REMODELLING BAMBANWALA CROSS REGULATOR

RD 133296 U.C.C.

BY

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GENERAL INTRODUCTION TO WEIR DESIGN

For a hydraulic structure built on permeable soil the main design problems emanate from the under-seepage due to the head across and the energy released by the water flowing over the structure. The design of hydraulic structures therefore involves two aspects :-

(1). Hydraulic calculations.

(2). Structural design

The first step in design are the hydraulic calculations. The quantities dealt with in this analysis are discharges, velocities, water slopes and water levels. Next follows the structural design of works covering the calculations and design of all the structural details, preparation of final drawings showing fixed and movable parts of the work. Most often an irrigation facility (with the exception of masonry or concrete dams) is constructed on a pervious alluvial soil foundation. In this case the fixed part of the work can be divided into the superstructure, and the under structure or substructure. The former comprises the piers, abutments, retaining wall, arches and all the rest of the upper part of the structure. The substructure generally consists of a continuous masonry or concrete foundation known as the floor or apron (which extends beneath all of the structure) and of the cut-off or drop walls, or sheet piles, masonry wells placed under the apron, in order to render it more effective and reliable, (Leliavsky, 1955). The present work deals essentially with the phenomena of under seepage/laminar viscous flow through the capillary inter-stices of the porous media in connection with seepage discharge, the energy loss measured by the head loss experienced by the water percolating through the medium under the structure, the stability of the structure against uplift pressure and the safety against excessive exit gradient/undermining by piping.

In a structure built upon a pervious soil with the water level upstream of the structure higher than it is downstream, enforced filtering or percolation will, occur in the under-lying permeable soil. The design of the apron is intimately connected with this filtering. Two sources of weakness are to guarded against :-

(i) Percolation may cause undermining of the pervious foundation which would necessarily be followed by the collapse of the whole structure. In particular the percolating water attains sufficient force to lift up soil grains at the

downstream end of the structure. This leads to increased porosity of the soil and hence a progressive intensification of seepage flow ultimately resulting in a rush of water and soil beneath the foundation of the structure, causing it to subside into the hollow so formed.

(ii) The floor, or apron may be forced upwards owing to the upward pressure of water filtering through the previous soil beneath the structure. Percolating water exercises an upward pressure on the structure until it emerges at the downstream end, and if the pressure in not counter balanced by the weight of the concrete or masonry above, the structure will fail by the rupture of part of its floor.

1.0 INTRODUCTION TO THE REMODELLING WORK

With this background of weir design, remodelling of Bambanwala Cross Regulator at RD. 133296 U.C.C. would be presented under the following phases :-

1) Problem with the existing structure
2) Possible remedies
3) Remodelling proposals and final design
4) Check of design
5) Execution of the work including planning/construction
6) Accidents on the work
7) Summary and conclusions

The structure in question was unsafe against both, the uplift pressures under the downstream floor and the exit gradient on account of being subject to a head across greater than design due to unforeseen conditions imposed on the structure sometime after construction.

1.1 HISTORY

The Upper Chenab Canal was constructed in 1905-12, off-taking from Marala Headworks on the River Chenab in Punjab and having a design discharge of 11694 Cusec. About 27 miles downstream of the off-take, U.C.C. trifurcated into :-

1) Main Line Lower
2) Rayya Branch
3) Nokhar Branch

The Index Plan is at Appdx-I. U.C.C was remodelled in 1949-51, subsequent to remodelling of Rayya Branch which came to be called the Bambanwala - Ravi - Badian - Depalpur Link (B.R.B.D Link) after enhancement of its capacity. As constructed and latest data of the channels at the said cross-regulator is as under :-

Name of Channel	U.C.C.C.M.L.U Initial/Final Figures	M.L.I..	Rayya Br. Latter B.R.B.D.	Nokhar Branch
Design Discharge (Cusec)	11694	9434/11375	1444/5140	622/718
Full Supply Level	781.21/783.16	776.48/775.11	N.A	N.A
Head Across feet	--	4.73	N.A.	N.A
Full Supply Depth (feet)	11.05/13	10.07/10.8	6.4/806	5.4/5.77
Bed Width (Feet)	240/320	65/190	3ε ,6	
Bed Level (R.L.)	770.16	766.41	769.10	773.35
D/S Floor Level (R.L.)	--	766.41	--	772.0
Crest Level (R.L.)	--	772.66	773.34	N.A

When B.R.B.D and U.C.C. were remodelled, no simultaneous remodelling or strengthening of the Bambanwala Regulator was carried out and it remained safe for a head across of 4.73 feet only. After partitioning of the Indo-Pakistan sub-continent, irrigation supplies to Upper Depalpur Canal System (refer Index Plan at Appdx-I) were severed due to cutting off of the same from Madhopur headworks in India. To restore supplies, Rayya branch was remodelled for an enhanced capacity of 5140 Cusec in 1949-51 and extended to join and feed the severed Upper Depalpur Canal System.

Accordingly U.C.C. the parent Channel was also remodelled for an increased capacity of 16850 Cusec as appearing on the above Table. B.R.B.D. Link as extended after partition, also served as a first line of defence along Indo-Pakistan Border. Due to this reason the channel was placed in first preference at Bambanwala for regulating available supply in the Upper Chenab Canal.

During summer when there was ample supply available in River Chenab at Marala for diversion into U.C.C., there was no problem in running off-takes at Bambanwala. It was only during the winter months of December - February when river supply dwindled to about 4000 Cusec, that running of B.R.B.D to its winter indent/defence requirement of 3700 - 3800 Cusec was not possible. Under such conditions it was essential to divert the entire supply in

the U.C.C. into B.R.B.D. requiring maximum heading up upstream and a dry downstream floor. U.C.C. from Marala to Bambanwala regulator falls in the jurisdiction of Marala Headworks Division U.C.C., which the author held during the period December 1986 - September 1989. This remodelling work was carried out in the Annual Closure of December 1988 - January 1989 during the incumbency of the author as Executive Engineer.

1.2 ANALYSIS OF THE EXISTING STRUCTURE

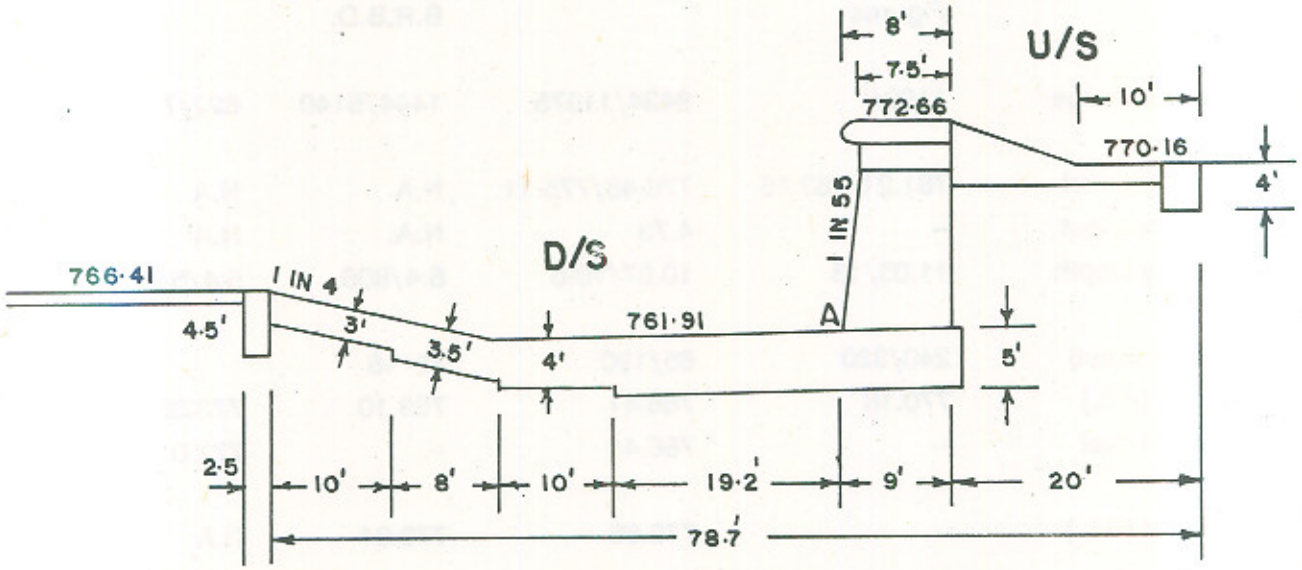


FIG.1 CROSS SECTION OF THE STRUCTURE AS CONSTRUCTED. (ALSO REFER APPENDIX II)

1.2.1 DESIGN EXIT GRADIENT

According to Khosla, exit gradient (GE) is given by the equation:

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

Where H = Head across
d = depth of downstream cut-off

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

where

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

b being length of floor

Here H = 4.73 ft.

d = 4.5 ft.

$$\alpha = \frac{b}{d} = \frac{78.7}{4.5} = 17.5$$

$$\lambda = \frac{1 + \sqrt{1 + (17.5)^2}}{2} = \frac{1 + \sqrt{306.86}}{2} = \frac{1 + 17.52}{2} \quad \text{or } \lambda = 9.26$$

$$GE = \frac{4.73}{4.5} \times \frac{1}{3.142\sqrt{9.26}}$$

or GE = 0.11 or 1 in 9.1

To safeguard against piping, the exit gradient must not be allowed to exceed a certain limit for different soils as given below :-

Nature of Soil	Factor of Safety	Safe Exit Gradient
Shingle	4 - 5	0.25 - 0.2
Coarse Sand	5 - 6	0.2 - 0.17
Fine Sand	6 - 7	0.17 - 0.14

1.2.2. SAFE HEAD ACROSS FROM EXIT GRADIENT CONSIDERATION

Luckily a liberally safe exit gradient of 1 in 9 resulted from a design differential head of 4.73 ft. for the structure as constructed. The structure was safe for a head across of 7.2 ft. for prescribed exit gradient of 1 in 6, for medium sand, by Khosla, as per the following calculation :

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

Here GE = 0.167
d = 4.5 feet
$\sqrt{\lambda}$ = 3.04 (as before)
$\therefore H = GE \times d \times \pi \sqrt{\lambda}$
= 0.167 X 4.5 X 3.142 X 3.04
or H = 7.2 feet

1.2.3 CRITICAL HEAD ACROSS FOR EXISTING STRUCTURE AFTER REMODELLING OF U.C.C./B.R.B.D

The critical Head Across, to which the structure would be subjected under limiting conditions, is calculated below as 12.4 feet :-

Discharge required at Head B.R.B.D. = 3800 Cusec, Gauge reading from Gauge - Discharge Table = 8.8 (given below), Bed Level of B.R.B.D = 769.10

Therefore supply level at BRBD Head	=	769.10 + 8.8 = 777.9
Depth over crest	=	777.9 - crest level
	=	77 .9 - 773.34
	=	4.56 ft
Minimum Modular Head (M.M.H)	=	0.2 x 4.56
(standard value = 0.2 x H)	=	0.91 ft
Water level in Main Line Upper U.C.C.	=	773.34 + 4.56 + 0.91 = 778.81
Design head across	=	778.81 - 766.41 (D/s Floor level)
	=	12.4 ft

DISCHARGE TABLE OF B.R.B.D

(Relevant extract)

Guage (Feet)	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
8	3001	3087	3194	3242	3392	3493	3594	3693	3802	3907
9	4041	4121	4230	4339	4450	4563	4676	4790	4906	5022
10	5140									

1.2.4 BLIGH'S CREEP CO-EFFICIENT

From Fig 1

Length of Creep = (4+3+7.25+25+5+1.5+4.5) + (20+9+19.2+10+8+10+2.5)

or L_B = 103.95 ft

Maximum Head Across = 12.4 ft

Bligh's Creep Co-efficient C_B = LB/H = 103.95/12.4

or C_B = 8.4

Permissible Values for Bligh's Creep co-efficient appear on the following Table :-

Class of Material	Safe Value
River beds of light silt or sand of which 60% Passes the 100-mesh sieve, as those of the Nile or the Mississippi rivers (Fine sand and silt).	18
Fine micaceous sand of which 80% passes a 75 - mesh sieve, as in Himalayan Rivers and Colorado River	15
Coarse-grained sands, as in Central and South India Boulders or shingle and gravel and sand mixed	4 - 6

The Minimum value of C_B prescribed for the nature of sand substratum in the instant case is 15.

Actual value of C_B being much less than that required, the structure is unsafe against piping.

1.2.5 BLIGH'S FLOOR THICKNESS REQUIRED

Worst conditions are at point A in the structure

Creep length up to Point A = (4+3+7.25+5) + (20+9) ft.

= 48.25 ft

Pressure @ point A = $\frac{12.4\ (1 - 48.25)}{103.95}$ = 6.7 ft.

Uplift pressure @ A = $\frac{6.7}{12.4}$ X 100 = 54%

Floor thickness required at A = $\frac{6.70}{2.24 - 1}$ X factor of safety

Specific gravity of floor material = 2.24

Required floor thickness = $\frac{6.70}{1.24}$ X 1.2

= 6.50 ft. against existing 5.0 ft.

Floor is inadequate to resist uplift pressure.

1.2.6 BLIGH'S SAFE HEAD ACROSS

Creep length = 103 . 95 ft.

Permissible creep co-efficient = 15

Safe head across = $\frac{103.95}{15}$ = 7 ft

1.2.7 LANE'S WEIGHTED CREEP CO-EFFICIENT

(giving 3 times weight to vertical creep in comparison to the horizontal)

Creep Length (4+3+7.25+1.5+4.5) + $\frac{(20+9+19.2+10+8+10+2.5)}{3}$

= 25.25 + 26.23 ft.

= 51.48 ft.

$$\text{Lane's Creep Co-efficient} = \frac{51.48}{12.4} = 4.3$$

Permissible Value of Lane's Weighted creep Co-efficient are as under :-

Material	CW Weighted creep ratio
Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0

Lane's Creep Co-efficient deriving from the Structure is 4.3 whereas the minimum required, for type of soil in the substratum in the instant case is 6.0

The Structure is therefore completely unsafe against piping.

Floor Thickness required.

Thickness at A:-

$$\text{Pressure} = 12.4 \times \frac{[1 - (4 + 3 + 7.25) + \frac{(20 + 9)}{3}]}{51.48}$$

$$12.4 \left(1 - \frac{28.91}{51.48}\right) = 5.5 \text{ ft.}$$

$$\text{Required Floor Thickness} = \frac{5.5 \times 1.2}{1.24}$$

$$= 5.30 \text{ ft against } 5.0 \text{ ft. existing.}$$

1.2.8 LANE'S SAFE OPERATIONAL HEAD ACROSS

Creep Length = 51.48 feet
Permissible co-efficient = 6
Safe head Across = $\frac{51.48}{6}$
= 8.67 feet

1.2.9 KHOSLA'S EXIT GRADIENT

b (width of structure) = 20+9+19.2+10+8+10+2.5
= 78.7 ft

d (depth of downstream cut-off) = 4.5 ft

$$\alpha = \frac{b}{d} = \frac{78.7}{4.5} = 17.5$$

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

$$\lambda = \frac{1 + \sqrt{1 + (17.5)^2}}{2}$$

$$= \frac{1 + \sqrt{307}}{2} = \frac{1 + 17.54}{2} = 9.27$$

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

$$\text{Exit Gradient} = \frac{12.4}{4.5} \times \frac{1}{3.142 \times \sqrt{(9.27)}}$$

$$= \frac{12.4}{4.5 \times 3.142 \times 3.05}$$

= 0.29 (1 in 3.05) which is in excess of the prescribed 0.167 (1 in 6).

1.2.10 KHOSLA'S UPLIFT PRESSURES

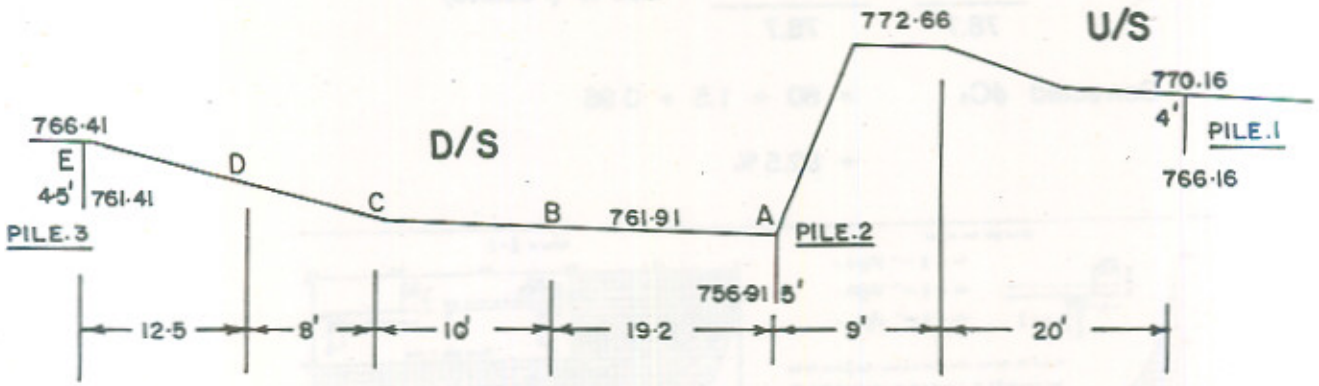


FIG.2 SIMPLISTIC LINE DIAGRAM OF THE WEIR.

Upstream Cut-off - Pile Line 1.

$$\frac{1}{\alpha} = \frac{d_1}{b} = \frac{4}{78.7} = 0.051$$

From Khosla's Curves (Fig.3) Top corner for sheet Pile at end :-

$\phi C1 = 100 - 20 = 80\%$ $(100 - \phi E)$

$\phi D_1 = 100 - \phi D = 100 - 14 - 86\%$

Thus although the lower curve in Khosla's diagram is labelled ϕE, it furnished both ϕE, ϕC on the principle of reversibility of flow.

Correction of floor thickness = $\frac{(86-80)}{4} \times 1.0 = 1.5\%$ (positive)

Correction for Mutual Interference of piles i.e. effect of downstream pile on upstream pile line

$$C_i = 19\sqrt{\frac{D}{b_1}} \times \frac{(D+d)}{b}$$

D = 770.16 - 761.41 = 8.25 ft.

d = 4.0 ft.

b_i = 78.7 ft.

$$\therefore C_i = 19 \quad \frac{8.25}{78.7} \times \frac{(8.25 + 4)}{78.7} = 0.96\ \%\ \text{(Positive)}$$

$$\therefore \text{Corrected } \phi C_1 = 80 + 1.5 + 0.96$$

$$= 82.5\ \%$$

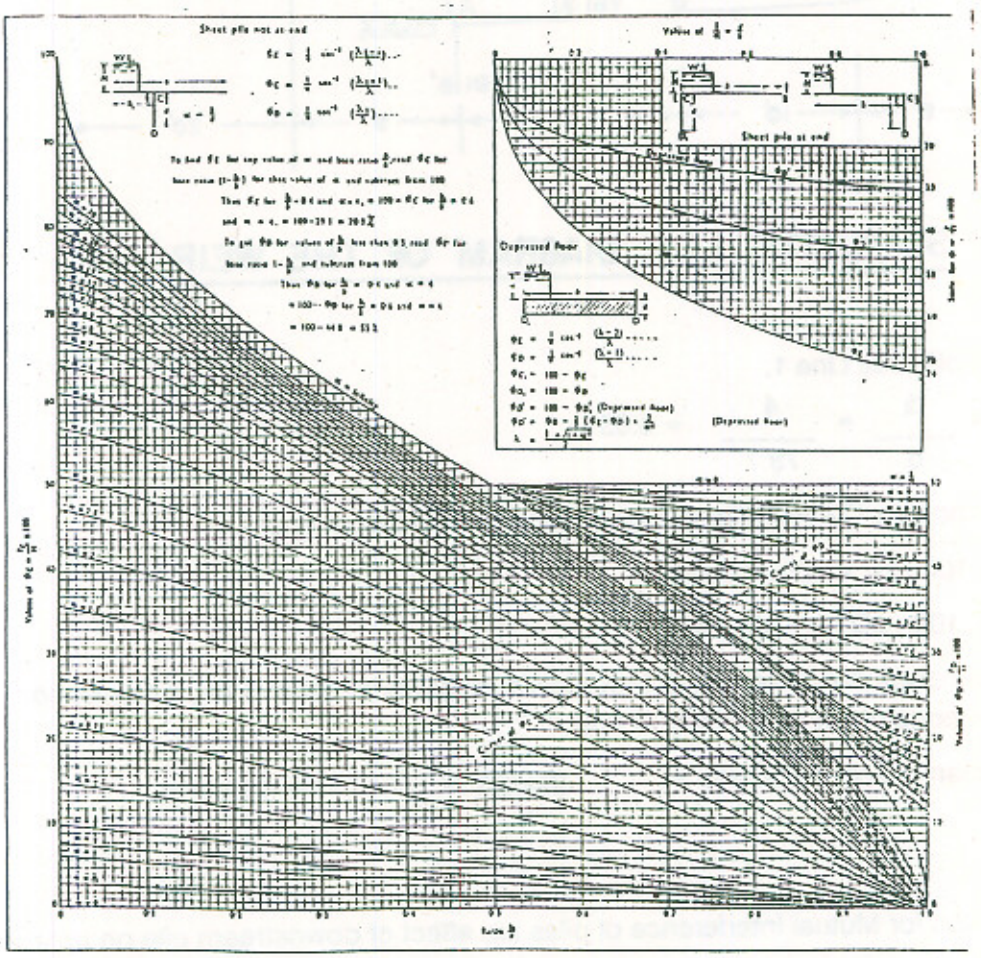


FIG -3

Pile Line - Toe of Drop Wall
Following S.K. Garg (p.553), 1983

$$\frac{b_1}{b} = \frac{29}{78.7} = 0.37$$

Following Khosla (p.145)
calculation for Deg Escape Regulator.

$$\frac{b_1}{b} = 0.37$$

$d = 761.91 - 756.91 = 5$ feet

$\alpha = \frac{b}{d} = \frac{78.7}{5} = 16$

From Khosla's Curve (Fig. 3)

$\phi D_2 = \phi A = 100 - 45 = 55\%$

$d = 766.41 - 756.91 = 9.5$ feet

$\alpha = \frac{b}{d} = \frac{78.7}{9.5} = 8.5$

From Khosla's curves

$\phi D_2 = \phi A = 100 - 44 = 56\%$

Thickness required at A (to resist uplift pressure)

$$t_A = \frac{\phi A}{100} \times \frac{H}{G-1} \times \text{Factor of safety}$$

Here H = 12.4 ft.

G = Specific gravity of floor masonry = 2.24

F.O.S. = 1.2

$$\text{Required } t_A = \frac{55}{100} \times \frac{12.4}{2.24 - 1} \times 1.2 = 6.60 \text{ feet}$$

against existing thickness of 5.0 feet.

Floor at A is unsafe against uplift

Pile line 3 - DOWNSTREAM CUT-OFF

$$\frac{1}{\alpha} = \frac{d_2}{b} = \frac{4.5}{78.7} = 0.057$$

From Khosla's curves (right hand top of Fig. 3)

$\phi E_3 = 21\%$

$\phi D_3 = 14\%$

Correction for floor thickness $= \frac{3}{4.5}(21-14) = 4.7\%$ (-ive)

Correction for interference of piles is NIL,

Since D = 766.16 - 766.41 ≈ 0

Slope Correction - reference Fig. 4

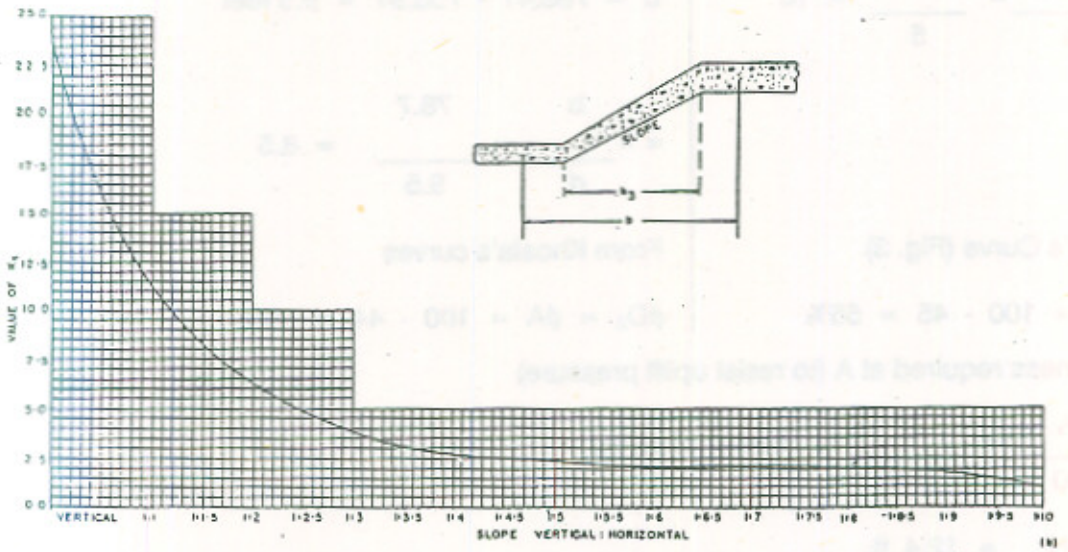


FIG 4 UPLIFT PRESSURE DISTRIBUTION UNDER FLOOR WITH DIFFERENT SLOPES - CORRECTION FOR SLOPING UNDER-SIDE OF FLOOR

$$C_x = \frac{b_3}{b} \times Ks$$

Here b_3 = 10 + 8 = 18 feet,

b = 19.2 + 9 + 10 + 8 + 10 = 47.2 feet

& $Ks = \frac{766.41 - 761.91}{18} = \frac{4.5}{18} = \frac{1}{4}$

or Ks = 2.75 (from Fig. 4)

& $C_x = \frac{18}{47.2} \times 2.75$ = 1% (negative, upslope)

ϕE_3 Corrected = 21 - 4.7 - 1 = 15.3% = 15.5%

Floor Thickness Required :

a) Pressure at point E = $\frac{15.5}{100} \times 12.4$ = 1.92 feet

Floor thickness required = $\frac{1.92}{2.24 - 1} \times 1.2 = 1.86$ feet

against existing 3.0 feet

b) Point B

Pressure at point A = 0.55 x 12.4 = 6.82 feet.
Pressure at point E = 1.92 feet
Creep Length between A and end of structure = 19.2 + 1.5 + 3 - 2.5 = 51.7 feet

Pressure at point B = $6.82 - \frac{(6.82 - 1.92)}{51.7} \times 19.2 = 5$ feet

(according to Khosla, Bligh's Creep Theory holds between sheet Piles,

Thickness required at B = $\frac{5}{1.24} \times 1.2 = 4.8$ ft.

against 4.0 feet existing

c) Point C

Pressure at point C = $6.82 - \frac{(6.82 - 1.92)}{51.7} \times 29.2$

= 4.05 feet

Thickness required at C = $\frac{4.05}{1.24} \times 1.2 = 3.92$ feet

against 3.5 feet existing

d) Point D

Pressure D = $6.82 - \frac{(6.82 - 1.92)}{51.5} \times 37.2 = 3.29$ feet

Thickness required at D = $\frac{3.29}{1.24} \times 1.2 = 3.2$ feet

against 3.0 feet

Downstream floor is unsafe against uplift pressure all along except at downstream end (Point E).

1.2.11 KHOSLA'S SAFE HEAD ACROSS

1) Against Exit Gradient

Safe Head = 7.2 feet (as already calculated)

2) Against Uplift Pressure

Uplift Pressure at A = 0.55 H (as already computed)

Floor thickness required $= \frac{.55\ H}{2.24 - 1} \times 1.2 = 0.53\ H$

Head Across for thickness of 5 ft. is given by $0.53\ H = 5$

$H = \frac{5}{0.53} = 9.4$ feet

1.2.12 OVER ALL SAFE OPERATIONAL HEAD ACROSS

Permissible Head Across

Bligh	-	7 feet
Lane	-	8.9 feet
Khosla (Uplift)	-	9.4 feet
Khosla (Exit Gradient)	-	7.2 feet

Therefore, overall safe operational head across for the structure was 7 feet, whereas the structure had to be subjected to a head of 12.4 feet, under worst conditions.

2.0 DESIGN REQUIREMENTS

Something had therefore to be done to meet the situation and to be able to run B.R.B.D Link Canal to its winter demand which very nearly matches the available river and hence Upper Chenab Canal supplies.

This problem gave food for thought about the alternate measure to remedy the situation.

2.1 POSSIBLE REMEDIES

Various solutions were considered and examined before accepting the adopted course. The pros and cons of various possibilities including the reasons for non-adoption or otherwise appear below :-

2.1.1 FEEDING B.R.B.D. THROUGH MARALA - RAVI LINK CANAL (also off-taking at Marala Headworks).

This way Bambanwala Regulator could be by-passed in periods of low river supplies and instead of using U.C.C., M.R. Link could be used to feed B.R.B.D Link through a sub-link at its chainage 302000 to connect B.R.B.D at its chainage 188000. The objections to the proposal are :-

i) M.R. Link is inherently silted up badly in its head reach i.e. up to chainage 95000. Silting up is of the order of 7-8 feet, while the channel has widened by about 200 - 250 ft. in the same Reach. M.R. Link is a large channel with design discharge of 22000 Cusec. Running low supplies in a seriously widened channel (with existing width of 500-550 feet) entails a large loss of supply by seepage due to very high wetted perimeter for such a small depth of flow. Wastage of precious supply in the shortage period could be ill-afforded.

ii) Due to silting up of M.R. Link and rise of supply levels, seepage from M.R. Link in winter could seriously affect wheat crop in adjoining areas. M.R. Link normally runs between 15th April and 15th September each year and any regular running of the canal in winter would be strongly resisted by farmers and would be a decision difficult to implement politically.

iii) By using M.R. Link as above, any chance of opening Nokhar Branch at Bambanwala regulator to winter watering of wheat crop would be lost. Although the said canal is non-perennial, still traditionally some 2-3 waterings are allowed in each Rabi crop on all Non-perennial channels in Punjab.

2.1.2. CONSTRUCTING SUBSIDIARY WEIR D/S BAMBANWALA REGULATOR

This proposal appears on the following line diagram (Fig-5)

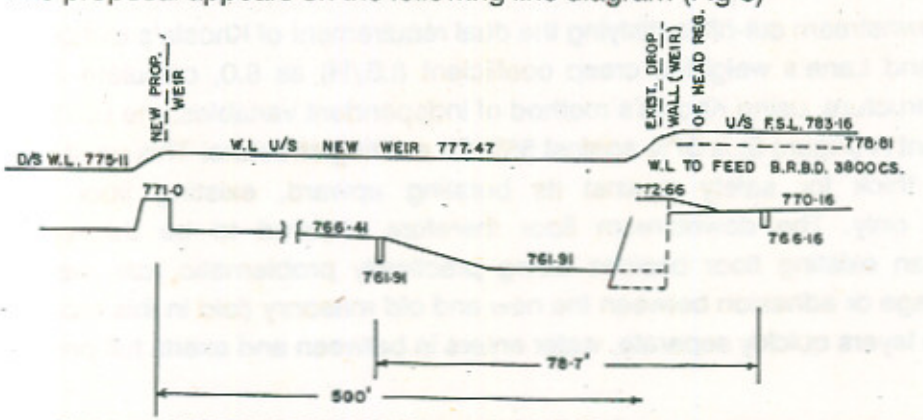


FIG.5 : LINE DIAGRAM SHOWING SUB-SIDIARY WEIR PROPOSAL.

Considering the old structure as not to be fit for any addition or alteration, it was proposed to construct a new regulator about 500 feet downstream of the existing structure. This new structure would provide a pond of water between the two regulators to reduce the differential head across the old structure. The proposal was, however, dropped due to the cost involved in the construction of a full-fledged new regulator needing extensive sheet pilling and canal diversion since this work could not have been completed in the limited annual closure period (18 - 19 days).

2.1.3 REMODELLING OF THE EXISTING STRUCTURE

Having considered and rejected external proposals, there was no alternative to remodelling of the existing structure for safety against uplift pressure and exit gradient imposed by critical head across.

3.0 REMODELLING PROPOSALS FOR THE STRUCTURE

3.1 INTRODUCTION

Having proved that remodelling of the existing structure was the only possible course, the under mentioned possibilities thereunder, were examined and compared before concluding about the most feasible proposal :

1) Deepening downstream cut-off and thickening of the downstream floor.

2) Deepening upstream cut-off.

3) Extending upstream floor.

Selection from amongst these depend upon the following criteria :

3.2 DEEPENING DOWNSTREAM CUT-OFF

Depth of the downstream cut-off, satisfying the dual requirement of Khosla's exit gradient as 1/6 or lesser and Lane's weighted creep coefficient (LB/H) as 6.0, calculate to 16 feet. Analyzing the structure, using Khosla's method of independent variables, the uplift pressure at the worst point A (Figure 3) is 57% against 55% for existing structure. This uplift requires a floor 6.4 feet thick for safety against its bursting upward, existing floor thickness being 5.0 feet only. The downstream floor therefore required to be thickened also. Thickening of an existing floor besides being practically problematic, can never result in perfect bondage or adhesion between the new and old masonry (laid in this case in 1905-10). If done, the layers quickly separate, water enters in between and exerts full pressure,

which only the combined thickness of the two could resist (Khosla, 1936). Deepening of the downstream cut-off was therefore not decided upon.

3.3 DEEPENING UPSTREAM CUT-OFF

Requisite depth of upstream cut-off was calculated for 'creep/piping' consideration and found to be 15 feet against the existing 4 feet. The structure (with deepened upstream cut-off) was analyzed for uplift pressure, with the following results :-

"Pressures under the upstream floor reduced appreciably i.e. from 83% to 65% at its upstream end, while reduction in pressures under the downstream floor is nominal i.e. from 55% to 53% at the worst point (just downstream of the drop wall). This uplift required a floor thickness of 6.4 feet for safety against rupture whereas the existing thickness is just 5 feet. The structure would also continue to be unsafe against piping by excessive exit gradient even after upstream cut-off had been deepened.

Reduction of uplift pressure under the upstream floor is of no consequence since these pressure are more than counter-balanced by the load of water on it. The above results, are exactly as stated by Khosla, when he asserts, (Khosla, 1936), that the upstream cut-off is much less important than that at the downstream end of floor since while the latter provides safety against scour and exit gradient, former is necessary for scour only.

3.4 EXTENSION OF THE UPSTREAM FLOOR

Extension of the upstream floor remains the only workable measure to increase length of creep for safety against piping and exit gradient. The provision of maximum possible creep length upstream of the barrier (crest) increases the upstream floor length which is itself free from the effects of uplift pressure and also reduces pressures on that part of the floor which must be provided downstream. This is so because a larger percentage of the total creep having taken place upto the barrier, the residual heads on the downstream floor are reduced. At the face of it this proposal is costlier since considering a relative ratio of vertical to horizontal creep as 3:1, a 3 feet extension of horizontal floor is equivalent to 1 feet deepening of a cut-off. Design for the work is at Appdx-II.

3.5 REQUIRED EXTENSION OF FLOOR

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

$$\therefore \lambda = \frac{H^2}{GE^2 \times d^2 \times \pi^2}$$

$$= \left[\frac{12.4}{0.167 \times 4.5 \times 3.142}\right]^2$$

$\therefore \lambda = 27.57$

also $1 + \sqrt{1 + \alpha^2} = 2\ \lambda$ (from 1.2.1)

$1 + \alpha^2 = [2(27.5) - 1]^2$

or $1 + \alpha^2 = 2933$ or $\alpha = (2933)^{1/2} = 54.15$

But $\alpha = \frac{b}{d}$ so $b = \alpha d = 54.15 \times 4.5$ or $b = 244$ ft

In actual execution, extension of about 170 ft. was provided giving a total floor length of 248.70 feet.

3.6 DEPTH OF UPSTREAM CUT-OFF

Discharge intensity (q) = $\frac{16850}{320}$ = 52.66 cusec/foot width

Lacey's formula - $R = 0.9 \frac{(q^2)^{1/3}}{f}$

R = scour depth

f = Lacey's silt factor = 1.1

$R = 0.9 [(52.66)^2]^{1/3} = 12.2$ feet

Now ds = XR - dn

Where ds - scour depth below bed)

X = multiplier (prescribed by Khosla as between 1.25 and 1.75 for upstream scour)

dn = normal depth of flow

factor (x) of 1.5 was used for design

da = 1.5 (12.2) - 13

= 5.3 feet

Upstream cut-off depth provided = 6 feet

As far as the downstream scour is concerned, the multiplier factor had remained nearly constant at 1.5 every since initial construction with depth of the cut-off remaining unchanged and only full supply depth varied with discharge. Downstream cut-off depth was

not altered in the instant work and therefore, the factor. Although prescribed factor is 1.75-2.5 (for river works), yet the structure has safely lived with downstream scour, obviously on account of controlled discharges, lesser variation/flow concentrations, etc. in canals, as compared with rivers, which uncertainties, the multiplier factor is intended to cover up. Besides this, R.S. Varshney prescribes factor of 1.0 R - 1.25 R for upstream and 1.25 R - 1.5 R for downstream scour respectively, (Varshney, 1979).

4.0 ANALYSIS OF THE REMODELLED STRUCTURE

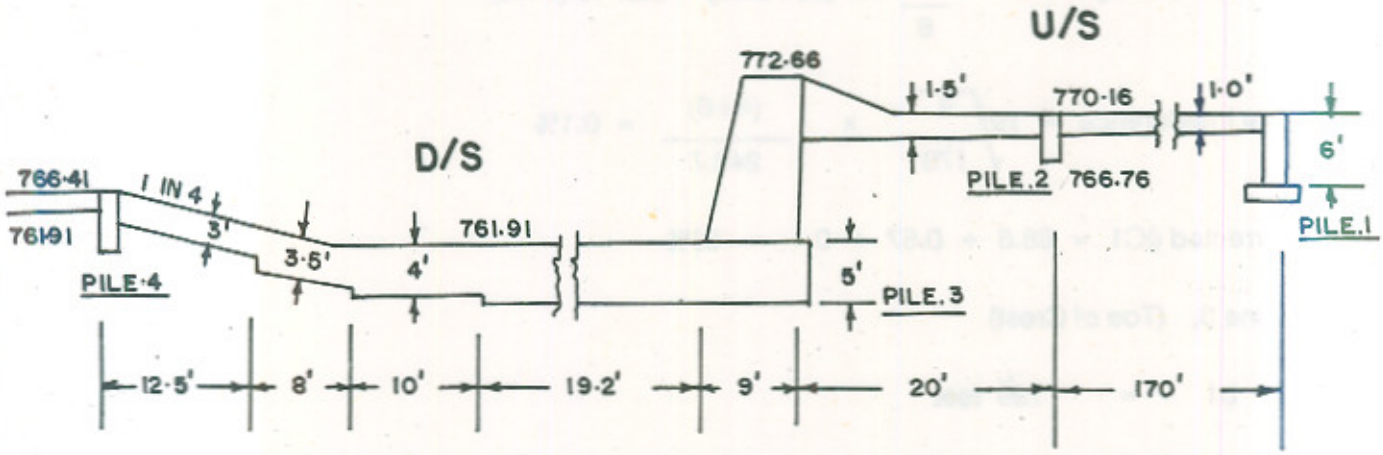


FIG.6 SECTION OF THE REMODELLED WEIR.

4.1 BLIGH'S CREEP CO-EFFICIENT

$$LB = 248.7 + (6+5) + (3+3) + 7.25 + 5 + (1.5+4.5) = 283.95 \text{ ft.}$$

$$C_B = \frac{283.95}{12.4} = 23 \text{ against minimum prescribed } C_B = 15$$

4.2 LANE'S WEIGHTED CREEP CO-EFFICIENT

$$L_C = \frac{248.7}{3} + (11+6+12.25+6) = 118.15 \text{ feet}$$

$$C_W = \frac{118.15}{12.4} = 9.5 \text{ against prescribed minimum } C_W = 6.0$$

4.3 KHOSLA'S UPLIFT PRESSURES

Pile 1, upstream cut-off $\frac{1}{\alpha} = \frac{d}{b} = \frac{6}{248.7} = 0.025$

From Khosla's curves (Fig.3) $\phi E_1 = 100 - 11.4 = 88.6\%$

$\phi D_1 = 100 - \phi D = 100 - 8 = 92\%$

Thickness correction $\frac{1}{6} \times (92 - 88.6) = 0.57\ \%$ (+ive)

Mutual interference $= 19\sqrt{\frac{4}{179}} \times \frac{(4+6)}{248.7} = 0.1\%$

Corrected $\phi C1 = 88.6 + 0.57 + 0.1 = 89\%$

Pile Line 3, (Toe of Crest)

$b1 = 199$ feet

$b = 248.7$ feet

$\frac{b1}{b} \doteq \frac{199}{248.7} = 0.8$

$d = 5.0$ feet (as before)

$\alpha = \frac{b}{d} = \frac{248.7}{5} \approx 50$

From Khosla's curves

$\phi D_3 = \phi A = 27\%$

Floor thickness required at A $= \frac{27}{100} \times \frac{12.4}{2.24} \times 1.2 = 3.25$ FT.

against 5.0 ft. existing.

Pile Line 4 - Downstream Cut-off

$$\frac{1}{\alpha} = \frac{b}{d} = \frac{4.5}{248.7} = 0.02$$

From Khosla's Curves :-

$\phi D_4 = 8\%$

$\phi E_4 = 13\%$

Correction for floor thickness $= \frac{3}{4.5}(13-8) = 3.33\%$ (-ive)

Correction for mutual interference of piles is neglected since these are very far apart.

Correction for Slope = -1% (Calculated earlier)
Correction $\phi E_4 = 13 - 3.33 - 1 = 8.67\% \approx 9\%$

Floor thickness required $= \frac{9}{100} \times \frac{12.4}{1.24} \times \frac{1.2}{2.24 - 1} = 1.08$ feet,

Against 3.0 feet existing
Floor is all along safe against uplift pressure.

It has been seen that although he could have done with a lesser extension of the upstream floor from consideration of Bligh's Creep Co-efficient, restriction of exit gradient is altogether essential for safety.

4.4 CHECK-LENGTH OF DOWNSTREAM FLOOR (Surface Calculations)

Discharge at weir = 11375 Cusec

Waterway at weir = 18 x 8.5 ft. = 153 ft.

(18 bays of 8.5 ft. each)

Therefore Discharge intensity (q) $= \frac{11375}{153} = 74.3$ Cusec/ft.

Upstream full supply level = 783.16

Downstream full supply level = 775.11

Therefore head across (HL) = 783.16 - 775.11 = 8.05 ft.

From graph (Fig.6) Ef2 = 12.05

Now Ef1 = Ef2 + HL = 12.05 + 8.05 = 20.1

From (Fig.7)

d_1 = 2.3 feet

d_2 = 11.5 feet

d_1 and d_2 being the hyper-critical and sub-critical depths of water upstream and downstream of the point of formation of the standing wave.

Height of standing wave = $d_2 - d_1$ = 9.2 feet

Length of the downstream floor = 5 ($d_2 - d_1$) = 46 feet against

48.7 feet existing

Hence the downstream floor is adequate.

5.0 ACTUAL EXECUTION OF THE WORK

5.1 DESIGN ADOPTED

The design finally adopted for the work "Remodelling Cross Regulator at RD 133296 U.C.C." appears at Appdx-II and has the following components :-

1). Extension of the upstream floor by 170 ft. with 1.0 ft. thickness comprising 0.65 ft., cement Masonry over 0.35 ft. thick cement concrete 1:3:6 - to provide additional length of creep.

2). Construction of a 6 ft. deep upstream cut-off at the end of the extended floor - for safety against scour.

3). Construction of a masonry side encasing wall on the right end of the extended floor, connecting the regulator with the right side off-take i.e. Nokhar Branch - having bottom level as that of the upstream cut-off to check lateral seepage.

4). Construction of a baffle 0.8 ft. in height in cement Masonry 1:3 over downstream cut-off - to reduce the head across the structure, as an added precaution.

5.2 PROPOSALS FOR EXECUTION

The work could, obviously, only be done during the canal closure period. Besides the quantum of the work, what was perplexing was the disposal of seepage recharge in the closed canal and the lowering of the sub-soil water level at the work site by about 10 ft. for the purpose of laying foundations. The Upper Chenab Canal is in part cut and fill in the reach between Marala and this regulator. The area between U.C.C. and Marala Ravi Link canals is completely charged with seepage from the link canal in the Head reach of about 19 miles (due to reasons explained earlier). From observations during previous canal closures it was known that initial seepage discharge at this cross-regulator is about 100 Cusec tapering down to 60 Cusec towards the end of the closure. Obviously the regenerated supply at any stage of canal closure could not be disposed off at the work site, since the by-passing capacity was not enough. Even if pumping arrangements were made, adjoining areas could not be flooded. Accordingly it was decided to attend to the recharge in the canal at the following two points indicated on a line diagram. (Appdx-III)

i). Syphon RD 90275 U.C.C.

Recharge between head and syphon site RD 90275 U.C.C. i.e. about 18 miles apart (approximately 70 Cusec initially) would be disposed of into a drain passing beneath the canal, by erecting a bund in the canal bed (on pacca floor on top of syphon) and cutting the right bank of the canal. The capacity of the drain, which is almost dry in the relevant period, is ample, as also, the working head available for discharging of canal recharge into a drain passing underneath the canal bed.

ii) Just Upstream of the work site

Regenerated discharge between the bund at RD 90275 and the work site at RD 133296 U.c.C (about 7 miles distance) was anticipated to be about 27 Cusec and was planned to be removed through a side drain (after cutting the left bank of the canal), across syphon at RD 977 of the B.R.B.D Link and into Main Line Lower (about 500 ft-downstream of the said regulator) through an existing inlet. Luckily the syphon was designed for 30 Cusec discharge i.e. more than the requirement of the proposal.

5.3 DEWATERING

For this work, dewatering was necessary to lower the sub-soil water level by 10 ft. to obtain working conditions. This had to be done by the following two means :-

1) Open pumping through diesel pumps since there is a general load shedding programme in the country in relevant period, due to closing down of Hydel Power Stations. This system offers the facility of supplementing capacity with additional units if so required.

2) Well point system or individual pumps at water level or even individual wells with pumps placed on top of wells.

Both systems had to be used conjunctively to obtain general lowering of water table and immediate relief for laying foundations. While the open pumping system is fairly simple, dewatering by wells is complicated requiring use of bore-hole logs (for strainer length, location, size etc.), complex formulae for interference of several wells (to fix up well facing), Leliavsky, 1960 states. "Comparing various possibilities under wells, it is seen that while discharge and dewatered areas are nearly the same in both cases, the lift on the pump for single well is greater than for system of several wells meaning that required H.P is less in the second case. However, with a number of wells, spacing thereof is very important, so as not to throttle each other's discharge. The spacing of individual wells or well-points is a function of two factors : difference in level between natural and depressed sub-soil water levels and Darcy's co-efficient of permeability. What is required in the field is not an elaborate estimate of equipotential surface corresponding to a well arrangement, but a simple rule for well spacing so as not to materially reduce individual well discharges. A working rule given by Sichardt prescribes a minimum spacing between adjoining wells as $10\pi d$ or about thirty diameters.

In our case, a minimum required spacing of about 11 ft. results for well diameter of 4 inches - which was ensured for the eleven number of wells installed over length of about 500 ft. (Leliavsky, 1960).

For use of single wells with pumps at water level, a relatively larger diameter bore is necessary to house the pump, which makes the bore costlier and more time-consuming then a well with a pump at its top. We could ill-afford the resulting delay although we would have had a positive pressure always and the head could therefore attain any reasonable value.

For the work in question, we were restrained by time from having the collector system wherein a number of well points work by suction i.e. vacuum and discharge into a common pipe or collector, from which water is evacuated by means of pump of larger air and water capacity. Although this system is the most efficient and in commonest use now-a-days, it would have required a much larger number of borings in comparison to individual wells. Under the circumstances besides diesel pumps, individual wells with pumps on the

surface i.e. suction type with maximum permissible lifting head of 26 ft. were used on the work since maximum lowering of water table required lifting heads below the limiting vacuum.

The expected difficultly with dewatering led to the following initial proposals :-

i) Upstream cut-off depth was proposed as 6 ft. all along and in case existing scour was deeper than this depth, the wall was to be founded on sand filling in the scour pit.

ii) Side encasing wall was proposed to be founded on wells despite fear of hitting some obstruction, notably, stone, dumped against scour regulator.

With extra dewatering we were able to achieve the following improvements :-

i) Foundation concrete of the upstream cut-off wall was kept at 6 inches below scoured bed at all points. The top level of concrete foundation for masonry wall was, however, kept at the same level throughout (R.L. 764.16) bottom level varied from 763.16 to 762.70 for worst scoured zone.

ii) Side wall was constructed on 'Open Foundations'. This made its very execution possible since while excavating for foundations, we hit an abandoned head regulator floor of Nokhar Branch which was not appearing on our drawings. This floor would have obstructed sinking of wells and there would not have been time for switch-over to the present proposal.

6.0 PLANNING FOR THE WORK

Due to the size and nature of the work, it had to be properly planned after administrative approval and release of funds. No extension in the closure period could be provided, neither could the main work be continued after re-opening of the canal. Various activities (Pre+closure) were identified, their linkages established, time required for execution assessed and a working plan prepared under the Critical Path Method besides the traditional Bar Chart Programme.

6.1 CONSTRUCTION PLANNING

Construction planning including monitoring project progress during execution is carried out traditionally in the form of Bar Diagrams. However, due to limitations of this method, Critical Path Method (CPM) is being increasingly applied these days to all modern construction projects. The two approaches are described below (Siddiqui, 1979) :-

6.1.1 Bar Diagrams are excellent diagrammatic representations of activities which can be easily understood by all levels of construction crew (from managerial to the field staff). In these diagrams one bar or line (usually horizontal) represents only one activity. The bar diagram although looking fine on paper, usually presents limited information. It does not give clear information of activities necessary for or linked to any other activity. Bar diagram prepared for this work as at (Appdx-IV) does not indicate that activities C.D.E. and K are necessary before activity L can be taken up.

6.1.2 Critical Path Method (CPM) for planning, scheduling and monitoring is a comparatively newer concept (having been developed in USA in mid 1950's), but has already proved its worth as a management technique in all big projects. This technique relates the sequence and logical relationship of activities required to achieve the project. The backbone of the CPM is a graphic model of the project prepared in the form of an arrow network diagram, each arrow representing an activity. The graphic model provides a clear picture of the various stages of construction. It focuses attention to the critical activities which determine the project duration.

After identifying activities, their linkages and estimating completion time for each, the arrow diagram is completed. There would be more than one path or chain of activities in the diagram to reach the end point from the point of start. The path whose time is the longest is called the Critical Path. This means that unless and until all activities on that path are completed, the project cannot be completed. Earliest event time is defined as the earliest time at which any event can occur. It is the sum of the duration along the path of activities leading into any Event.

Late Event Time is the latest time at which any event can occur without delaying the project completion. When more than one activity converges into a event, the 'late event time' is taken as the lesser of the time calculated for all the activities.

6.1.3 CPM APPLICATIONS

This method finds the following applications in projects :-

1) Man-power progress

2) Monitoring progress

3) Cost control

The net-work diagram for Critical Path analysis of the work in question may be referred to as Appdx-V.

6.2 PRE-CLOSURE ACTIVITIES

Besides the arrangement of material and equipment by the contractor as for all works, the following were clearly the activities which could be taken up before the actual canal closure (refer to line diagram showing schematic details as Appdx-III) :-

1) Stockpiling of earth for bunds at RD 90 and 133 U.C.C.

2) Supplying Empty Gunny Bags and filling with earth for lining these bunds.

3) Part cutting of canal banks at the two cut sites leaving only the bare minimum necessary for accommodating the low canal supply just before canal closure.

4) Improving drains along-side the work site.

5) Boring Tubewells (on canal berms only) for lowering sub-soil water level in the pit. Prior to this, test bores had to be made for bore-hole logs to decide strainer location, depth of bore etc. (Appdx-VI).

We were badly crammed in space available for stacking/collecting material at site and had to earmark sites for different material. The site plan is at Appdx-VII.

6.3 TENDERING/ALLOTMENT

Annual canal closure was scheduled for December 1988 - January 1989, tenders were invited on Ist September 1988. A detailed bill of quantities was supplied to each tenderer which spelt out of the full requirement of material and equipment necessary for the work. In view of the special nature of the work, a condition was inserted requiring the Tenderer (if successful) to collect all material and machinery at site and to get the same inspected by the officer incharge, at least one week before commencement of the closure. This was also spelt out in the acceptance letter issued to the lowest tenderer on 30th November, 1988. In terms of the said letter, he was informed that in case of failure to collect material etc. the work would not be commenced and that he would be penalized for lapse of Annual Development Programme (A.D.P) Funds. To allow him sufficient time to make all arrangements, acceptance letter was issued to him on 30th November, with closure fixed for 26th December 1988 - January 1989. The site plan for stacking/collecting material, the C.P.M. Plan, schedule of daily labour requirement (Appdx-VIII) besides the approved design, detailed drawing and the approved project were given to him when he received (and acknowledged) the acceptance letter.

6.4 ACTUAL EXECUTION

The pre-closure activities already listed having been duly executed before 25th December 1988 the following was roughly the schedule of actual progress on the work, with its main components as detailed :

CANAL CLOSURE AT HEAD - 26/12/88 AT 6.0 AM

1.	Bund and Cut at RD 90	26/12/88
2.	Bund and Cut at Rd 132-133	26-27/12/88
3.	Open Pumping started	27/12/88
4.	Installation of Tubewells in bed	27/12/88
5.	Dismantling D.B. flooring, stone in silted position/removal etc.	28-30/12/88
6.	Preparation of sub-grade, sand filling, levelling etc.	28-30/12/88
7.	Excavation for foundation of upstream cut-off.	29/12/88
8.	Laying of stone apron upstream of cut-off	29/12/88-6/1/89
9.	Construction of upstream cut-off	30/12/88-6/1/89

10.	Construction of floor	31/12/88-16/1/89
11.	Construction of downstream baffle	30-31/12/88
12.	Excavation for side wall	30/12/88-3/1/89
13.	Construction of side wall upto initial water level in canal	1/1/89-17/1/89
14.	Earth-filling behind side wall	11/1/89-24/1/89
15.	Dismantling bed Tubewells, removing Pumps, site clearance	17/1/89
16.	Cutting of bunds	18/1/89
17.	Making up cuts in Banks of canal strengthening continued after opening of canal	17-20/1/89
18.	Reopening of canal after extended closure	18/1/89

7.0 SPECIAL FEATURES/PROBLEMS OF THE WORK

These were as under :-

7.1 HUGE QUANTITY OF CEMENT MASONRY

The 170 ft. extension of the upstream floor had to have a thickness of 1 ft. as under :

a) Cement concrete - 0.35 ft.

b) Cement masonry - 0.65 ft.

Provision of total thickness as cement concrete would have been more workable due to quicker 'mass concreting', but this would have been costlier (by about Rs. 50000/-).

Provision of Brick Masonry made the work labour intensive and required stricter and more dispersed supervision for quality control. The very completion of about 36000 cubic ft. of Brick masonry requiring about 5,40,000 number bricks and 3800 cwt. Cement in a period 15 days only (leaving the first few days for dewatering, dismantling, surface preparation etc.) looked 'challenging'. the entire Canal Division's Engineering staff was deployed on the work requiring day and night watch for quality control. Obviously the work proceeded round the clock and complete lighting arrangements were made at the site by the contractor.

7.2 SHIFTING OF MATERIAL TO WORKING PIT

As already explained all construction material was carried to the site and collected along both banks. After closure of the canal, material had to be rehandled into the pit using ramps prepared after dismantling the parapet of the right parapet wall (above road level). The material had to be steadily carried to match the consumption since there was no collection point, with scour pit just upstream of the floor extension. The work on the extreme right panel and the side encasing wall could only be completed after several shiftings of ramps.

7.3 EXCAVATION FOR SIDE WALL

Excavation had to be done from an average berm level of 776.50 down to foundation level of 763.91. There was a serious, side sloughing problem which could be overcome only due to constant operation of the four tubewells bored on the right berm and use of Back-hoe excavator.

Foundation concreting for the side wall could not be done simultaneously, in the entire length, since we had always to allow for ramp across the wall alignment for carriage of construction material to the work pit. To allay fears of differential settlement of the plain foundation cement concrete an R.C.C i : 2 : 4 slab, 1 ft thick was introduced with its bottom level of 766.16 (refer Appdx-IX).

7.4 WATCH OF QUALITY AND PROGRESS ON THE WORK

The work was kept under special vigil as regards quality of the material, workmanship and progress. As said earlier all engineering staff under the canal Division were deployed on the work. Progress was reviewed nightly, in meetings between the Departmental Officers and contractors (including sub contractors) in the site office. Beside

the programme for the next day, any shortfall in the targeted progress was especially discussed. The progress and performance of the work was watched by senior officers of the Department and the Consultants.

8.0 ACCIDENTS ON THE WORK

8.1 BREACH IN THE BUND JUST UPSTREAM OF THE WORK PIT

A 2 ft. freeboard had been provided in fixing top level for protection against overtopping by wave action. During the night between 1/1/89 and 2/1/89 due to a violent rain/wind storm, this bund breached despite best efforts to control the situation. This resulted in flooding of the pit, drowning of pumps and tubewells in bed, deposition of silt on the prepared sub-grade for floor. As per agreement executed between the contractor and the Department, the former was responsible for the damages to the work during construction and had therefore, to bear the loss. The accident caused a set-back to progress, eventually leading to an extension in the closure period. However, despite this set-back, courage was not lost, rather extra efforts were put in to repairing bund, dewatering, surface preparation etc. and the work was again under active execution from 3/1/1989.

8.2 OVERTURNING OF SIDE WALL FROM 2 FT. ABOVE FLOOR LEVEL

The side wall was constructed upto full supply level in the canal by 18/1/89, when the canal was re-opened to initial discharge after annual closure. The side wall was completed upto its design top level and with earth back-fill by 24/1/89, (Appdx-IX). The following events in quick succession, led to the saturation of the back-fill upto a height even above the supply level in the channel :-

1) Closing and Pulling out of dewatering tubewells on the berm (causing sub-soil water level to rise).

2) Opening of Nokhar Branch causing seepage into wall back-fill.

3) Raising of supply level (due to raising of canal indent) resulting in capillarity and seepage into back-fill.

The back-fill being of clayey material exerted much higher pressure than expected due to swelling on saturation. The density of clay soil is known to be 115 lbs/ft^3 & 180 lbs/ft$_3$ in dry and saturated conditions respectively. The flattening of angle of repose added to the pressure exerted by the back-fill and the side wall overturned above the reduced level of 772.16 and fell into the running canal. A cross section of the wall showing the damage is

at Appdx-X. Although the masonry was 'Green' i.e. not having attained sufficient strength due to having been laid on the average just 12 days before mishap on 1/2/89, yet the failure occurred due to excessive back-fill pressure exceeding the assumption of the type design used for the side wall (Appdx-XI). No cracks appeared in the wall before failure and the entire wall fell at once, and not by parts. This proves that the foundation was intact and that it was not a construction fault. A foundation settlement should have resulted in spiltting/cracking of the masonry or its falling apart.

At the time of the collapse there was an appreciable cushion of water over the floor i.e. 7 ft. to absorb the impact of the fall and prevent damage to the floor. The thickness of the wall at the rupture point was 3 ft. but did not create problem although being greater than the height of the crest above the floor i.e. 2.5 ft. on account of the following :-

The overturned wall lay in a width of about 17 ft.; the two extreme right hand side i.e. Hydratomat bays are lying closed since long as also indicated by the berm in front of these. Beyond this the wall thickness was less than the crest height above the floor i.e. 2.5 ft.

The fallen wall therefore posed no problem concerning Surface Flow. Regarding under-seepage, the boxing of the floor and side wall was intact from the bottom level of the wall (763.91) to a level of 2 ft. above the floor level of 770.16 and lateral creep was checked in as far as the least creep length to under the downstream floor was concerned. The matter was discussed at the highest level in the department and the above views about the mishap, endorsed. It was also agreed that there was no immediate concern and that remedial work could safely be taken up in the next Annual Closure i.e. during January, 1990.

Central Design Office on being referred to in the matter prescribed construction of a sloping brick pitched side wall above floor level i.e. 770.16. The design is at Appdx-XII. The work was carried out in the subsequent canal closure. There is reportedly no abnormality on this regulator since then.

9.0 SUMMARY AND CONCLUSIONS

Techniques of weir design have been evolved largely on the basis of study of causes which led to their failure. Before advancement in soil mechanics, weirs and barrages were designed principally on the basis drawn from previous experiences which were not based on a fundamental understanding of the hydraulic and mechanical principles involved.

9.1 The main considerations in weir design as have been explained, are :-

i) The exit gradient which measures the tendency to "Piping".

ii) The uplift on the structure, particularly on the downstream floor.

iii) the dynamic action of water causing scour-holes adjacent to the work or washing away material of the floor.

9.2 The major problem with the designer in the field of such structures as weirs and barrages is to deal with an infinite variety of sub-grade materials which are heterogeneous accumulations with unknown patterns of stratification from natural processes. This variation is generally taken care of by providing a liberal margin of safety in such structures. One should not be lured by false sense of economy into designing with, lower factors of safety (against Khosla's/Terzaghi's exit gradient), than those prescribed (4-7) especially when much higher factors are in use of Egypt (11 - 15) as stated by Leliavsky (Leliavsky, 1948).

9.3 Upstream pile is provided for safety against scour and its depth is controlled solely by the scour depth. (Khosla, 1936). Upstream pile is not materially effective in reducing uplift pressures under the floor; the reduction in pressures under the upstream floor resulting from increased depth of upstream pile is of no importance since these pressures are more than counter-balanced by the load of water on the upstream floor.

9.4 Downstream pile is essential in respect of scour and exit gradient and therefore much more important than the upstream pile. In principle if there is no downstream pile (simple flush floor), the exit gradient would be "Infinite". (Khosla, 1936).

9.5 Deepening of the downstream cut off results in :

i) reduction in exit gradient for safety against undermining of the foundation soil and

ii) increase of uplift pressures all along the floor requiring greater floor thicknesses;

A balance has to be struck between these conflicting results of deepening of the downstream pile.

9.6 For such a structure, requiring safety against exit gradient, although it may be theoretically economical to deepen the downstream cut-off and simultaneously thicken the downstream floor, yet in practice there are restrictions :

i) Cut-off can be practicably deepened to a certain limit only.

(ii) The fresh top layer of masonry or concrete and the existing floor would lack bond or adhesion of mortar no matter what precaution might be taken. Proper bonding is, however, very important since while thickness of a floor

may be adequate, if monolithic, it will become unsafe as soon as the layers separate permitting water to enter in between. This water would exert full hydrostatic pressure which anything less than the full thickness of floor cannot safely stand.

9.7 Creep Theories assume water to be percolating along the underface of the structure (or base profile) while Khosla considers flow in Stream Lines and therefore susceptible to mathematical treatment, (Garg).

9.8 Creep Theories give no importance to the location of cut-off or the exit gradient value, being only interested in the length of creep; Khosla on the other hand proves downstream cut-off to be essential for safety against undermining due to excessive exit gradient, postulating an 'Infinite' exit gradient for a simple flush floor i.e. without downstream cut-off (Khosla, 1936). Khosla has also given Curves and Tables for uplift pressures under different structure geometries.

9.9 Creep Theories prescribe creep co-efficient for various types of soil for fixing safe head against failure due to seepage erosion (piping) while Khosla provides solutions for uplift pressures (at key points) for safety against floor ruptures, and exit gradient for safety against undermining of the foundation soil.

9.10 Bligh's Creep Theory find use even after Khosla. Lane's weighted creep theory is used no-where except in U.S.A. while Khosla's method finds general use despite its limitation i.e. empiricism, assumptions of infinite depth of soil and istropism of the media etc. (S.P. Garg, 1983.)

9.11 Safety cannot to provide against 'piping' by providing sufficient floor length as stated by Bligh, but can be obtained by keeping Khosla's exit gradient well below the critical value i.e. the exit gradient may not be safe even if the average hydraulic gradient of Bligh (i.e. 1/c) is safe. This was verified with respect to remodelling of the typical structure discussed in this paper.

9.12 Controlling seepage and hence the uplift in any hydraulic structure such as weir or barrage still stands as a great challenge to engineer, because water will always find the path of least resistance through previous strata, joints and seams as they really exist, rather than as they are assumed to be for the purposes of analysis. For this reason, provision of pressure pipes may be considered as an essential part of newly constructed hydraulic structure or a useful addition to an old one.

9.13 The following important conclusions are also drawn from the remodelling work :-

i) A clause of the general agreement from used for the contract executed with the contractor in the instant case, provides that no payment is to be made for

bad, unsound or imperfect work until the contractor reconstructs or re-erects such work. In the present case this clause was stretched to cover the breach of the protective bund upstream of the work site, which was therefore repaired by the contractor at his own cost. To avoid legal complications, it is suggested to provide a special clause in the agreement, covering damages/reconstruction of such enabling works which may fail due to non-watching or strengthening by the contractor.

ii) Approved type design should not be used mechanically, but rather adopted after ascertaining the site conditions, soil type and behavior under the conditions to be imposed. This would help avoid accidents such as the present over-turning of the side wall due to excessive back-fill pressures exerted by saturated clayey soil not taken cognizance of in the general design based on the ordinary soil back-fill height of the wing wall.

The present documentation including compilation of design for a remodelling of a typical cross regulator on a Main canal would be helpful for general design of weirs besides tackling similar problems elsewhere. A case known to the author and which had been pended till outcome of this work, is about a cross-regulator on the Lower Chenab off-taking from river Chenab at Khanki Headworks. This paper would give a go-ahead to the remodelling of Chenawan Regulator at RD 40200 L.C.C.

10.0 ACKNOWLEDGMENTS

(i) The author is highly obliged to Mr. Waqar Hussain Waraich, Executive Engineer, Marala Headwork Division, U.C.C. and Mr. Dilawar Khan Kirmani, Sub Engineer for thorough assistance & supply of record despite their busy official routine,

(ii) The author is also grateful to Mr. Aslam Tabassum A.D.E. ISRP Design Office, I&P Department for assistance on computer for the Paper.

(iii) Thanks are also due to Mr. Muhammad Zubair Sheikh, Sub Engineer Chakbandi Division, Lahore for help in preparing Annexures & compiling the Paper.

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6. "Irrigation & Hydraulic Design" Vol-III by Serge Leliavsky . 1960.
7. "Irrigation Canals-Planning, Design, Construction & Maintenance" by I.H. Siddiqui, 1979.
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11. "Irrigation Engineering and Hydraulic Structures" by S.K. Garg, 1983.

12.0 PROJECT PICTORIAL

Some typical photographs showing the various components of the project have been added.

LIST OF APPENDICES

I. INDEX PLAN
II. APPROVED DESIGN FOR THE WORK
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VIII. SCHEDULE OF DAILY LABOUR REQUIREMENT
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X. X-SECTION SHOWING DAMAGE TO WALL
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EXCAVATION FOR LAYING OPEN FOUNDATIONS FOR RIGHT ENCASING WALL BY BACK-HOE EXCAVATOR MACHINE.



RIGHT SIDE WALL UNDER CONSTRUCTION SHOWING ALSO A RAMP FOR REHANDLING CONSTRUCTION MATERIAL INTO THE WORK AREA



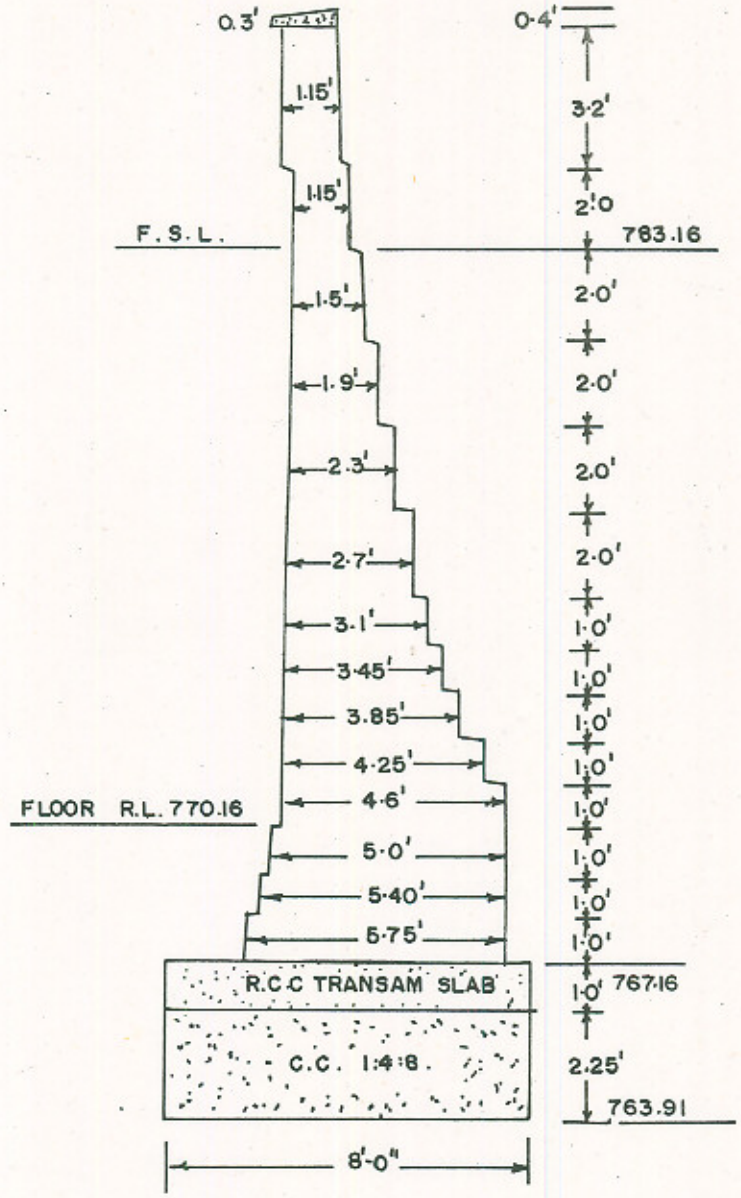
TUBEWELLS INSTALLED ON RIGHT BERM OF THE CANAL FOR DEWATERING.

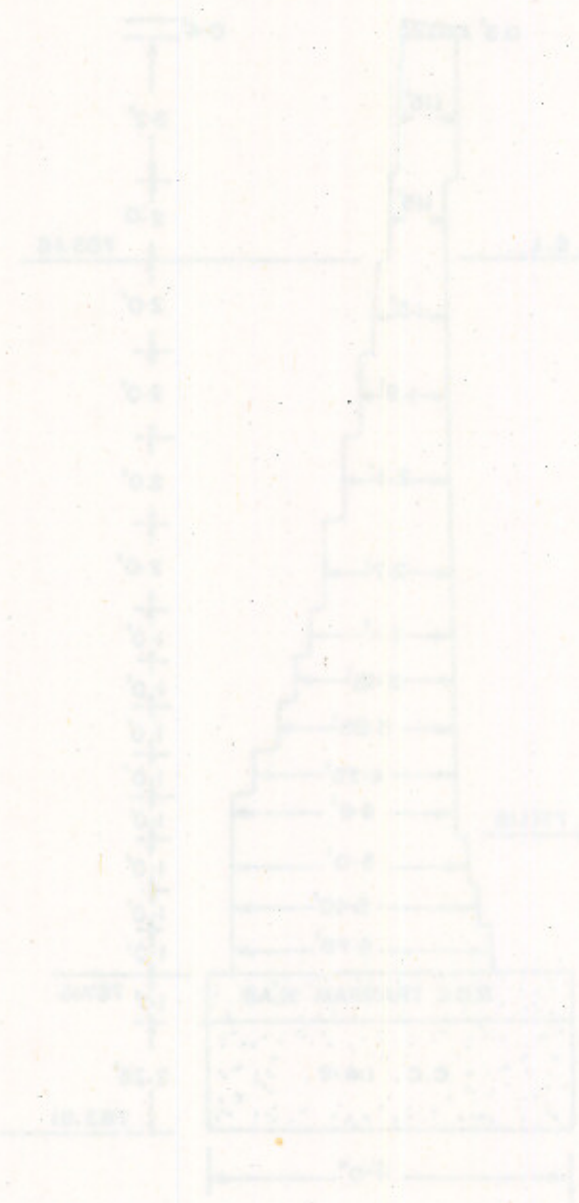


U/S CUT-OFF – WORK IN PROGRESS.

APPDX. IX

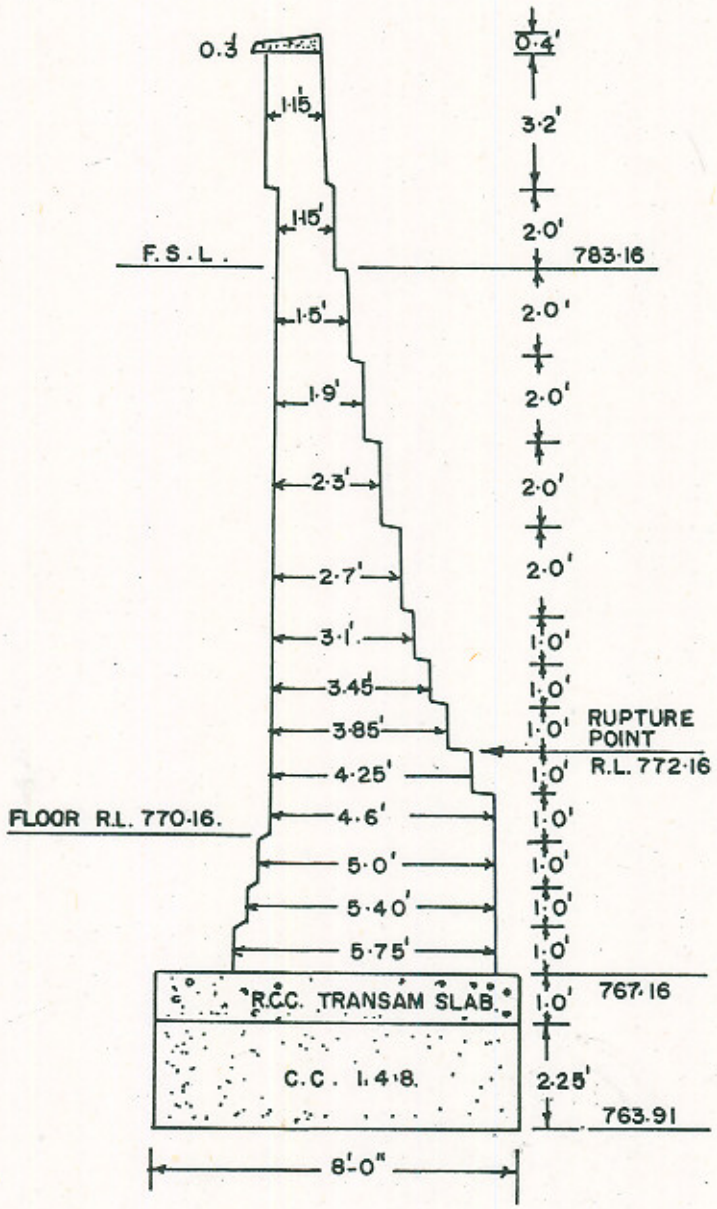
SIDE WALL X-SECTION AS CONSTRUCTED.





APPDX. X

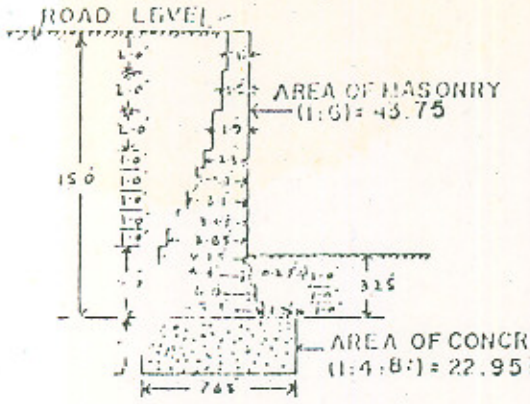
X-SECTION SHOWING DAMAGE TO WALL.

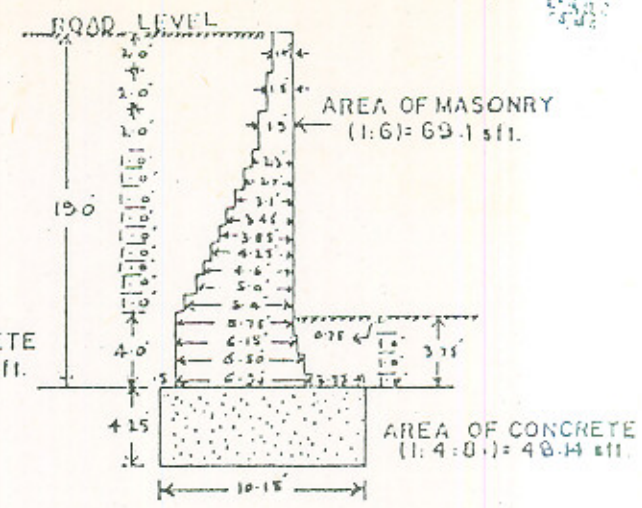


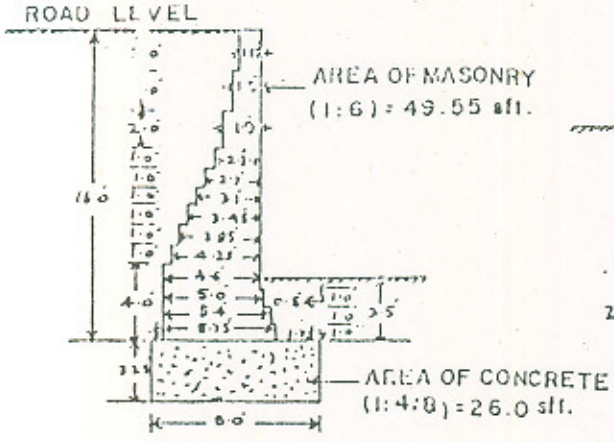


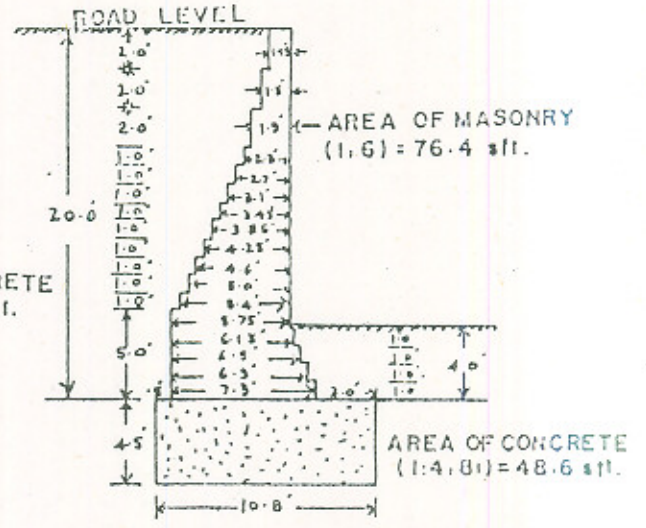
WING WALL

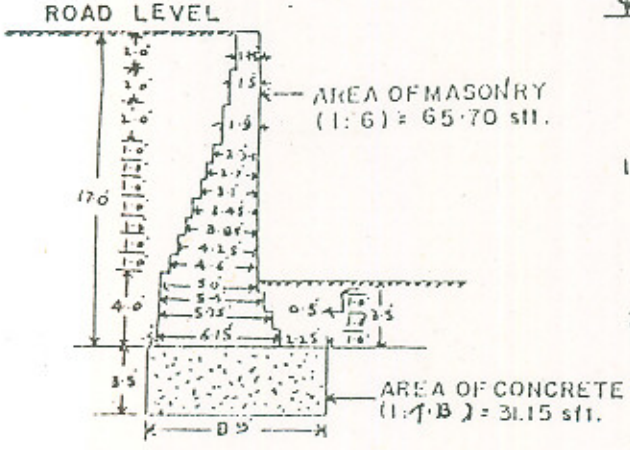
C.D.O. DRG: NO. W-13
SHEET NO. 2

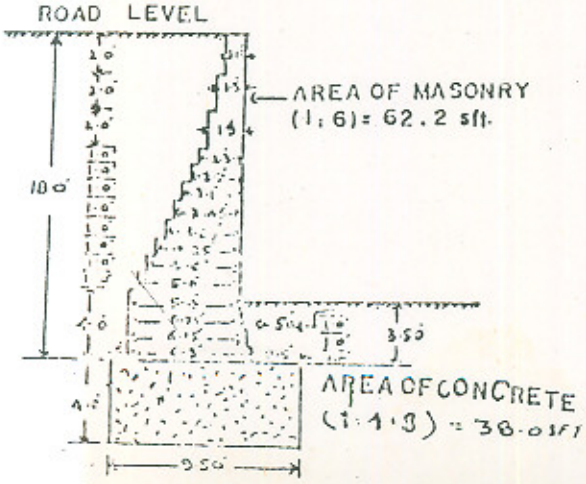












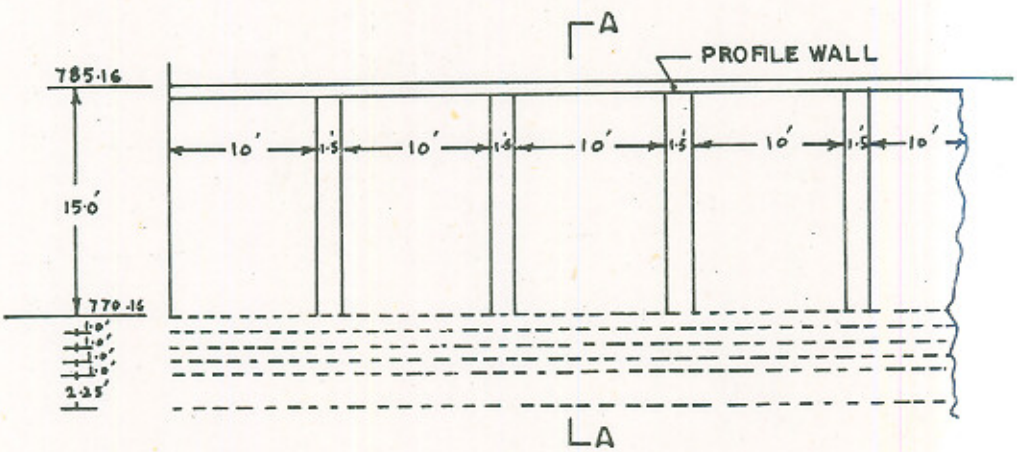
1. WEIGHT OF MASONRY 120 LBS PER CFT
   Ø "ANGLE OF REPOSE 35° 100 " "
   WEIGHT OF CONCRETE 120 " "
   VALUE OF Ø FOR WORKING OUT DEPTH OF FOUNDATIONS = 35° AND W = 62 LBS PER CFT.
2. SAFE TENSIL STRESS OF 1:6 MORTAR ACCORDING TO ROBERTSONS TEST IS 100 LBS/□" AFTER 7 DAYS WITH A FACTOR OF SAFETY OF 4 PERMISSIBLE STRESS FOR 1:6 MAY BE TAKEN AS 25 LBS/□" OR 3600 LBS/□'.
5. ATTENTION IS DRAWN TO THE JOINT MASONRY AND CONCRE... THE CONTACT SURFACE OF CONCRETE MUST BE THOROUGHLY NICKED AND CLEANED WITH A WI... BRUSH IMMEDIATELY BEFORE STARTING MASO...

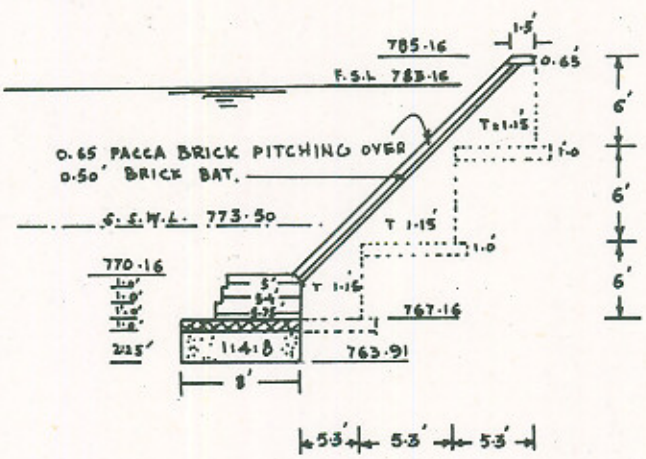
CHIEF ENGINEER		SD. J.H.D. BEDFORD 28.7.3
DIRECTOR	C.D.O.	SD. KANWAR SAIN "
S.A.E.	C.D.O.	SD. RAGHBIR SINGH "
J.A.E.	C.D.O.	SD. P.S. SABU "

T.BY/ M. RAZZAQ



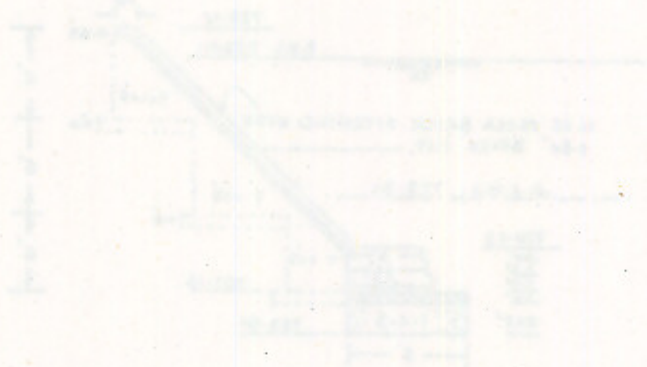
REVISED DESIGN APPROVED BY C.D.O. FOR SIDE WALL.





SECTION ON A-A





REMODELLING BAMBANWALA CROSS REGULATOR (RD 1333296 U.C.C.)
PAPER NO. 541 BY USMAN AKRAM

ERRETTA

PAGE	LINE NO.	INCORRECT	CORRECT
112	6	PREVIOUS	PERVIOUS
118	5	PRESSURE @ POINT A = $\frac{12.4\ (1 - 48.25)}{103.95}$ = 6.7 Ft	= $12.4\ [1 - \frac{48.25}{103.95}$ = 6.7 F_T
119	14	PRESSURE = $12.4 \times \frac{[1-(4 + 3 + 7.25) + (20 + 9)}{51.48}$	$12.4\ \ 1 - \frac{(4 + 3 + 7.25 + 5) + (20 + 9)}{51.48}$
124	4	& KS = $\frac{766.41 - 761.91}{18} = \frac{4.5}{18} = \frac{1}{4}$	& SLOPE =
130	22	d_a = 1.5 (12.2) - 13	d_s =
131	FIG 6	PILE 2 766.76	PILE 2 766.16
136	10	TO FIX UP WELL FACING	(TO FIX UP WELL SPACING)
146	29	PREVIOUS	PERVIOUS