wide, 46.5′ high, 4′ thick. There are 17 sets of wheels on each gate, each wheel being 31 inches in dia and 14 inches wide. The gate has got 18 No. horizontal beams. The dead weight of gate is 370 Kips. The seals are on the upstream face of the gate. The side and top seals have a thin stainless steel sheath on sealing surface.

There will be always contact between the side seals and seats surface when the gate is fully closed or open. A gap is created along top seal when the gate is opened more than a few inches. The geometry of service gate-shaft above the top of gate passage way is shown in Fig. 23b.

The model included portion of Tunnel 4, 480 feet of U/S tunnel, 700 feet of D/S tunnel, transitions to the gate passage way, gate structures, and the gate passage way. Piezometers were placed at several key locations on the transition, pier walls, the U/S face of gate structure the U/S and D/S walls of gate shaft, the two faces of the horizontal members of the gate structure.

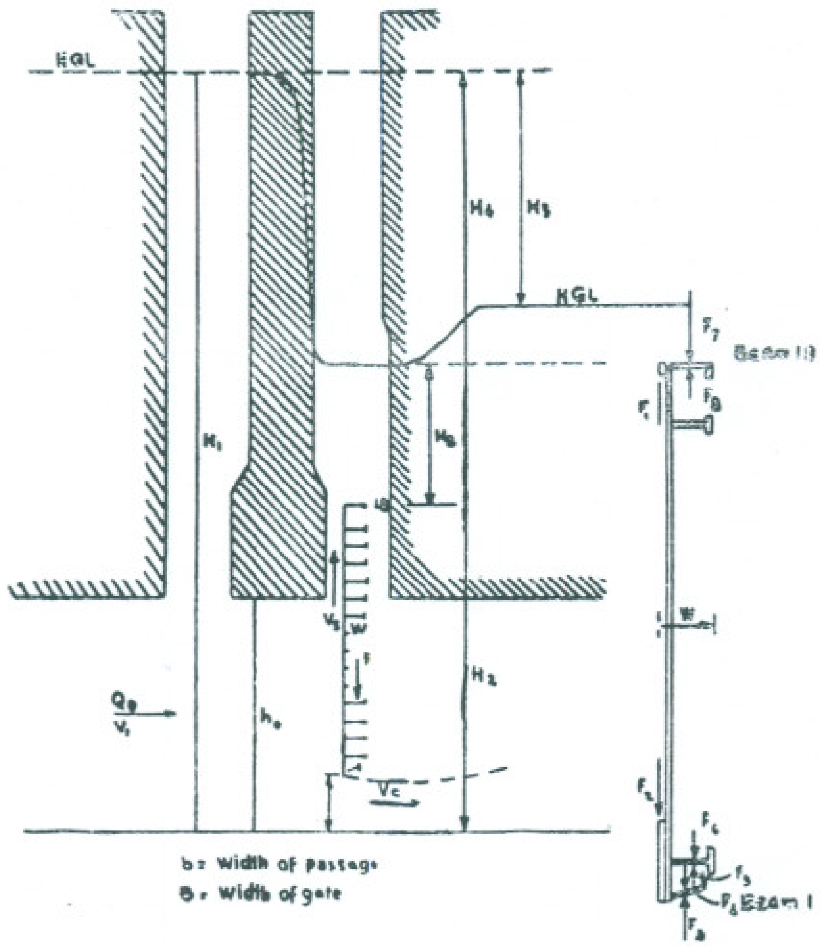
Two identical models of the service gates. were constructed from brass plate with thickness of brass plate identical to the model thickness of the members of the gate structure with exceptions of side and face plates of gate which were thicker to provide structural strength to the model. The wheels of the gate were also chosen from the commercially available ball bearings with sizes slightly smaller in dimensions than the true scale dimensions. Hard moulded rubber was glued to the face plate to form the seals. A plate was added to the bottom end of the gate to act as a hold back plate for bottom seal. One gate was equipped with piezometers to measure pressures on the various horizontal members of the gate and the other was equipped with the flush mount pressure transducers. Gate position indicaters and force indicators were calibrated. The seals of the model gates were not in contact with the seal surface to eliminate additional unknown amounts of friction forces.

C-6.2 Dimensionless characterizations :—

The geometric hydraulic and hydrodynamic variables are identified in the difinition sketch. Fig. 24. The velocity head

Fig. 24

TARBELA DAM PROJECT
HYDRAULIC MODEL STUDIES
DEFINITION SKETCH



$\frac{Vc^2}{2g}$ an important flow parameter was used as non-dimensioning parameter. The critical velocity Vc was at the vane contracta of the jet flowing beneath the gate. The contraction co-efficients for submerged and free jets were determined on the model 1/69.6 scale (described earlier with the modification that it had a new gate structure which included total length of the shaft to ground level). The contraction coefficient for both free and submerged conditions is plotted in Fig. 25.

Another non-dimensionalling parameter used in analysis is H_4 (difference in levels in the bulkhead and service gate wells which in turn is dependent on Vc and gate position). The total hydrodynamic downpull F was conveniently split into eight components.

(1) F upward force on the top seal clamp bar.

The downward force on the top seal clamp bar is included as part of the downward force on the top of beam No. 18. The pressure head acting on the clamp bar was measured at peizometer No. 3-32 at the corner formed by the clamp bar with U/S face plate of the gate. The pressure head H_{3-32} is a function of H_1 and H_4 and so the pressure coefficient C_{ts} was defined as

$C_{ts} = \frac{H_1 - H_{32}}{Hy}$, where H_{32} is the pressure head measured at point 3—32.

C_{ts} is plotted against %age gate opening in Fig. 26 which shows a discontinuity at 40% gate opening as at this gate opening (17 feet) the geometry of shaft wall changes affecting the pressure at the top seal quite significantly. At 75% gate opening H_{32} equals H_2 and C_{ts} equals unity.

(2) F_2 Forces on the bottom seal plate :—

The force F_2 is the downward force acting on the top edge of the plate (3′ above the bottom seal) backing the bottom seal. The pressures were observed at piezo 3-30 at the corner of U/S face of gate with the backing plate for bottom seal.

Fig. 25
TARBELA DAM PROJECT
HYDRAULIC MODEL STUDIES
CONTRACTION COEFFICIENTS FOR SERVICE GATE

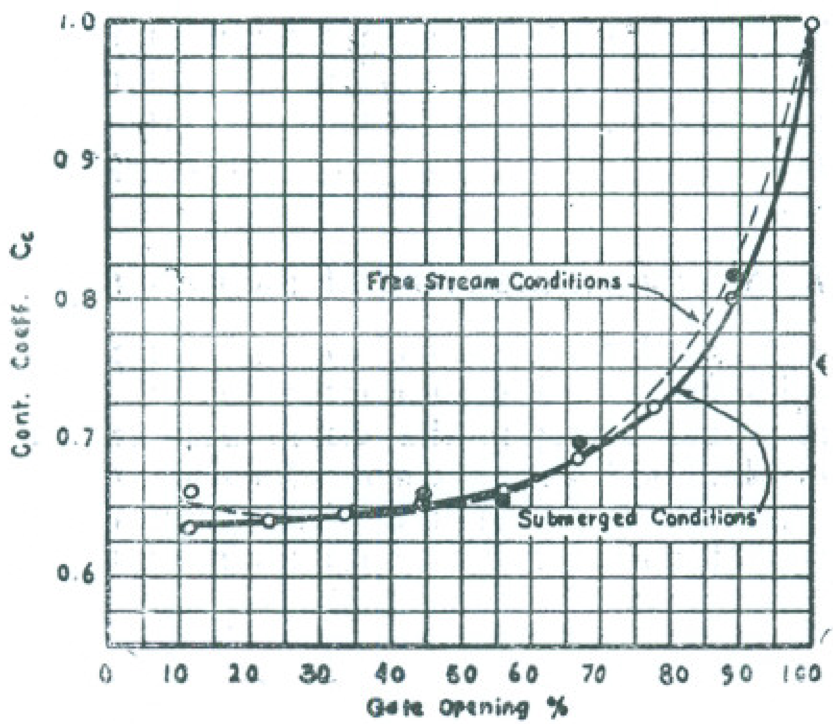
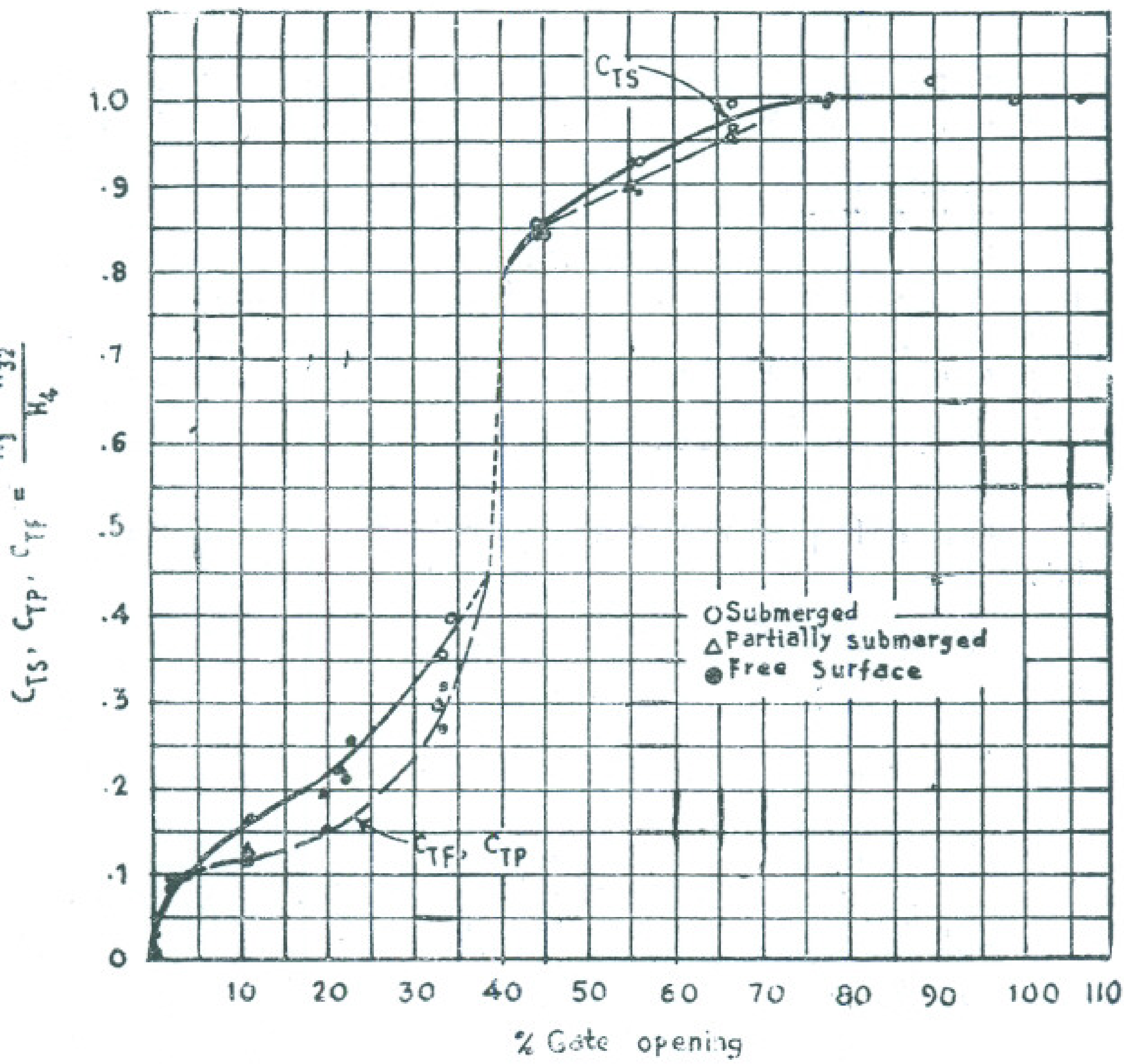


Fig. 26

TARBELA DAM PROJECT

Hydraulic Model Studies

Variation of Pressure Heads on top Seal Clamp Bar



The pressure at point 3-30 is strongly influenced by the flow pattern near the bottom of gate and so $V_c^2/2g$ is used as non-dimensionalizing factor. Moreover the pressure head H_1 is the ambient pressure head the pressure coefficient is written as :—

$$C_{bs} = 2g \times \frac{(H_1 - H30)}{V_c^2}$$

If the top of plate is above the top of gate passage the discontinuity is bound to occur at 92% gate opening Fig 27.

<u>Force F_3,F_4</u>

F_3 is the upward force on the bottom beam including the upward force on bottom seal. The pressure coefficient for the pressure head on bottom of beam is defined as it is in the region of same field as F_2

$$C_{bis} = \frac{2\,g\,(H_1 - Hb_1)}{V_c\,^2}$$

Where H_{b1} is the pressure recorded at the bottom of beam I.

Similar force F_4 is the downward force on the top of beam I, the coefficient is defined as

$$C_{is} = \frac{2g\,(H_1 - H_{t1})}{V_c^2} \text{ See Fig. 28}$$

where H_{t1} is the recorded pressure head on top of the beam I. The pressures on the bottom of beam I are significantly less than the acting pressure at the top of beam.

<u>Force F_5 and F_6.</u>

The force F_5 and F_6 represent the unbalanced forces in the vertical direction acting on bottom and top of beam II. It was found on the model that the unbalanced forces on beams 2-17 were not significant.

<u>Forces F_7 and F_8.</u>

Variations in pressure heads occur between the top and bottom sides of beam 18. The forces F_7 and F_8 are directly

Fig. 27

TARBELA DAM PROJECT

Hydraulic Model Studies

Variation of Pressure Heads on the Bottom Seal Plate

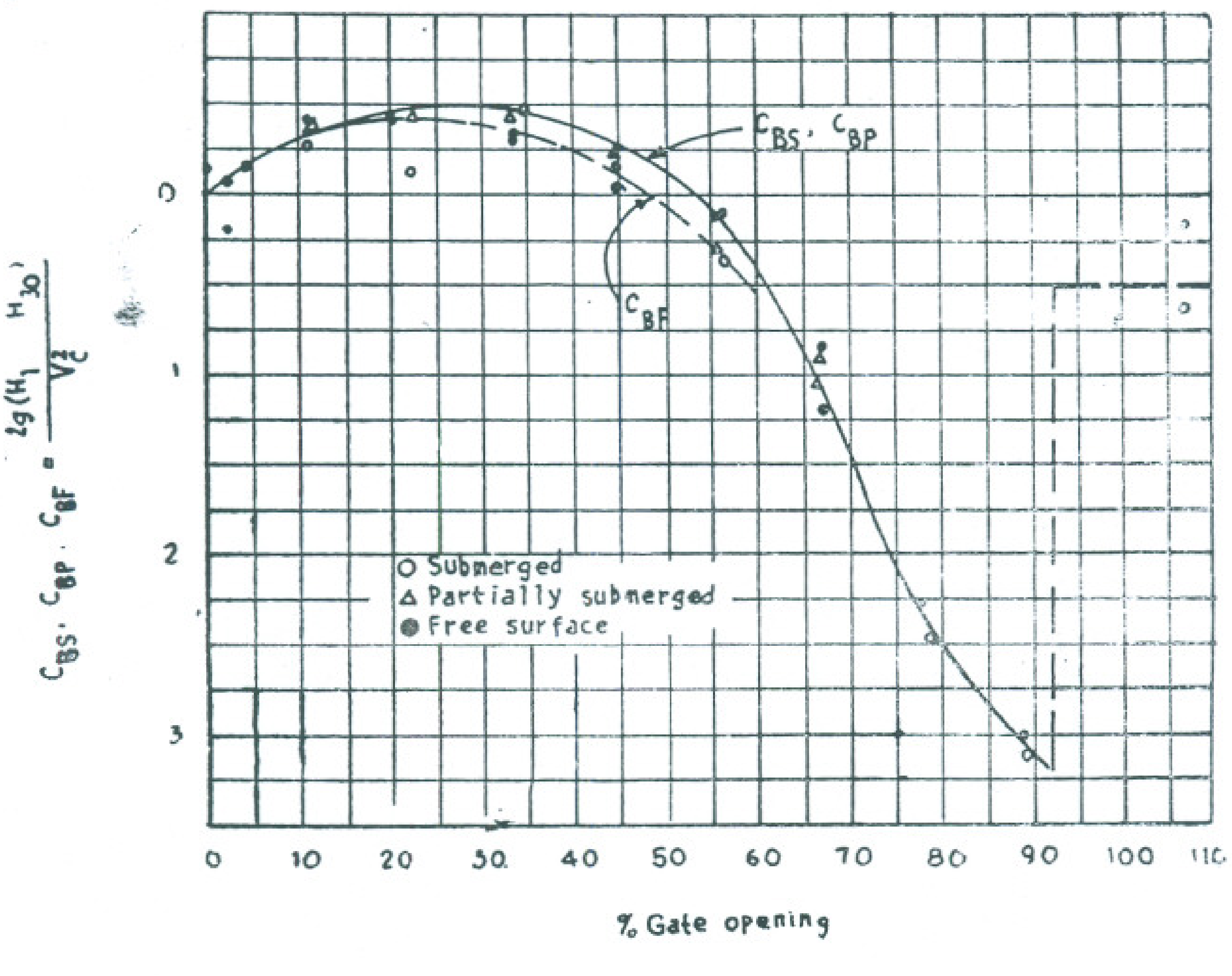
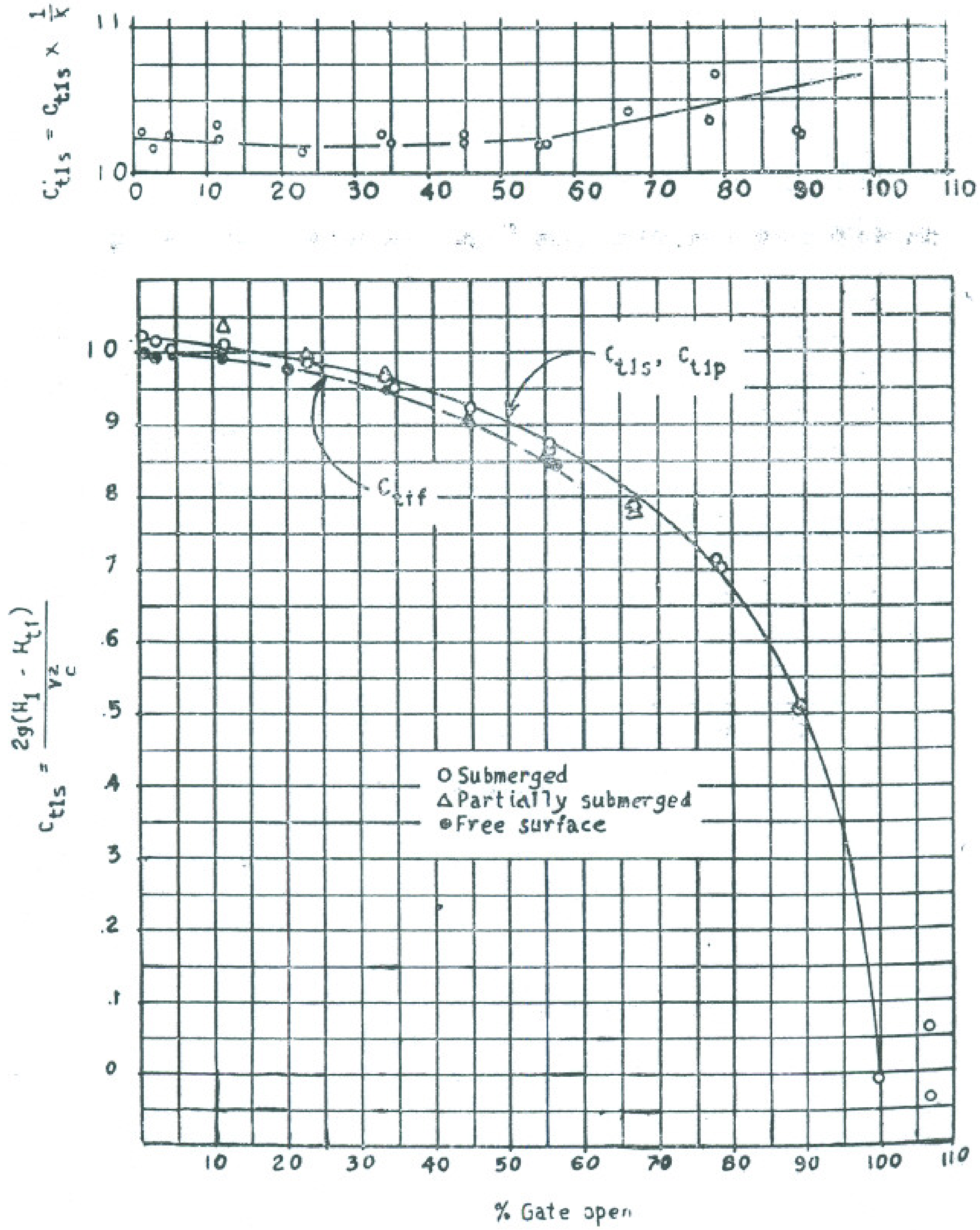


Fig. 28

TARBELA DAM PROJECT

HYDRAULIC MODEL STUDIES

Pressure Head Coefficient for top side of Beam I



related to the head in the service gate well above the level of the beam. Although the significant difference from hydrostatic pressure distribution may not be expected, the magnitude F_7 and F_8 differ simply because areas differ.

The pressure coefficient for the bottom of beam 18 is defined as :—

$$C_{b18} = \frac{H_1 - H_{b18}}{H_4}$$

and for the top the coefficient is

$$C_{t\ 18S} = \frac{H_1 - H_{t\ 18}}{H_4}$$

H_4 was used as non-dimensionalizing factor as Vc is not a directly relevant flow variable.

The coefficients are plotted in Fig. 29.

The above forces F_1 - F_8 were measured for the following flow conditions D/S of the gate as observed from 1 : 69.6 scale model :—

(a) The gate being completely submerged below the water level in the service gate well.

(b) The gate partially submerged where the water level in the D/S transition was higher than the level of the bottom seal.

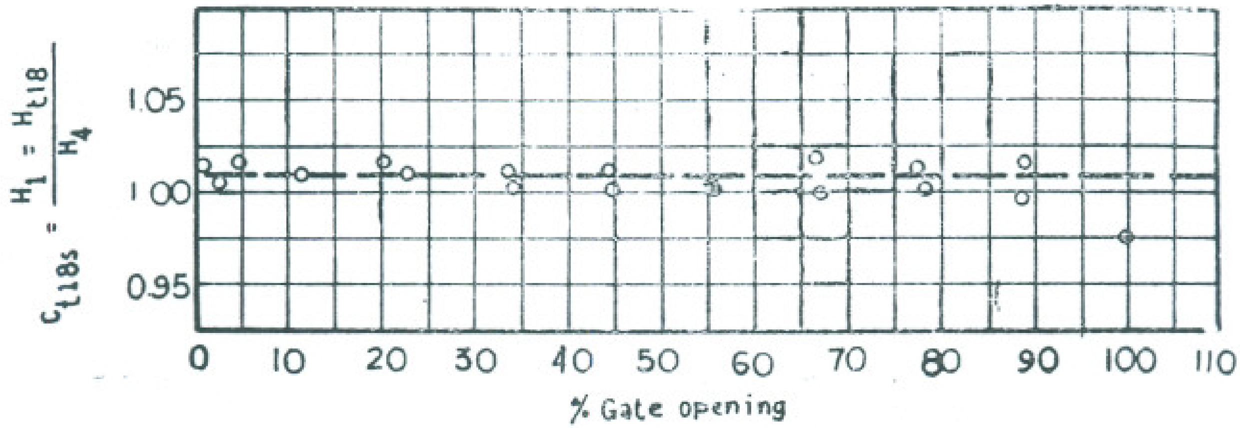
(c) The free surface flow as the water level in the D/S transition at the vena contracta is below the bottom level of the bottom seal.

For partially submerged conditions H_3 will be less than zero, H_2 is the depth of water immediately D/S of gates, whereas the hydraulic conditions U/S of the gate were the same as for the submerged gate. The effective pressure on the top beam was measurably small for the unsubmerged top beam. The weight of water contained between the beams because of the water cascading from the flow through the top seal gap may be included as part of the downpull and the estimate of cascading flow for 17 beams was 26 Kips.

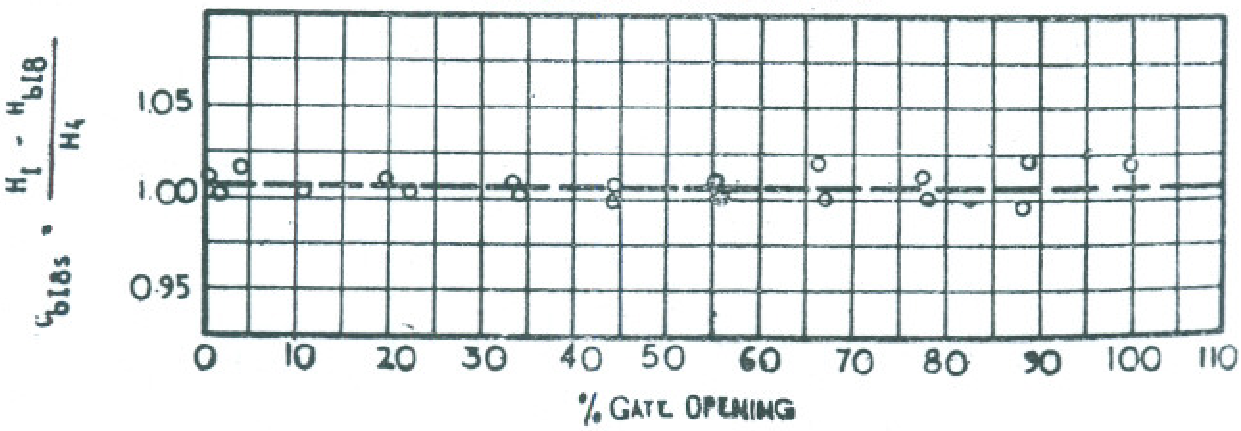
Fig. 29

TARBELA DAM PROJECT
HYDRAULIC MODEL STUDIES

Pressure Head Cofficients for the top side of Beam 18



PRESSURE HEAD COFFICIENTS FOR THE BOTTOM SIDE OF BEAM 18



For free surface flow conditions the magnitude of H_2 was considered to be the depth of flow at the vena contracta. It was observed that the pressure head acting on the top of the beam I was fluctuating about zero. For a gate opening of ·25′ the pressure on the bottom of beam I was approaching vapour pressure. For gate openings of I foot the measured heads were essentially atmospheric.

C-6.3 The Pertinent conclusions from the study are :—

I. By utilizing the pressure head coefficients for downpull forces F_1 - F_8 for the three hydraulic conditions in the D/S transition as given in Figs. 26-29 the separate forces may be calculated as follows :—

(a) The pressure coefficients, H_4 or $\frac{V_c^2}{29}$ together with H_1 being known, the average piezometric head on the particular part of gate with reference to sill level is as given below :—

$$H_i = (H_1 - C_i \,.\, H_4)$$

(b) Substract the pertinent elevation of the gate part above sill level 'z' from head H_i and multiply by γ A_i to obtain the force.

$$F_i = (H_i - Z_i)\ \gamma\ A_i$$

II. Low pressures (upto-32′) were identified at the U/S wall of shaft of service gate in the vicinity of top seal area due to large velocities of flow through the gap and the sharp corner forming at the junction of U/S wall of service gate shaft and the roof of passage way.

III. There were three regions of sub-atmospheric pressure (upto -32 feet) for free surface flow conditions :—

(1) The wall of the passage just D/S of the gate groove for the service gate.

(2) The divider wall within the D/S transition near the floor and even on the floor.

(3) The top seal surface on the U/S wall of the service gate.

IV. In case one gate is fully closed and the other fully open sub-atmospheric pressures just D/S from the U/S nose of the divider pier i.e. in the wake of separation were observed.

V. The head loss coefficients for the headloss (H_5) between the beginning of the U/S transition and the tail end of D/S transition and the headloss H_4 i.e. the difference in water levels in the bulk head gates well and service gate well are plotted in Fig. 30.

VI. The fluctuations in water level in the left and right shafts in the service gate shaft were negatively correlated due to vortex shedding at the tail end of divider wall. The maximum downpull due to this cause was 16 kips.

VII. Gate oscillations were small and it was concluded that these will not occur on prototype due to proto friction of the side seals.

VIII. Unexpected large uplift force did not result because of gate motion.

IX. A sharp increase in downpull force (180 kips) was recorded near gate closure at about .25 ft. for unsubmerged flow conditions.

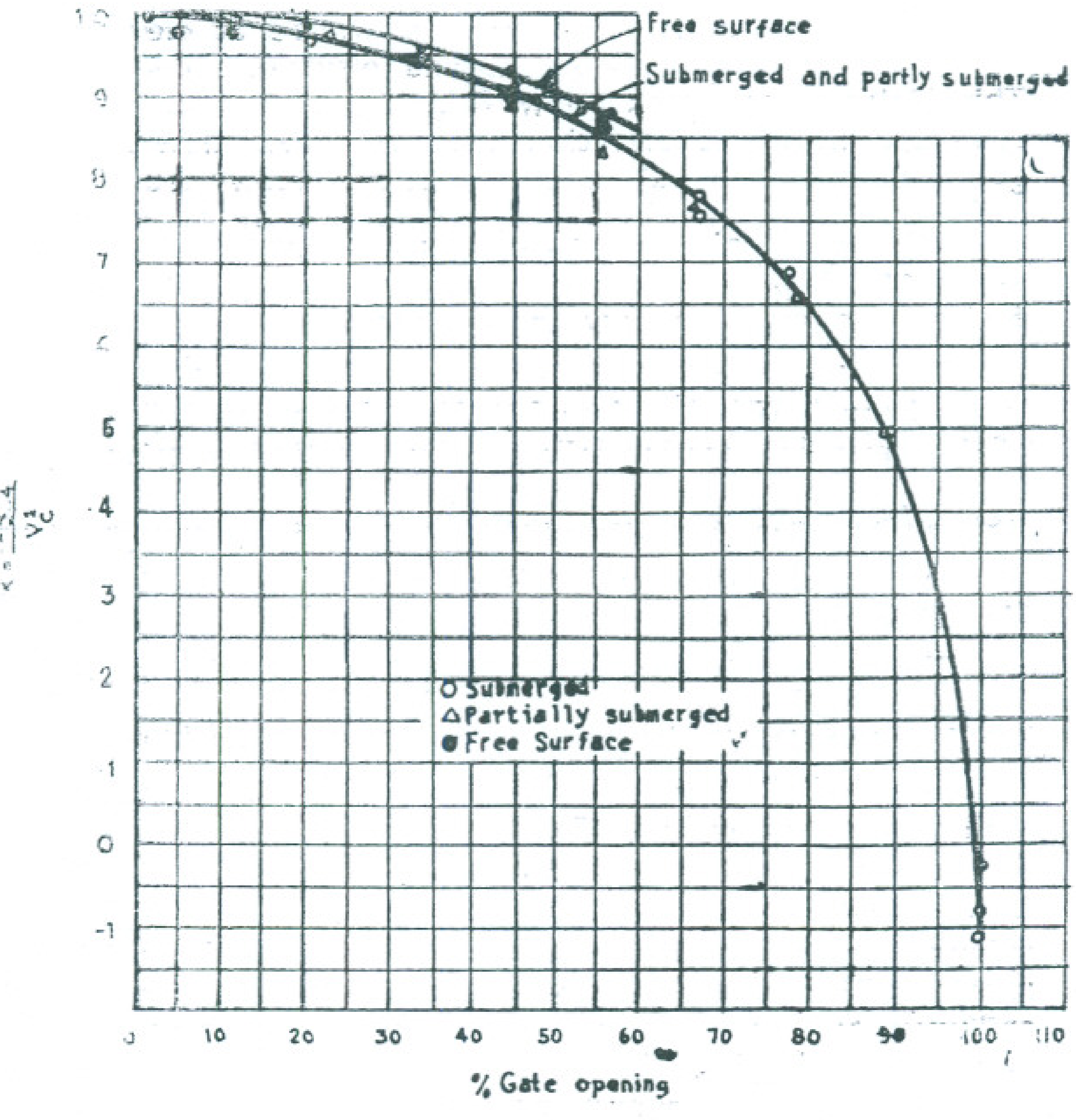
X. A small increase in downpull force of the order of 60 kips was recorded near full open position of gate.

XI. Force fluctuations resulted in the range of 2′ - 36′ feet probably as a result of water level fluctuations in the gate shaft.

. . .

Fig. 30

TARBELA DAM PROJECT
HYDRAULIC MODEL STUDIES
Variation of K with Gate Position



D-1.0 Hydraulic model studies on Service and Auxiliary Spillway of Tarbela Dam

Two gate controlled spillways on the left bank will discharge into an outlet channel excavated in Daldara, a natural side valley which returns to the Main Indus D/S of Dam. The outlet channel will have a bottom width of 400′ U/S of Service Spillway and 500′ D/S of Service Spillway and will be at elevation 1080 U/S of Service Spillway out fall and elevation 1060 D/S of it. This Daldara has a rock below R.L. 1100. The crest of both spillways will be at elevation 1492 with over flow shapes and gate sections identical in cross-section. Flow from Service spillways will be controlled by 7 No. radial gates 50′ wide and 58′ high. The Auxiliary spillways will have nine radial gates 50′ wide 58′ high. The design capacity of service and Auxiliary Spillways is 654,000 and 840,000 cusecs respectively.

The original design of the service spillway specified a 2250feet long channel STA 8+30 to STA 30+80 set at a 1% slope D/S of the over flow section followed by a vertical transition curve from STA 30+80 to 32+51.58 on equation $\left\{Y = \frac{x}{100} + \frac{x^2}{100}\right\}$ to a chute at a slope of IV: 2H ending in a flip bucket. Fig. 31-a

The design of the auxiliary spillway specified a transition curve from STA 9+36.20 to 11+11.20 and following $Y = \frac{x^2}{100}$ immediately D/S of over flow section followed by a chute IV : 2H ending in a flip bucket. Fig. 31b.

As planned, the auxiliary spillway will be operated only when the discharges greater than design discharge of service spillway must be passed or when service spillway is under repair. The limitations on the operation of the auxiliary spillways to extended periods is due to shorter length of Auxiliary Spillway and the rock foundation is less resistant than that of service spillway and the auxiliary spillway bucket, is in close proximity to Auxiliary dams I and 2-a and could endanger these dams.

The model studies on service spillways, auxiliary spillway and Dal Dara channel were carried out first at Irrigation Research Institute in 1963-1965 then in 1967 and 1970-71 and during 1970 to 1972 at the Alden Laboratories in USA. The models and their results are briefly discussed below :—

D-1.1 Experiments at Irrigation Research institute on Spillways 1963-1965.

Two models were made for tests on spillways :—

(1) A general model (scale 1/120) including both the spillways, with proper approach conditions and the Dal Dara into which both discharge. The river where the Dal Dara channel out falls was also represented. The model was used for the study of flow conditions, through the crest structure, chutes and the flip buckets. This model dealt briefly with the scour problem. For scour tests, the erodible channels consisted of Haro sand whereas the banks were modelled with clay.

(2) A partial model of spillway to a scale of 1 : 64 for observing pressures on the chute and co-efficient of discharge.

D-1.2 Model Results and conclusions

Important conclusions are listed below :—

(1) Separation occurred at the round nose piers and this discontinuity in flow contributed to the strength and intensity of the vortices which formed adjacent to the piers during partial gate operation. The model tests showed that the 20′ radius head-wall performed unsatisfactorily and the 40 feet radius head-wall with 40 feet radius piers perform satisfactorily. The latter configuration but modified to 35′ radius was used on both spillways.

(2) Flow in the chute of the Auxiliary spillway was quite satisfactory, after a slight modification, required in the shape of the side walls at the vertical curve. There were the usual diamond wave pattern in the chute, and rooster tails downstream from the piers, but these disturbances ironed out rapidly with no unusual concentration of waves along the training walls.

(3) For symmetrical operation a discharge of 24,000 cusecs was required to spring the jet free of the bucket.

(4) Experiments indicated optimum bucket rudius of 50 feet and a lift angle of 35 degrees for both the spillways. For

both spillways bucket radius of 50′ was adopted. The flip angle of service spillway was kept at 35 degrees whereas 30 degree flip angle was adopted for Auxiliary Spillway.

(5) A symmetrical or unbalanced gate operation is not recommended for the auxiliary spillway because of the resulting poor flipping flow conditions at flip bucket.

(6) An eddy developed along the promontary at the junction of outlet channel of Service Spillway and Dal dara which was drawn into the region of low-water surface elevation under the jet.

(7) As the water level rises above the lip level of bucket it interfered with the jet, violent oscillations occurred as the jet alternately drowned out and sprang free. Consequently the bucket lip was raised from 1202 to 1220 for both spillways which means :—

(a) the service spillway could be operated upto 500,000 without degradation of channels by erosion.

(b) with auxiliary spillway bucket lip at 1220, the auxiliary spillway operated satisfactorily at 450,000 but experienced surging at 450,000 when the combined flow of two spillways was at 800,000 cusecs.

(8) The equations of vertical and horizontal curves of chutes and side walls respectively was finalized as a result of studies on this model.

(9) Severe overtopping of the side walls of the chute of Auxiliary Spillway was observed with an end gate closed and adjacent gate open. In a number of adjacent gates on one side of auxiliary spillway were closed, the jet was not able to spring free at all points across the bucket.

D-1.3 Two Spillway models on 1/80 natural scale at Nandipur in 1967.

In 1967 two spillways (service and auxiliary) were modelled on 1/80 natural scale separately primarily to investigate scour problem in the vicinity of the flip buckets. These models provided information for use in the establishing design of under ground protection walls abutted to foundation blocks of flip buckets of spillways.

Even if the prototype rock characteristics in respect of scour are known (which they are not) there is no foretelling how any model material could be produced to model the prototype. The model tests with a representative material are guide to judgement of relative merits and demerits of different alternative schemes tested on a model. For non-homogeneous and layered materiel such as exist at Tarbela it is not possible at this time to obtain a model material that would correctly reproduce prototype erosion. Any direct extension of results to predict lateral extent of scour is not warranted.

At the service spillway bucket the strike of the rock is roughly parallel to the Dal Dara: the dip 50° to 55° away from the bucket. The rock is mainly fractured and jointed dolomitic limestone with an intrusion of basic rock. The soundest rock, encountered along the exit channel from the bucket is located farthest from the flip bucket at the side of the Dal Dara; a bed of dolomite, some 50 ft. thick, Dwg. 302 BG 218. At the auxilary spillway bucket, the strike of the rock is roughly paralled to the bucket lip; the dip is 45° to 50° away from the bucket and the rock, a limestone which slopes away from the bucket lip, is generally not so sound as the rock at the service spillway.

The natural rock was representated by a mixture of clay and aggregate in ratio of 50% each in the regions of service spillway and in a ratio of 25% clay and 75% aggregate in areas of Auxilary spillways to reflect the poorer quality of rock in the vicinity of flip bucket of auxiliary spillway. The mix was placed in the model in the layers watered and compacted with wood hand tampers. The overburden in Daldara was moulded in Haro Sand. The mixture of clay and aggregate had a near vertical angle of repose even when submerged with the scoured face exhibiting an irregular rock like appearance.

The studies were instituted to aid in answering the following questions:—

1. What will be the pattern of scour at the flip buckets of the service and auxiliary spillways?

2. Will scour tend to undermine the flip bucket monoliths?

3. If protective walls are required to protect one or both of the spillway buckets from scour, how deep and extensive should these walls be?

The maximum discharge during the record period of about 100 years was about 680,000 cfs in 1929, slightly more than service spillway capacity of 650,000 cfs. While with the dam constructed, a discharge of this magnitude could always be produced artificially by opening the spillway gates at design resevoir level. Naturally or artificially produced, such a discharge would not persist for long. During the period of record several ten day periods have occurred in which the flow averaged between 400,000 and 500,000 cfs. Taking into account a discharge capacity of 100,000 cfs for one irrigation tunnel and neglecting powerhouse flow, it seems reasonable to use 400,000 cfs as the maximum discharge which the service spillway would be expected to pass for extended periods.

Tailwater rating curves for the service and auxiliary spillways are given in Fig. 32. The two curves in each figure relate to different conditions of scour in the spillway outlet channel. The "high tailwater" curve corresponds to a condition of no scour, i. e., the channel is as shown in the project plan. The "low tailwater" curve corresponds to a condition of scour to rock along the entire length of the channel. Values were taken at Sta. As 60+00 for the service spillway and AS 20+00 for the auxiliary spillway.

To obtain useful results from the scour models in the time available a flexible test programme (Table IV, V) was adoped in which each test run was based upon the results of preceding runs. The entire test programme for each model was accomplished without remoulding the bed. Generally, each model was operated for long periods at discharges ranging from 50,000 to 400,000 cfs the minimum and maximum sustained discharges discussed above under Criteria Discharge. Operation at design discharge was for short periods only, Initially, high tailwater levels were maintained. Later, after the scour pattern had begun to develop, tailwater was dropped to retrogressed levels reflecting what would happen with time in the prototype,

The procedure was aimed at determining qualitatively how the scour pattern in the prototype would approach the bucket

Table IV

TEST PROGRAM—SERVICE Spillway MODEL

Total hours model operated	Duration (hrs)	Discharge cfs	RUN DATA: Water surface Elevation Pt. A	Pt. B	Pt. C	Under jet	Velocity in Eddies fps Left	Right
1	2	3	4	5	6	7	8	9
High Tailwater								
0	5	50,000	1172	...	...	...	...	...
5	5	100,000	1176	1178	1178	...	...	...
10	5	200,000	1184	1188	1191	...	14.4	...
15	1	400,000	1197	...	...	...	19.2	...
16	11	400,000	1197	1213	1203	...	26.4	...
27	6	400,000	1197	1210	1200	1193	24.0	...
33	12	400,000	1197	1211	1201	1194	25.2	...
45	9	400,000	1197	1211	1201	1194	26.7	...
54	1/4	650,000	1214	1229	1214	1202+	...	...
54.25								
Tailgate lowered to prevent submergence of bucket.								
55	3/4	650,000	1205	1219	1212	1200	28.5	...
High tailwater resumed.								
60	5	400,000	1197	1215	1206	1196	28.9	...
65	5	200,000	1184	1197	1197	1192	20.4	...
70	5	100,000	1176	1190	1191	1191	7.7	...
75	5	50,000	1172	1186	1189	1190	...	...

Table IV

TEST PROGRAM—SERVICE SPILLWAY (Continued)

Total hours model operated	Run Data						Velocity in Eddies (fps)	
	Duration (hrs.)	Discharge (cfs)	Water surface Elevations					
			Pt. A.	Pt. B.	Pt. C.	Under jet	Left	Right
1	2	3	4	5	6	7	8	9
First Retrogressed Tailwatet—Deposited material downstream of scour hole removed.								
80	5	50,000	1125	1136	1136	...	...	...
85	5	100,000	1130	1139	1141	...	18.4	8.8
100	15	200,000	1136	1149	1153	...	20.4	12.4
110	10	400,000	1148	1173	1172	...	25.2	16.2
120	10	400,000	1148	1174	1172	...	25.6	16.5
126	6	400,000	1148	1176	1172	...	26.0	16.8
136	10	400,000	1148	1177	1173	...	26.2	17.1
160	24	400,000	1148	1177	1173	...	26.3	19.4
Second Retrogressed Tailwater—Deposited material downstream of scour hole removed again. Tailgate at lowest setting. Stoped wall installed.								
170	10	50,000	...	...	...	...	...	...
180	10	50,000	...	...	...	...	...	...
190	10	400,000	1116	1162	1161	...	28.0	18.3
191	1	650,000	1125	1182	1177	...	...	...

Note:—Pts. A, B and C located on Bed Contour Plots.
Velocities are maximum values measured where eddies acted upon banks.

Table V

TEST PROGRAM—AUXILIRRY SPILLWAY MODEL

RUN DATA					Velocity in Eddie (fps)	
Total hours model operated	Duration (hrs.)	Discharge (cfs)	Water Surface Elevations			
			Pt. A	Pt. B	Left	Right
High tail-water						
0	7	50,000	1175	1175	...	...
7	5	100,000	...	...	...	...
12	15	200,000	...	...	...	...
27	5	400,000	1198	1206	16.8	22.8
32	5	400,000	1196	1208	18.6	24.8
Retrogressed Tailwater—Deposited material downstream of scour hole removed. Stoped wall installed.						
	20	50,000	...	1158	...	...
57	10	400,000	1170	1178	23.3	28.2
67	1	840,000	...	...	...	...

Notes: Pts. A and B located on Bed Contour Plots.
Velocities are maximum values measured where eddies acted upon banks.

TARBELA DAM PROJECT, HYDRAULIC MODEL STUDIES
SERVICE SPILLWAY TAILWATER RATING CURVES

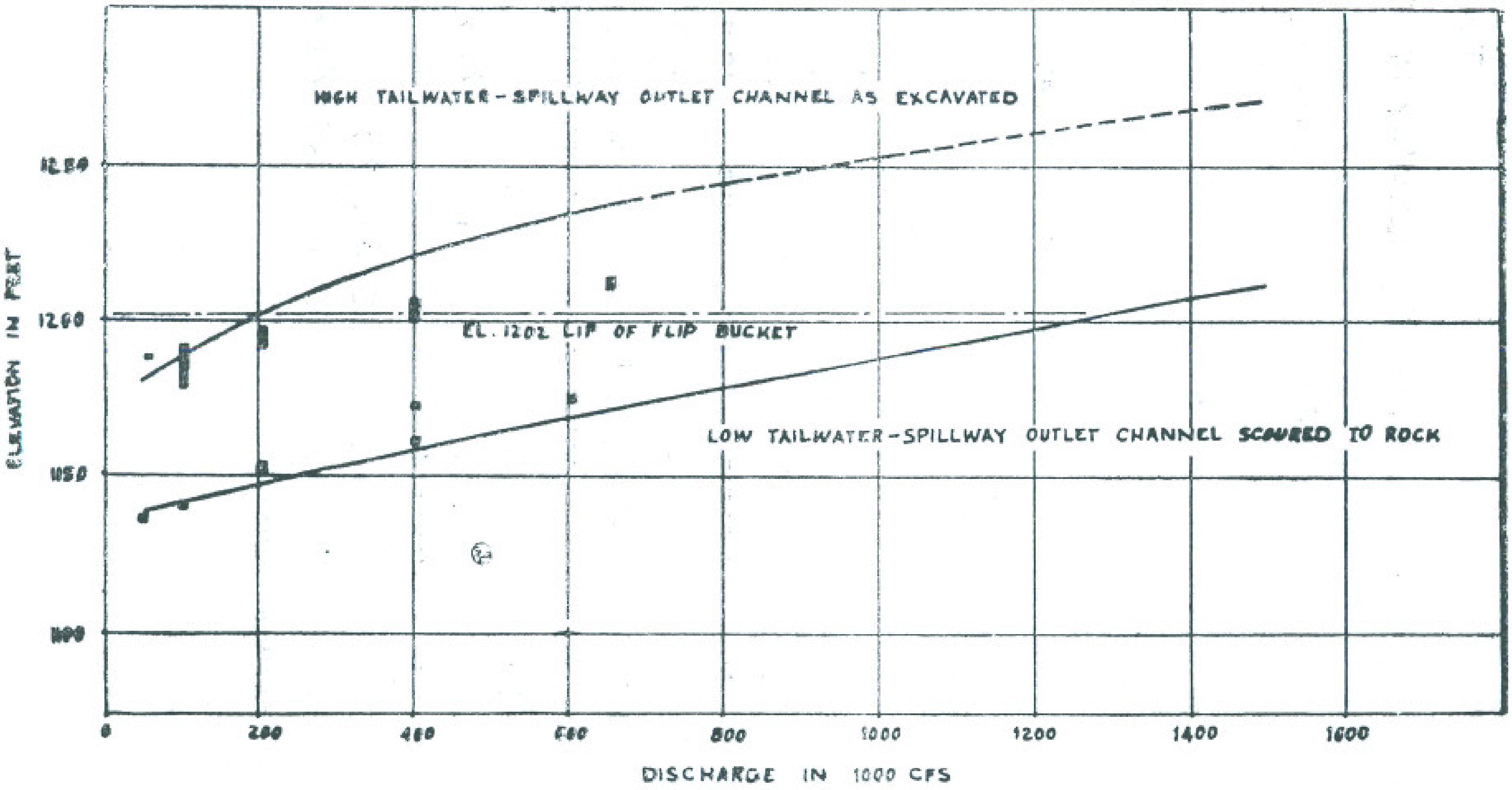


Fig. 32

NOTES:-

1) HIGH TAILWATER CURVE FROM CALCULATED WATER SURFACE PROFILES ASSUMING NO SCOUR IN DAL DERA

2) LOW TAILWATER CURVE FROM CALCULATED WATER SURFACE PROFILES ASSUMING RETROGRESSION IN DAL DERA UP TO ROCK LEVEL

with quantitative indications of relative rates of scour at different stages in the process and the maximum depth of scour near the bucket.

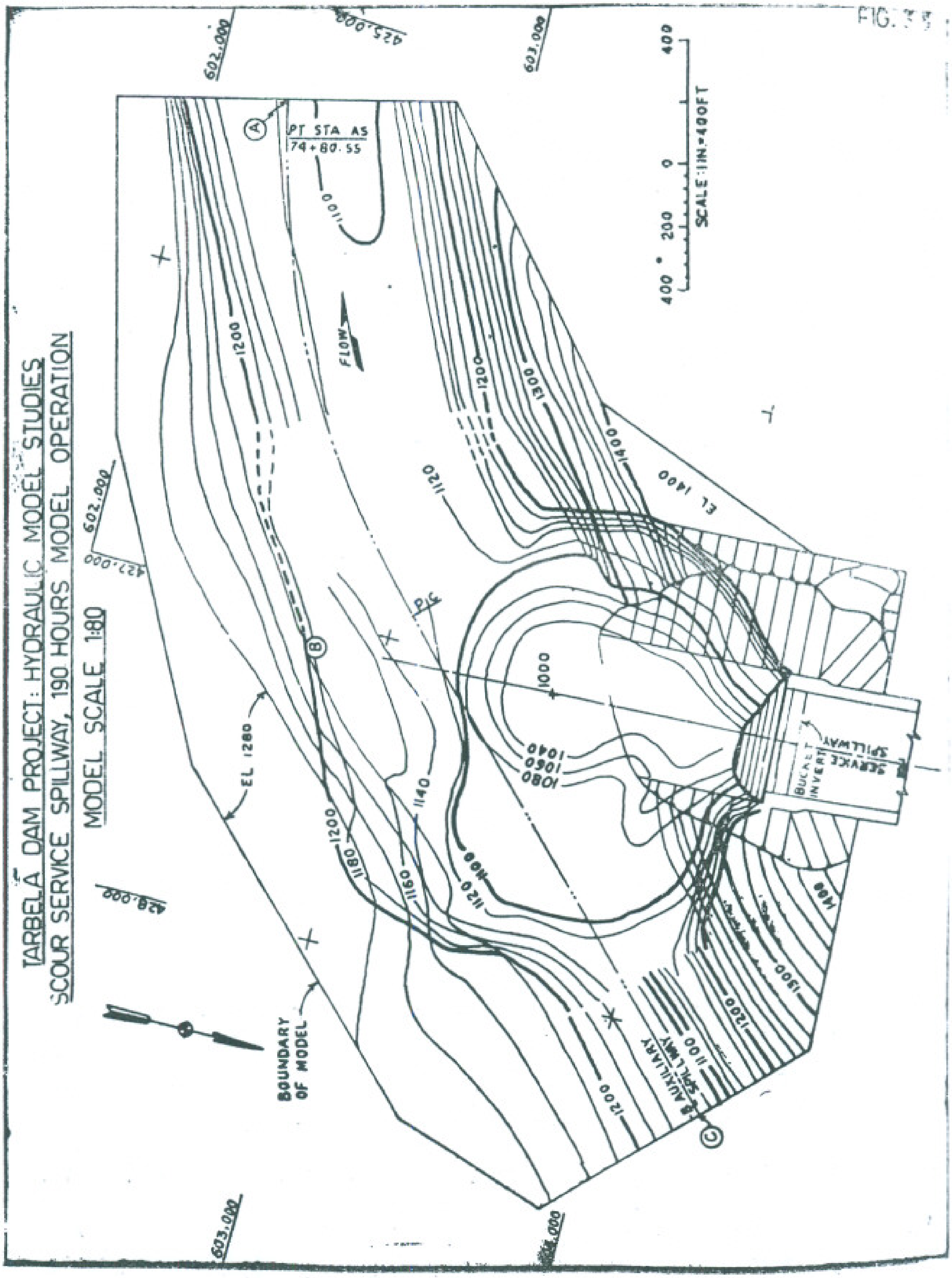
After a total of 75 hours of operation at high tailwater, the excess scoured material was removed from the model and testing at low tailwater begun.

The flow conditions had indicated a serious eddy at the left end of the jet impingement area at the junction of left bank of outlet channel of service spillway and Daldara. The eddy intensity is a function of the energy density in jet impingement area and the extent of the eddy. The eddy conveyed water from high elevation ride up area D/S of plunge pool to the region of low water surface elevation under the jet. Near the end of test the velocity of 29 fps on the left side eddy and 18 fps on right side eddy were observed for a discharge of 400,000 cusecs.

During the low tailwater runs the promontory to the left of the impact area continued to scour at a slow rate; the bank at the right scoured at an accelerated rate. Most important, the scour hole worked back toward the face of the flip bucket. While no portion of the flip bucket had been exposed during the high tail-water runs, after 85 hours of operation at low tailwater (160 hours total), the face of the bucket was uncovered to the level of the base.

During the final portion of the service spillway test programme a proposed form of scour protection, a sloped wall, was represented in the model, The prototype wall, shown in Dwg. 31NY400, would be constructed by excavating a series of tunnels or cross drifts from inclined shafts. Concreted, the drifts form an apron-like, sloping wall which, if the scour hole extended back towards the bucket would armor the erodible material against further scour. In the model, the face of the wall where exposed by the scour pattern was formed in mortar. The wall, placed at a slope of 1 on 1 intersected the face of the flip bucket at El. 1165, ten feet above the base.

A proposed form of protection, a wall at a slope of 1 on 1 was represented in each model. After 190 hours of operation at high and retrogressed tailwater levels, the wall in the service spillway model was exposed to El. 1050, (Fig. 33) about 150 ft.


TARBELA DAM PROJECT: HYDRAULIC MODEL STUDIES
SCOUR SERVICE SPILLWAY, 190 HOURS MODEL OPERATION
MODEL SCALE 1:80
SCALE: 1 IN = 400 FT
PT STA AS
74+80.55
FLOW
EL 1400
EL 1280
BOUNDARY OF MODEL
SERVICE SPILLWAY
BUCKET INVERT
AUXILIARY SPILLWAY
602,000
603,000
425,000
427,000
428,000

ɔelow the lip of the bucket. Exposure of the wall was limited o the area between the corners of the bucket.

Flow patterns in the auxilliary spillway model were similar .o those observed in the service spillway model. Eddies scoured he banks on both sides of the impact area; however in the uxiliary spillway, both eddies were active at both high and low ailwater and the eddy on the left was stronger than the one on he right. By the end of the test programme, the fixed bed o the right of the impact area had been exposed and the romontory to the left scoured even at back of the lip of the ucket.

The face of the flip bucket in the auxilliary spillway model was xposed after 37 hours of operation, all at high tailwater levels. t this point the flip bucket protection wall was installed, the xcess material removed and operation resumed at low tailwater. esting was continued for 30 hours more untill the bed configura- on shown in Fig, 34 was reached. The wall was exposed to l. 1060 near the spillway centerline; some 120 ft of wall were xposed beyond the corners of the flip bucket.

The shorter time required to expose a substantial portion of rotection wall in the auxilliary spillway model implies an equal eed for protective works at both spillways. In other words, if rotective works are provided at the service spillway, they should so be provided at the auxilliary spillway despite the fact that ıe later will be infrequently operated.

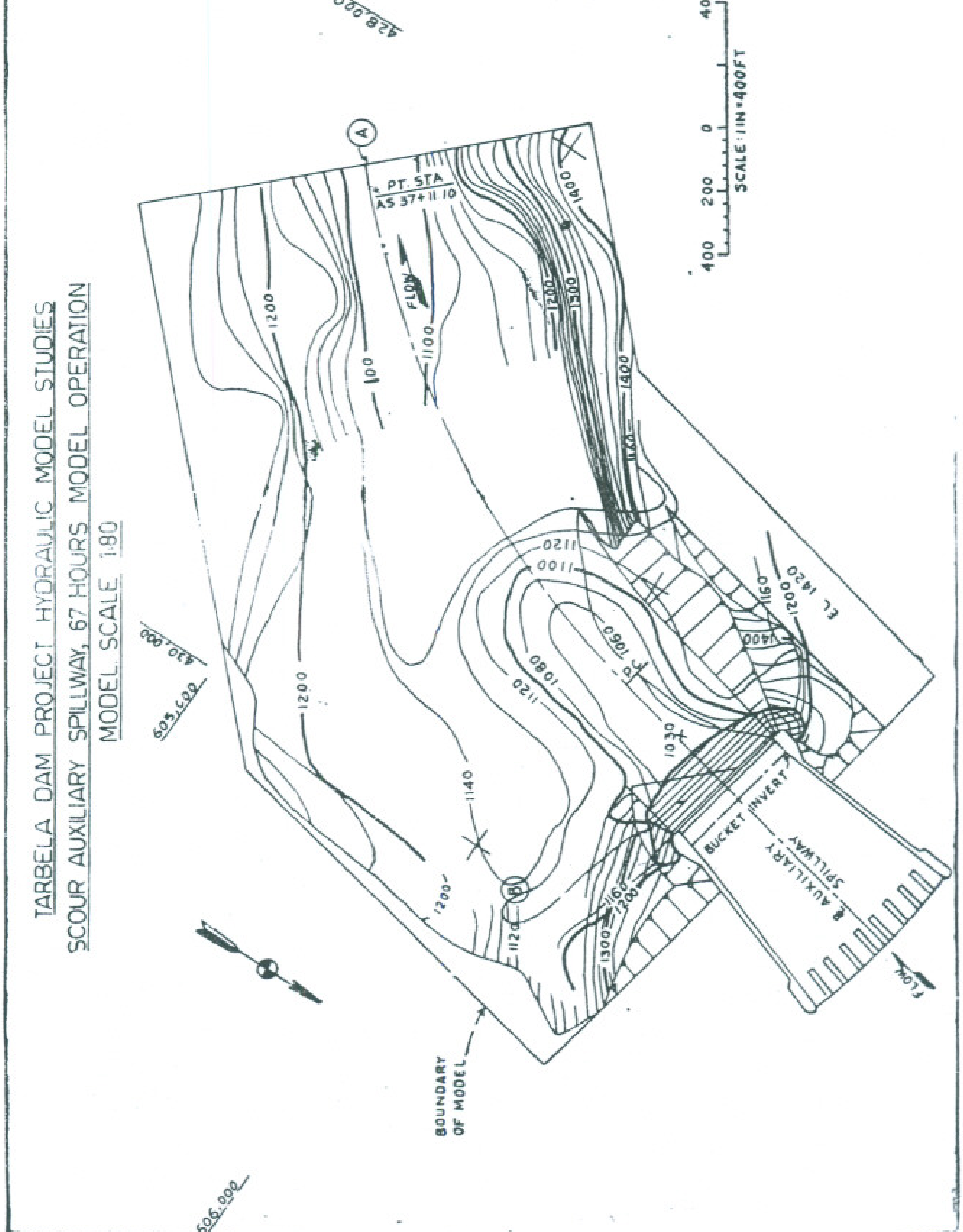
)-1.4 1975 Demonstration Experiments at Irrigation esearch Institute.

In these experiments the bucket of service spillway was set ıck by 500′. The rock was represented in clay only. The over ırden was represented by Haro sand.

1. At high discharge Q 200,000 cfs, bellies can from in the de benches in the areas of week rock (the size depending on ıeir erodibility) due to the flipping jet partly impacting on the ənch. These have to be repaired off and on.

2. Highly turbulent flow conditions with bulked flow and ater hump at tail end of outlet channel of service spillway

FIG. 34


TARBELA DAM PROJECT HYDRAULIC MODEL STUDIES
SCOUR AUXILIARY SPILLWAY, 67 HOURS MODEL OPERATION
MODEL SCALE 1:80
SCALE: 1 IN = 400 FT
400
200
0
200
400
PT. STA
AS 37+11.10
FLOW
BOUNDARY OF MODEL
BUCKET INVERT
℄ AUXILIARY SPILLWAY
EL 1420
428,000
430,000
605,600
606,000

about 50 feet higher than the water level in Daldarra are noticeable. We have to live with this condition of flow. Local bellies can form in the left bank of Daldarra opposite the outfall of service spillway and also of tunnel 5 when high continuous discharge run from each. These may not present serious problem and can be taken care of when needed.

3, Heavy erosion of the banks of Daldarra is likely to occur above the point where it outfalls into the river (It has actually happened).

4. Downstream of the auxilliary spillway, the back roller on the left side erodes the bench rapidly and if the rock nose on left abutment of Auxilliary spillway is not very resistant, the scour can be very near the toe. This aspect has to be given more consideration.

D-2.0 Model tests at Alden Research Laboratories; 1970-72.

The model of two spillways and their common outlet channel Daldara was constructed to 1/80 scale to explore further the ultimate extent of plunge pool development by utilizing the model material simulating the pertinent properties of prototype rock and over burden. The model at Alden Laboratories was supposed to be more refined than the Nandipur model.

The initial concept of A. R. L. O. was to develop (step by step) a scour hole configuration by removing material from high velocity areas such that at no place of the periphery of scour hole the velocity does not exceed upper limit of 20 fps on prototype to be followed by development of scour hole configuration with peripherial velocity less than 15 fps. As the configuration widened a high velocity eddy developed in the left side of jet drawn in under the jet similar to the one described in case of 1/80 scale model at Nandipur. The velocity eddy along the promontory dividing the service spillway outlet channel and Daldara could not be reduced below 35 fps. Various measures ware tried to reduce the severity of this eddy by reducing the discharge per feet run or increasing the width of the jet impingement zone. In the model tests the eddy was reduced by spreading the jet impingement zone longitudinally by placement of blocks in the spillway bucket. The idea of blocks in tbe bucket was rejected by TAMS.

The next decision was to use a moveable bed model with material cohesive to stand on steep slopes. Exploratoty flume tests were done to determine resistance to erosion of cement-sand mixtures in various ratios. A mixture of 1 part cement to 60 parts sand by volume was selected as a result of these studies. The mixture when tried on 1/80 model proved to be more resistant to scour than is expected for the geological formation at the site. The stabilized scour hole on the model was narrow but deep with bottom R. L. 957 and was located right of the axis of the channel. The depth and lateral extent of the scour and high velocities in the scour hole indicated that the cement-sand mixture was too strong.

A 1 : 80 cement-sand mixture was used to model the rock contours. The test programme run in the model is given in table No. VI. At run No. 5 of the scheduie a large piece of promontory on left bank of outlet channel at junction of Daldara fell into the scour hole. Following the cave in the eddy on the left side of the Jet increased in size and intensity and so the cut back of left bank of the service spillway continued steadily. After 57 hours of the model operation a larger portion of the left bank of the service spillway had eroded. The contours of the scour hole and the exposure of the apron are shown in of Fig. 35. The scour hole bottom near the left end of bucket was at R. L. 1039. The main scour pit upto R. L. 993 was near the junction of right hank of Daldara with the right bank of outlet channel of service spillway and thus the scour hole was right of center-line whereas a great mound formed to the left of the center line and a trench circumscribing the mound (on left, U/S and D/S of it) formed by the main eddy current, a great cut back of left nose and exposure of 45° protection wall of the spillway bucket. TAMS remarked on 1:80 cement-sand test that :—

"The test with 1:80 mixture was unrepresentative in that failures occurred by tension and from internal pore pressure. Whether the characteristic of ease of erosion by wave action and velocity is representative is difficult to assess. What did seem significient was that when the left bank of the service spillway channel had enlarged sufficiently for an eddy to form, it continued to enlarge as a result of eddy current velocities and of wave action, a situation consistent with the Nandipur

TABLE VI

SCHEDULE OF MODEL OPERATION
SERVICE SPILLWAY II TESTS

Rock contours represented by 1.80 cement-sand mix
Lip of flip bucket at Sta. S.S. 36+83:81

Run No.	Discharge cft.	Duration of run hr. min.	TOTAL TIME RUN AT 100,000 cfs.	200,000 cfs.	400,000 cfs.	600,000 ± ofs.	Total time run all discharges
1	200,000	0-10		0-10			0-10
2	200,000	0-20		0-30			0-30
3	400,000	0-10		0-30	0-10		0-40
4	400,000	1-20		0-30	1-30		2-00
5	400,000	4-52		0.30	6.22		6-52
6	637,000	0-20		0-30	6-22	0-20	7-12
7	[620,000	1-16		0-30	6-22	1-36	8-28
8	632,000	4-32		0-30	6-22	6-08	13-00
9	627,000	7-00		0-30	6-22	13-08	20-00
10	620,000	19-50		0-30	6-22	32-58	39-00
11	630,000	17-10	0-00	0-30	6-22	50-08	57-00

Note. All times are model times.

TARBELA DAM PROJECT
HYDRAULIC MODEL STUDIES
SCOUR HOLE CONTOURS AFTER RUN
NO.11 OF THE SS-11 TEST SERIES

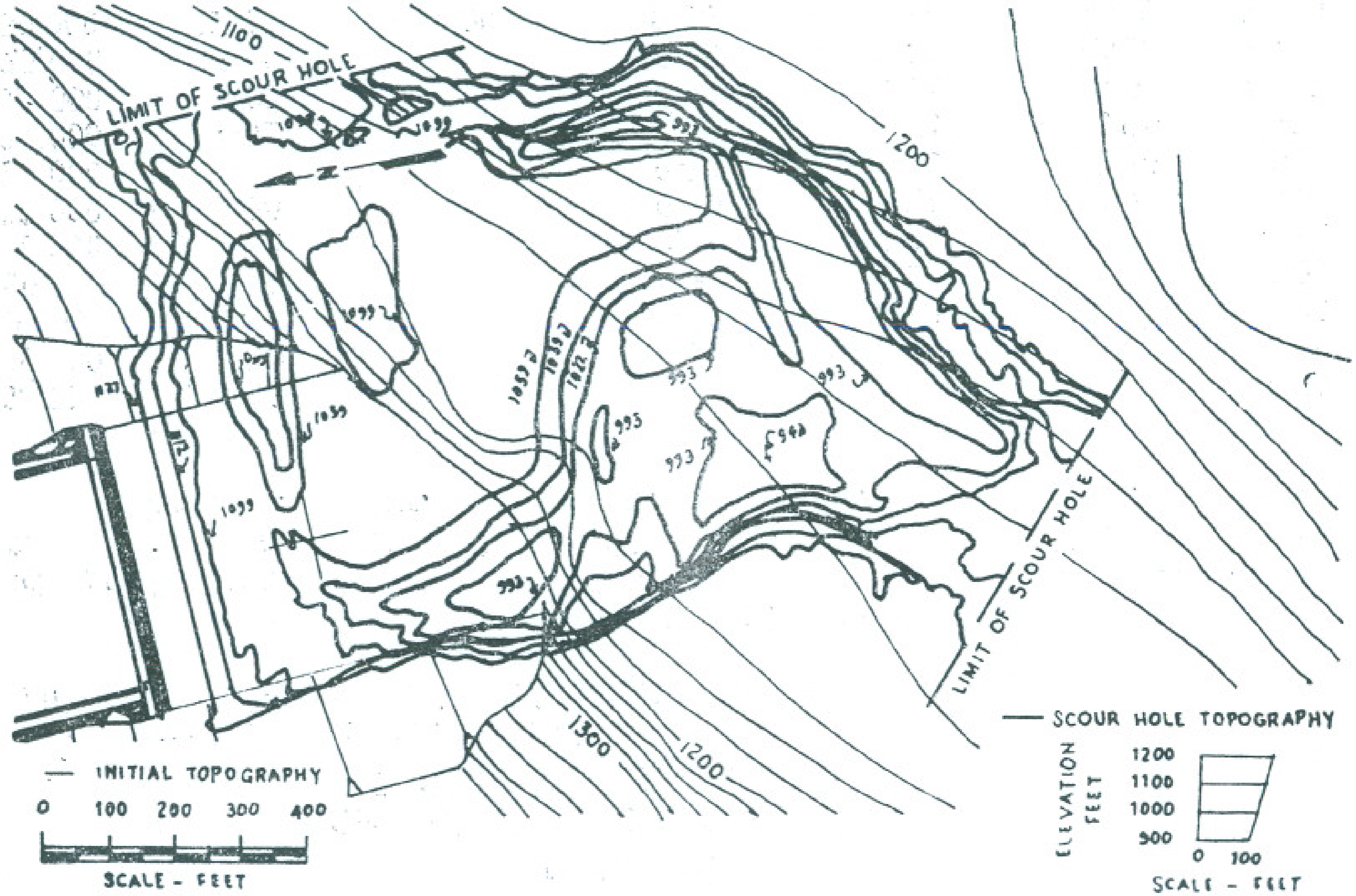


Fig. 35

test results. The close proximity of the left end of jet impingement zone to the nose of the land in between the two channels (exit channel of service spillway and Daldarra) is not favourable."

TAMS then decided to run a new test on the service spillway model with the upper one-percent slope chute shortened by 500 feet. The level of the bucket was raised by 5 feet, to Elevation 1207 ft. The foremost objective of the new bucket location was to gain additional natural rock to the left of the spillway, in the "nose" between the spillway bucket and the Dal Darra channel. The upstream location of the spillway bucket served also to increase the distance from the ride-up area of water on the left bank of the Dal Darra to the depressed pool level under the jet. The test was run with the 1 : 80 cement-sand mixture downstream from the spillway, which mixture on this occasion seemed to be more cohesive than in the preceding test series. The scour hole is shown after operation for 218 hours Table vii Figure 36, which shows that very little scour had taken place at the protection wall; in fact, scour along the centerline profile of the scour hole was less severe than had been computed at the time of original design. After an additional 200 hours of testing under a variety of conditions, the exposure of the 45° wall had preceded only down to elevation 1070. In view of the model test indications, the decislonswere to locate the spillway bucket STA 31+83.81 500 ft. upstream of the location STA 36+83.81 shown in the contract drawings, and to retain the original protective apron design, that is, with its base at elevation 1050 which was based on Nandipur Tests.

Shortening the spillway chute by 500 ft. and increasing the channel width by in increasing the diversion angle of channel walls with the channel central line resulted in increased velocities in the eddies formed on the right and left side of the jet impact area. Velocities of return eddy were measured at the junctions of the service spillway outlet channel and the Dal Dara for the 20° and 25° angle cut backs at a discharge of 400,000 cusecs. At the D/S nose on left bank of outlet channel velocity of 15 fps for the 20° cut back and 18 fps for the 25° cut back were measured. The corresponding velocities on the D/S nose of right bank of outlet channel of service spillway were 19 fps and 22 fps measured inside the outlet channel.

TABLE VII

SCHEDULE OF MODEL OPERATION
SERVICE SPILLWAY III TESTS

Rock Contours represented by 1:80 cement-sand mix
Lip of flip bucket at Sta. S, S. 31+83.81

Run No.	Discharge cfs.	Duration of run hr. min	TOTAL TIME RUN AT 100,000 cfs.	200,000 cfs.	400,000 cfs,	600,000 ± cfs,	Total time run all discharges
1	200,000	0-30		0-30			0-30
2	400,000	1-30		0-30	1-30		2-00
3	400,000	5-00		0-30	6-30		7-00
4	590,000	1-30		0-30	6-30	1-30	8-30
5	623,000	4-30		0-30	6-30	6-00	13-00
6	601,000	6-00		0-30	6.30	12-00	19-00
+7	613,000	7-05		0-30	6-30	19-05	26-05
+8	605,000	40-12		0-30	6-30	59-17	66-17
9	637,000	23-31		0-30	6-30	82-48	89-48
10	637,000	13-22		0-30	6-30	96-10	103-10
11	640,000	43-55		0-30	6-30	140-05	147-05
12	640,000	71-00		0-30	6-30	211-05	218-05
13	100,000	15-00	15.00	0-30	6-30	211-05	233-05

Note. All times are model times.

+During Run 7 and Run 8 power failures occured. The exact times of the power failures are unknown. The errors in the duration of Runs 7 and 8 are estimated to be minus 11 hours to plus 10 hours.

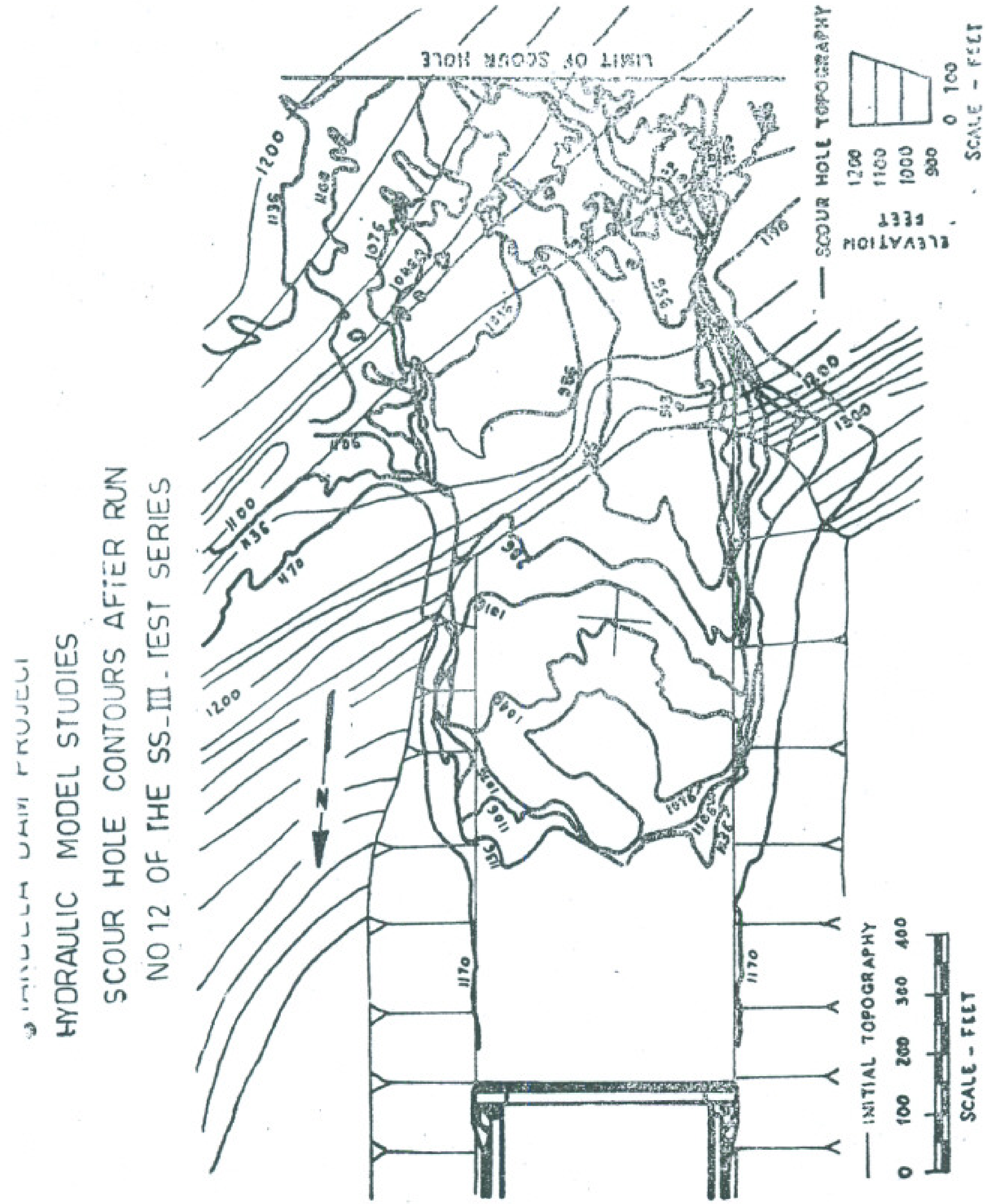

TARBELA DAM PROJECT
HYDRAULIC MODEL STUDIES
SCOUR HOLE CONTOURS AFTER RUN
NO 12 OF THE SS-III. TEST SERIES
LIMIT OF SCOUR HOLE
SCOUR HOLE TOPOGRAPHY
ELEVATION FEET
1200
1100
1000
900
0 100
SCALE - FEET
INITIAL TOPOGRAPHY
0 100 200 300 400
SCALE - FEET
N

Fig. 36

Further studies showed that the lateral spread of the discharge jet as it left the flip bucket lip could be reduced by the addition of a wedge section at the D/S end of each side wall of spillway chute. The wedges deflected the edges of the jet toward the centerline of the service spillway outlet channel and thus alleviated the potential problem of erosion of the banks resulting from the outer fringes of the jet beyond the wall end. The jets of the two spillways will be constricted with each wall bearing inward 1.33′ at a 15° angle over a distance of 5 feet in the direction of the jet.

Conclusions of Auxilliary spillway studies at A.R.L.O. on 1/80 scale model of Auxilliary Spillway with rock represented by 1 : 80 cement-sand ratio and over burden by 1 : 120 cement sand ratio.

(1) A left bank eddy developed with locally great velocities across the rock nose to the left of the spillway bucket so long as this nose held—it shielded the flip bucket. Test with rock contours without over burden indicated velocity of this eddy as high as 40 fps.

(2) At great depths in the scour hole, the right bank area was strongly under cut by the river currents.

(3) A moderate discharge of 250,000 cusecs was run for 200 hours on the model. The scour hole resulting from a sustained discharge of 250,000 cusecs is moderate in magnitude. Discharge of this order was actually run during 1975 operations to permit development of the spillway channel in between auxilliary spillways and service spillway which was cut at R.L. 1180 and to give an indication of the rate of erosion of left bank immediately D/S of auxilliary spillway.

(4) Attempts to protect the left bank against scour for discharges above 300,000 met with little success.

D-3.0 Prototype Performance of Spillway and Daldara Channel.

The hydraulic performance of the spillways was very satisfactory and according to the predictions of model. The flip

:tion of Service spillway was excelent—air was entrained well by e upper and lower nappes of the deflected jet which impacted ı the rock edge and upstream of it creating a plunge pool ›wnstream of the flip bucket. On the teft side downstream of e plunge pool a massive hard rock ledge is so orientated to ·flect some jet flow towards softtest rock which has been eroded the left bank tunnel access road. The entrance conditions ›re satisfactory only the rock noses extending some 50 feet :o the reservoir beyond the slope paving deflected surface flow the side bay causing an eday in the corner, Removing part the rock may be needed. It was indicated by Irrigation ;titute - 1965 report that the entry to the service spillway ›uld be bell-mouthed. The recommendations of Irrigation search Institute were not implemented.

The auxilliary spillway operated very well. The plunge ›l has developed considerable erosion of the loose material. e erosion has occurred on the left side of the plunge pool as ›dicted from model tests and more is likely. It is limited by rock ledge on the left downstream flank.

The Dal Darra channel has degraded considerably. A bar ;hown by model tests at Nandipur has formed at the outfall :he Daldarra channel into the river. The left bank erosion r the out fall was heavy as predicted from the model, so ch so that the spillway had to be closed to shift the electric ›er situated on the river bank being eroded.

.0 FIRST FILLING OF RESRVOIR-PROTOTYPE

Performance and the damages.

Reservoir filling was started on July 1, 1974, in accordance ›rocedure for first filling of Reservoir (Design memo-R/2 sup- nent No. 3 final report and Appendix). The regulation of ›s of Tunnel 1, 2, 3A, 3B, 4A, 4B during the period July, ;ust and September, is graphically illustrated in Figure 37 (a). chronological statement of events is recorded below:—

'-1974. Cell 18 of cellular Coffer dam collapsed. (Central gate T-2 opened. Flank gates closed to lessen the impact of tunnel discharge on remaining cells.

'-1974. Rate of filling slowed to allow iate evacuation.

26-7-1974. Motorcycle (also described as machine gun or clapping noise) from airvents of outlet gates heard.

27-7-1974. First attempt to close Tunnel 2. (gate 2) without success.

31-7-1974. Hindu Kash earth quake felt at site (Richter scale 6.5).

2-8-1974. Pieces of concrete aggregate fragments and stones noticed for first time from Tunnel 2 outflow. Grating of primary aeration through 3A noted missing.

8-8-1974. Second attempt to close central gate (G2) of T-2, without successs, gate jammed.

9-8-1974. T.J.V. reported considerable amount of solids being washed out of RDA-20.

13-8-1974. Severe cavitation and erosion of chute of T4 and damage to steel liners of T3a noted.

15-8-1974. Lowered service gates of tunnel 3 and 4 and T-2 (G.2) Re-bars observed at tunnel 2 outlet.

18-8-1974. Tunnel T-2 (G2) gates stem broke.

20-8-1974. Lowered bulk head gates of tunnel 3 and 4.

21-8.1974. Upstream portion of tunnel 2 caved in, Loud noise heard, at T-2 intake. Flow suddenly increased three fold. Slugs of brown water appeared off and on together with large quantities of rock debris. High level vibrations in entire right abutment area.

22-8-1974. Reservoir level peaked at 1462.30. Emergency unwatering of reservoir commenced. Bulkhead gates of Tunnel 3 and Tunnel 4 and gates (G 1, 2, 3)of T_1 raised G (1) of T-1 got stuck at 7 feet.

23-8-1974. Large cavity in the free drawing fill noticed.

26-8-1974. Cell 17 collapsed.

27-8-1974. Cell 16 collapsed.