PAPER No. 400

MODEL TESTS FOR ASSESSING THE CAUSES OF EXCESS UPLIFT PRESSURE UNDER DOWNSTREAM FLOOR OF BAYS NO. 31-40 OF MARALA BARRAGE AND REMEDIAL MEASURES

By

S. M. AYOOB*, P.S.E. I & CH. MOHAMMAD ALI**, T. K.

INTRODUCTION

In the design of hydraulic structures on pervious foundation, the forces to be taken into consideration include those due to surface flow and seepage flow through the foundation medium. Seeping water in foundation of hydraulic structure has a certain residual force at each point along its path of flow which is proportional to the pressure gradient at that point. At the tail end this force is upwards and will lift up the sand particles if its force is more than the submerged weight of the latter.

The stability of hydraulic structures such as the downstream apron of a weir on a permeable foundation, the d/s face of an earth dam, the downstream end of a single wall cofferdam etc., is jeopardised by excessive residual uplift pressure of water percolating through the foundation tending to lift up the structure or when seepage emerges out at the exit under high pressure developing a velocity sufficient to carry away the foundation material-phenemenon called piping or sand boiling *i.e.* progressive washing away of the material from the downstream, is experienced, thus undermining the structure.

Creep theory

Among the earliest investigations to understand the effect of percolating water on the stability of a structure constructed on sand foundation, were those started by the British Engineers in India under the leadership of Clibborn and Beresford[1] consequent on the failure of Khanki Weir in 1895 and Narora Weir in 1898. Mr. Bligh[2] proposed an empirical theory in 1910 known as "Bligh's Creep Theory".

He assumes that seepage flow creeps along the contact of base profile of the weir with the sub-soil and that certain length of creep had the same resistance effectiveness in reducing residual head irrespective of vertical or

**Director, Irrigation Research Institute, Lahore.*

***Principal Research Officer, Hydraulics, Irrigation Research Institute, Lahore.*

horizontal or any other direction of flow lines. He called the loss of head per unit length as creep coefficient and assigned a safe value for 'C' for different classes of soil which varies from 5-9 for boulders and shingle, to 18 for river bed of finer silt and sand as those of Nile or Mississipi.

Lane's[3] conclusions leading to his weighted creep theory are based on a study of nearly 300 hydraulic structures on pervious foundations. According to Lane's theory more weight is given to vertical cut-offs (1/3 to horizontal and one to vertical).

Potential theory

In 1932 Warren Weaver[4] worked out analytical solutions for the calculation of uplift for some simple profiles making use of principles of irrotational flow, conformal mapping and the application of Laplace equation with the given boundary conditions. For many of the practical complicated profiles, it becomes either too involved or it is impossible to find analytical solutions. Khosla[5] has evolved, however, a method of solution of such cases, known as Khosla's method of independent variables which is based on the basic principles *i.e.* the foundation is assumed homogeneous and the flow is treated irrotational (two-dimensional). In Khosla's method of independent variables, complex profile is split into simple components which are amenable to mathematical solution. Khosla's potential theory is not applicable to anisotropic heterogeneous medium under hydraulic structures.

The uplift pressure results[6] for different weir profiles with Khosla's method compared well with those obtained from electrical analogy and the deviation does not exceed 2%. Bligh theory gives high values of Phi (residual head as percentage of total differential head) at points on the upstream side and low values at points on the d/s side of weir profile and the difference of Phi values at these points from those obtained by electrical analogy is 12%—18% whereas the corresponding difference for middle part of profile is of the order of 5%. The variation of Phi values obtained from Lane's method and electrical analogy ranges from 0—18%.

Some work has been done in Irrigation Research Institute to find out how non-homogeneity of substratum affects the uplift pressures and this is the subject matter of this paper.

Marala Barrage

To improve silt entry condition in M. R. Link and U.C.C. besides river training, pond at Marala was raised by constructing a new Barrage (completed by 30-9-1968) 1116 ft. downstream of the old weir. New Marala Barrage spans 4475 ft. between the abutments, and comprises 66 bays (1 to 13 on left and

60-66 on right are undersluices bays). For keeping a check on the safety of the Barrage, 10 pressure pipes were installed along Piers 11, 23, 35, 47 and 60 and these pressure pipes in each pier were supposed to monitor the uplift pressure fluctuations under profile of the Barrage.

The Barrage has been designed for the following effective head of water against uplift pressures :—

Undersluices:

U.S.W.L.	=	+812.0	Normal pond level.
D. S. W. L.	=	+787.0	Top of d/s Rubble apron.
H	=	25.0 ft.	

Normal Openings:

U. S. W. L.	=	+812.0	Normal pond level.
D. S. W. L.	=	+790.5	Top of lip wall at d/s end of floor.
H	=	21.5 ft.	

High uplift pressures under d/s floor of bay 35.

The actual effective head-across up to December, 1968 ranged between a maximum of 16.0 to a minimum of 8.0 ft. Piezometers adjunct to Piers 11, 23, 47 and 60, indicated that the Phi values for all points were within the design values. However, for pier 35, Phi values for all points downstream of pile F-3 were 40-49 in the period February 1969—April 1969 and increased beyond 51 in May 1969.

The pressure pipe data was also observed by the staff of Irrigation Research Institute during the period 27-10-1969 to 5-11-1969. The theoretical and observed Phi values for points 4 to 10 at Pier No. 35 on 5-11-1969 are compared below :—

	Pt. 4	Pt. 5	Pt. 6	Pt. 7	Pt. 8	Pt. 9	Pt. 10
Theoretical ..	65	62	51	48	32	21	..
Observed ..	66.72	66.33	62.24	59.9	54.24	50.83	1.19
Difference ..	+1.72	+4.33	+11.24	+11.9	+22.24	+29.83	..

The actual number of bays besides bay No. 35 subjected to high uplift pressures could not be established excepting by drilling through the downstream concrete floor.

The technique of drilling through concrete floor of Marala Barrage and the bore hole data are discussed separately by Mr. Ramiz Ahmad Malik, Deputy Project Director, Marala Barrage.

Hydraulic Model studies were taken up for assessing the causes of excess uplift pressure under downstream floor of bays 31-40 of Marala Barrage and remedial measures.

One of the suggested remedial measures was weighting of the d/s concrete floor with mass concrete or dummy piers with spacings that the nominal reinforcement in the concrete floor can take up the tensile stresses on the concrete. The effect of weighting d/s floor on the performance of hydraulic jump and energy dissipation was determined on the 1/36 natural scale sectional model of Marala Weir in a glass flume.

UPLIFT PRESSURES

Marala Standard Bays

The formulae for determining uplift pressures under hydraulic structures for simple case of level floor with single sheet pile not at end are given in C. B. I. Publication No. 12 and the graph plot of variables of these formulae is reproduced as Fig. 1. Mr. Khosla has evolved a method of solution of problem of determining uplift pressures for complicated profiles known as Khosla's method of independent variables. In this method a complex profile of structure is split up into elementary standard sections, considered as flush floor fitted with single sheet pile not at end which are treated as independent of the others. Pressures at key points (junction points of floor and the pile line, the bottom of pile line and the bottom corner in case of depressed floor) are read from Fig. 1.

The readings of junction points are then corrected for :—

(1) the mutual interference of piles.

(2) the floor thickness.

(3) the slope of floor.

The method is explained below with an example of Marala Barrage Fig. 2 (a-d):—

Effective depth of sheet pile 4 upper most $=795-773.5=21.5'$

Effective depth of sheet pile No. 3 $=(795+61/16)-764=34.8'$

Effective depth of sheet pile No. 2 $=789.5-756.5=33'$

Effective depth of sheet pile No. 1 $=789.5-760.5=29$,
(near Tail end)

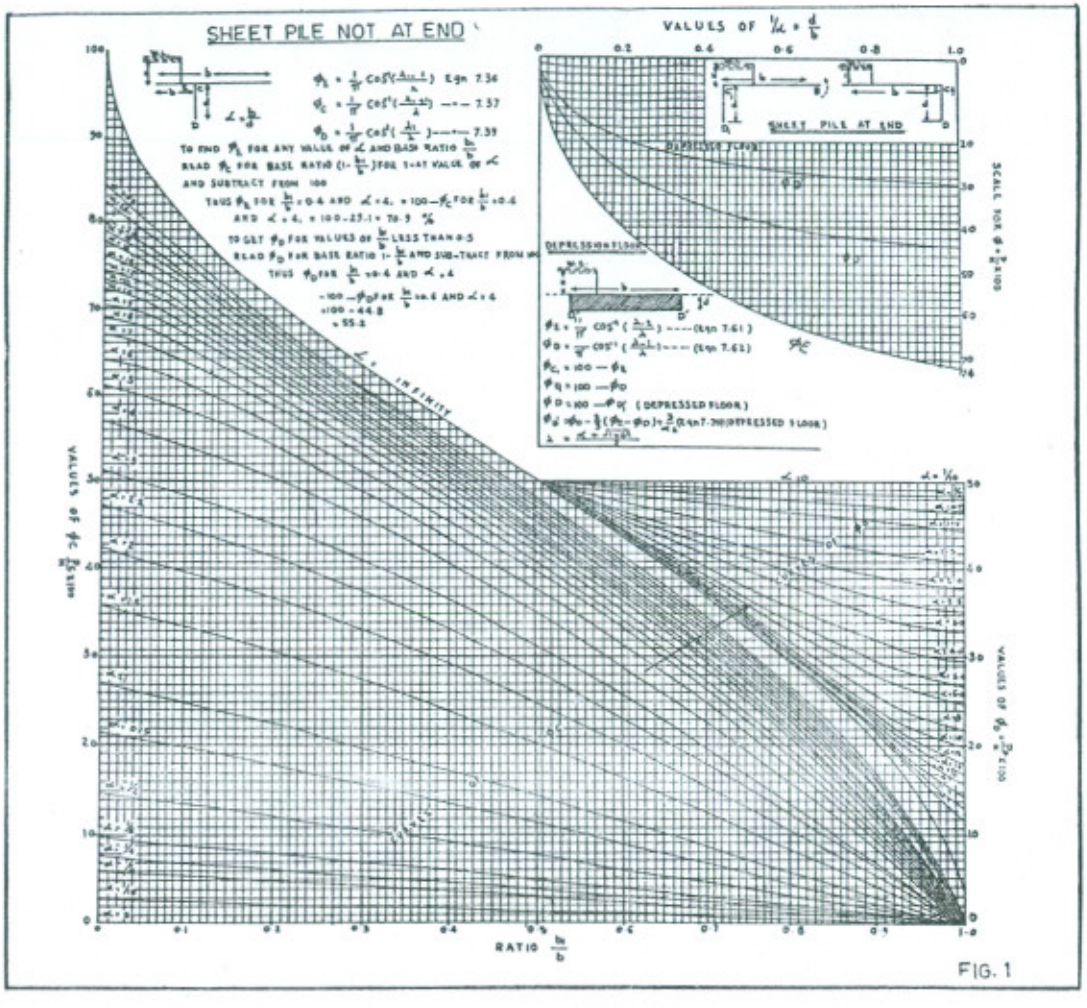

SHEET PILE NOT AT END
VALUES OF $\frac{t}{L} = \frac{d}{b}$
SHEET PILE AT END
DEPRESSED FLOOR
$\phi_A = \frac{1}{\pi}\cos^{-1}(\frac{\alpha_{n-1}}{\alpha_n})$ Eqn 7.36
$\phi_C = \frac{1}{\pi}\cos^{-1}(\frac{\alpha_{n-2}}{\alpha_n})$ ---- 7.37
$\phi_D = \frac{1}{\pi}\cos^{-1}(\frac{\alpha_1}{\alpha_n})$ ---- 7.39
TO FIND ϕ_A FOR ANY VALUE OF α AND BASH RATIO $\frac{b_1}{b}$
READ ϕ_C FOR BASE RATIO $(1-\frac{b_1}{b})$ FOR 1-AT VALUE OF α
AND SUBTRACT FROM 100
THUS ϕ_A FOR $\frac{b_1}{b}$ = 0·4 AND α = 4, = 100 − ϕ_C FOR $\frac{b_1}{b}$ = 0·4
AND α = 4, = 100 − 27·1 = 72·9 %
TO GET ϕ_D FOR VALUES OF $\frac{b_1}{b}$ LESS THAN 0·5
READ ϕ_D FOR BASE RATIO 1− $\frac{b_1}{b}$ AND SUB-TRACT FROM 100
THUS ϕ_D FOR $\frac{b_1}{b}$ = 0·4 AND α = 4
= 100 − ϕ_D FOR $\frac{b_1}{b}$ = 0·6 AND α = 4
= 100 − 44·8
= 55·2
$\alpha = \infty$ IN LIMIT
$\phi_L = \frac{1}{\pi}\cos^{-1}(\frac{\alpha_{n-1}}{\alpha_n})$ ---- (Eqn 7.61)
$\phi_D = \frac{1}{\pi}\cos^{-1}(\frac{\alpha_1}{\alpha_n})$ ---- (Eqn 7.62)
$\phi_C = 100 - \phi_A$
$\phi_A = 100 - \phi_D$
$\phi_D = 100 - \phi_{D'}$ (DEPRESSED FLOOR)
$\phi_{d'} = \phi_D - \frac{\lambda}{\alpha_n}(\phi_A - \phi_D) + \frac{\lambda}{\alpha_n}$ (Eqn 7.70 DEPRESSED FLOOR)
$\lambda = \alpha - \alpha_{(n-1)}$
VALUES OF ϕ_A FOR $\frac{b_1}{b}$ RATIO
SCALE FOR $\phi = \frac{p}{p_0} \times 100$
VALUES OF $\phi_D = \frac{p}{p_0} \times 100$
RATIO $\frac{b_1}{b}$

FIG. 1

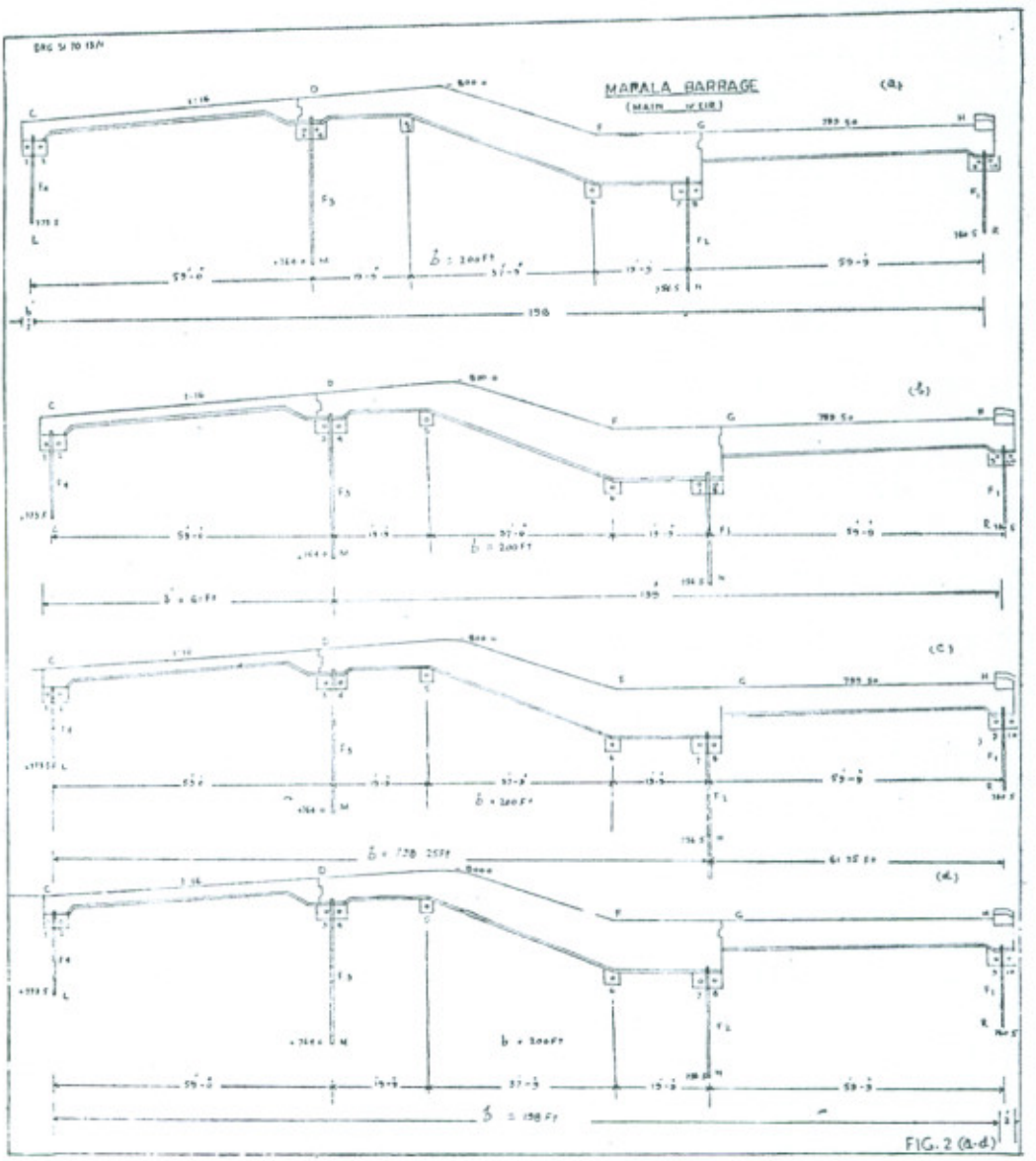

MAHALA BARRAGE
(MAIN W.SIR)
(a)
(b)
(c)
(d)
59'-0"
19'-9"
37'-9"
19'-9"
59'-9"
156
b = 200 Ft
b = 198 Ft

FIG. 2 (a-d)

Consider Points 1 and 2—Fig. 2(*a*):

$b = 200'$

$b_1 = 2'$

$b_1/b = 2/200 = .01$

$d = 21.5$

$b/d = 200/21.5 = 9.3$

ϕ_E of Fig. 1 or ϕ_1 of Fig. 2 $= 100 - 1.5 = 98.5\%$

ϕ_C of Fig. 1 or ϕ_2 of Fig. 2 $= 70.5\%$

ϕ_D of Fig. 1 or ϕ at R. L. $= 773.5$

(Toe of sheet pile F4) $= 79.0\%$

ϕ_1 correction for floor thickness

$$= \frac{(98.5 - 79)}{21.5} \times 4 = 3.6$$

ϕ_2 Correction for floor thickness

$$= \frac{(79 - 70.5)}{21.5} \times 4 = 1.58$$

Correction for interference of a pile

$$= 19 \sqrt{\frac{D}{b_1}} \cdot \frac{D+d}{b}$$

where :

D = depth of sheet pile whose effect is to be determined. This depth is with reference to some fixed R. L., say top of floor.

d = depth of sheet pile at which effect is to be determined. This depth is with reference to some fixed top of floor.

b_1 = Horizontal distance between piles.

b = Total length of impervious floor.

ϕ_2, correction for interference of upper intermediate sheet pile F3

$$= 19 \sqrt{\frac{31.0}{59}} \cdot \frac{31.0+21.5}{200} = 3.60$$

ϕ_1 eorrected value $= 98.5 - 3.6 = 94.9\%$

ϕ_2 corrected value $= 70.5 + 1.58 + 3.60 = 75.68$

Consider Points 3 and 4 of Fig. 2(*b*):

Calculations for pressure at points 3 & 4 below the underside of floor shall be as under :—

$b = 200$

$b_1 = 61$

$b_1/b = 61/200 = 0.305$

Depth of sheet pile $= (795+61/16)-764 = 34.8$

$$\frac{b}{d} = \frac{200}{34.81} = 5.75$$

$\phi_3 = 74\%$

$\phi_4 = 50.5$

ϕ at R. L. 764 bottom of sheet pile F3 $= 60.9\%$

ϕ_3 correction for floor thickness $= (13.1/34.8)\times 5.55 = 2.09\%$

$$\left.\begin{array}{l}\phi_3 \text{ correction for interference} \\ \text{of sheet pile No. 4}\end{array}\right\} = 19\sqrt{\frac{24.80}{57}}\,\frac{24.5+34.8}{200} = 3.72$$

ϕ_3 corrected value $= 74-3.72-2.09 = 68.19$

Corrections for $\phi 4$

1. For thickness of floor $= \dfrac{10.4\times 5.55}{34.8} = 1.66\%$

2. Due to interference of sheet pile F2 $= 19\sqrt{\dfrac{42.3}{77.25}}\,\dfrac{42.3+34.8}{200}$

$= +5.42$

Corrected value of ϕ at point 4 $= 50.5+5.42+1.66 = 57.58\%$

Consider Point 5

$$\text{Correction for slope} = \frac{b_3}{b_1}\times \text{K}$$

where :—

b_3 = length along horizontal of slope.

b_1 = distance between piles.

k = correction factor from graph.[7]

$$\frac{37.75}{77.25}\times 4.75 = 2.33\%$$

This will be+ive because the slope is downward. To calculate ϕ at point 5 :—

$$\frac{b_1}{b} = \frac{80.75}{200} = .404$$

b/d =200/5.75=34.8

ϕ_5 = 56% (read ϕ_D for base ratio of 0.6 *i.e.* 34.8 and substract from 100).

Corrections for ϕ_5

1. Due to mutual interference of piles

   (*a*) due to sheet pile F3

   here D=800−764=36

   d=5.75

   b=19.75

$$\text{Correction} = 19 \sqrt{\frac{36}{19.75}} \cdot \frac{36+5.75}{200} = -5.35$$

   (*a*) due to sheet pile F2

   here D=800—756.5=44.5

   d =5.75

   b =57.5

$$\text{Correction} = 19 \sqrt{\frac{44.5}{57.5}} \cdot \frac{44.5+5.75}{200} = +4.2$$

Corrected value of ϕ_5=56+2.33+4.2—5.35=57.18

Consider Point 6

b_1=118.5, b_1/b=118.5/200=.592, α =200/10.5=19.0

ϕ_6=45%

1. Correction for slope=2.33%
2. Correction for interference of sheet pile F3

$$= 19 \sqrt{\frac{25.5}{57.5}} \cdot \frac{25.5+10.5}{200} = -2.28$$

3. Correction for interference of sheet pile F2

$$= 19 \sqrt{\frac{33}{19.75}} \cdot \frac{33+10.5}{200} = +5.33$$

ϕ_6 corrected value=45.00+2.33—2.28+5.33=50.38

Consider Point 7

$b_1 = 138.25$, $b_1/b = 138.25/200 = .691$

$\alpha = \frac{200}{33} = 6.06$

$\phi_7 = 48\%$

$\phi_8 = 28\%$

ϕ at R. L. 765.5 $= 39.1\%$

ϕ_7 correction for floor thickness $= (8.9/33) \times 10.5 = 2.83$

ϕ_8 correction for floor thickness $= (11.1/33) \times 10.5 = 3.53$

ϕ_7 correction for interference due to sheet pile F3

$$= 19 \sqrt{\frac{25.5}{77.25}} \times \frac{25.5+33}{200} = 3.12\%$$

ϕ_8 correction for interference due to sheet pile at F1

$$= 19 \sqrt{\frac{29}{59.75}} \times \frac{29+33}{200} = 4.1\%$$

ϕ_7 (corrected value) $= 48 - 3.12 - 2.83 = 42.05$

ϕ_8 (corrected value) $= 28 + 3.53 + 4.1 = 35.63\%$

Consider Point 9

$b_1 = 198$, $b_1/b = 198/200 = 0.99$, $\alpha = 200/29 = 6.9$

$\phi_9 = 33.5\%$

$\phi_{10} = 2.00$

ϕ at R. L. 760.5 $= 23.0\%$

ϕ_9 correction for floor thickness $= (10.50/29) \times 6.25 = 2.26$

ϕ_9 correction for interference due to sheet pile F2

$$= 19 \sqrt{\frac{33}{59.75}} \cdot \frac{33+29}{200} = -4.37$$

ϕ_{10} correction for floor thickness $= 21.0/29 \times 6.25 = +4.52$

ϕ_9 corrected value $= 33.5 - 2.26 - 4.37 = 82.87\%$

ϕ_{10} corrected value $= 2.0 + 4.52 = 6.52\%$

LIMITATIONS OF METHOD

The above described method for calculating the uplift and exit gradient is admissible under the following limitations :—

(*i*) The flow is two-dimensional.

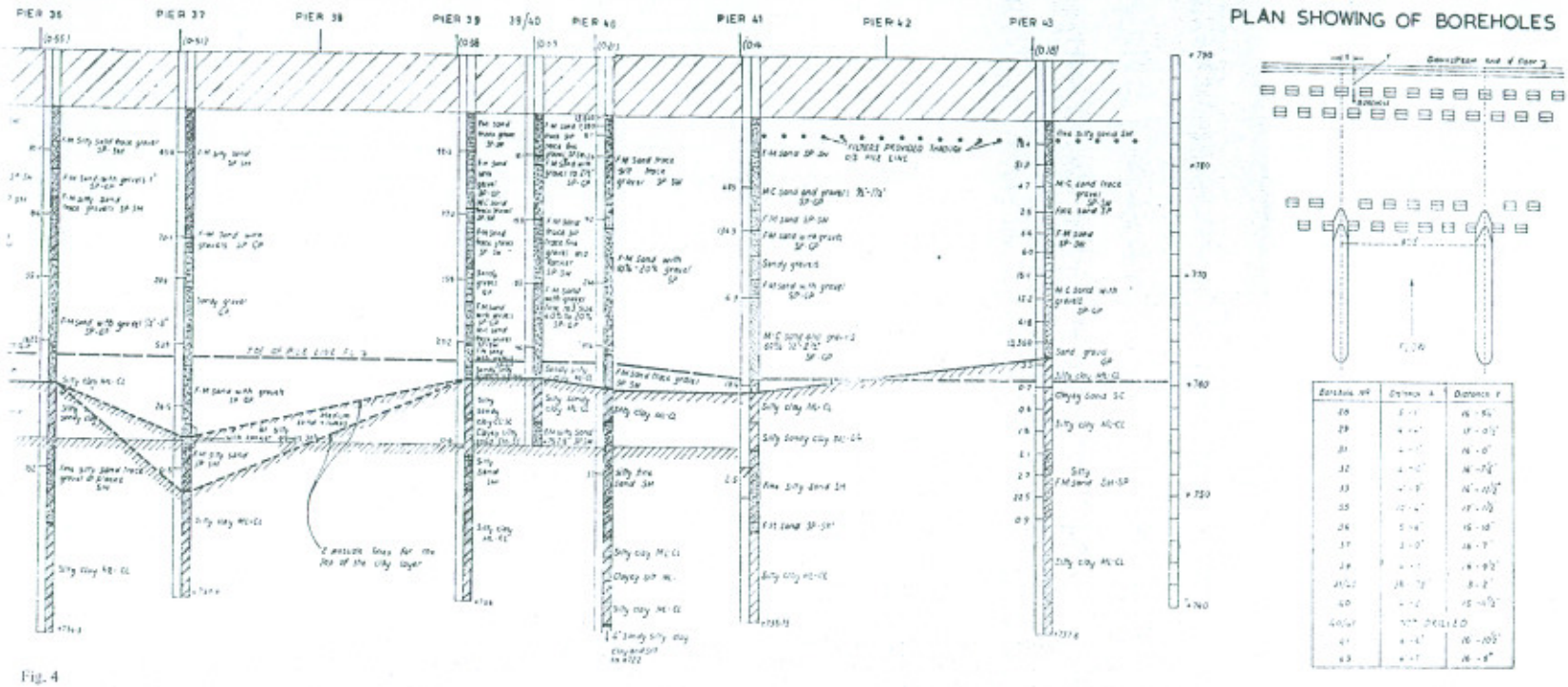

PIER 36
PIER 37
PIER 38
PIER 39
39/40
PIER 40
PIER 41
PIER 42
PIER 43
PLAN SHOWING OF BOREHOLES

Fig. 4

PAKISTAN MARALA BARRAGE DOWNSTREAM SUB–SOIL INVESTIGATION PIER 26 TO 43

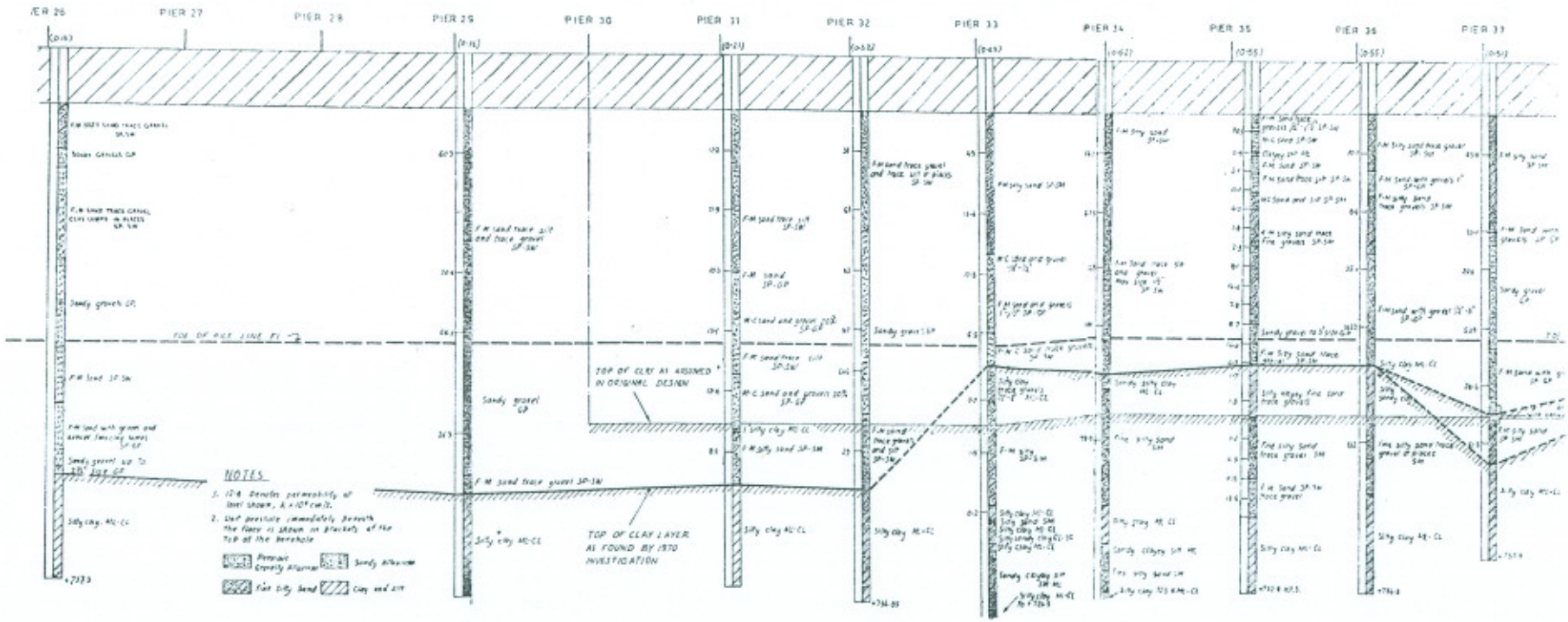


Fig. 4

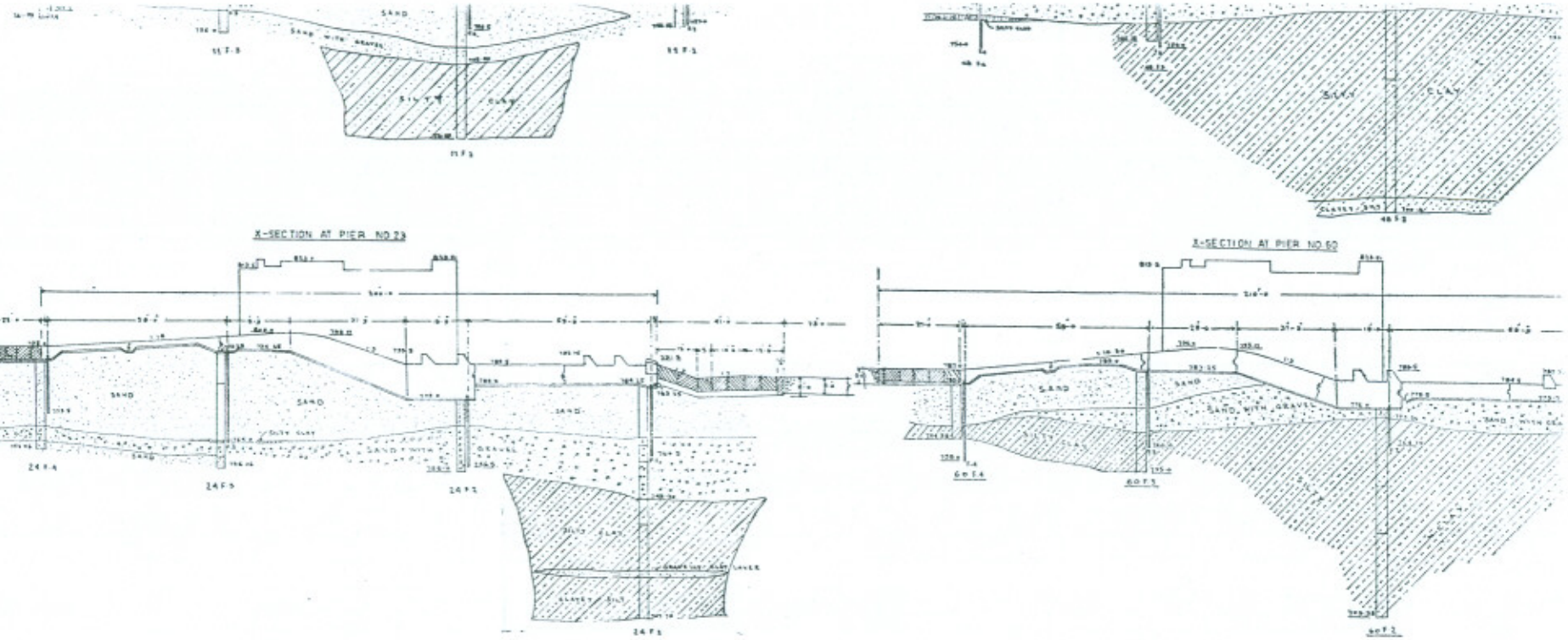
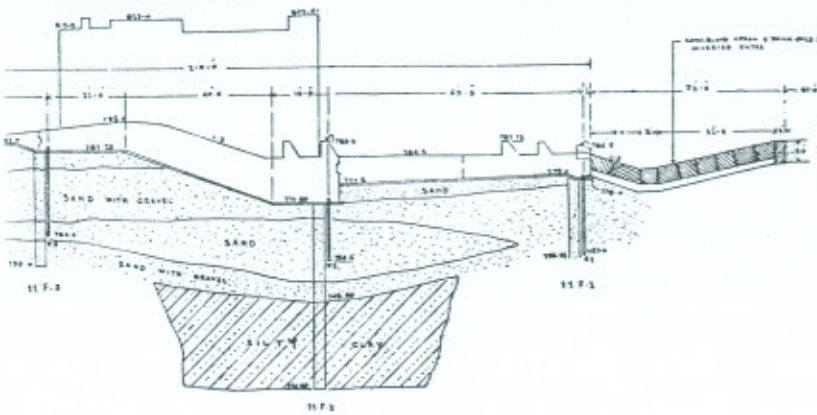
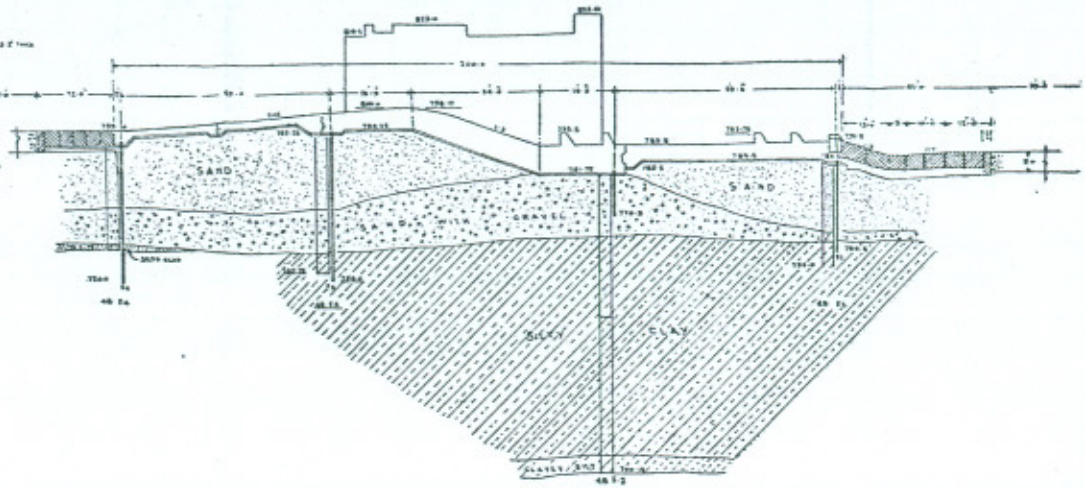

X-SECTION AT PIER NO 23
X-SECTION AT PIER NO 60
X-SECTION AT PIER NO 35
SAND
SAND WITH GRAVEL
SILT
CLAY
SILTY CLAY
SANDY WITH GRAVEL
CLAYEY SAND
NOTE

Fig. 3

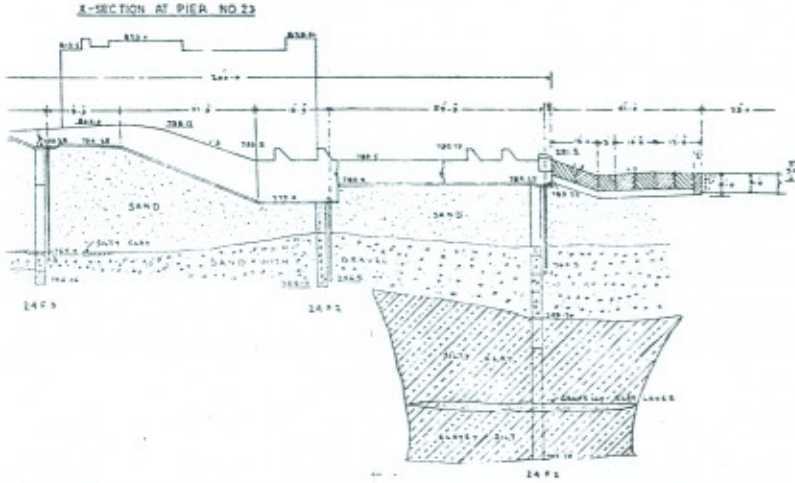
X-SECTION AT PIER NO.11



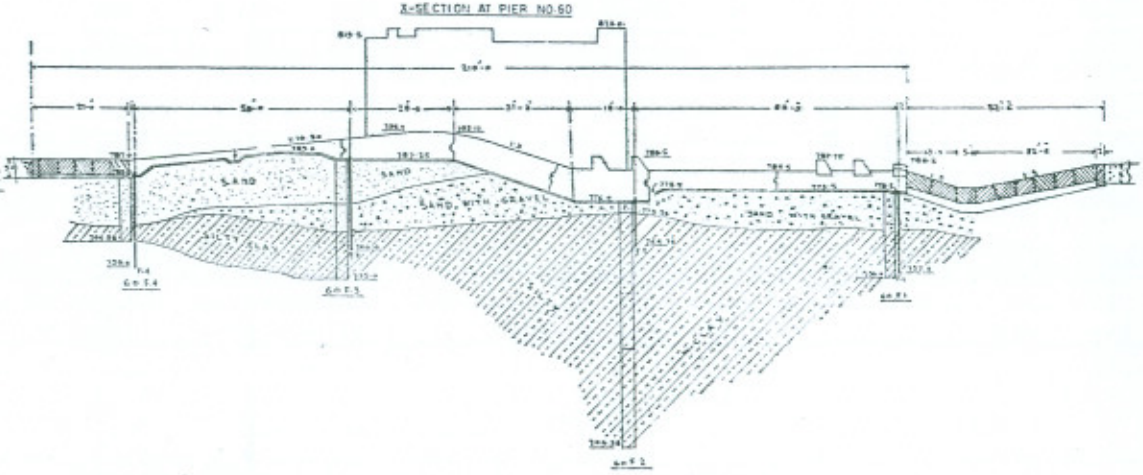
X-SECTION AT PIER NO.67



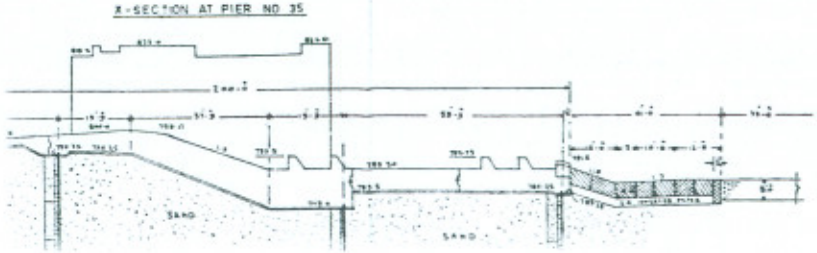
X-SECTION AT PIER NO.23



X-SECTION AT PIER NO.60



X-SECTION AT PIER NO.35



NOTE

(*ii*) The substratum is homogeneous, isotropic and deep.

(*iii*) There is no silt blanket or semi-impervious layer upstream or d/s of the impervious floor.

(*iv*) There is no temperature gradient in the direction of or against the flow.

Dr. Mushtaq Ahmad[7] has discussed in Paper No. 294 submitted in the Punjab Engineering Congress 1952, the correction to be applied for silt blanket.

GEOLOGICAL SUBSTRATUM AT MARALA BARRAGE

Indus valley construction company observed bore hole data (in July 1965) along 4 rows of sheet piles and the bore holes were opposite each 4th pier. Deep bores were made along F_2 (d/s intermediate sheet pile) for determining the thickness of clay layer. The soil profile indicated a thick layer of clay and the top level of this clay is at R.L. 740 at the left abutment and rises to R. L. 775 near the right abutment. In view of the sub-soil investigation Coode and Partners readjusted the bottom level of the sheet piles and the revised bottom R. L. of the sheet piles are given in Fig. 3 which reveals that the u/s and d/s sheet piles are penetrating the clay near piers 47 and 60 and all the 4 sheet piles in between piers 11 to 35 are in sand above clay.

FURTHER GEOLOGICAL INVESTIGATIONS AFTER THE CONSTRUCTION OF THE BARRAGE

In order to investigate the causes of excessive uplift pressures, at site, drill holes through the downstream concrete floor on the line between two rows of deflector blocks were started by Indus Valley construction company on 21-1-1970 and completed on 19-4-1970. Geological section, ϕ-values computed from observed piezometric pressures through borings, the order of permeabilities, the top of impervious layers, the tip of downstream sheet pile F_1 are given in Fig. 4. The bottom R. L. of sheet pile F_1 (at tail end) is 760.5 against top elevation of impervious layer in bays 31, 32, 33, 34, 35, 36, 37, 38, 39, 40 at 752, 747, 755, 757, 758, 759, 754, 759 & 758 respectively. The soil in between tip of sheet pile and the clay layer is of low permeability. Thus it is evident that the impermeable layer lies close enough to the tip of sheet pile to severely restrict the drainage under the sheet piling. Drainage right of pier No. 40 appears to be ample (probably due to weep holes in sheet pile F_1) although the upper surface of relatively impervious layer is above R. L. 760.5.

VI. THE HYDRAULIC MODEL REPRODUCTION OF DESIGNED AND PROTOTYPE PHI VALUES AND CAUSE OF HIGH PRESSURE ON THE DOWNSTREAM FLOOR OF BAY NO. 35

A sectional model of weir representing bay No. 35 of Marala Barrage was constructed in seepage study 0.4′ wide glass flume on 1/80 natural scale. The details of experimental glass flume and model technique are given in Chapter IV of Central Board of Irrigation Publication No. 12. The water entering the model was cleared of impurities by glass wool filters before introducing into the model. The water levels upstream and downstream of the gate were reproduced at R. L. 812 and 789.5 respectively. The water levels in the pipes were observed in manometric bank observing all possible precautions.

PROOF RUN TESTS

(*a*) In order to check up the designed Phi values on the model, a test was made with homogeneous media under the structure. The results are plotted in Fig. 5 where in the designed Phi values and the prototype observed Phi values have also been superposed. It will be seen that there is fair agreement between model and the designed values.

(*b*) In an attempt to obtain prototype reproduction of Phi values on the model, the prototype geological stratification as based on 1965 contract borings (Fig. 3) was simulated on model. The clay layer in the foundation of Bay No. 35 as reproduced on the model was downstream of gate line 45′ in depth and the sand gap between sheet pile F-4 and clay layer was 15′. A shingle sand layer in depth of about 7′ overlaid the clay layer. The seepage flow lines with 1965 stratification are depicted in Fig. 6 (*a*) which indicate that seepage flow above clay layer follows sandy shingle (the path of least resistance) and impermeable clay layer deflects the flow lines away from toe of downstream floor. The Phi values as recorded from the hydraulic model are compared with prototype and theoretical values based on Central Board of Irrigation Publication No. 12 in Fig. 6-b. The model Phi values are below the prototype values, but higher than the theoretical values. The Phi values at point 8 recorded on Hydraulic model are 12% lower than prototype value and 10% higher than theoretical values. The model Phi value at point 9 is 19% lower than prototype value and 14% higher than theoretical value. The clay layer in full depth was extended towards sheet pile F-1 in a step of 3.0′ in each test and model Phi values were observed after maintaining pond level at 812 and tail level at 789.5 and are plotted in Fig. 6-*b* which reveals that as the sand gap between clay lens and downstream and sheet pile is reduced the pressures above end sheet pile

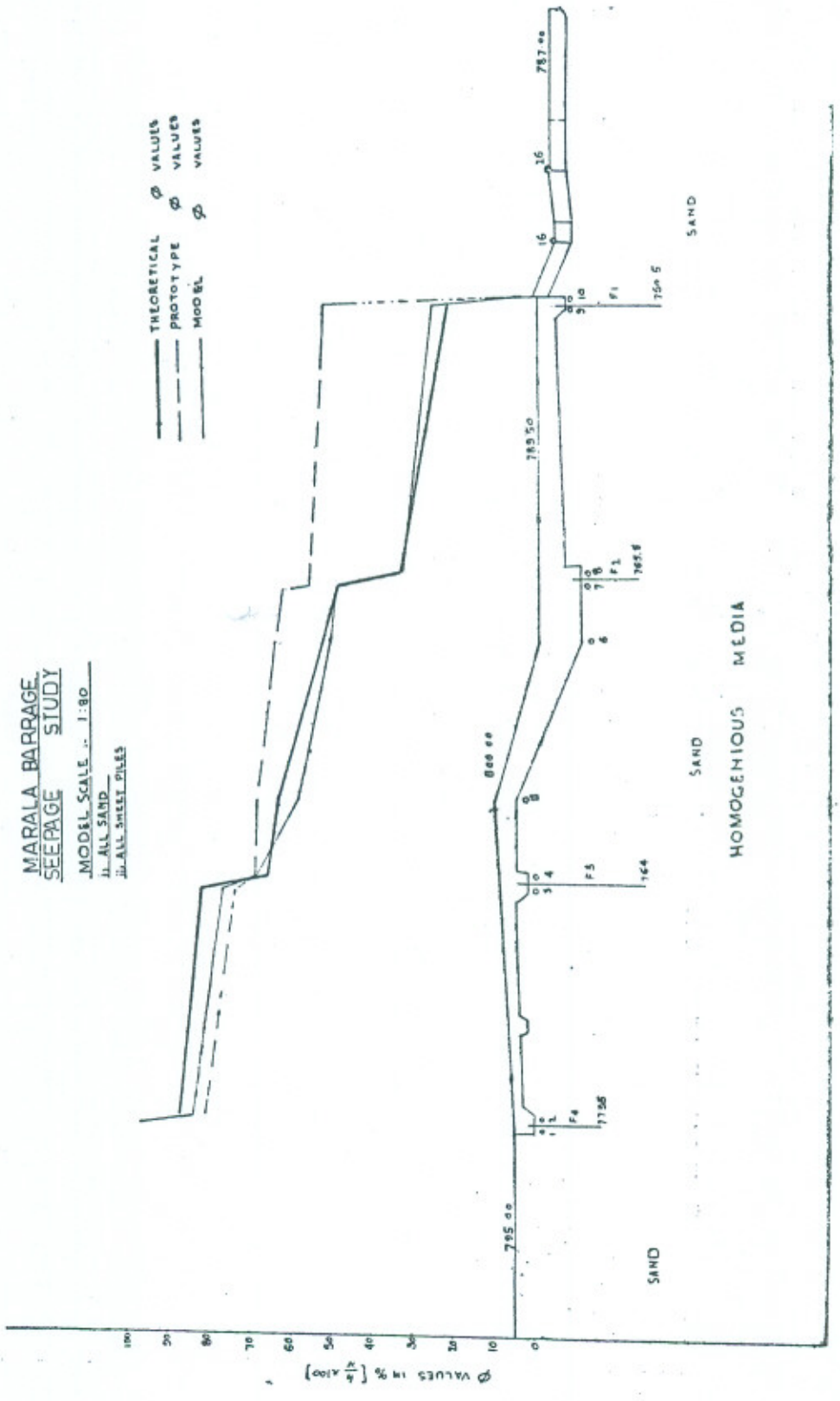

MARALA BARRAGE
SEEPAGE STUDY
MODEL SCALE :- 1:80
i. ALL SAND
ii. ALL SHEET PILES
THEORETICAL Ø VALUES
PROTOTYPE Ø VALUES
MODEL Ø VALUES
SAND
HOMOGENIOUS MEDIA
SAND
SAND
787.00
785.50
800.00
795.00
Ø VALUES IN % [h/H x 100]

Fig. 5

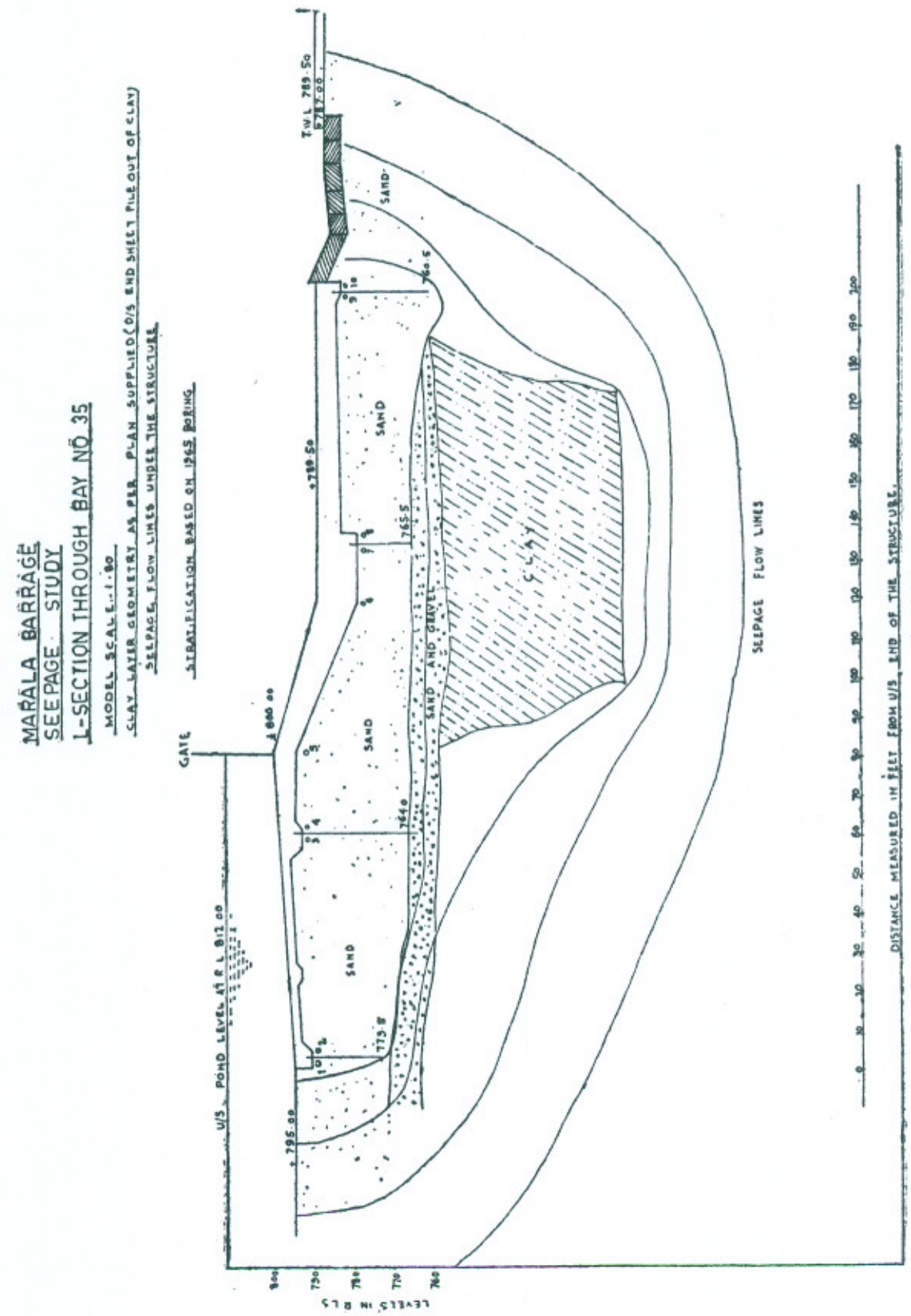

MARALA BARRAGE
SEEPAGE STUDY
L-SECTION THROUGH BAY NO 35
MODEL SCALE 1:80
CLAY LAYER GEOMETRY AS PER PLAN SUPPLIED (U/S END SHEET FILE OUT OF CLAY)
SEEPAGE FLOW LINES UNDER THE STRUCTURE.
STRATIFICATION BASED ON 1965 BORING
GATE
U/S POND LEVEL AT R.L 812.00
T.W.L 789.50
SAND
CLAY
SAND AND GRAVEL
SEEPAGE FLOW LINES
LEVELS IN R.L.S
DISTANCE MEASURED IN FEET FROM U/S END OF THE STRUCTURE.

Fig. 6 (*a*)

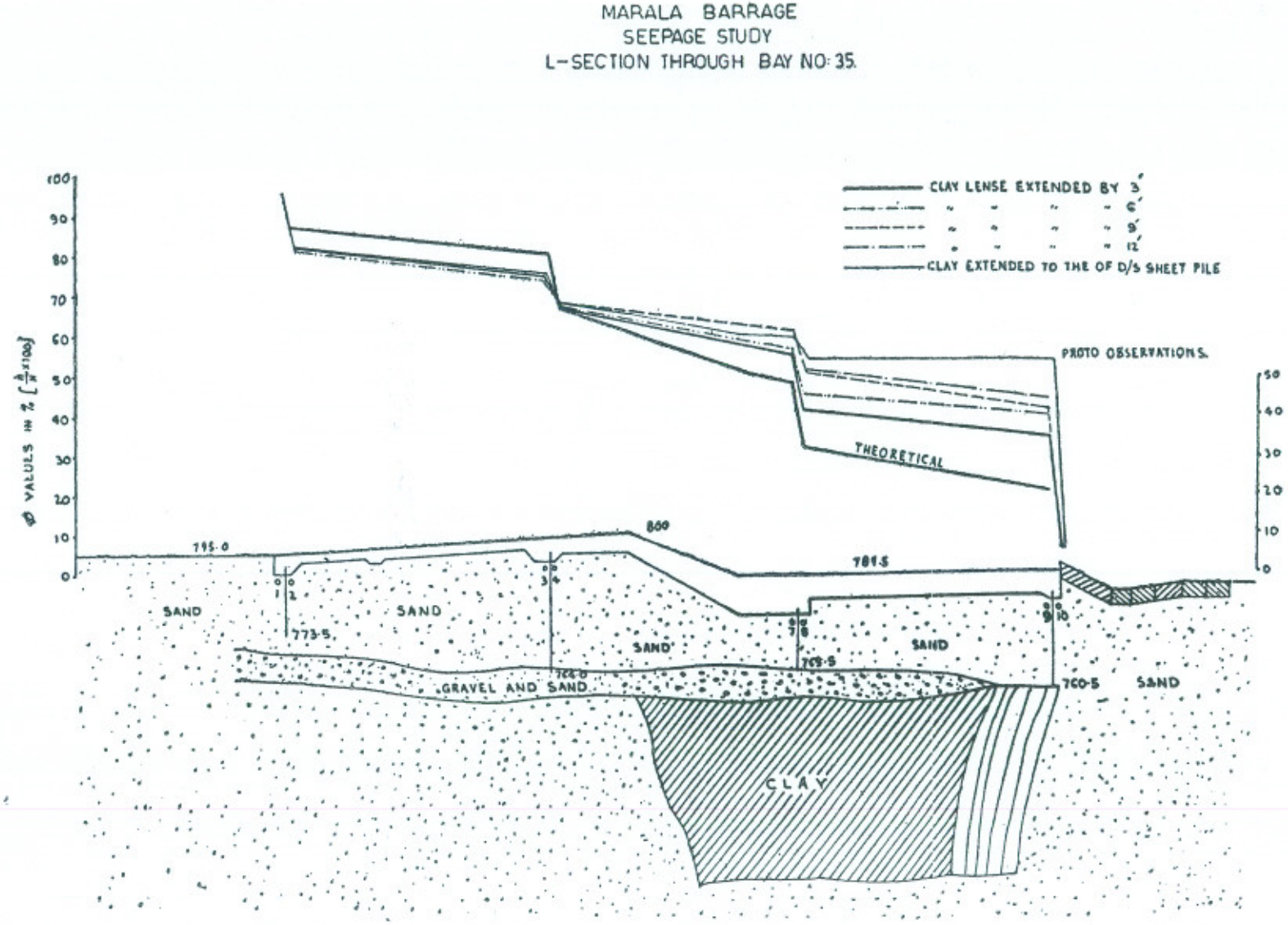

MARALA BARRAGE
SEEPAGE STUDY
L-SECTION THROUGH BAY NO: 35.
CLAY LENSE EXTENDED BY 3'
" " " " 6'
" " " " 9'
" " " " 12'
CLAY EXTENDED TO THE OF D/S SHEET PILE
PROTO OBSERVATIONS.
THEORETICAL
Ø VALUES IN % [h/H x 100]
795·0
800
781·5
773·5
769·5
760·5
SAND
SAND
SAND
SAND
SAND
GRAVEL AND SAND
CLAY

Fig. 6 (*b*)

build up and when this gap is replaced by clay the prototype Phi values are reproduced.

(*c*) The prototype geological stratification as based on 1965 contract borings and borings in 1970 as plotted approximately in Fig. 4 was next simulated on the model. In the absence of bore hole data the extent of clay layer was fixed arbitrarily. The upper surface of relatively impermeable layer below the tips of sheet pile F-1 (tail sheet pile) and F_2 (lower intermediate sheet pile) is 4.5 and 5.5 feet respectively.

The critical operating conditions (pond at R. L. 812 and tail at R. L. 789.5) were reproduced on model. The water levels of pressure recording points were observed by observing usual precautions. The seepage flow lines are depicted in Fig. 7 (*a*) which indicate that impermeable clay layer deflects the flow lines away from base of downstream floor. The Phi values as recorded from hydraulic model are compared with prototype values and theoretical values based on Central Board of Irrigation Publication No. 12 in Fig. 7 (*b*). The model Phi values are below the prototype values, but higher than the theoretical values. The Phi values at points 8 and 9 recorded on hydraulic model are 5% and 11% lower than prototype values and 19-20% higher than the theoretical values. These results indicate that the model stratification is not in accordance with the prototype.

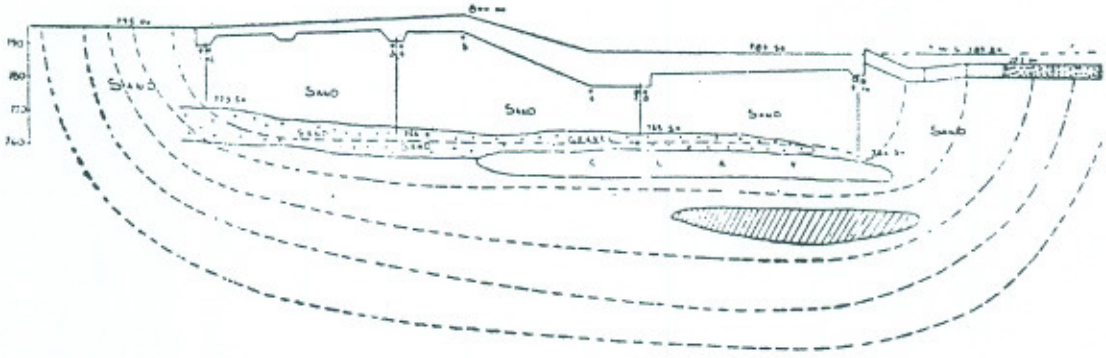


Fig. 7 (*a*)

The above test was repeated by replacing sand in between two layers by clay. The Phi values by assuming that the fine medium sand, sandwiched between two clay lenses is assumed of permeability nearer to that of clay are also superposed in fig. 7-*b* which indicates that the Phi at point 8 is not affected whereas Phi value at point 9 has gone up by 4.5% but is still short of prototype value.

In the next test the sand in gap between toe of tail sheet pile (F_1) and upper layer of clay was replaced by clay *i.e.* flow section at toe of F_1 is plugged with clay. The recorded Phi values are also superposed in Fig. 7-*b*. The Phi values of this test exactly conform to the prototype values. Thus, the build-up

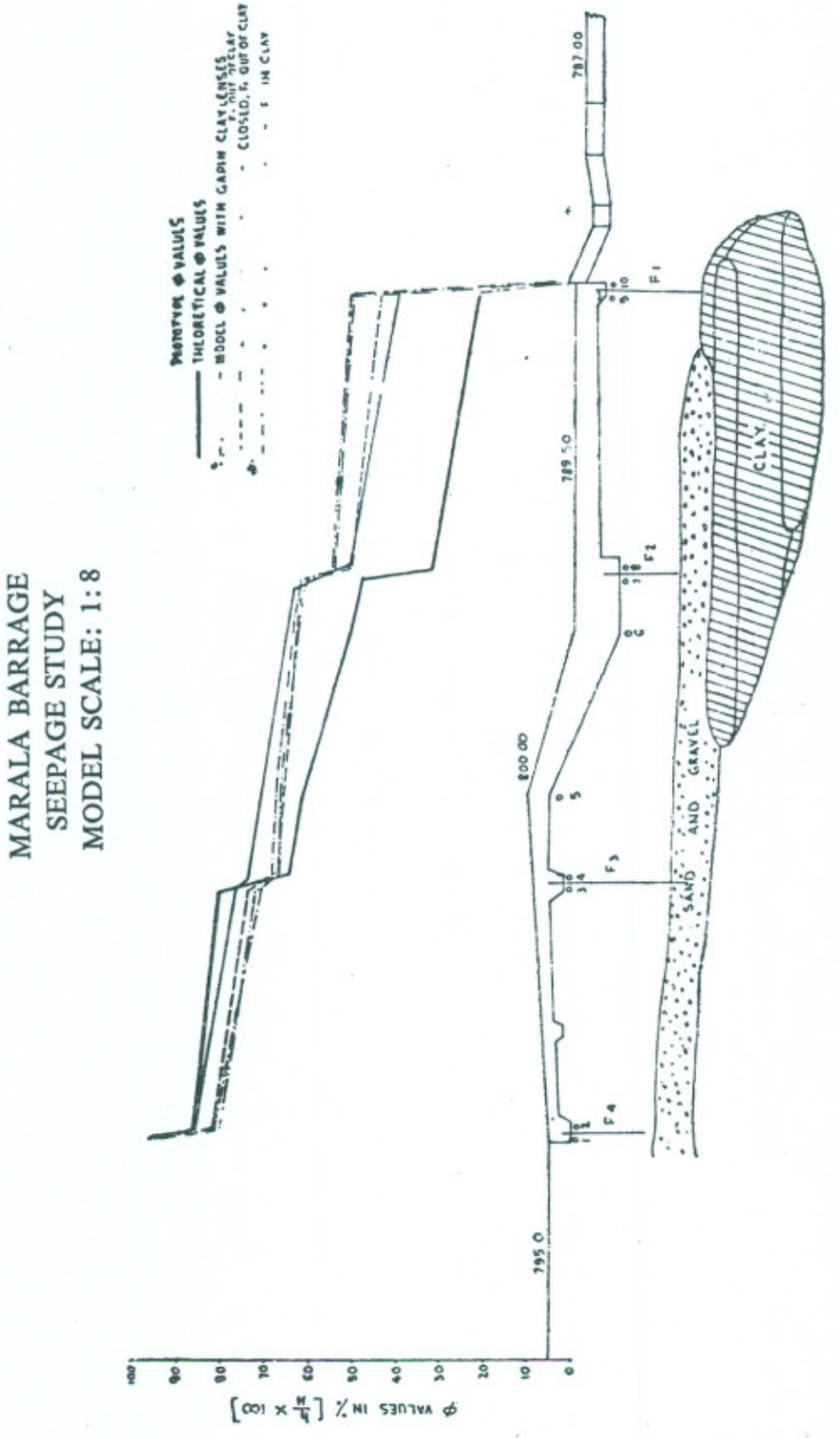

MARALA BARRAGE
SEEPAGE STUDY
MODEL SCALE: 1:8
PROTOTYPE Φ VALUES
THEORETICAL Φ VALUES
- MODEL Φ VALUES WITH GAPS IN CLAY LENSES
- CLOSED, IE. OUT OF CLAY
- - - IN CLAY
787.00
789.50
800.00
795.0
F1
F2
F3
F4
CLAY
SAND AND GRAVEL
Φ VALUES IN % [k/k × 100]

Fig. 7 (*b*)

of pressure on the downstream floor of bay No. 35 is due to the fact that the sheet pile F_1 is penetrating the impermeable layer and the drainage under this sheet pile F_1 is obstructed. This is also substantiated by the observed constant head permeability tests in bore holes. The order of permeability of silty clay and fine medium silty sand below R. L. 760.5 (toe of sheet pile F_1) is 1.3×10^{-3} cm/sec., to 1.7×10^{-3} cm/sec. and 1.1×10^{-3} to 4.3×10^{-3} cm/sec respectively whereas the permeability of sand is of the order of 10^{-1} cm/sec.

The seepage flow lines when flow section under sheet piling F_1 is blocked are shown in Fig. 7-*c* which indicates that stream tubes expand as these leave the clay layer and the flow lines are deflected way from bottom of weir profile in the vicinity of toe of sheet pile F_3.

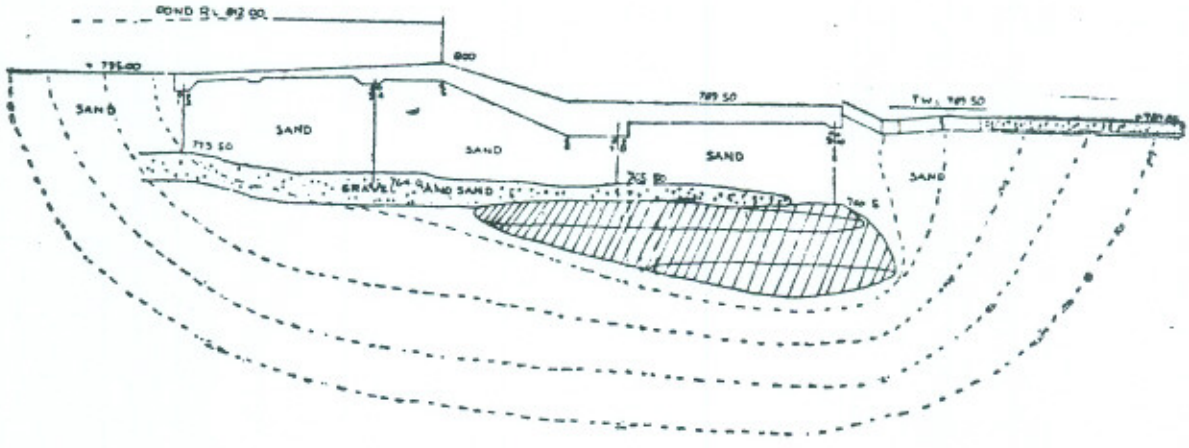


Fig. 7 (*c*)

CLAY LENS IN FOUNDATION OF THE WEIR UNDER THE UPSTREAM GLACIS

To study the effect of clay lens under the upstream floor on upstream glacis a clay lens with hypothetrical geometry as shown in Fig. 8-*a* was represented on the model, whereas the clay lens under the downstream floor was replaced with sand. The clearance between the top of clay lens and tip of sheet piles F_3 and F_4 was of the order of 3 feet, and the tests with the following changes were made:

(*a*) F_3 and F_4 both clear of clay.
(*b*) Test (*a*) above but F_4 piercing into clay lens.
(*c*) Test (*a*) but F_3 piercing into clay lens.
(*d*) F_3 and F_4 both piercing the clay lens.

The Phi values at 10 points are plotted in the Fig. 8-*a*. The tests were repeated in presence of 63.0′ long upstream impervious floor in continuation of the upstream glacis assuming that the existing flexible apron will be converted into impervious floor by cement grouting. The results with upstream impervious

floor are plotted in Fig. 8-*b*. The following conclusions can be drawn from the Figs—8-*a* and 8-*b*.

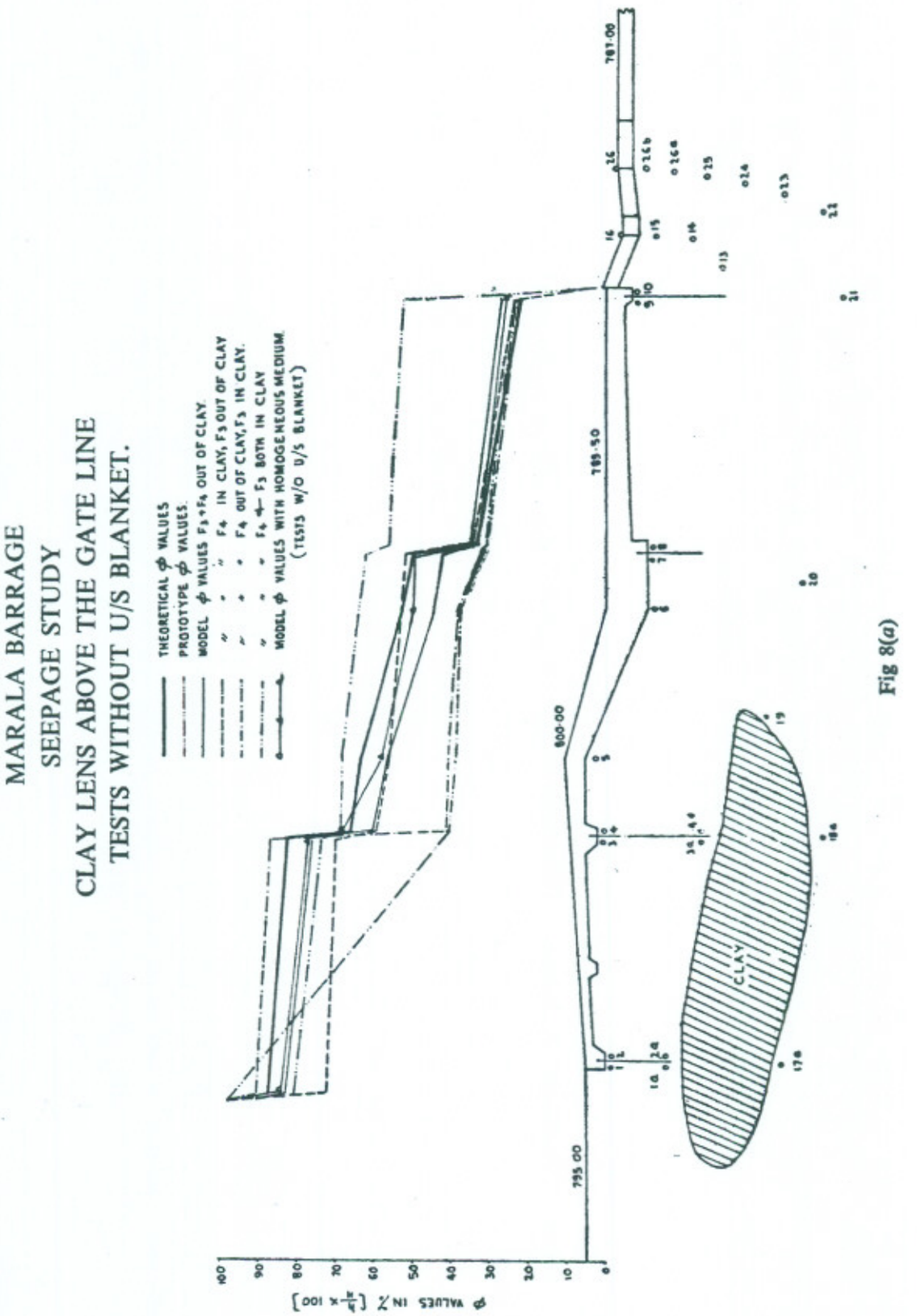


Fig 8(*a*)

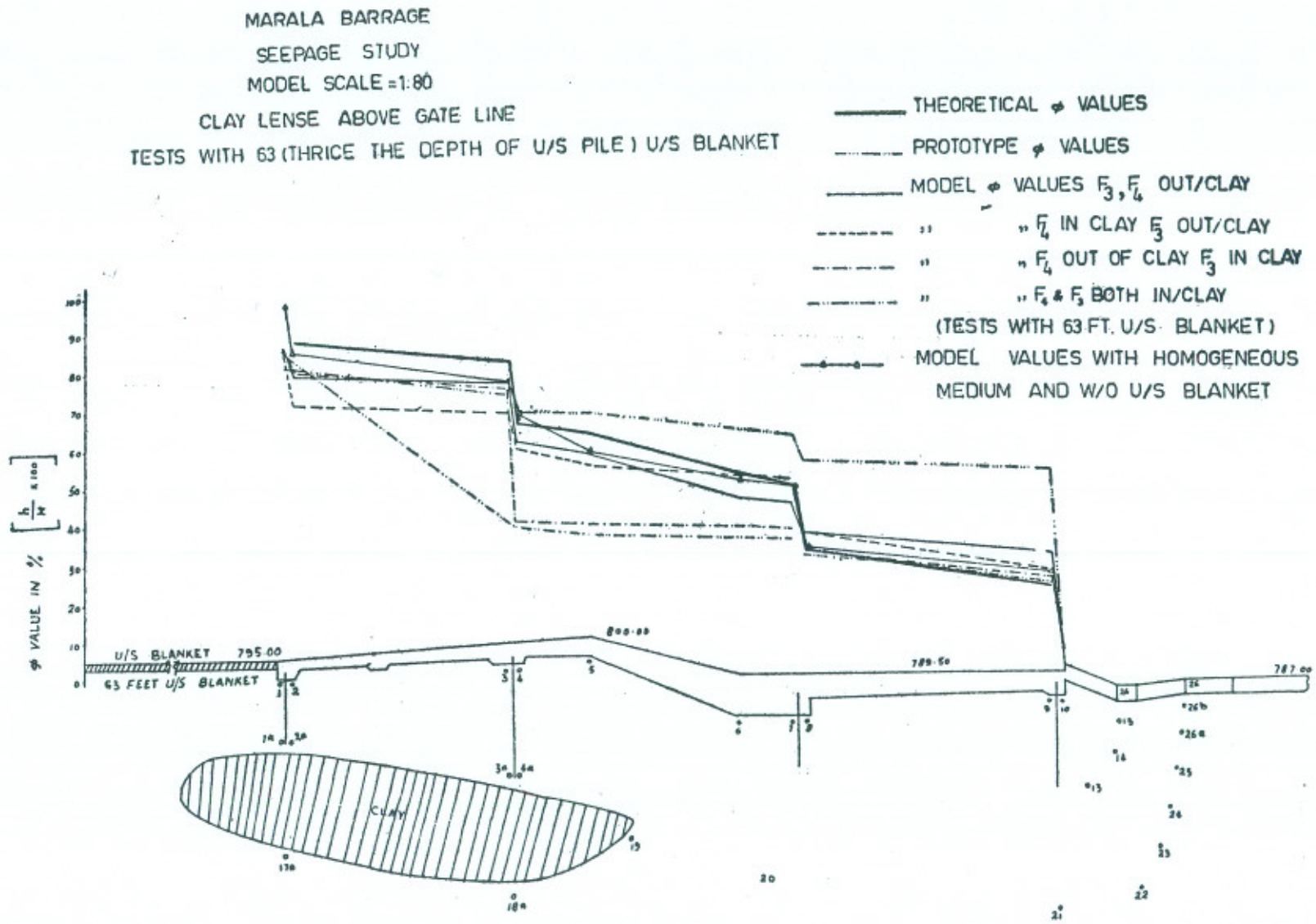

MARALA BARRAGE
SEEPAGE STUDY
MODEL SCALE = 1:80
CLAY LENSE ABOVE GATE LINE
TESTS WITH 63 (THRICE THE DEPTH OF U/S PILE) U/S BLANKET
THEORETICAL φ VALUES
PROTOTYPE φ VALUES
MODEL φ VALUES F3, F4 OUT/CLAY
" F4 IN CLAY F3 OUT/CLAY
" F4 OUT OF CLAY F3 IN CLAY
" F4 & F3 BOTH IN/CLAY
(TESTS WITH 63 FT. U/S BLANKET)
MODEL VALUES WITH HOMOGENEOUS MEDIUM AND W/O U/S BLANKET
φ VALUE IN % [h/H x 100]
U/S BLANKET 795.00
63 FEET U/S BLANKET
800.00
789.50
787.00
CLAY

Fig. 8(*b*)

(*a*) Clay layer under the upstream glacis or upstream floor obstructing lines of flow tends to reduce pressure significantly on the upstream and downstream glacis, and provides comparatively lesser relief on pressures, acting on the downstream floor.

(*b*) If the sheet pile F_4 or even F_3 is embedded into clay (or rock) and the clay cuts off all the flow lines under the weir, no pressure will be transmitted to the downstream floor and if the downstream sheet pile is clear of clay, the line on the downstream floor will follow the fluctuations in tail water level, but if clay stratum is not very deep, the flow lines after being deflected by the clay lens will rise up and touch the base of the downstream floor, which will be subjected to pressure depending on the number of flow tubes, rising towards the downstream floor.

(*c*) The effect of upstream silt blanket in presence of clay lens in reduction of uplift pressures is not significant specially when F_3 or F_4 is piercing the clay lens.

THE REMEDIAL MEASURES

For lowering Phi values under d/s floor

After proving the model different possible remedial measures effecting the uplift pressures were tested on this model.

I. EFFECT OF U/S FLOOR ON REDUCTION OF PRESSURES ON D/S FLOOR

No u/s impervious floor has been provided at Marala Headworks. There is flexible apron 63′ in length, that can be converted into impervious floor by grouting and building a new flexible apron. The 4 lengths of u/s floor tested on the model are equal to D_1, $2D_1$, $3D_1$ and $4D_1$ where D_1 is the effective depth of sheet pile F_4. The model phi values for these tests are plotted in Fig. 9 which indicates that lowering in Phi values at points 5 to 10 is as given below:—

Lowering of Phi values with respect to prototype Phi values

Test	Piezometer Points					
	5	6	7	8	9	10
Length of u/s floor .. D_1	6	7.5	7.5	8	8	0
Length of u/s floor .. $2D_1$	8	9	10	10	10	0
Length of u/s floor .. $3D_1$	9	10	12	12	12	0
Length of u/s floor .. $4D_1$	10	13	13	14	14	0

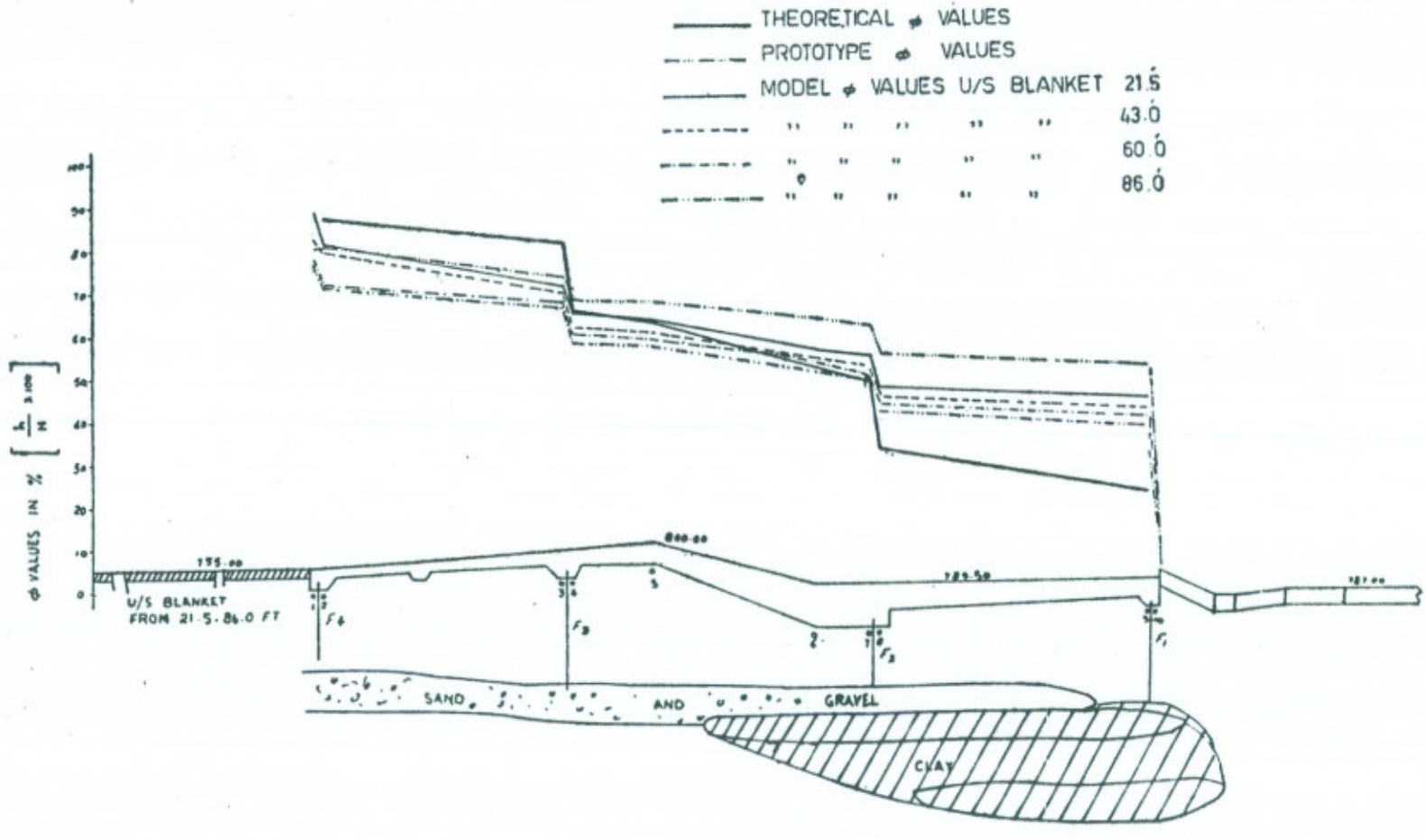

MARALA BARRAGE SEEPAGE STUDY
MODEL SCALE = 1: 80
TEST WITH DIFFERENT LENGTHS
OF U/S BLANKET.
THEORETICAL ϕ VALUES
PROTOTYPE ϕ VALUES
MODEL ϕ VALUES U/S BLANKET 21.5
" " " " " 43.0
" " " " " 60.0
" " " " " 86.0
ϕ VALUES IN % [h/H x 100]
U/S BLANKET
FROM 21.5-86.0 FT
SAND
AND
GRAVEL
CLAY

Fig. 9

Thus with u/s floor equal to depth of u/s sheet pile the reduction in Phi values on the d/s floor is of the order of 8% and further increase in length of u/s floor by D_1 results in lowering of pressures on d/s floor by 2%.

Any increase in the length of u/s floor results in the increase of overall length of the structure and means (if sheet pile depth is not changed) increase in ratio of α (the ratio of total length of structure and depth of sheet pile), and base ratio b_1/b of Fig. 1 and the effect of increase of α and base ratio on pressures is in opposite direction. The theoretical reduction of pressures for homogeneous media for profile of Fig. 2 by addition of 86′ u/s floor on points 8 & 9 is 6% and 4% respectively.

Contribution of u/s floor in reduction of Phi values as compared to sheet piles.

In order to determine the contibution of u/s floor equal to $4D_1$ in reduction of uplift pressures in case the u/s end sheet pile alone or along with intermediate sheet piles are not provided in the design, the following tests were done :—

(*a*) **No u/s end sheet pile (F4) without and with u/s floor.**

Phi values of the two tests are plotted in Fig. 10.

In the absence of sheet pile F_4 Phi values go up all along the structure (the rise of Phi values is more pronounced along base of u/s slope) and thus F_4 is responsible for deflecting flow lines away from base profile of structure and towards the bottom of clay layer under d/s floor and the contribution of u/s floor in lowering uplift pressures all along the structure in absence of F_4 gets doubled up as the flow lines get deviated away from base of structure at a higher point.

(*b*) **F_4, F_3 & F_2 sheet piles not provided, without and with u/s blanket.**

Phi values of the two tests are plotted in Fig. 11.

In the absence of sheet pile F_4 and u/s floor the intermediate sheet piles (F_3 & F_2) do not reflect on the Phi line but in the presence of u/s floor the intermediate sheet piles are effective in lowering pressures on d/s floor to the extent of 10%.

(*c*) **(a) and (b) repeated with homogeneous media.**

Phi values of the four tests are plotted in Fig. 12.

The uppermost sheet pile F_4 is effective in lowering the Phi value at point 8 by 2% and at point 9 by 4%. The intermediate sheet piles F_3 & F_4 have little effect in lowering pressures on d/s floor but build-up pressures under

MARALA BARRAGE SEEPAGE STUDY
MODEL SCALE = 1: 80 U/S END SHEET
PILE F_4 REMOVED TESTS WITHOUT
AND WITH BLANKET.

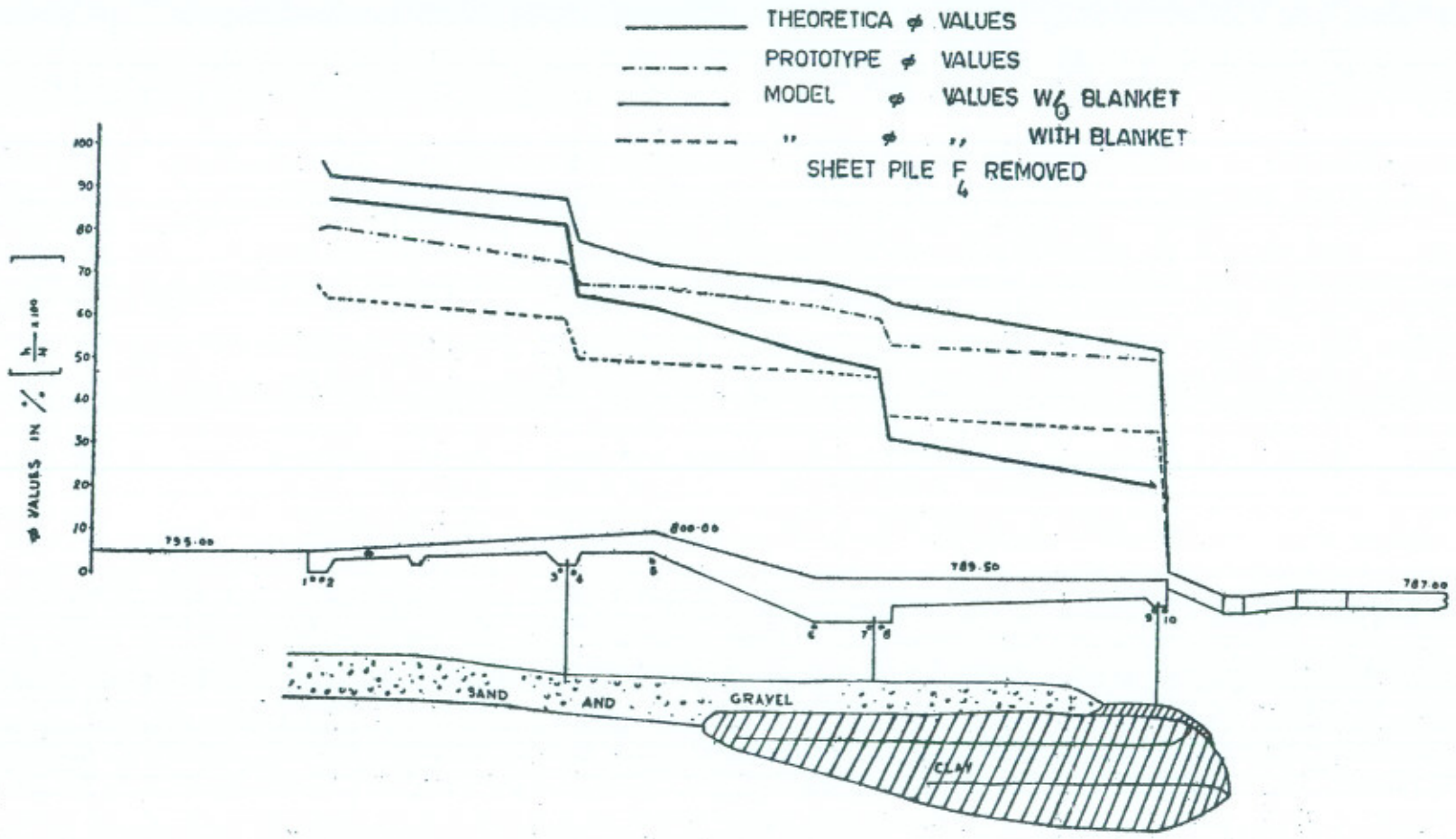


Fig. 10

MARALA BARRAGE
SEEPAGE STUDY
MODEL SCALE = 1: 80
F_2, F_3, & F_4 SHEET PILES
REMOVED TESTS WITHOUT
AND WITH U/S BLANKET.

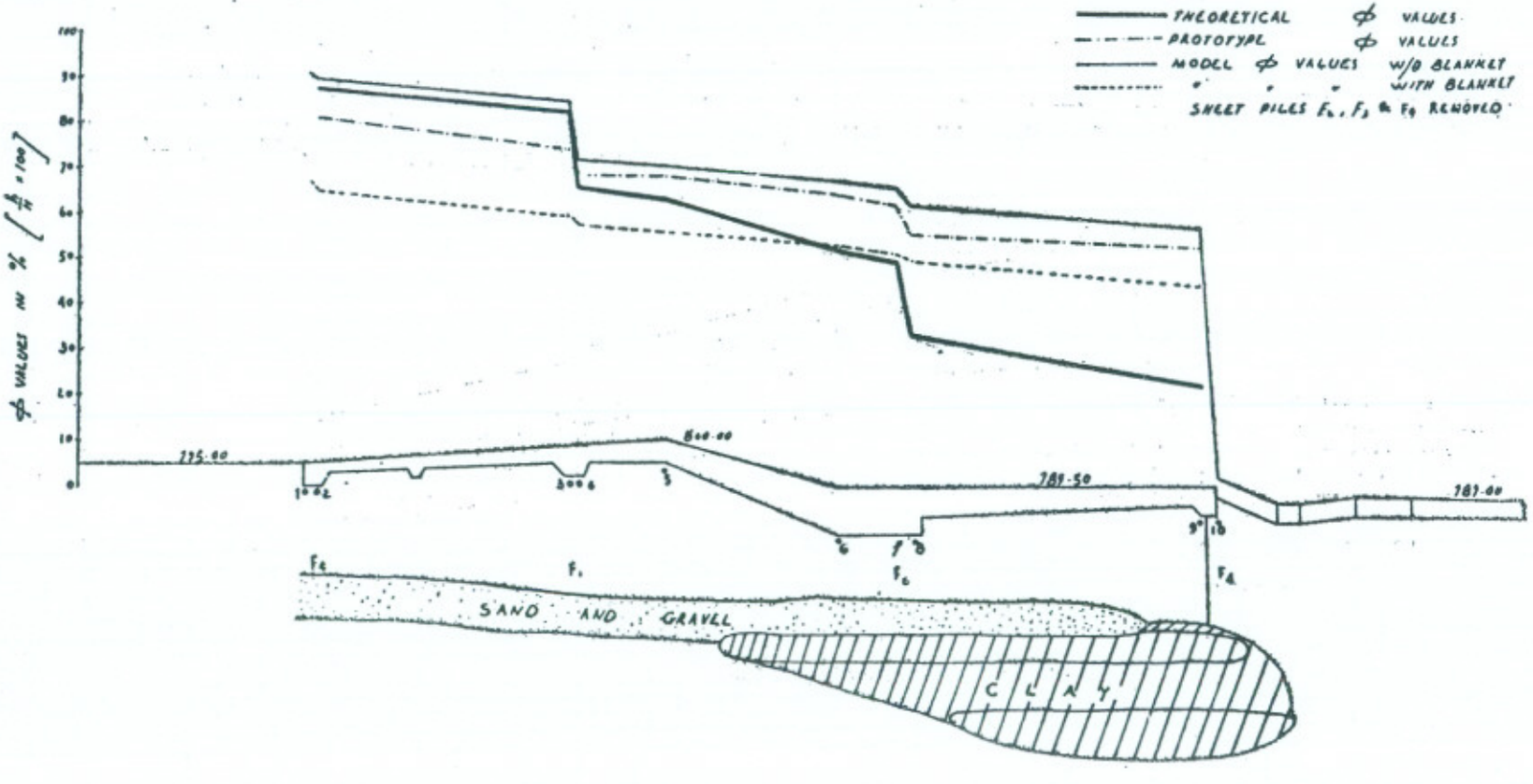


Fig. 11

MARALA BARRAGE
SEEPAGE STUDY
MODEL SCALE = 1: 80.

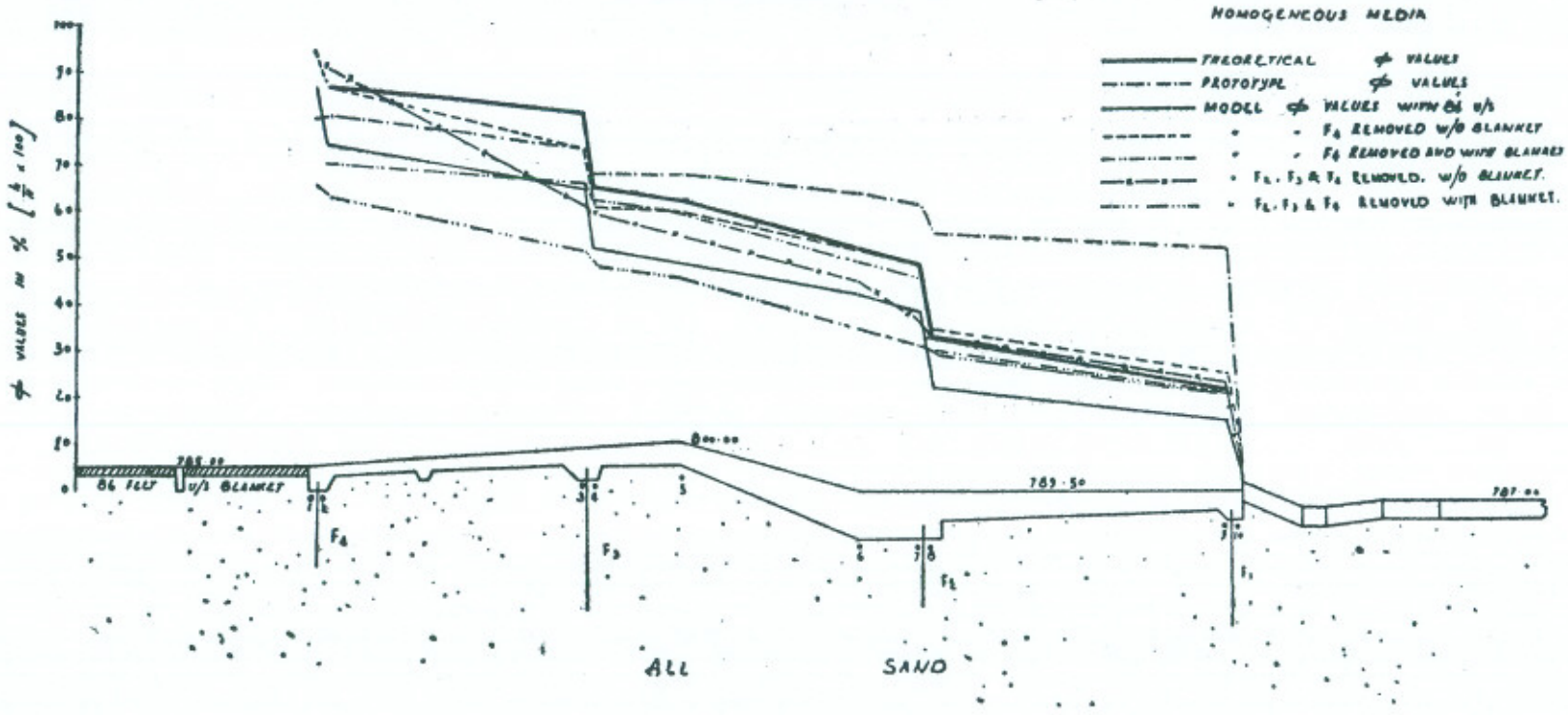


Fig. 12

the floor of work u/s of the sheet piles. Sheet pile F_2 builds up the uplift pressure on d/s glacis by 7.5 to 12 %.

The 86.0′ length of u/s floor is effective in lowering pressures in presence of all the four sheet piles by 14, 10, 10 and 7.0 per cent at points 5, 6, 8 and 9 respectively and the corresponding lowering in absence of sheet pile 2-4 is 9, 9, 4 and 3 and in absence of single sheet pile F_4 is 0, 3, 5 and 5 per cent, thus indicating that addition of u/s floor is more effective in the presence of sheet piles, especially u/s end sheet pile F_4.

II. STOPPAGE OF DRAINAGE THROUGH SEEPAGE FLOW SECTION IN BETWEEN TIP OF SHEET PILE F_2 AND CLAY LAYER

The drainage flow in the gap between sheet pile F_2 and clay layer was stopped on hydraulic model by assuming that this flow section is plugged on prototype by cement grout after drilling through the floor and Phi values were observed.

The Phi values of pressures at points along bottom profile of weir are compared with design and prototype observed values in Fig. 13. It can be

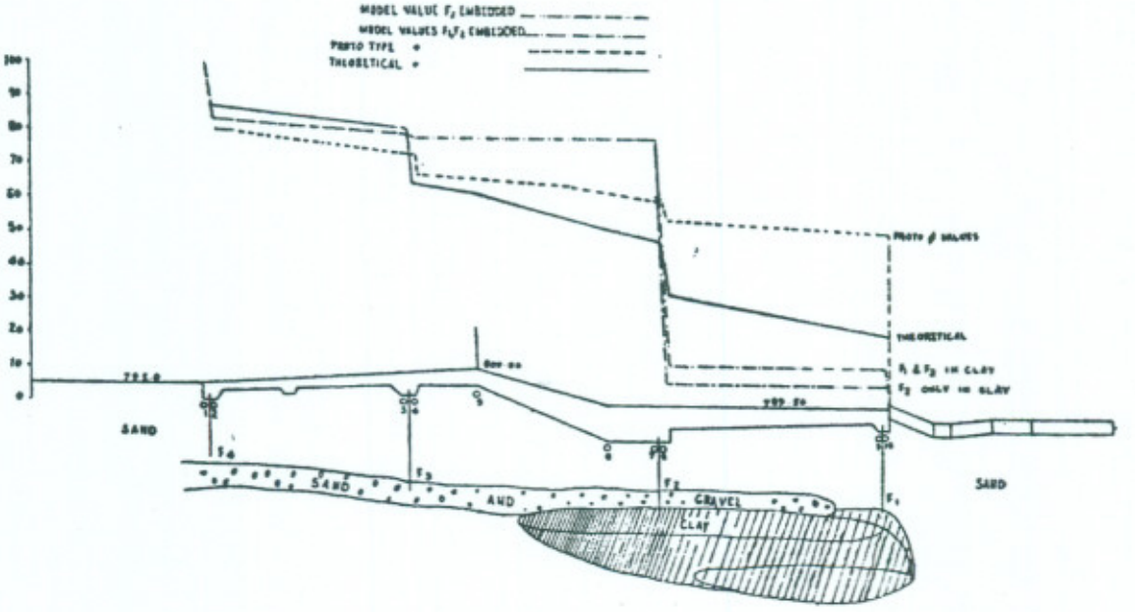


Fig. 13

concluded from this figure that the blockage of seepage flow in gap between tip of sheet pile F_2 and clay lense has resulted in—

(*a*) reducing Phi values on downstream floor to less than half of design values.

(*b*) built up of pressures under bottom profile of downstream glacis and crest to values much above the prototype and theoretical design values.

(*c*) and the phi-values at points 8 and 9 increase by back feeding of seepage flow from d/s side from 6% to 12% when F_1 is taken out of clay and gap width for seepage flow is 4.5′.

III. POSITIONING OF RELIEF WELLS FOR LOWERING OF PHI VALUES UNDER DOWNSTREAM FLOOR

Drainage-well screen of diameter equivalent to 8″ on prototype and depth equal to 20′ to 80′ on prototype was tried at the following positions:—

1. The depth of drainage well tried in block apron was 80′.
2. Just upstream of sheet pile F_1. Depth of well=20′.
3. Just downstream of sheet pile F_2. Depth of well=20′.
4. Just upstream of sheet pile F_2. Depth of well=20′.
5. The relief well in between two rows of deflector blocks was kept 25′ in depths as if prototype relief wells are 3″ in dia and 11′ and $2\frac{1}{2}$″ centre to centre and 20′.0 deep the equivalent depth of 8″ dia well on basis of area of flow entering relief well works to be about 25.1′.

The Phi values of ten pressure points for the 5 tests are compared with theoretical and prototype values in Fig. 14. The perusal of the figure leads to the following conclusions :—

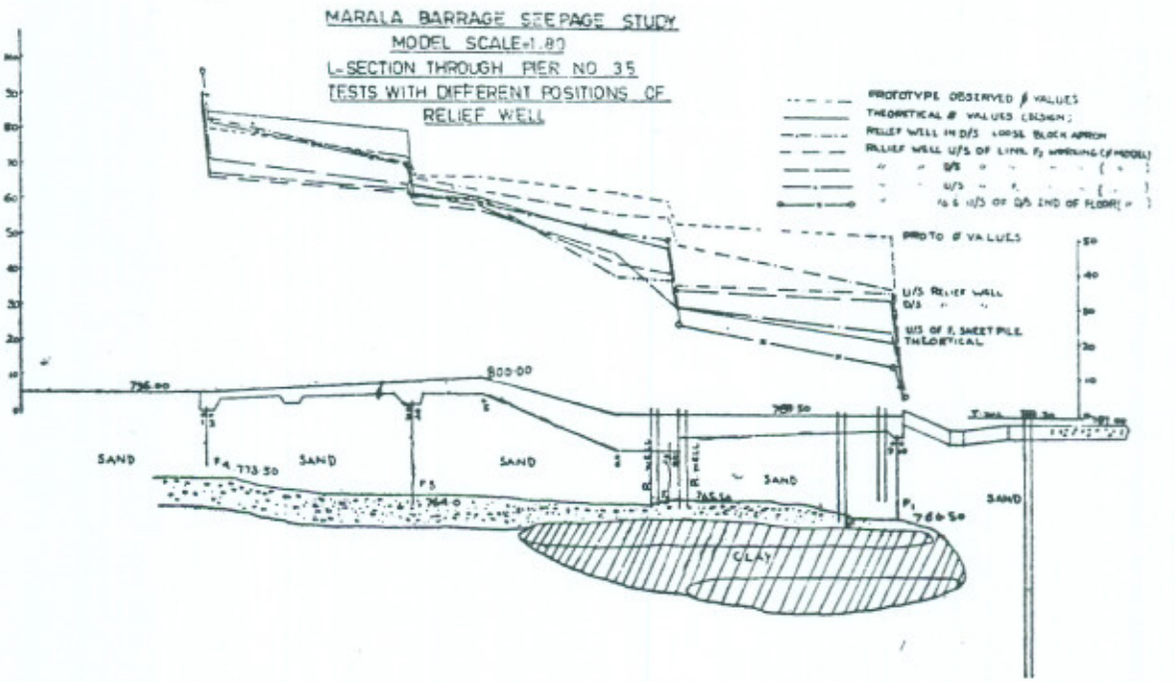


Fig. 14

1. As there is no restriction to drainage flow under the block apron and Phi values at point 10 are low so the relief well through block apron is not at all effective in reducing the pressures under the floor u/s of the sheet pile F_1. The clay layer under the concrete blocks and stone apron of left under-sluices of Rasul Barrage was blocking the escapage of seepage flow resulting in building of high pressures on the d/s floor and so 80′ deep relief well (8″ dia) was very effective in lowering the pressures to acceptable values, Fig. 15.
2. As the sheet pile F_1, is penetrating nearly impermeable clay, the drainage under the sheet pile F_1, is almost stopped and the flow lines curve around the clay layer and the pressure on d/s floor in between sheet piles F_1 and F_2 is the pressure at the bifurcation of seepage flowing on the two sides of clay lens minus same drop in pressure at sheet pile F_2. The relief well u/s of sheet pile F_1 is effective to reduce the Phi values under the d/s floor to almost design values.
3. The relief well nearer to the upstream face of sheet Pile No. F_2, is more effective in lowering Phi values upstream of sheet pile F_2, than the relief well near the downstream face of F_2, whereas in respect of pressures at downstream floor between sheet piles F_1 and F_2, the relief well immediately downstream of sheet pile F_2 is more effective than the relief well immediately upstream of sheet pile F_2.
4. The flow lines entering the relief well are obstructed by the sheet piles and maximum interference is when the relief well is located near to the sheet pile (as flow is out off from half circle) and so the lowering of Phi values at points 8 and 9 is maximum when the relief well is located in between two rows of deflector blocks as in this case the flow entering the relief well is from a wide angle.

IV. DRAINAGE THROUGH WEEP HOLES IN THE D/S END SHEET PILE

To lower the excessive uplift pressures on the D/s floor two holes with flow section equal to that of pipe of 8″ dia, were provided in the tail end sheet pile F_1 and the seepage flow drained into a 25′ long graded filter above R. L. 775 over laid by concrete blocks. The Phi values with and without u/s floor 86′.0 in length are plotted in Fig. 16 which shows :—

(*a*) The Phi values on the d/s floor get lowered by 20—25% and are only 4% above design Phi values. Further lowering of pressures

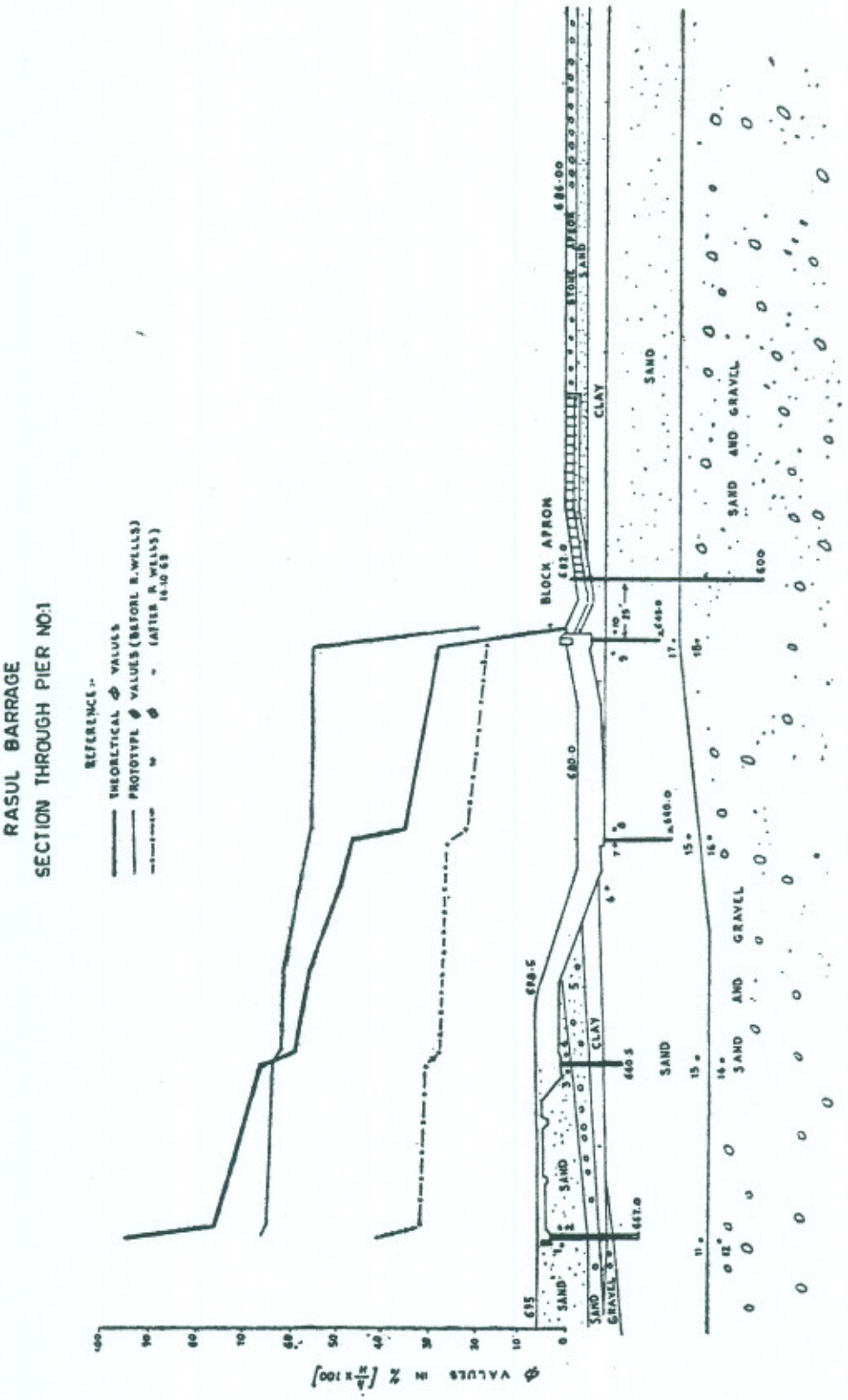

RASUL BARRAGE
SECTION THROUGH PIER NO:1
REFERENCE :-
THEORETICAL Φ VALUES
PROTOTYPE Φ VALUES (BEFORE R.WELLS)
" " Φ - (AFTER R.WELLS)
BLOCK APRON
CLAY
SAND
SAND AND GRAVEL
GRAVEL
Φ VALUES IN % [Φ/H x 100]

Fig. 15

MARALA BARRAGE SEEPAGE STUDY
MODEL SCALE = 1: 80 WEEP HOLE IN
THE D/S END SHEET PILE OPENING
INTO 25′ LONG GRADED FILTER
TESTS WITHOUT AND WITH BLANKET.

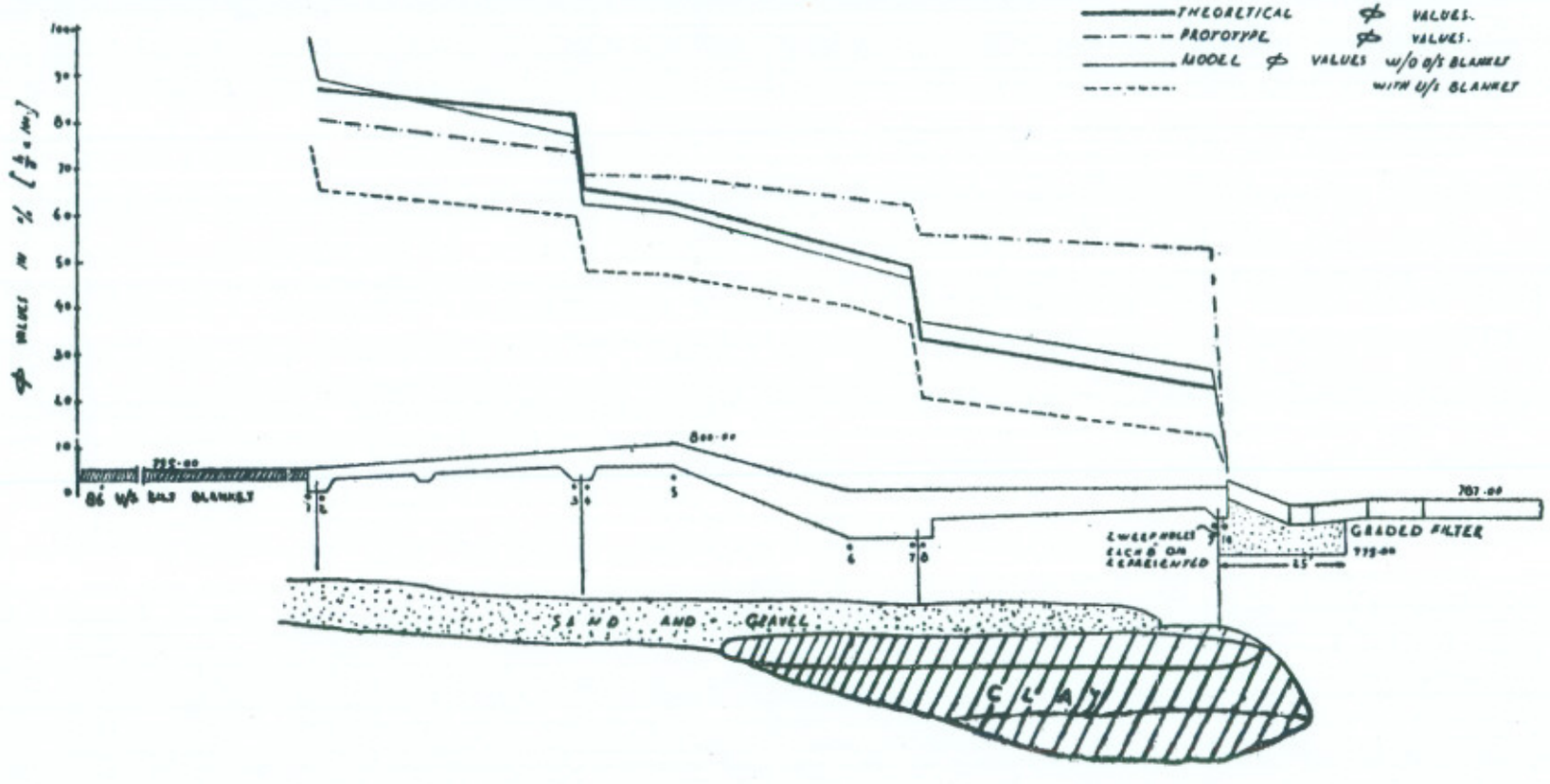


Fig. 16

on d/s floor is possible by providing one additional weep hole.

(*b*) The u/s floor is effective in further lowering of Phi values at points 8 and 9 by about 14%.

The solution could not be adopted as scheme involved enclosure Coffer Dam and subsequent dewatering of the working area and removal and replacing of large number of concrete blocks and appeared to be difficult and costly.

Conclusions

As a result of study of degree of effectiveness in lowering pressures on d/s floor by four alternatives tried on model it could be concluded that relief wells in between two rows of blocks at site of bore holes are more practical and effective provided carbonates and iron ions in seepage flow approaching the relief well are not excessive to cause incrustation.

EFFECT OF HIGH PRESSURES ON DOWNSTREAM FLOOR ON EXIT GRADIENT

By weighting down the downstream floor, the high pressures that have caused the trouble will still persist and so it is necessary to investigate whether the existence of such high pressures near the sheet pile F_1 gives rise to the acceptable exit gradient. Khosla requirement of exit gradient for Marala is 1/6.

To observe the exits gradient, new pressure observation points as shown in Fig. 17 (*a*), were added on the model. The Phi values for all the pressure points were observed for the following cases :—

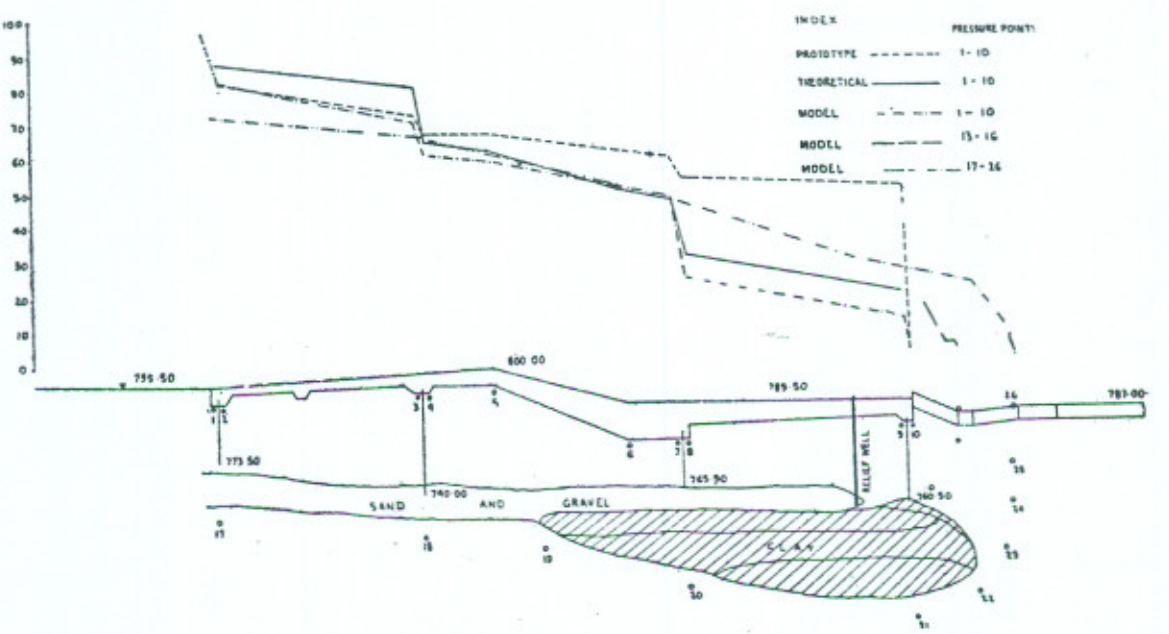


Fig. 17 (*a*)

Test A. Seepage flow section 4.5′ wide under sheet pile is plugged by clay. Phi value data is plotted in Fig. 17(*a*).

Test B. Same as 'A' but relief well located in between two rows of blocks and 25′ in depth is working. Phi value data is plotted in Fig. 17(*b*).

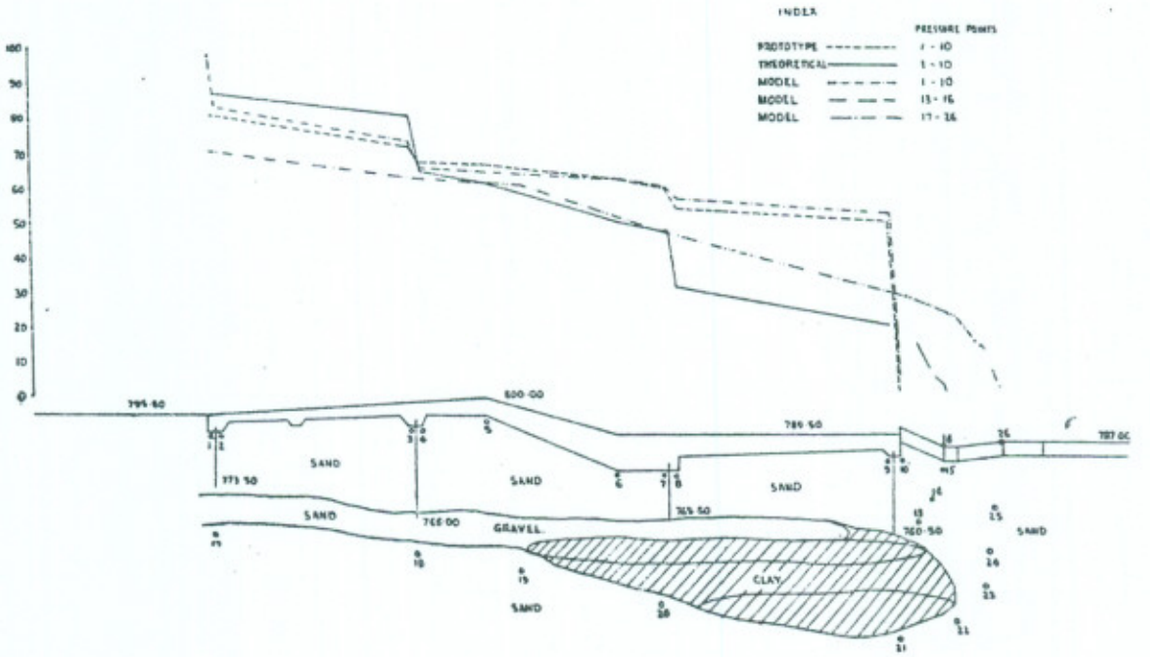


Fig. 17 (*b*)

Test C. It is same as test 'A' except that F_1 is out of clay and seepage gap is available for flow. Phi value data is plotted in Fig. 18(*a*).

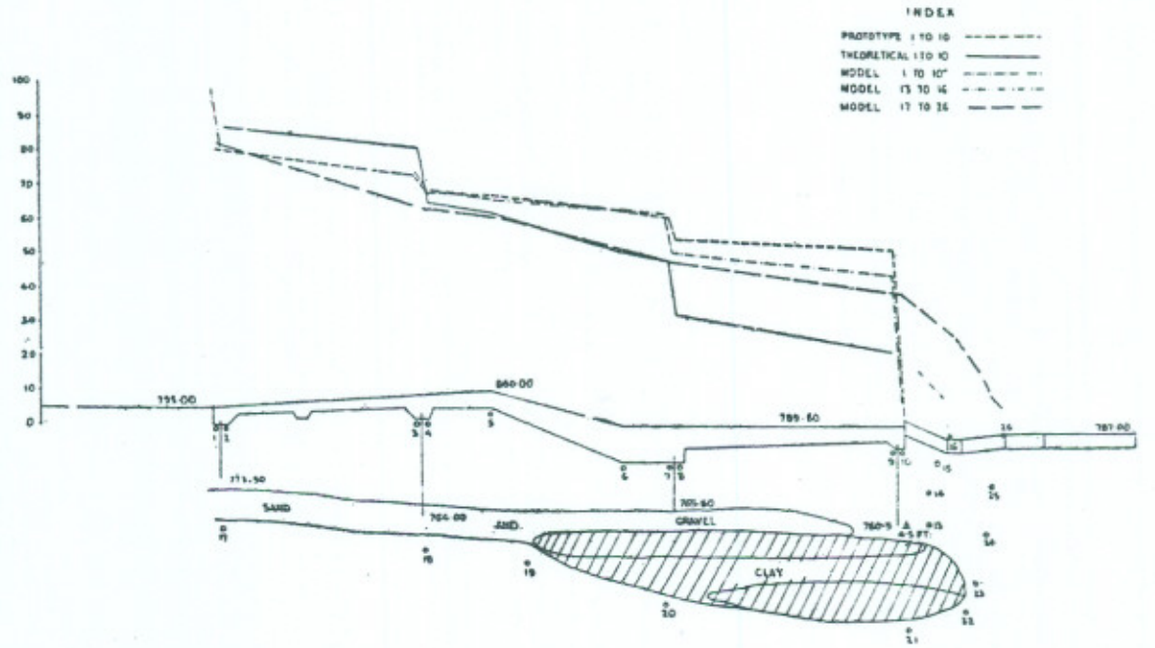


Fig. 18 (*a*)

Test D. Same as test 'C' but relief well in between two rows of deflector blocks. Phi value data is plotted in Fig. 18(*b*).

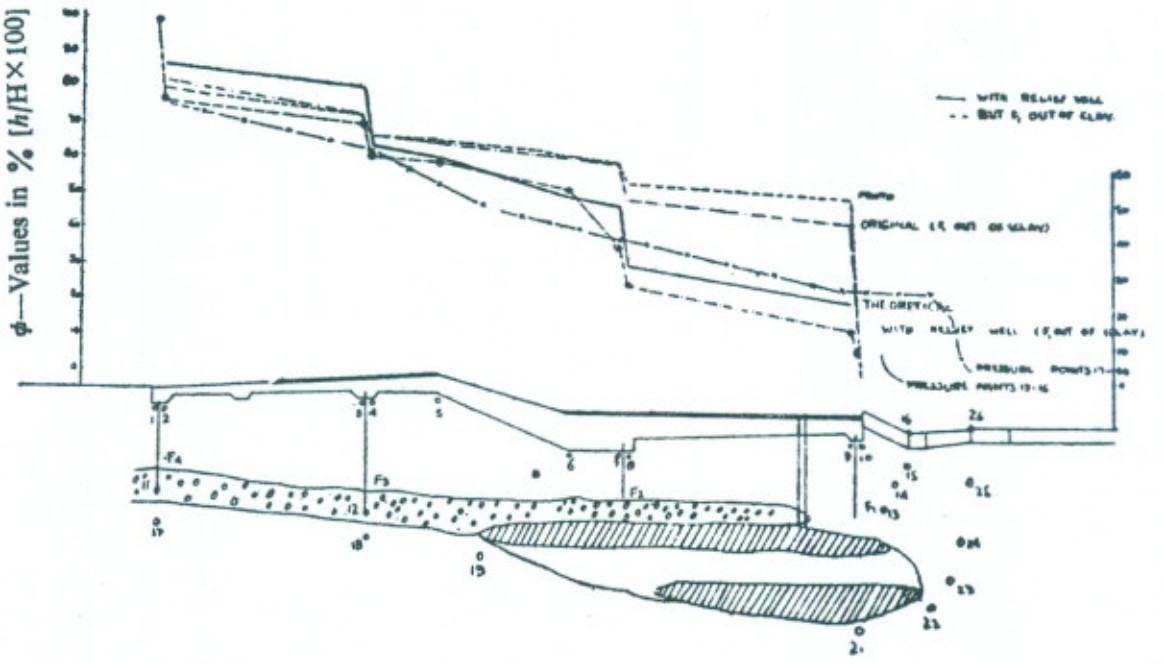


Fig. 18 (*b*)

The pressure exit gradient along the seepage path of flow through the gap in between toe of sheet pile F_1 and top surface of clay layer and the seepage path along bottom profile of clay lens is given in Table I.

It can be concluded from these tests that :—

(1) The Phi line around the bottom of clay layer points 17—26 is lower than the Phi line for points 1—10 under the profile of weir for tests A and C and *vice verse* for tests 'B' and 'D'.

(2) The exit gradient is function of drop in Phi value across the sheet pile F_1, residual Phi value d/s of sheet pile F_1 and effective depth of sheet pile. The exit gradient observed on model is flatter than the theoretical value of 1/8 for the four tests.

(3) The relief well does not reflect on the exit gradient if F_1 is penetrating the clay.

(4) Relief well is effective in lowering the exit gradient if sheet pile F_1 is not penetrating the clay layer and some seepage flow is taking place through the gap in between toe of sheet pile and clay layer.

TABLE—I

MARALA BARRAGE

SEEPAGE STUDY

Scale 1 : 80

S. No.	Test Conditions	Exit gradient of two consecutive points	
		Exit grade : points 15-16 of Fig. 17-18	Exit grad : points 25-26 of Fig. 17-18
A.	Seepage flow section in between tip of pile and clay layer choked with impervious clay ..	1/22	1/13.3
B.	As A and a relief well 25′ deep at 16.5′ u/s of d/s end of floor ..	1/27	1/13.3
C.	Seepage flow section between toe of sheet pile F_1 and clay layer fully open to flow ..	1/9	1/15.6
D.	As C but relief well 25′ deep in between two rows of blocks also in operation ..	1/14.3	1/27

EFFECT OF WEIGHTING DOWNSTREAM FLOOR ON THE PERFORMANCE OF HYDRAULIC JUMP AND CONSEQUENT SCOUR

Three main alternatives were tried on 1/36 natural scale model of bay 35, two piers and 35.0′ width of bays 34 and 36 in 4′.0 glass flume.

(*a*) **Thickening d/s floor by 4′ thick concrete layer:**

With design head across of 22.5′ the additional thickness of floor required to cater for Phi value of 60% at points 8 and 9 the unbalanced head of water to be counteracted is 5.6′ which is equivalent to 4′ thick concrete.

The weighting down of the downstream floor by raising the floor by 4.0′ affected adversely the discharging capacity of the bays under permissible retrogressed conditions by 20% and under accreded conditions by 15%. Moreover by raising the floor by 4′ the scour shifts nearer to the structure and scour increased at all discharges and the percentage increase in scour decreases with increase in discharge intensity.

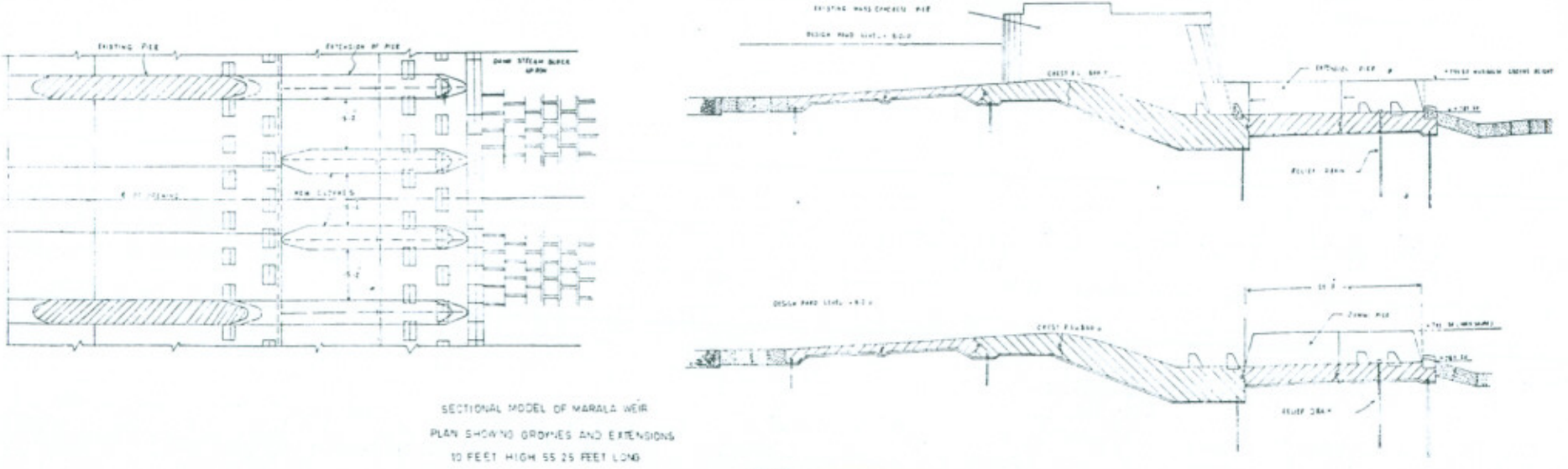
(*b*) **Providing single additional groyne from crest to tail end of d/s floor with top at cill level on the axis of the bay and extending the existing piers up to the d/s end of d/s floor:**

The decrease in design discharge intensity with one groyne per bay on axis of bay along with extension of existing piers for permissible retrogressed tail level is of the order of 15%.

It was noted that the portion of groyne over the glacis was affecting the flow pattern and performance of standing wave adversely as the flow obstructed by pier was deflected sideways and the area of turbulance in standing wave especially near the front of the jump decreased significantly with the result that the depth of scour doubled up. So in the subsequent tests the groynes were cut short at the u/s end and were located d/s of the impact rows of blocks.

(*c*) **Weighting d/s floor by depth of water on the downstream floor by raising the height of continuous lip to 4′.**

If it is assumed that the critical operating conditions are when the upstream water level is at R. L. 812 and downstream water level is at R. L. 792 (which is ever recorded minimum tail level downstream of new barrage and old weir in the period 1936—1962), when release downstream of Barrage is nil, the head across the Barrage will be 20.0 feet. The Phi value recorded by Irrigation Research Institute at Point No. 9 along Pier No. 35 is 51%. If factor of safety is assumed as 1.1, the assumed value of Phi for Point No. 9 works out to be 56%. The recorded value of Phi from recent bore holes adjusted for point



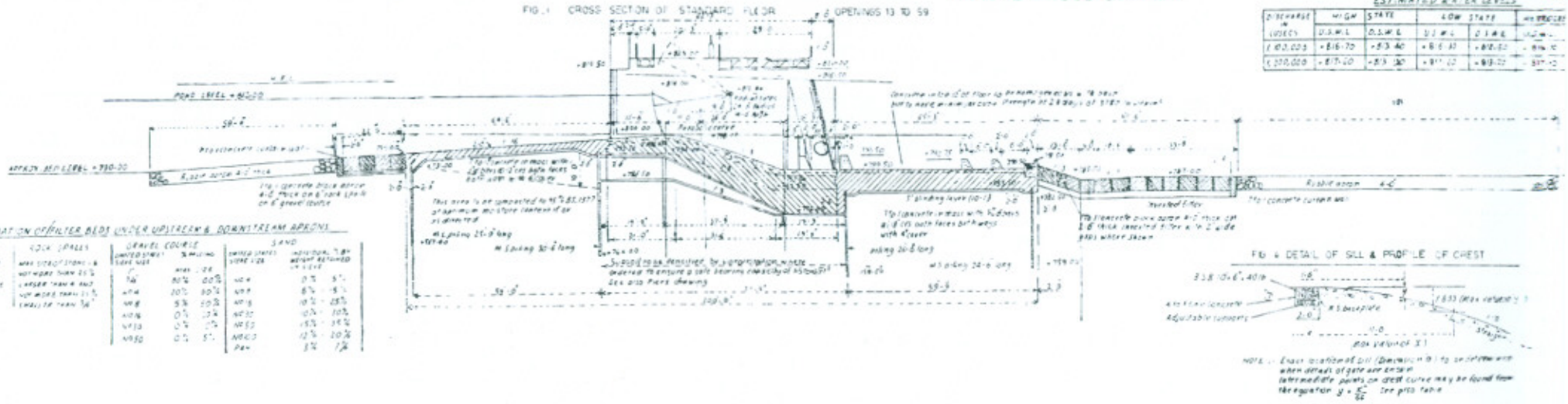
SECTIONAL MODEL OF MARALA WEIR
PLAN SHOWING GROYNES AND EXTENSIONS
10 FEET HIGH 55 25 FEET LONG

Fig. 19

PAKISTAN MARALA BARRAGE

FLOOR OF BARRAGE

TYPICAL CROSS SECTION



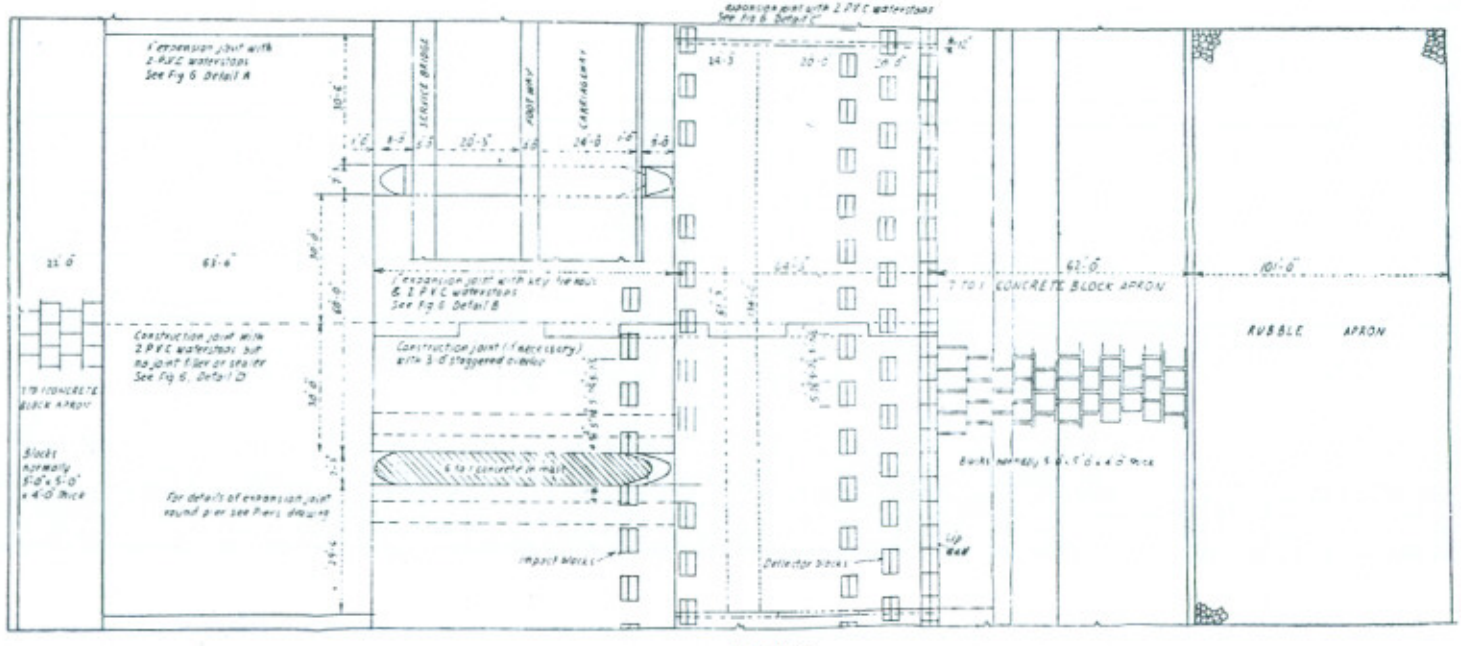


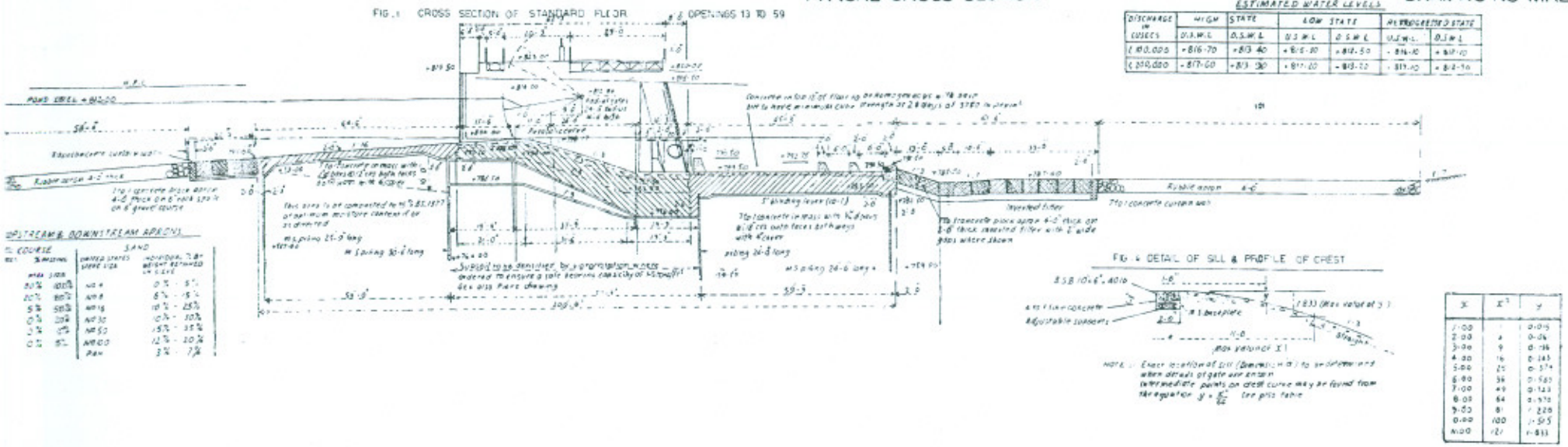
Fig. 19 (*a*)

MARALA BARRAGE

FLOOR OF BARRAGE

TYPICAL CROSS SECTION

DRAWING NO MRL NB 106



ESTIMATED WATER LEVELS

DISCHARGE IN CUSECS	HIGH	STATE	LOW STATE		REPRESENTED STATE	
	U.S.W.L.	D.S.W.L.	U.S.W.L.	D.S.W.L.	U.S.W.L.	D.S.W.L.
1,810,000	+816·70	+813·40	+815·30	+812·50	+814·10	+811·10
1,350,000	+817·00	+813·30	+817·00	+813·20	+817·00	+812·90

X	X^3	Y
1·00	1	0·019
2·00	8	0·076
3·00	9	0·136
4·00	16	0·243
5·00	25	0·379
6·00	36	0·583
7·00	49	0·833
8·00	64	1·370
9·00	81	1·220
10·00	100	1·515
11·00	121	1·833

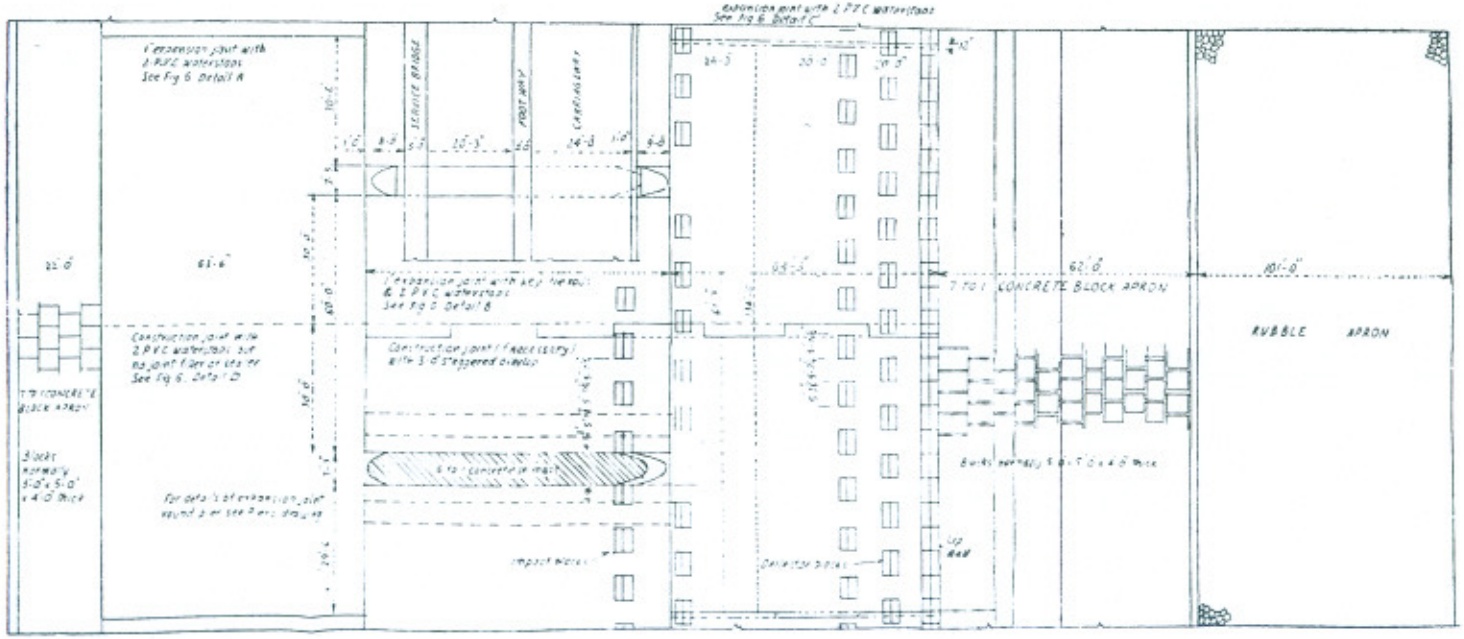


Fig. 19 (a)

No. 9 is 53.9%. Coode and Partners assumed factor of safety at 1.12 and thus assumed design value of Phi for point 9 to be 60%. and thus the uplift pressure at point No. 9 will be 12'.0 head of water. The thickness of downstream floor at joint 51'-3" from toe of downstream glacis is 5'-6" (equivalent to 7.84' head of water) over 3" binding layer (equivalent to say .25' head of water) and thus the additional depth of water required to counteract 12.0' head of water acting as uplift at point No. 9 is 4.0' which can be attained by raising 4.0 high baffle wall at end of downstream floor.

The weight of 4.0 baffle wall at end of downstream floor plus weight of deflector rows of blocks if spread over 39'-0 length of floor (downstream of joint) is equivalent to 1.5 head of water but it is assumed that weighting down effect of downstream baffle wall and deflector blocks does not extend beyond the joint 30' u/s of downstream end of impervious floor.

(*d*) **Weighting d/s floor by the groynes of different heights located 15'-2" apart beginning just d/s of downstream row of impact blocks and ending 4' short of lip wall.**

The consultants proposed 10 feet high groynes, 7.25 feet wide and 15'-2" apart. Every third groyne will be the extension of existing pier. The arrangement is shown in Fig. 19. The top of the groynes are 6 inches below the crest level. The upstream end of groynes is just downstream of 2nd row of impact blocks whereas downstream end of the groyne is curtailed 4 feet upstream of the lip wall. (This curtailment is to allow the flow to fan out at the end of downstream floor.)

The calculations indicated that if upstream water level is assumed at 814.0 (top of gates) and downstream water level at 789.5 and Phi at Point No. 9 at 60%, the height of groynes works out to be 9 feet. The corresponding height gets further reduced to 7.5 feet if upstream water level is assumed at R. L. 812, downstream water level at 789.5 and Phi value at 60%.

(*e*) **Combination of (b) and (c):**

In the presence of baffle wall the weight of depth of water on the downstream floor could be taken into account when calculating the height of groynes thus effecting some economy in the height of the groynes. The height of groyne can be reduced from 9' to 7.5' if the height of baffle wall is 3.0'.

The important tests relating to alternatives (*b*), (*c*) and (*d*) are discussed here briefly.

The actual dimensions of concrete apron, block apron, stone apron represented on model were in accordance with drawing No. MRL-106/2, Fig. 19(*a*).

The tests were with maximum q-300 cusecs, run for 8 hours (equivalent to 2 days on the prototype) and the maximum scours are recorded in table

below. The bed contours at the end of usual tests as recorded photographically or as surveyed are given in Fig. 20 (a, b, c & d). The percentage increase in scour with respect to base test is also given in table :—

Test	Test details		Max-scour below d/s floor level.	% increase in scour.
1.	Base Test	..	14.0	..
2.	Groynes 10′ in height (Fig. 19).	..	21.0	50.0
3.	Groynes 8′ in height, and 3′ high baffle wall.	..	17.5	25.0
4.	4′ high baffle wall	..	15.7	12.14

There is every likelihood of prototype scour being double of scour indicated by model. The existing d/s stone apron is designed to deal with scour to a depth of 38.0′ below apron level and so, for test 2 replenishment of stone may be called for after each major flood. Thus it is obvious that the design scour of 38.0′ is likely to be exceeded for 10.0′ high groynes.

The tail water levels for specific discharges for baffle wall of height 2.5, 3.0, 3.5, 4 and 4.5 feet at which surface disturbance over-baffle wall just disappears are plotted in Fig. 21. The corresponding tail water and discharge per foot run curve for keeping the front of hydraulic jump just at the toe of d/s glacis is superimposed. The minimum retrogressed curve as estimated by Coode & Partners (Barrage Consultants) is also plotted in Fig. 21 which shows that baffle wall of height of 4.0 feet is acceptable as its weir effect will be nill for assumed minimum retrogressed level for a specific discharge right up to 250 cusecs per foot run.

Test No. 3 with groyne height 8′ on alignment as shown in Fig. 19 plus 3.0′ high baffle wall was most promising and was further pursued by tapering the groyne at its d/s end as shown in Fig. 22.

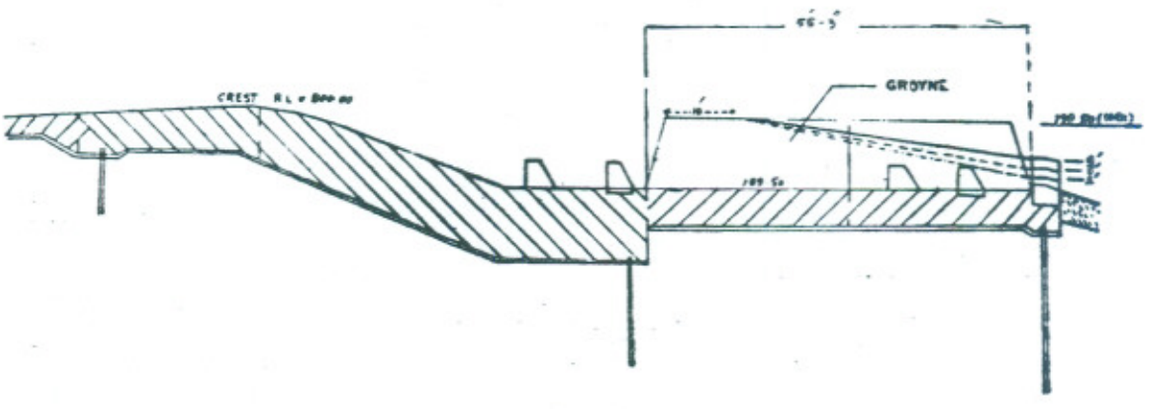


Fig. 22

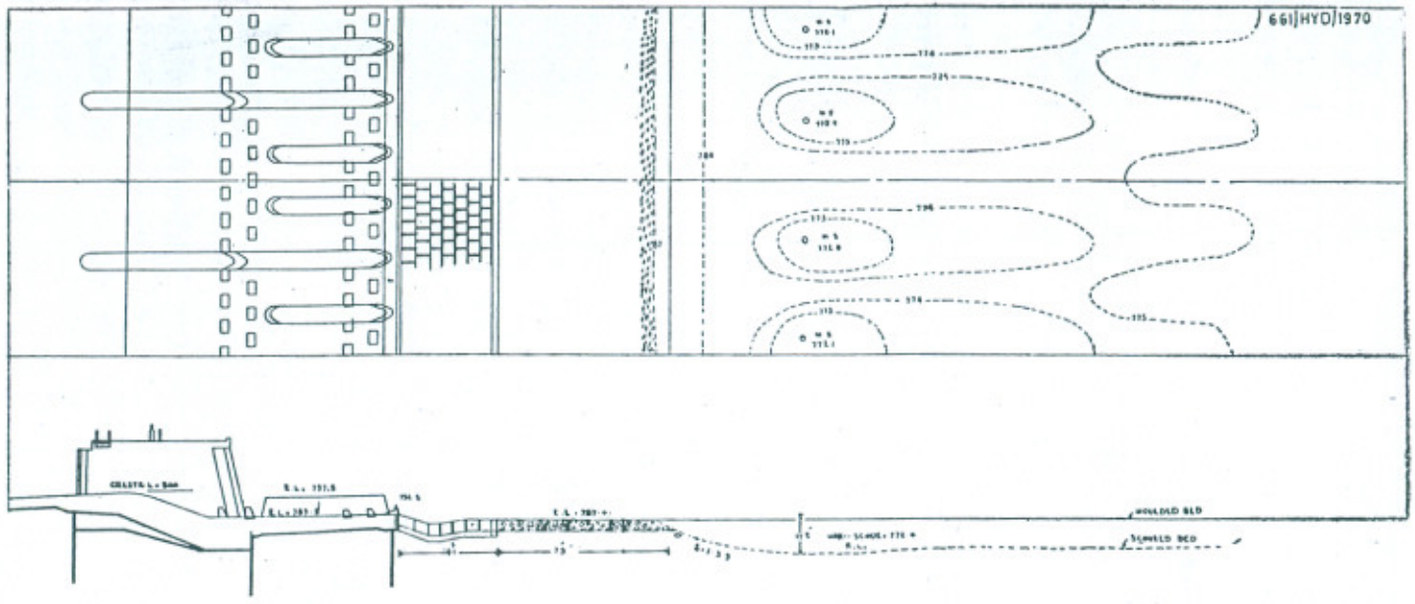


Fig. 20 (*c*)

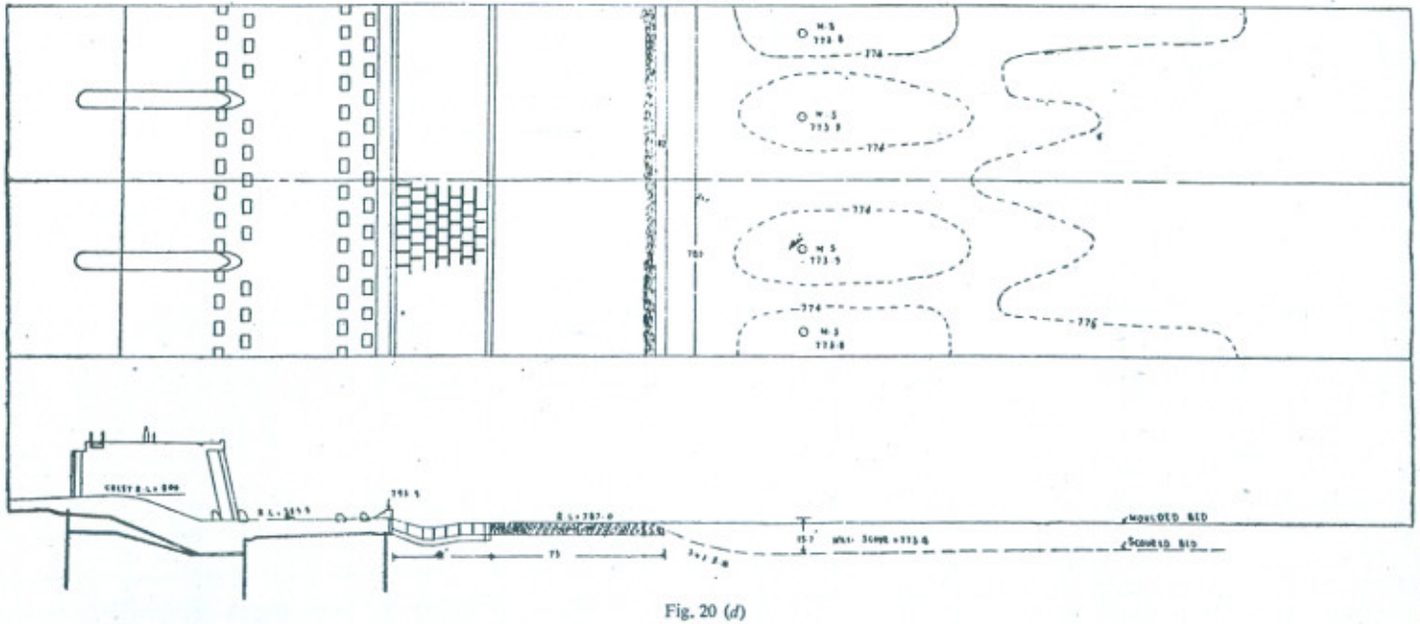


Fig. 20 (*d*)

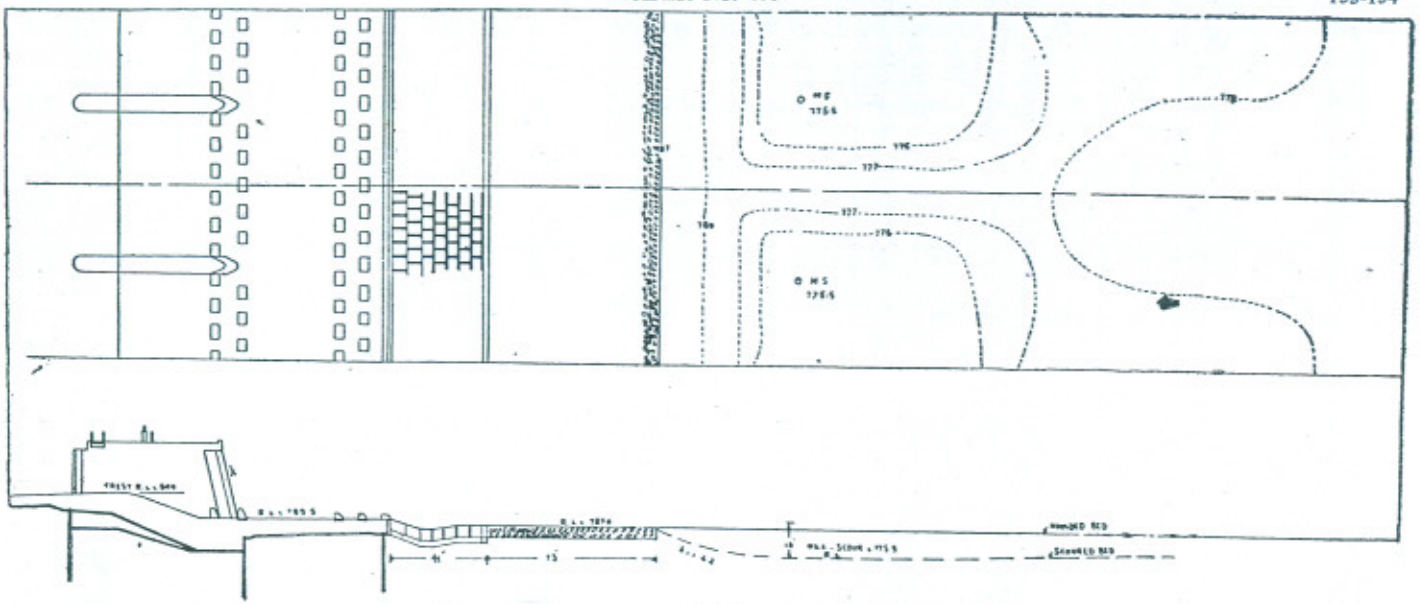


Fig. 20 (*a*)

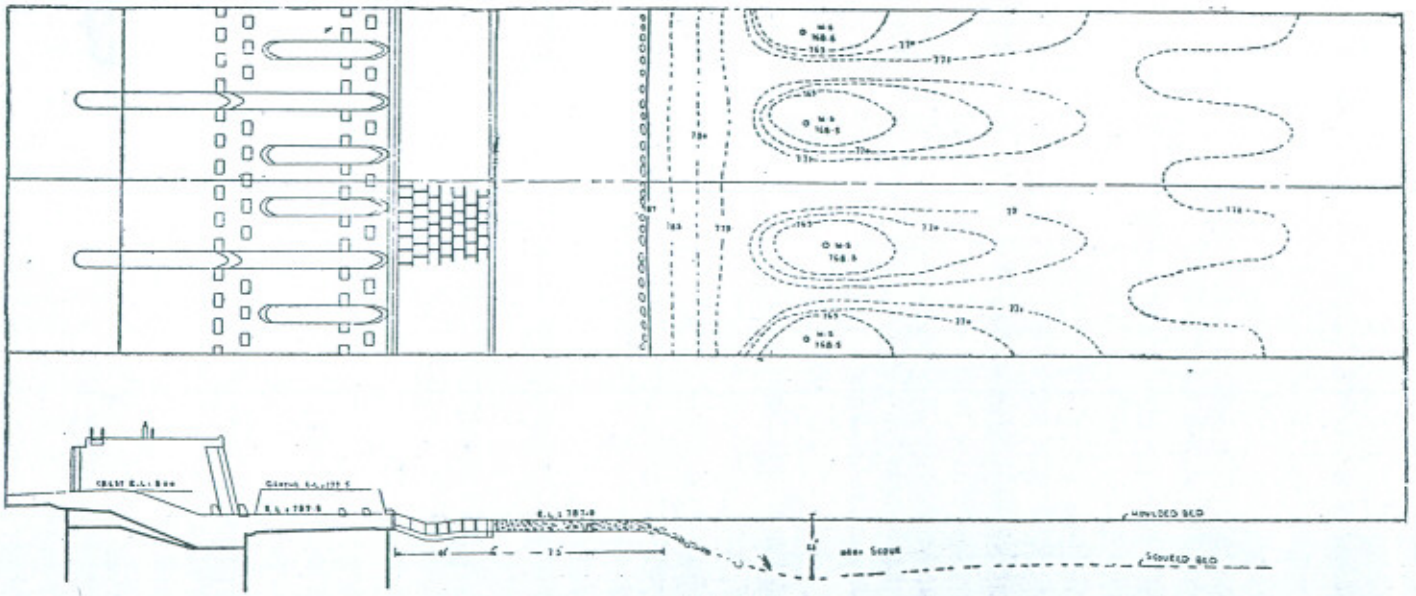
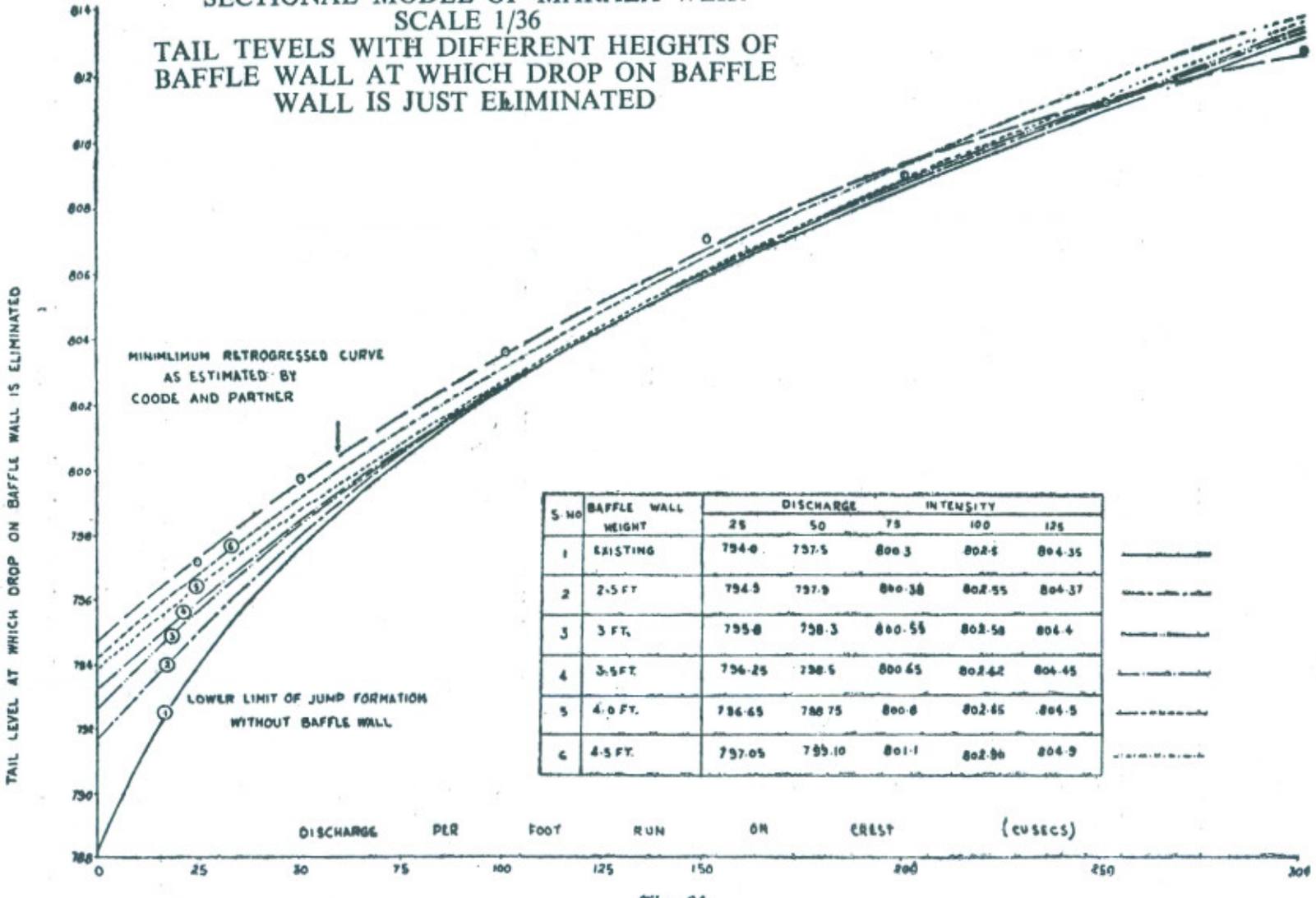


Fig. 20 (*b*)



S. NO	BAFFLE WALL HEIGHT	DISCHARGE / INTENSITY 25	50	75	100	125
1	EXISTING	794·0	797·5	800·3	802·5	804·35
2	2·5 FT	794·5	797·9	800·38	802·55	804·37
3	3 FT.	795·0	798·3	800·55	802·58	804·4
4	3·5 FT.	796·25	798·5	800·65	802·62	804·45
5	4·0 FT.	796·65	798·75	800·8	802·65	804·5
6	4·5 FT.	797·05	799·10	801·1	802·96	804·9

Fig. 21

The bed contours at the end of usual test with maximum Q=300 cs. run for 8 hours on the model were recorded. The maximum scour observed on the model and the percentage increase with respect to base test (Test No. 1) are given in the table below :—

Test No.	Description	Maximum scour.	%age increase in scour.
4.	Groynes 8′ high & 3′ high baffle wall	17.5	25%
5.	Same as 4, but length of groyne at the level top was cut short to 30′ and tapered down to baffle wall in a slope of 1 : 4.	16.25	18%
6.	Same as 4, but length of groyne at .. the level top was cut short to 20′ and tapered down to baffle wall in a slope of 1 : 6.	16.10	15%
7.	Same as 4, but length of groyne at .. the level top was cut short to 10′ and tapered down to baffle wall in a slope of 1 : 8.	15.4	10%
8.	Same as 4, but u/s top end of .. groyne (acting as a sharp crested weir) tapered down at a slope of 1 : 10.	15.0	7.12%

The percentage increase in scour with 4 feet high baffle wall alone at the end of d/s floor is about 12% as compared to corresponding increase of 10% with 3.0 high baffle wall at end of d/s floor along with 8′.0 high groyne 10′.0 long at top, tapering down at a slope of 1.8 to end of baffle wall.

Test No. 7 above was repeated with 2′ & 4′ high baffle wall at the end of d/s floor as shown in Fig. 23. The scour results are listed below:—

Test No.	Description	Maximum scour	%age increase in scour
9.	Test No. 4 above but the baffle .. wall height reduced to 2′.	15.75	12.5%
10.	Test No. 4 above, but the baffle .. wall height increased to 4′.	16.1	15%

It is evident that the optimum height of baffle wall in presence of 8′ high groynes 10′ long at level top and tapering down to baffle wall at a slope of 1.8 is 3′.0 from scour stand-point.

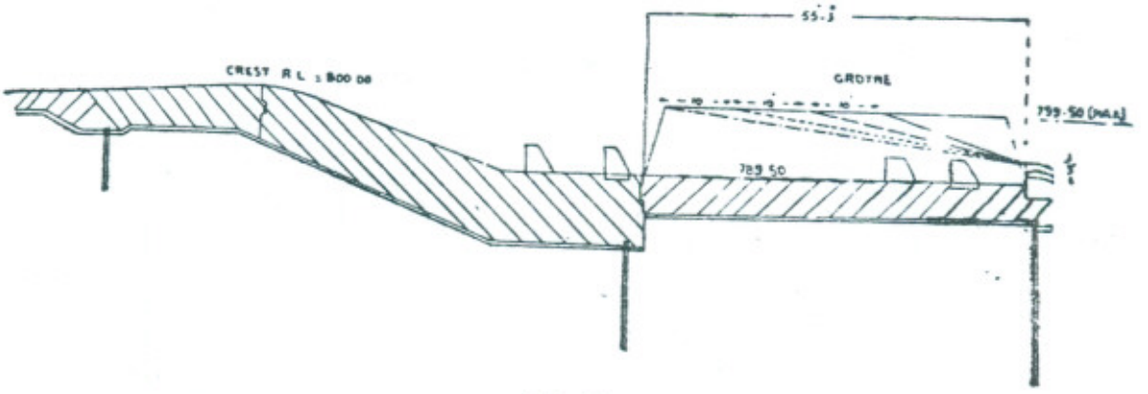


Fig. 23

The final recommendation as a result of model test was 8′ high groyne 10′ at level top and tapering down at a slope of 1:8 to end at baffle wall 3.0′ in height, and increase in length of stone apron proportional to percentage increase in scour.

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