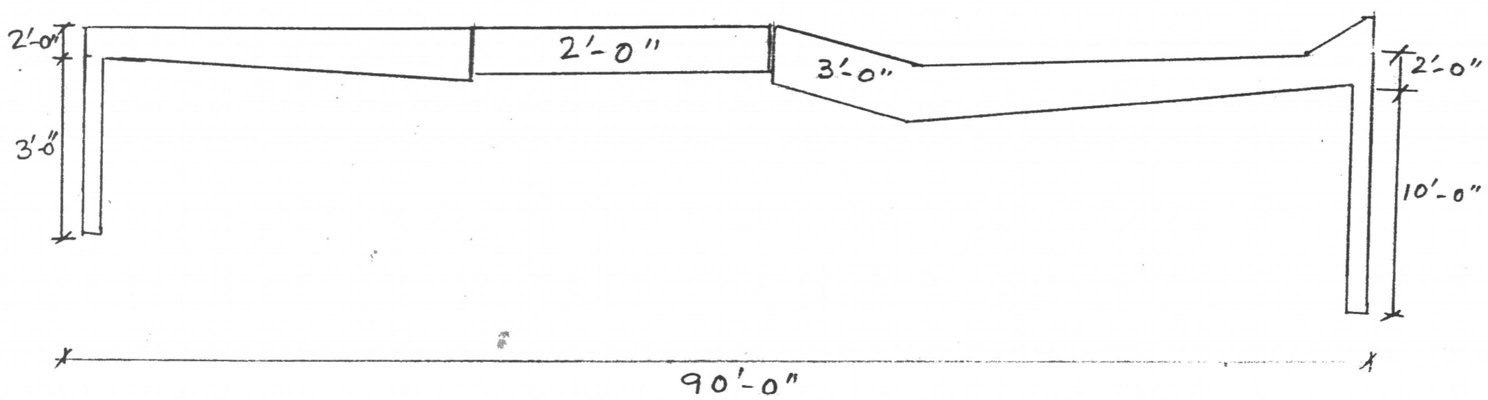
These volumes have been prepared to meet the partial requirements of training materials for 2nd and 3rd categories of training.

Any suggestions to improve the quality of contents will be greatly appreciated.

June, 1987

H.R. Khan
Water Resource Advisor.


R/S
C/S


2'-0"
3'-0"
2'-0"
3'-0"
2'-0"
10'-0"
90'-0"