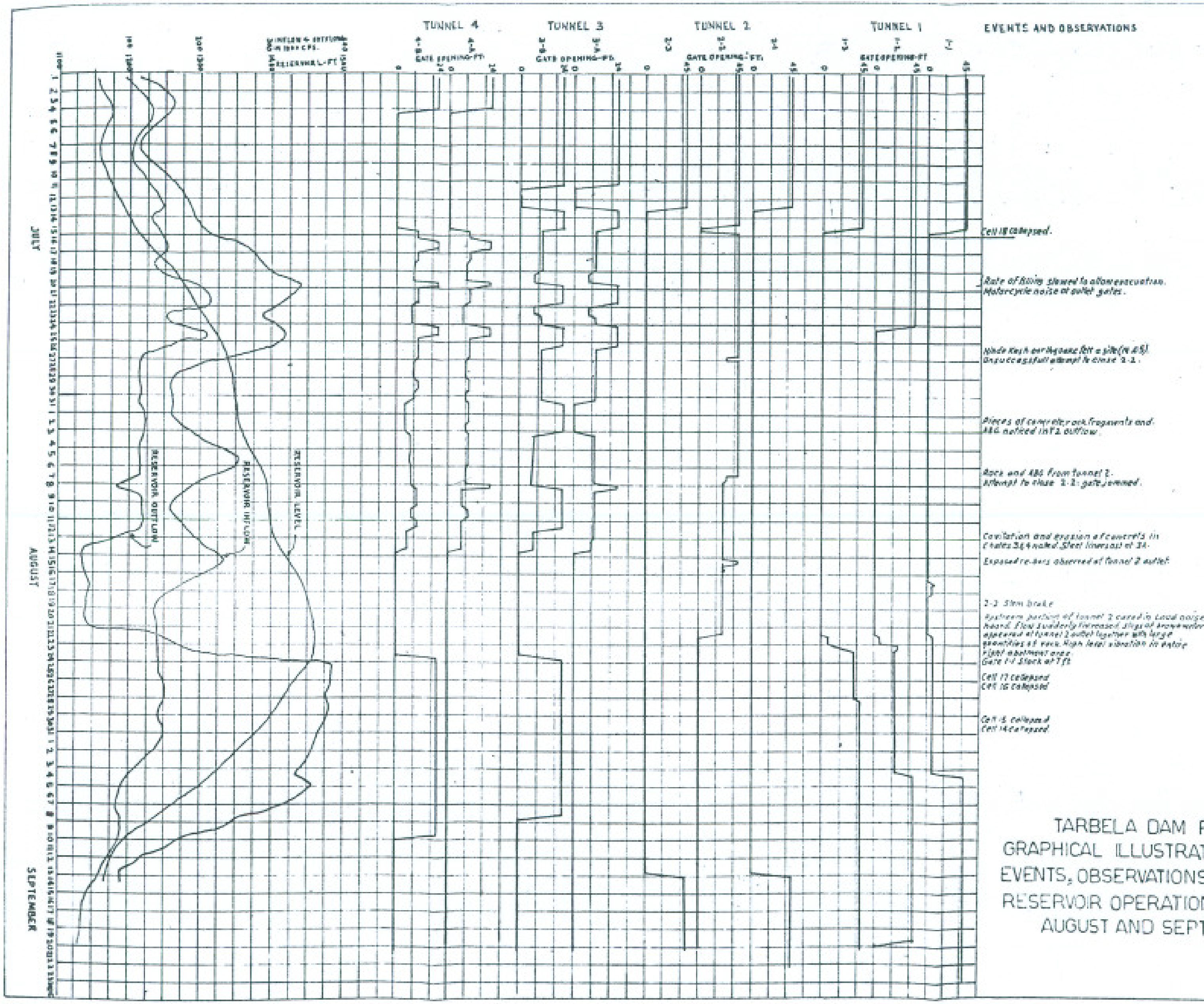
FIG:- 37- a

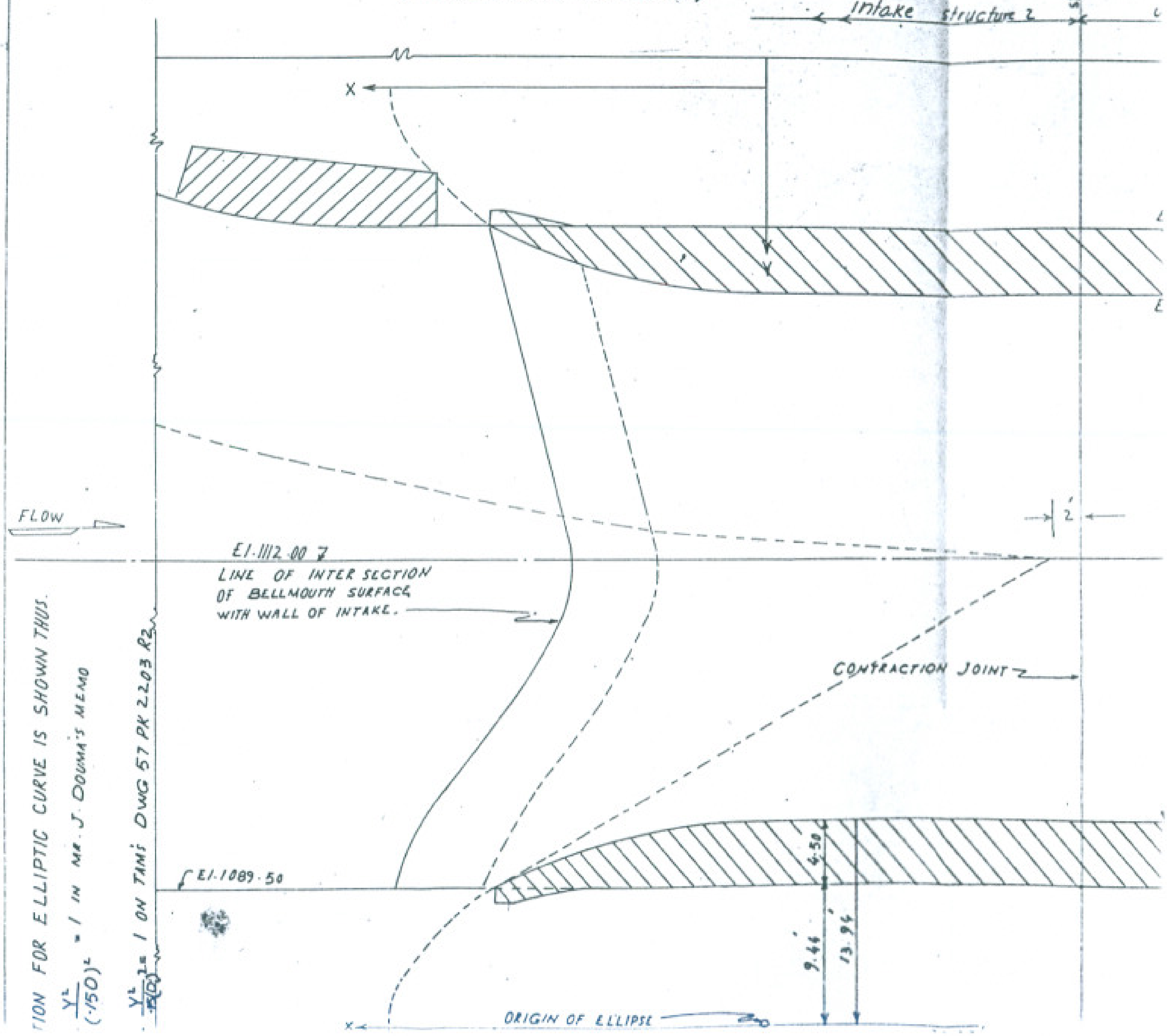


TARBELA DAM PROJECT
GRAPHICAL ILLUSTRATION OF MAJOR EVENTS, OBSERVATIONS, GATE AND RESERVOIR OPERATIONS DURING JULY AUGUST AND SEPTEMBER 1974

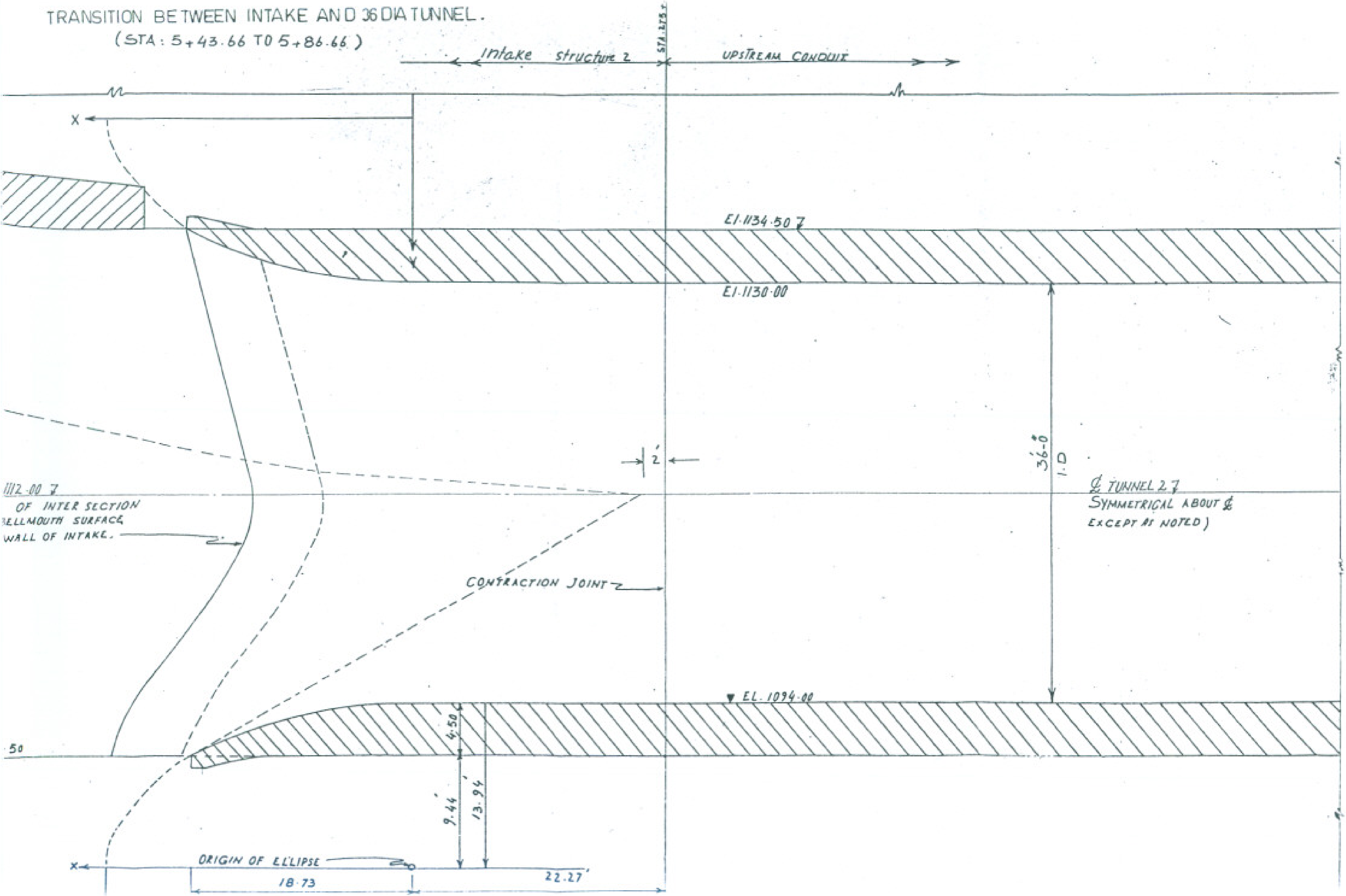
TARBELA DAM TUNNEL NO.2

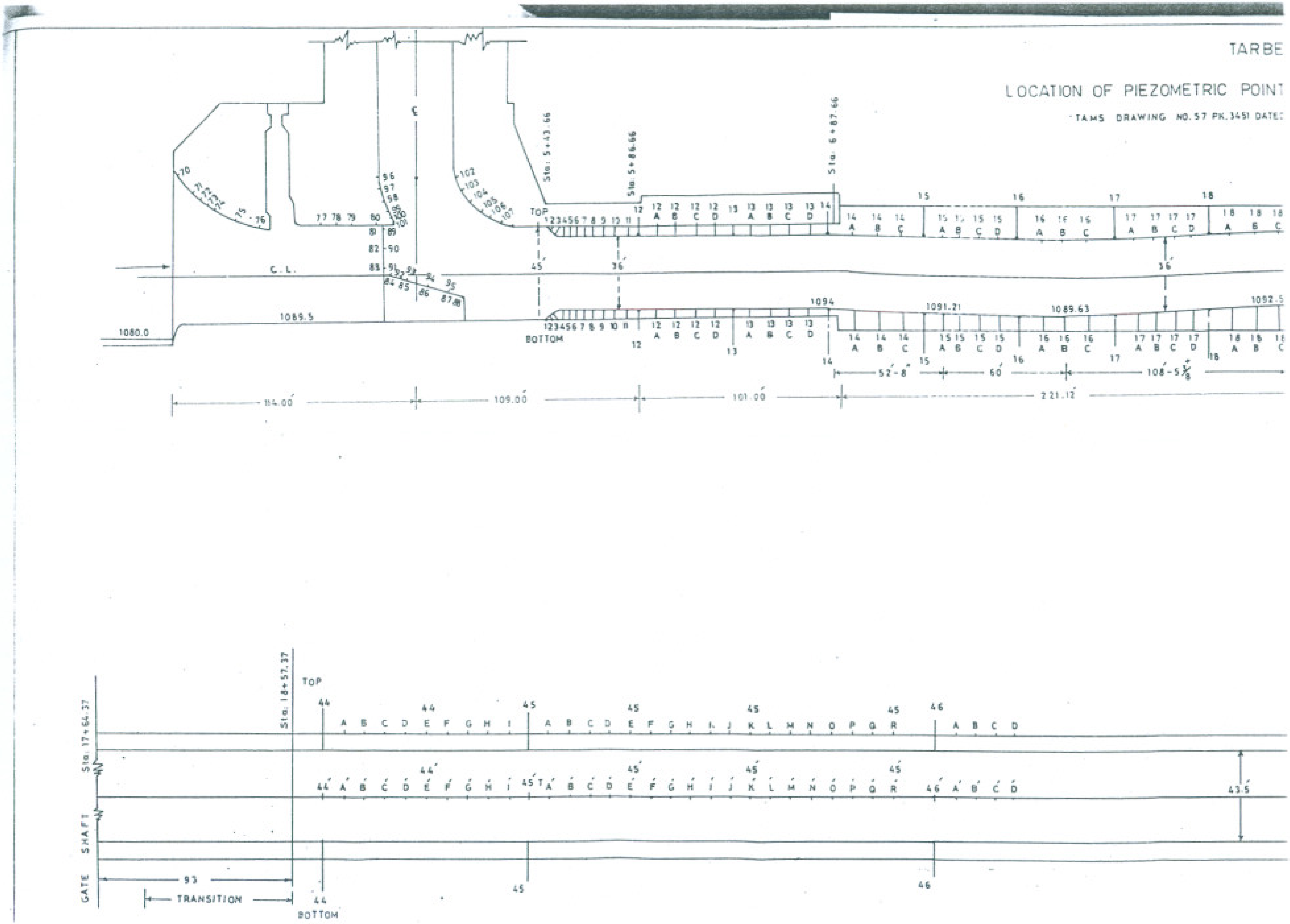
TRANSITION BETWEEN INTAKE AND 36 DIA TUNNEL.

(STA: 5+43.66 TO 5+86.66)



TRANSITION BETWEEN INTAKE AND 36 DIA TUNNEL.
(STA: 5+43.66 TO 5+86.66)



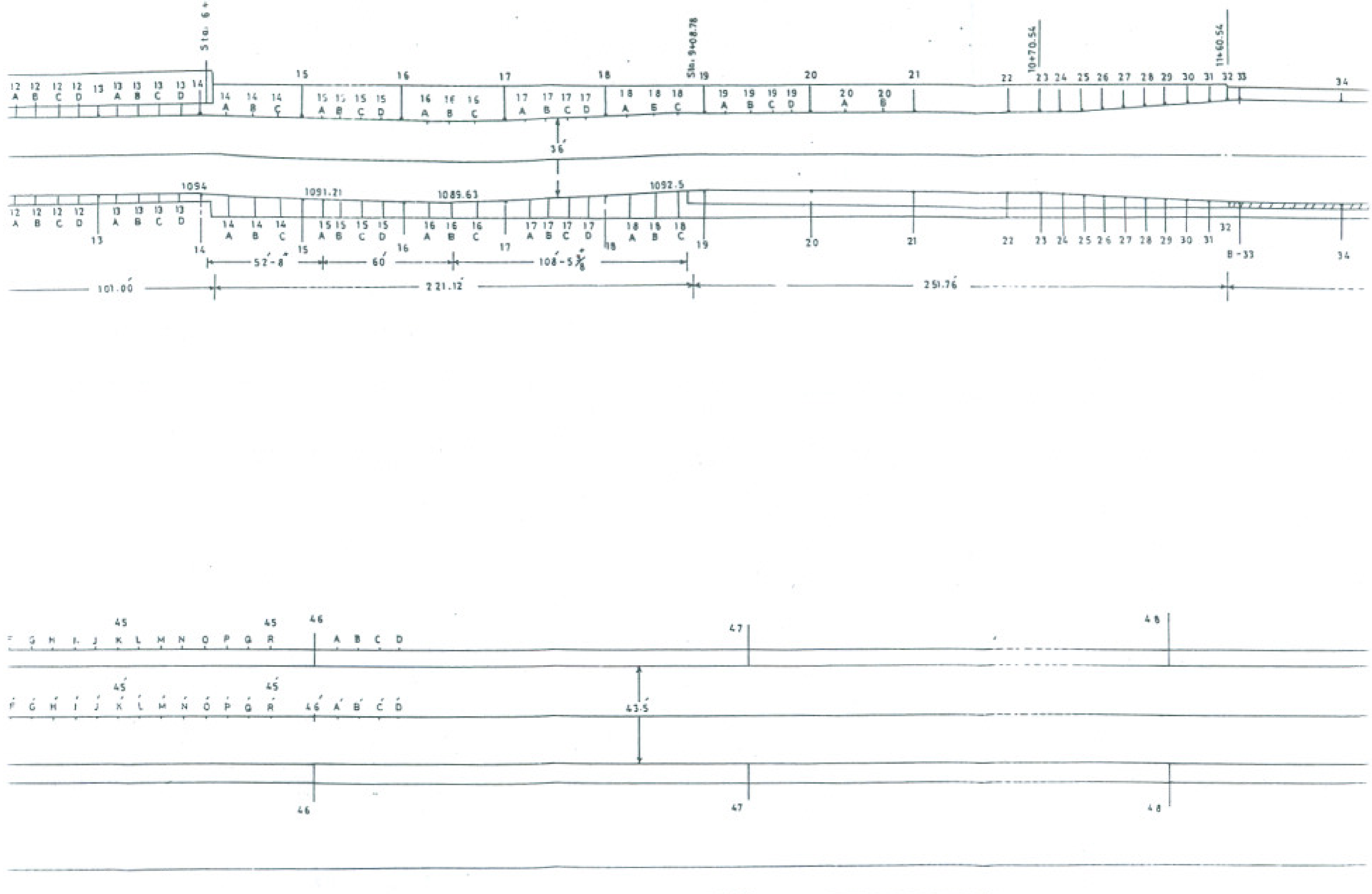

TARBE
LOCATION OF PIEZOMETRIC POINT
TAMS DRAWING NO. 57 PK. 3451 DATE
Sta. 5+43.66
Sta. 5+86.66
Sta. 6+87.66
TOP
BOTTOM
C. L.
1080.0
1089.5
1094
1091.21
1089.63
1092.5
52'-8"
60'
108'-5 3/8"
114.00'
109.00'
101.00'
221.12'
45'
36'
36'
Sta. 17+64.37
Sta. 18+57.37
GATE SHAFT
93
TRANSITION
43.5'

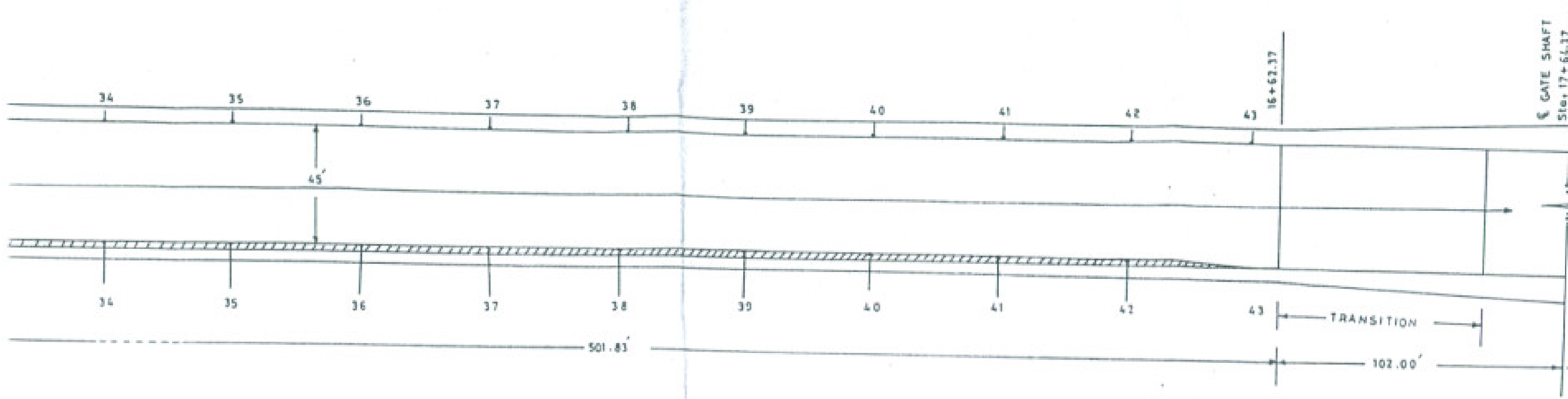
TARBELA DAM TUNNEL NO.2

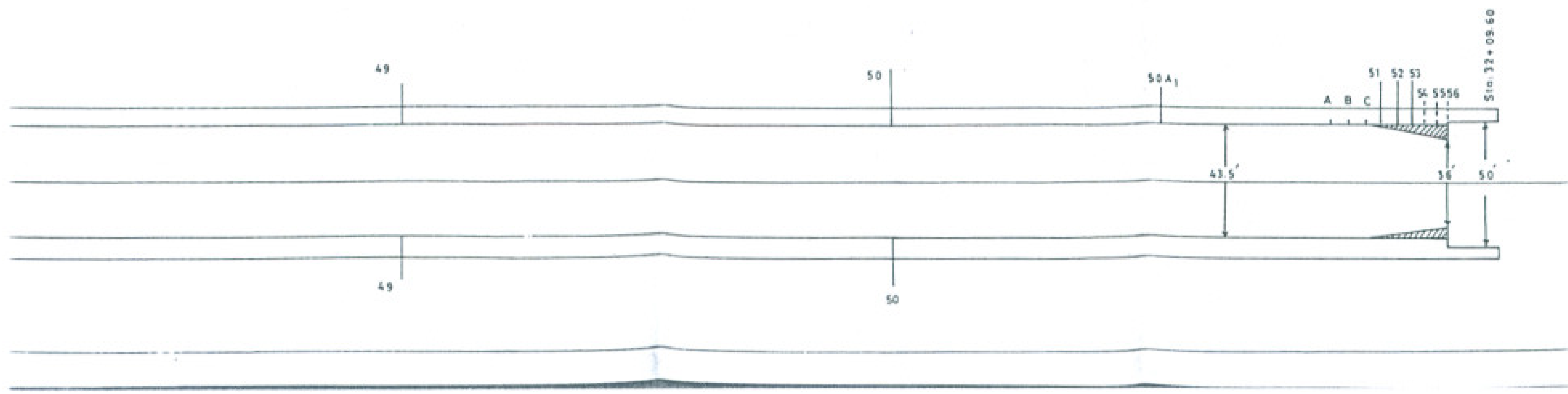
MODE SCALE=1/50

LOCATION OF PIEZOMETRIC POINTS ALONG THE TUNNEL SHOWN ON LONGITUDINAL SECTION

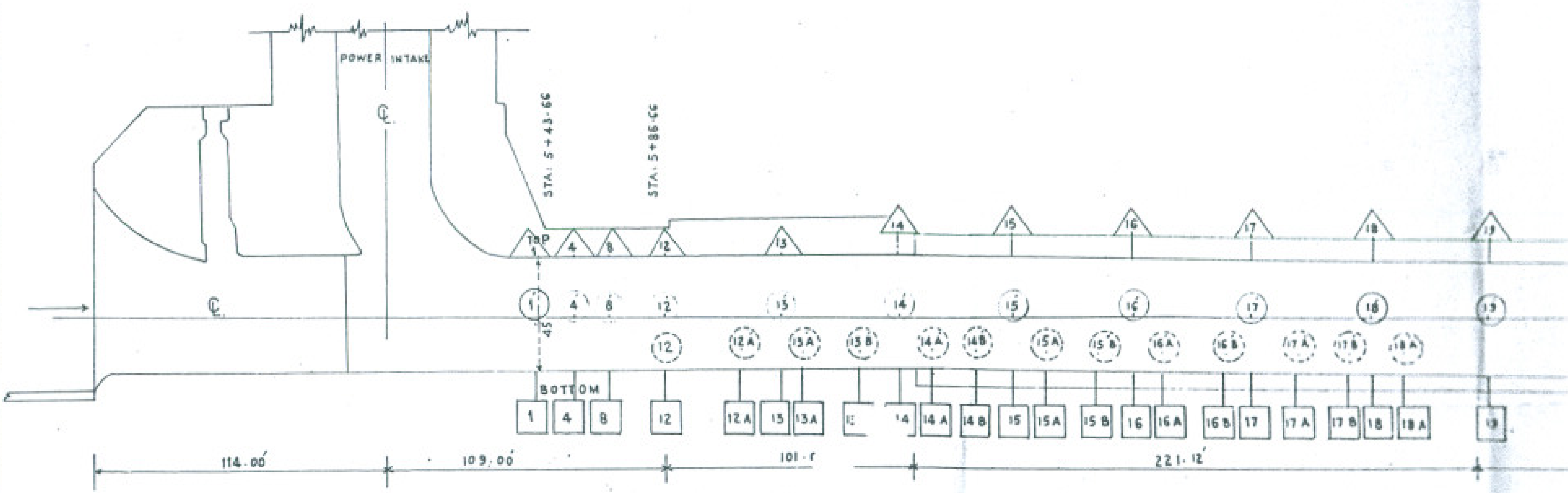
*TAMS DRAWING NO. 57 PK.3451 DATED NOV. 13, 1974 & NO.57 PK.3456 DATED JANUARY 11, 1975.




℄ GATE SHAFT
Sta. 17+64.37
16+62.37
GATE
CHAMBER
45'
TRANSITION
102.00'
501.83'
34
35
36
37
38
39
40
41
42
43


Sta. 32+09.60
49
50
50A1
A B C
51 52 53
54 55 56
43.5'
36'
50'

TARBELA DAM TUNNEL NO. 2
MODEL SCALE = 1/50
LOCATION OF PIEZOMETERIC POINTS ALONG THE TUNNEL
(ORIGINAL DESIGN)

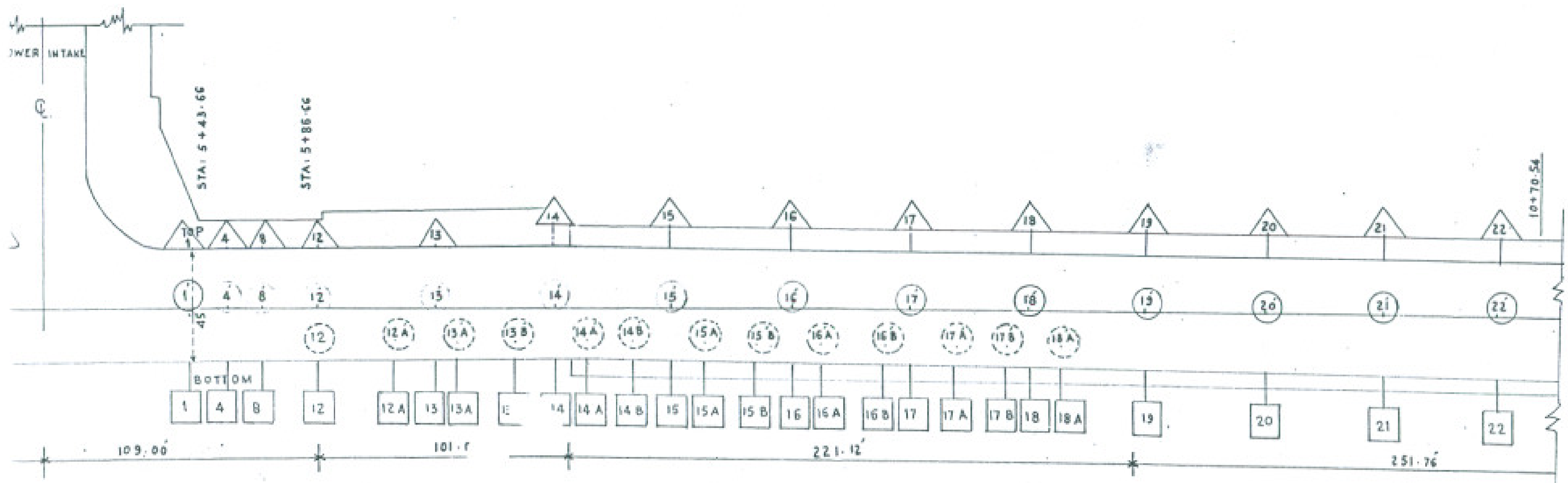


TARBELA DAM TUNNEL NO. 2

MODEL SCALE = 1/50

LOCATION OF PIEZOMETERIC POINTS ALONG THE TUNNEL

(ORIGINAL DESIGN)



feet per second and decreased after this peak, caused by a heavy rain, to 70,000 cubic feet per second. The maximum partial discharge in the channel to be closed was 22,500 cuhic feet per second. The water level upstream of closure dam rose from EL. 1119.5 to EL. 1128.6 by 9.1 feet. The water level downstream of the closure dam fell from EL 1119.5 to EL 1116.7 by 2.8 feet. The head cross was 11.9 feet. The crest elevation was increased during closure from EL. 1127.5 to EL. 1130.0, because the water rose more than expected. The crest width measured 95 feet and the length 500 feet. 35,900 cubic yards rocks as excavated and 440 cubic yards heavy boulders were used for the closure. Approximately 35% of the dumped material was washed away.

After completion the height of the closure dam was increased on a width of 30 feet to a level safe against over-topping by a flood with the discharge of 170,000 cubic feet per second.

Closure dam 2 has extended across island to the left river channel with cofferdam I-a safe against the above flood.

Closure dam of middle channel was made watertight in the same way as described for closure dam I-a.

B-2.3 Closure of left Channel.

The opposite bank i.e. left bank of left channel opposite closure bund was paved with 14,000 cubic yards of heavy boulders in a length of 500.

The construction of closure dam 3, closing the left river channel, commenced on the 8th. of October. The closure took 19 hours. The discharge of the river Indus decreased during closure dam 55,200 cubic feet per second to 51,200 cubic feet per second. The maximum discharge in the channel to be closed was 26,800 cubic feet per second. The water level upstream of the closure dam rose from elevation 1126.0 to elevation 1133.8 by 7.8 feet. The water level downstream of the closure dam fell from elevation 1126.0 to elevation 1121.2 by 4.8 feet. The head drop was 12.6 feet. The crest width measured 95 feet, the length 600 feet and the crest elevation 1136,5. 45,000 cubic yards rocks as excavated and 8,000 cubic yards heavy boulders between 1100 lbs. and 2200 lbs. were used for the closure. Only 6% of the material dumped was washed away, because the river bed in the closure bund area was well paved by nature with closely packed boulders. The average dumping rate was 2,800 cubic yards per hour.

After closure the closure dam 3 and the left bank were increased to a level safe against 170,000 cubic feet per second and made watertight.

B-2.4 Comparison of model and Prototype Results.

The water level in right channel at closure dam site at start of closure at a total river discharge of 63,300 cusecs was at EL. 1120.0 against elevation 1120 observed from model at a total river discharge of 60,000 cusecs.

The initial water level at start of closure in middle channel was observed to be at elevation 1119.5 at a discharge of 58,000 cusecs against 1119.2 observed from model at an incoming river discharge of 60,000 cusecs. The initial water level observed at site of closure in left channel was at elevation 1126.0 at an incoming river discharge of 55'200 cusecs against elevation 1126.6 observed on model at an incoming river discharge of 60,000 cusecs. Thus the agreement between model and prototype was close.

As already mentioned, the sequence of closure originally planned on the basis of model tests had to be changed at the last moment to take out the u/s rock plug at the entrance to diversion channel in dry conditions. The right channel which was originally scheduled to be closed after opening the diversion channel was closed without opening the diversion channel. An additional closure dam I-b had to be placed in the right channel u/s of the diversion channel mouth. The closure dam I-b was not removed before closing the central channel but was allowed to be washed away by flow and the residual cofferdam resulted in more discharge and higher water levels in the central channel. Moreover the discharge variation in the river during the closure was also excessive. It was 60,000 cusecs during closure of the right channel and increased to 90,000 cusecs during closure of central channel and again dropped to 50,000 cusecs during closure of the right channel.

The dumping rate of the material was 26000 to 2800 cubic yard per hour against anticipated rate of 1000 cubic yard per hour.

The hehaviour of the bed of three channel in relation to scouring was also different. The bed of the left channel lined by nature with boulders (Disc shaped placed at a uniform upturned angle) there was almost no scour even at the final stage of closure and the losses of stone were not more than 6%. The roughness of bed of diversion channel was also not truly represented on model.

All these factors had a significant influence on performance of model and the prototype. The model data as regards water levels head drop and material quantity is compared with the prototype data below :—

Right Channel.

	Prototype	Model	Difference
U/S water level EL.	1129.7	1128.1	+ 1.6 ft.
D/S water level EL.	1113.3	1114.0	— 0.7 ft.
Head drop elevation.	16.4	14.1	— 2.3 ft.
Quantity of material C. Yards.	69,900	52,000	+ 174000
Central Channel.			
U/S water level EL.	1128.6	1126.0	— 2,6 ft.
D/S water level EL.	1116.7	1116.1	+ 0.6 ft.
Head drop elevation.	11.9	9.9	+ 2.0 ft.
Quantity of material C. Yards	35900	30500	+ 5400
Left Channel.			
U/S water level EL.	1133.8	1132.2	+ 1.6 ft.
D/S water level EL.	1121.2	1121.6	— 0.4 ft.
Head drop elevation	12.6	10·6	+ 2.0 ft.
Quantity of material dumped in C. yards.	53000	95500	— 42500

The closure of left channel which was drawing 26800 cusecs at the time of closure (when the total incoming river discharge was 55,200 cusecs) did not present any particular problem as opposite bank was protected with heavy boulders, bed of this channel at this site was ideally paved by nature, average dumping rate was 2800 cubic yards per hour, heavy boulders between 1100-2200 lbs. were used for final gap, heavy stone was bouldozed from both banks when final gap was being closed and the diversion channel had a large capacity (750,000 cusecs) and its bed was at a level lower than bed of river at intake of diversion channel.

B-3.0 Model Studies on diversion channel—Buttress structure.

Purpose of Study :—

During stage II the river will be confined to newly constructed diversion channel. At the buttress dam the channel is 695 ft wide (corresponding to 27 buttresses at 5 ft each and 28 openings at 20 ft each). The channel contracts to a minimum width of about 375 ft some 2000 ft downstream of the buttress dam. At this point it was expected that the contractor will construct a haul road bridge with girders from the dis-assembled Indus River Bridge.

From a point about 180 ft downstream of the buttress dam a cellular cofferdam bounding the power house excavation forms the right side of the channel. Upstream of the cofferdam the right bank is excavated in rock and overburden with a 175 ft long warped transition from the buttress dam right abutment to the excavated slope.

The left bank of the channel between the buttress dam and contractor's bridge is excavated in overburden to a 1 on 3 slope. To protect this slope, which marks the west end of the stage I embankment, from scour, the basic design provided a wall 42 to 52 high running from the buttress dam downstream along the toe of the slope. The first 1800 ft of this wall is free standing. The remaining 500 ft of wall is backfilled. At discharges greater than about 600,000 cfs. (Diversion design discharge = 750,000 cfs) the free standing wall would be overtopped over the greater portion of its length.

In previous tests for stage II river diversion at Tarbela Dam site conducted in 1964 it was presumed that the bed and banks of diversion channel will be excavated in rock. The actual excavation of the channel at site indicated that banks of diversion channel for most of length of diversion channel in the head reach are in over burdens Fig—7 and this new feature was incorporated in the model in 1970 studies.

The studies were conducted during 1954 and again in 1970 on 1/80 and 1/60 scale natural scale models. At first, the scope of the study was limited to the measurement of water levels to provide structural design data. Initial observations, however, led to a broadening of the scope of the programme to include the study of modifications to the project features appertinent to the diversion channel.

The observations and the recommendations from these studies are summarised below :—

(*a*) The coarse material stays in the river bed whereas only finer material from the shoal at in-take of diversion channel enters the diversion channel which passes down the buttress structures.

Erosion of Diversion Channel Bank along Kaira Camp.

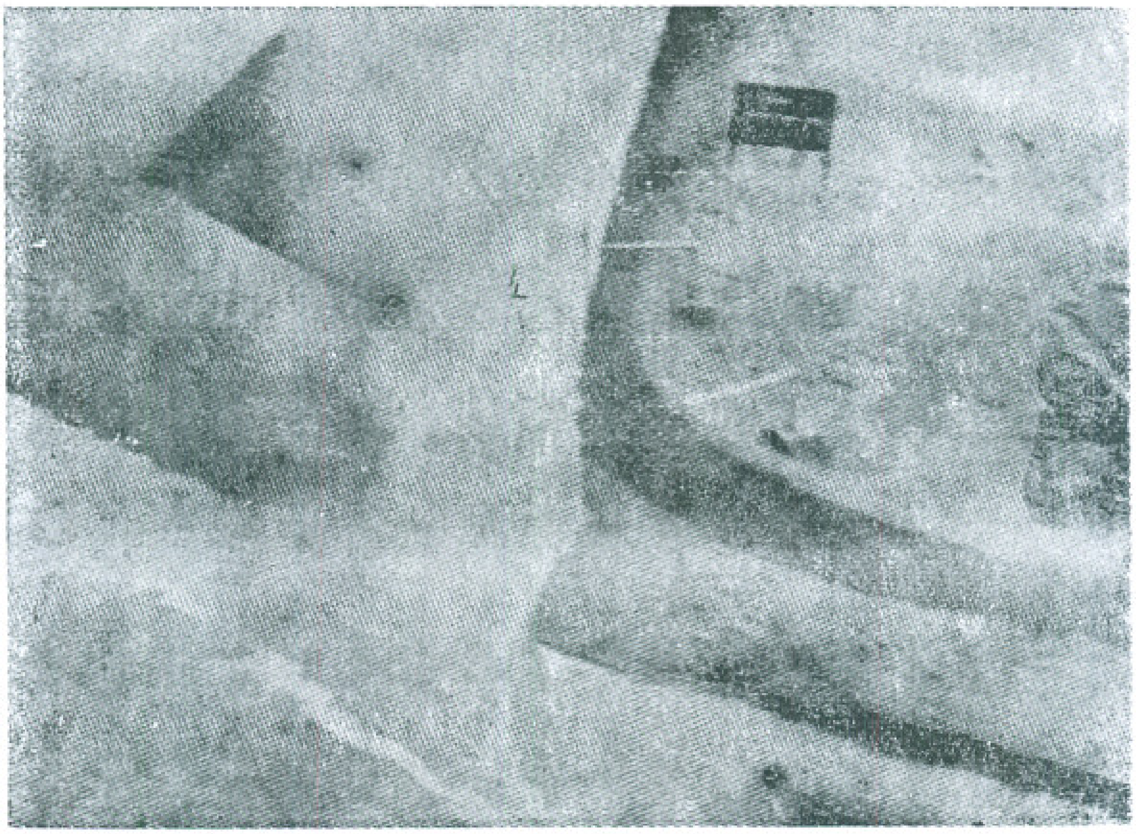
(*b*) On the right bank of the diversion channel T.J.V. constructed a camp which was provided against high water with a cofferdam. The bank erosion along Kiara Camp was discernable between discharges of 200,000 and 500,000 cusecs. At discharge about 500,000 cusecs bank gets submerged and erosion becomes insignificant. The erosion is maximum at river discharges 200,000 to 400,000 cusecs. The velocity along the bank is maximum at a river discharge of 400,000 cusecs and the order of maximum velocity is 15.9 fps. The finer part of material forming the bank gets washed out but the heavier component stays and paves the slope of the banks at a flat slope which varies from point to point. There is no significant erosion of outer bank of diversion channel in the vicinity of Kiara Bank.

Erosion of Diversion Channel.

(c) The flow is confined to the diversion channel upto 400,000 cusecs. At river discharge of 200,000 cusecs and upto bank full discharge the finer material in the banks gets washed down by high velocity flow which results in slipping of channel bank in areas where the channel is in loose material, The slipping was between stations 25+00 to 50+00 on left bank station 20+00 to 50+00 to and 65+00 to 80+00 on the right bank. The magnitude of erosion was maximum between sta 65+00 to 80+00 of right bank. The slipped banks get stabilized due to paving by heavier material. Flow over the high ground between left bank of diversion channel and the coffer dam 'C' starts above 400,000 cusecs and at 500,000 cusecs and above the flow enters the diversion channel through tunnels leading to diversion channel Fig 800. At 600,000 cusecs north West Corner of coffer dam 'C' is subjected to attack from concentrated flow (flowing all along the coffer dam 'C' but gaining velocity while approaching this corner) returning to the diversion channel. The flow leaving the North West Corner of coffer dam 'C' and entering the diversion channel at its left bank and the flow sweeping out from right bank embayment between sta 65+00 to 80+00 meet together to improve the flow distribution at buttress Structure. There is some deposition along the left bank in its head reach and in the introdes of the band at end of one hydrograph peaking at 400,000—500,000 cusecs. Some of the material eroded from outer bank of the channel deposits opposite the tunnel intake here the channelhas widened due to curved alignment of tunnel intake plug, No deposition in the bed of rocky portion of diversion channel was noticed at the end of hydrograph peaking at 750,000 cusecs.

Junction at Tunnel intake Channel

(d) During stage II diversion the tunnel intake channel will be closed off at its junction with the diversion channel by a rock plug left in place along the diversion right

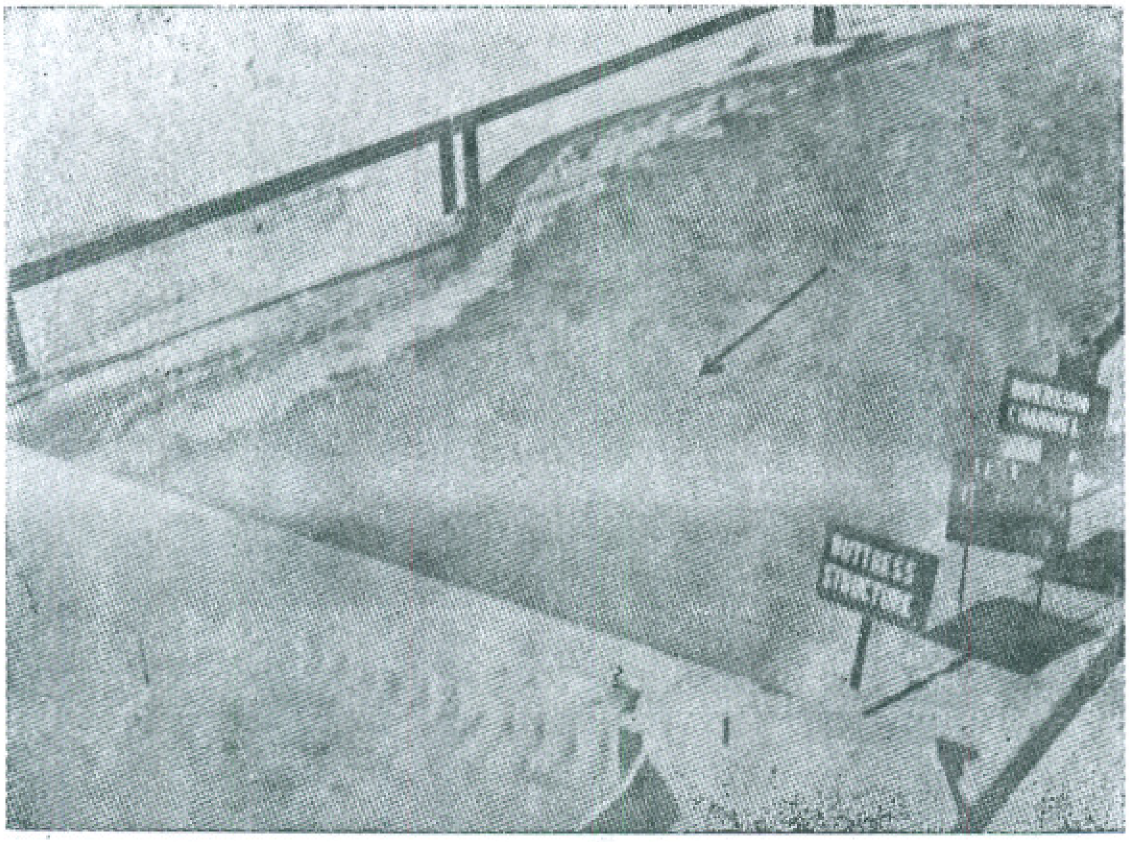


TARBELA RIVER DIVERSION MODEL

Scale - 1/80

Flow along extreme end of cofferdam 'C'

Fig. 8-a



TARBELA RIVER DIVERSION MODEL

Scale = 1/80

Test for junction at tunnels intake channel. Flow conditions along plug at discharge = 600.000 cfs.

Fig. 8-b

bank. In the original design the layout of plug at the junction was not stream-lined. Apart from visible flow disturbance, an intermittent surging action with as much as 16 feet of water level fluctuation at a design flow of 750,000 cusecs was observed. The surge was reflected back into the main channel. The junction was streamlined to reduce the surge produced by the branch channel. With the junction modelled in final form the flow conditions were found to be satisfactory. fig.—8-b.

The Conveyor Bridge Pier:

(*e*) The contractor erected an askew bridge pier just u/s of the buttress dam to carry the conveyor from the main barrow areas across the diversion channel. The pier on the right located closer to the buttress dam and in the area of highest velocities disturbed the flow more than on the left. Streamlining the pier to proper shape reduces the intensity of disturbance.

(*f*) A severe eddy at the square right abutment of the buttress-structure produced scour hole. A 175′ long warped transition from the right abutment to the right excavated slope eliminated the undesirable eddy.

(*g*) To stabilize the askew hydraulic jump over the d/s apron of Buttress structure it was finally decided to depress the apron by 5 feet as if it is not depressed the hydraulic jump moves d/s off the apron. (Fig. 9, 10).

Left Channel Wall:

(*h*) In view of the observed concentration of flow at the right side of the channel it appeared possible that the velocities on 1:3 sloped left bank of the channel d/s of buttress structure without the wall might be low enough to permit the rip rap protection. With the wall omitted the velocities on the slope at 600,000 cusecs (fig. 11) where not more than 17 fps which increased to 21·8′ near left u/s transition to proposed contractor's bridge 20,000′ d/s of buttress structure. The zone (A) material (maximum size 4000 lb.) specified for the main and auxiliary dams would be sufficient over most of the length of the slope except at the extreme d/s end where rip rap Type 'B' (maximum size 6,000 ft.) could be used.

(*i*) At the left abutment of buttress dam if 150′ length (battered to 1:4 at d/s end) of left wall is retained an eddy on left side of buttress structure which feeds a return flow back into the left end bay of the buttress structure is completely eliminated.

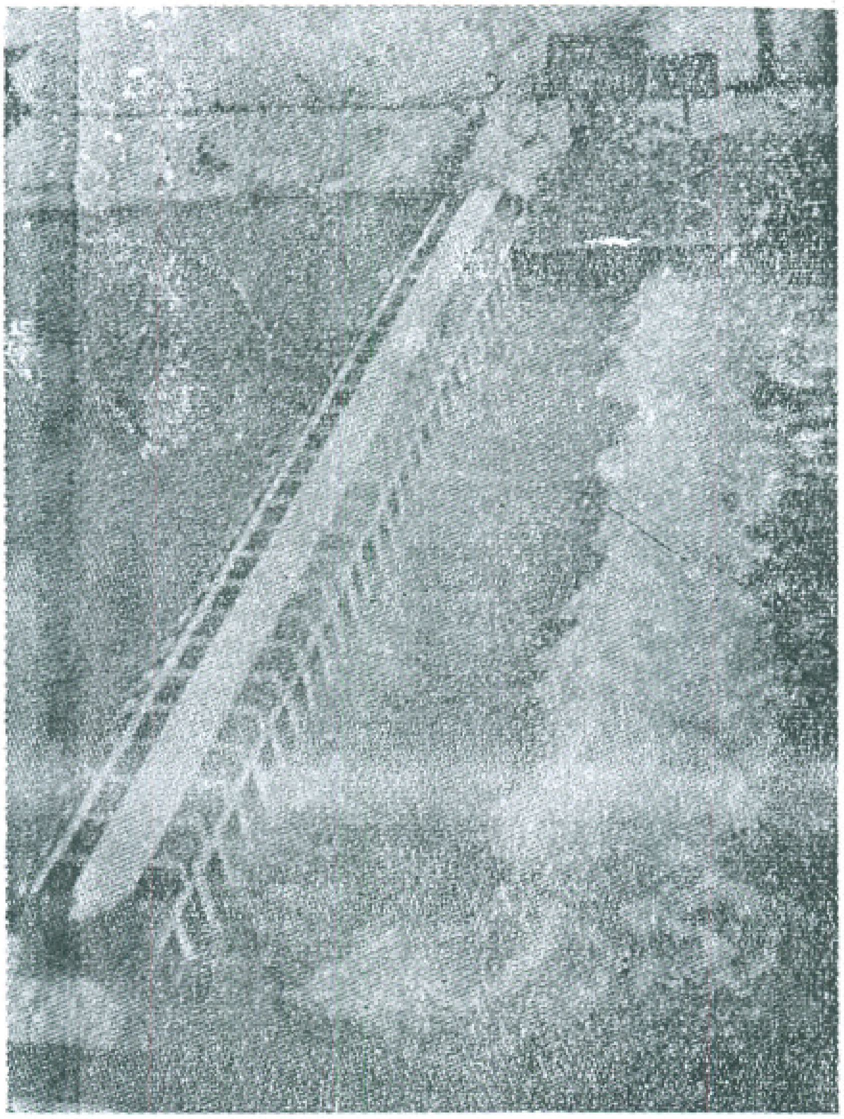
(*j*) With a 200 feet radius of curvature the transition section u/s of the proposed Contractors bridge gives satisfactory hydraulic performance.

(*k*) The proposed contractor's bridge was located 2,000′ d/s of the Buttress Dam and orientated normal to the diversion channel base line. With the bridge installed in the model, it becomes evident that to avoid large differential heads and excessive disturbance of flow the piers would have to be skewed to align these parallel to flow (optimum skew angle was found to be 9°). The flow through the piers is shown in Figure 12 (a) whereas the surface profile through the piers is as shown in Figure 12 (b). The contractos's bridge over the diversion channel 2000′ d/s of buttress structure was dropped ultimately.

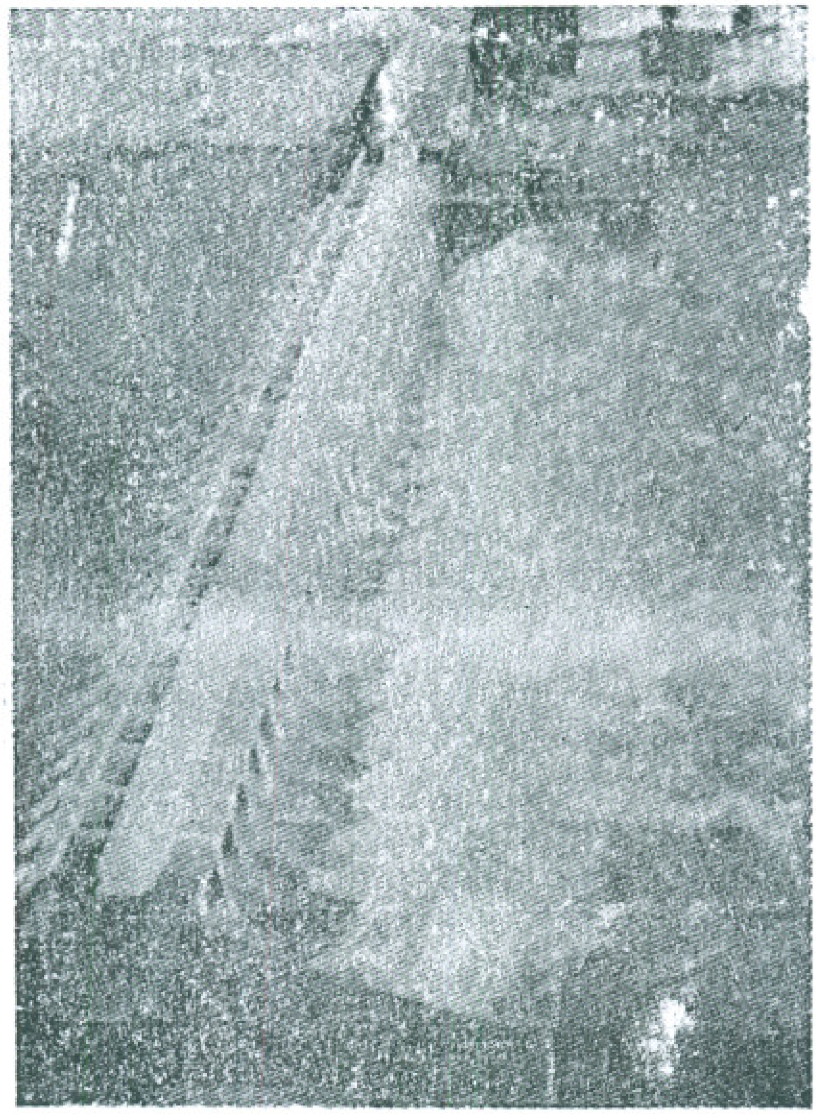
The hydraulic performance of the diversion and the buttress dam as constructed at site presented no serious problems needing discussion.

B-5. Experiment on flow leaving the diversion channel.

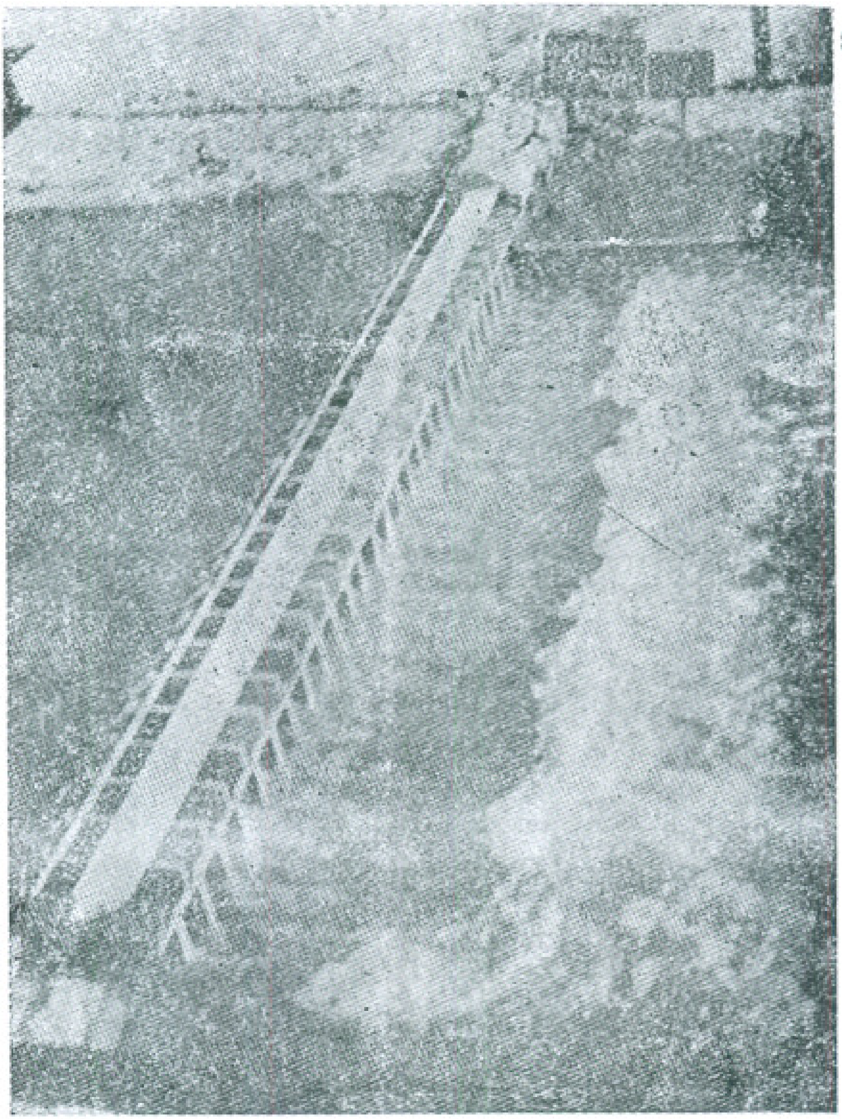
Four different positions of cofferdam E_1, E_2, E_3, E_4, (figure 13) were tested and finally on the basis of the data on high velocity of flow hugging along the face of the cofferdam for the shortest distance, the cofferdam alignment E_4, was recommended. Protection for a limited length of about 1000 feet were recommended. No problems were experienced.



TARBELA RIVER DIVERSION MODEL
Scale = 1/80
Tests for Buttress Dam Apron.
Flow conditions at discharge = 750000 cfs.
Apron horizontal at EL. 1090.
Fig. 9



TARBELA RIVER DIVERSION MODEL
Scale = 1/80
Tests for Buttress Dam Apron.
Flow conditions at discharge = 750,000 cfs.
Apron depressed to EL. 1085
Fig. 10



TARBELA RIVER DIVERSION MODEL
Scale = 1/80
Tests for Buttress Dam Apron.
Flow conditions at discharge = 750000 cfs.
Apron horizontal at EL. 1090,

Fig. 9

TARBELA RIVER DIVERSION MODEL
Scale = 1/80
Tests for Buttress Dam Apron.
Flow conditions at discharge = 750,000 cfs.
Apron depressed to EL. 1085

Fig. 10

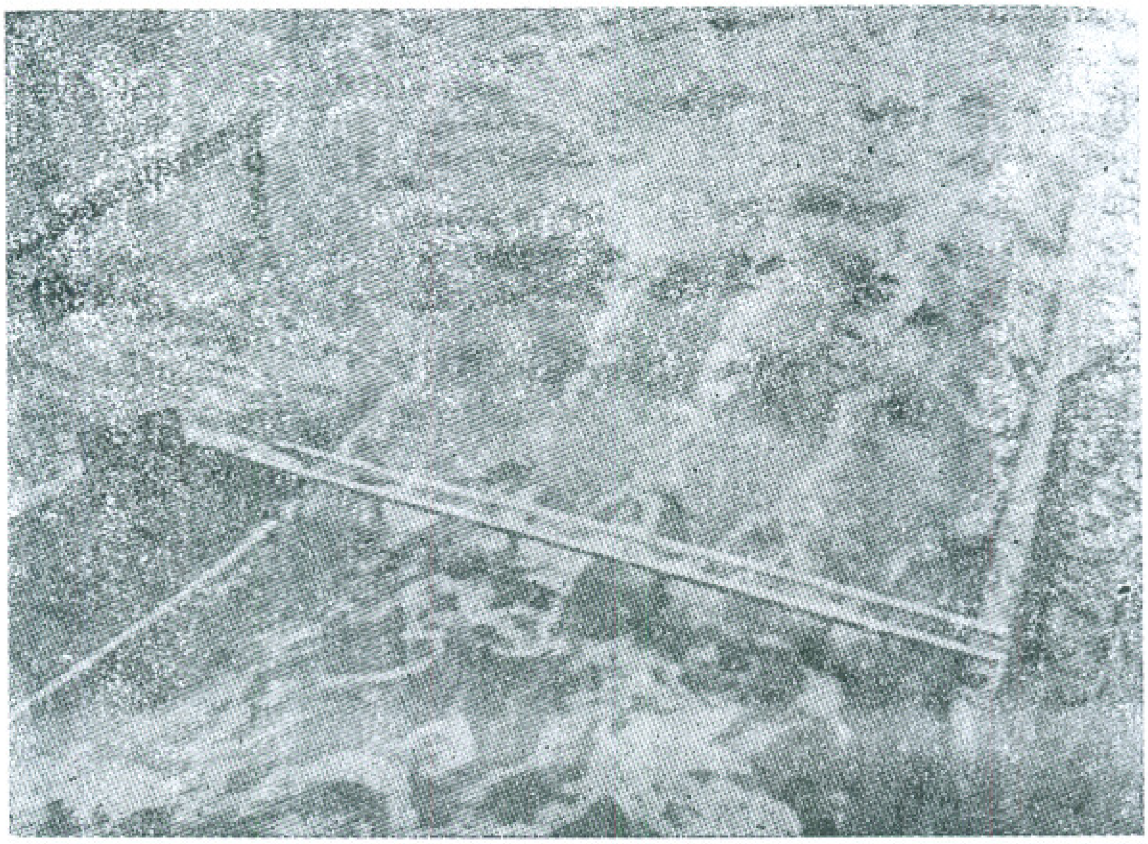
C-1.0 Experiments on power and Irrigation Tunnels their intake and outlet structures for stage III diversion

The four power and irrigation tunnels will serve for river diversion during the final stage of construction. The Stage III diversion will be achieved by removing the cofferdam just upstream from the tunnel portals and then permanently closing all gates of the Buttress Structure. The Tunnels follow curved alignment.

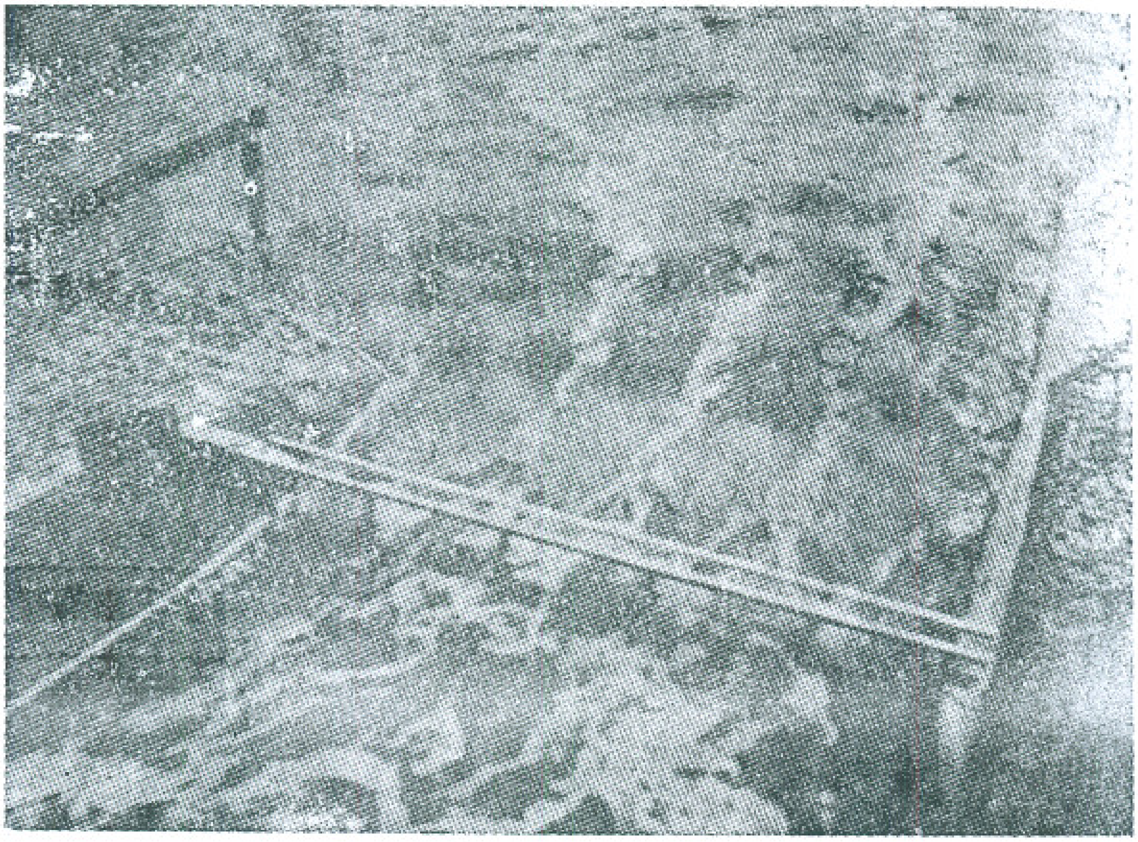
All the tunnels from intake to the Central gate chamber are 45 feet in diameter with transition for changes in shape from rectangular to circular and *vice versa*. The finished surface will be of concrete. Downstream from the central gate chamber Tunnel 1,2 and 3 which eventually serve as power penstocks, are fitted with steel liners having an inside diameter of 43.5 feet. Tunnel 4 which is strictly for irrigation releases, will be 36 feet in diameter. By means of a Y-branch and appropriate transition near the downstream end, tunnel 3 and 4 each terminate in two rectangular conduits 16 feet wide by 24 feet high. Each of the four rectangular conduits are fitted with a radial gate for regulation of the discharge. Downstream from the radial gates the flow expands over chutes leading to stilling basins provided for tunnel 3 and tunnel 4.

The stilling basin for tunnel 4 is to be a permanent structure while that for tunnel 3 will require partial dismantling at a later date, at which time this conduit will be extended to supply water to the last four units of the power house. In addition to the radial gates, 3 and 4 will each be initially equipped with two gates in central gate chamber. Tunnel 1 and 2 on the other hand, will operate uncontrolled as piers and gates were not scheduled for installation in the central gates chambers untill the tunnels are no longer needed for diversion. During Stage III diversion, the work will be in progress on the first four power house units.

Provision must thus be made to convey the high velocity flow from tunnels 1 and 2 to the river without interrupting work on the power house and without damage to the stilling basin of Tunnel 3. Towards the close of the fiood season of Stage III, the intake gates of tunnels 1 and 2 are scheduled to be



TARBELA RIVER DIVERSION MODEL
Scale = 1/80
Test with contractor's Bridge.
Flow through piers at discharge = 750,000 cfs.
Fig. 12-a



TARBELA RIVER DIVERSION MODEL
Scale = 1/80
Test with contractor's Bridge.

Flow through piers at discharge = 750,000 cft.
Fig. 12-a

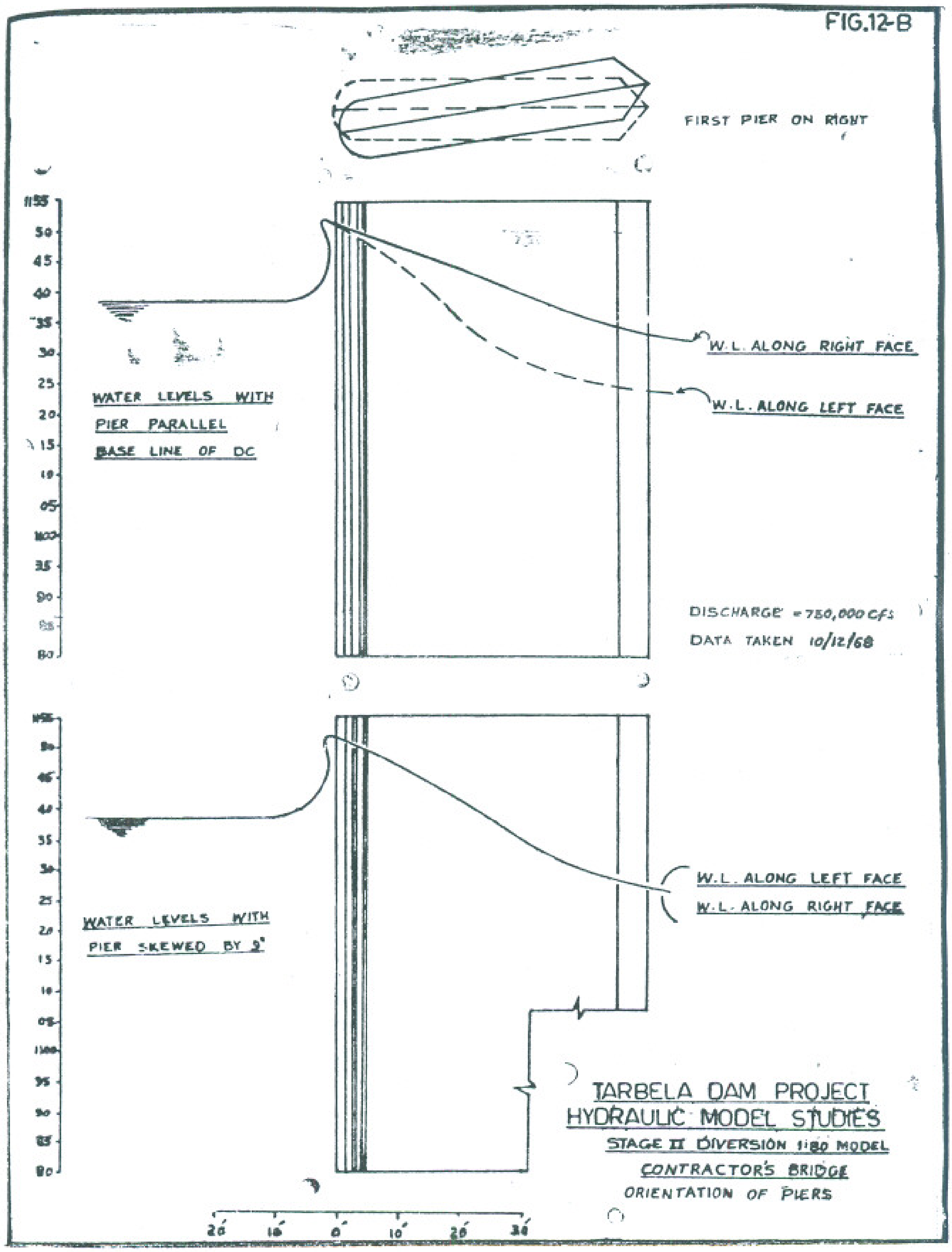


FIG.12-B
FIRST PIER ON RIGHT
W.L. ALONG RIGHT FACE
W.L. ALONG LEFT FACE
WATER LEVELS WITH
PIER PARALLEL
BASE LINE OF DC
DISCHARGE = 750,000 cfs
DATA TAKEN 10/12/68
W.L. ALONG LEFT FACE
W.L. ALONG RIGHT FACE
WATER LEVELS WITH
PIER SKEWED BY 9°
TARBELA DAM PROJECT
HYDRAULIC MODEL STUDIES
STAGE II DIVERSION 1:80 MODEL
CONTRACTOR'S BRIDGE
ORIENTATION OF PIERS

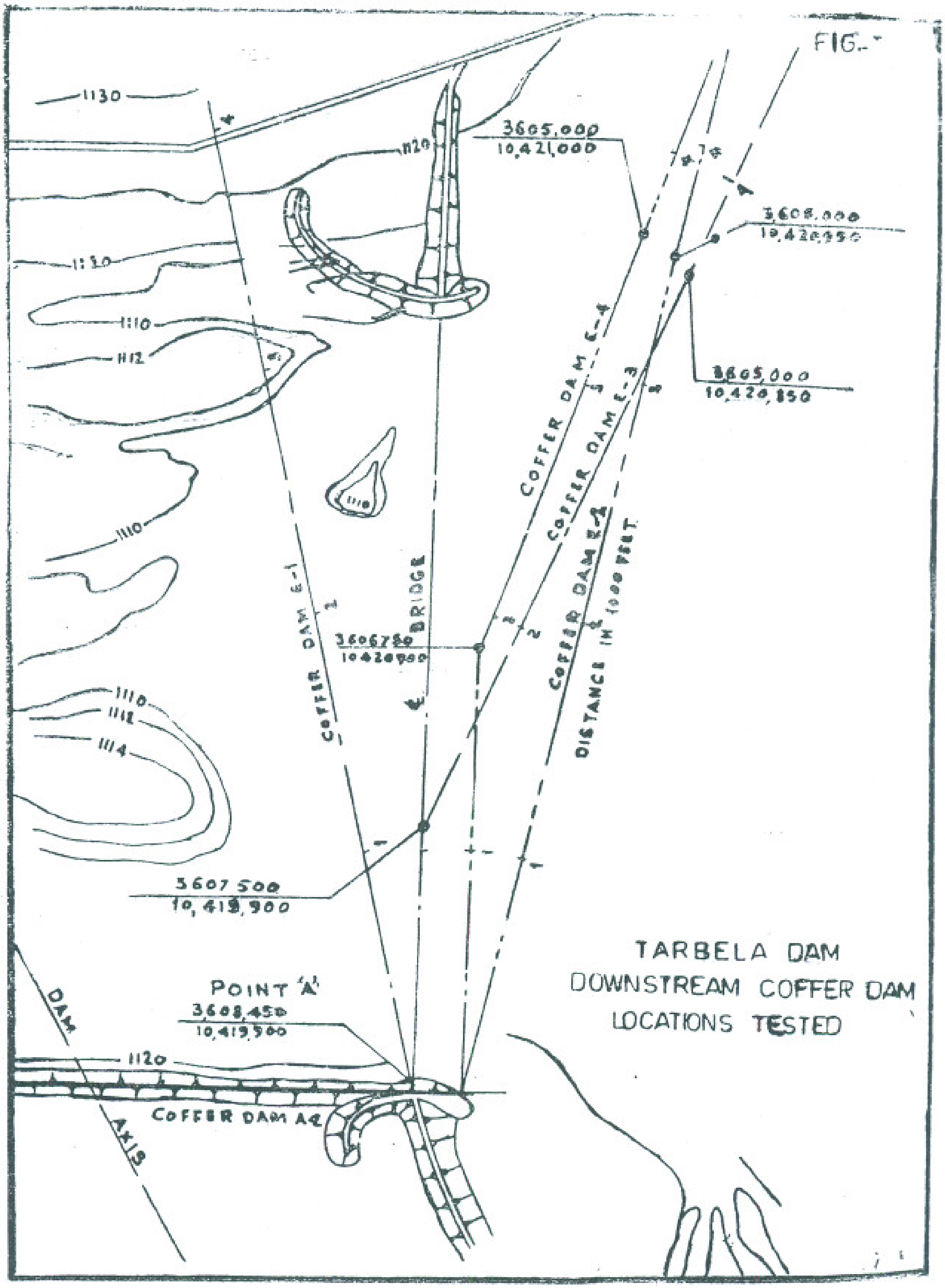


FIG.
1130
1120
1110
1112
1114
3605,000
10,421,000
3608,000
10,420,950
3605,000
10,420,850
3606750
10420900
3607 500
10,419,900
POINT 'A'
3608,450
10,419,500
COFFER DAM E-1
BRIDGE
COFFER DAM E-4
COFFER DAM E-3
COFFER DAM E-2
DISTANCE IN 1000 FEET
DAM AXIS
COFFER DAM A2
TARBELA DAM
DOWNSTREAM COFFER DAM
LOCATIONS TESTED

closed, after which the piers and gates will be installed in the central gate chambers. Normally water releases after this time will be handled entirely by Tunnels 3 and 4, the main embankment of the dam will be nearly completed and the spillways will be ready to accommodate excess flow.

C-2.0 Model Tunnels.

The hydraulic model studies for Stage III diversion on tunnels 1 and 2 (later to be used as power tunnels) and irrigation Tunnels 3 and 4 relating to different problems as visualised or faced by the Engineers at planning, design, construction and operation stages were carried out at the Irrigation Research Institute, at Lahore and Nandipur and at CSU Laboratories at Fort Collins as desired by the consultants with the following distribution :—

(i) Studies in the pre-detailed design stage were carried out on the tunnel intake structures, outlet flow condition of Tunnels 1 and 2 and performance of stilling basins of tunnels 3 and 4, at Irrigation Research Institute, at Lahore and Nandipur (1964-1965).

(ii) Studies for the detailed design of inlet, outlet and the central service gate chamber and general flow conditions through the tunnels were carried out at CSU, Fort Collins, (1964-1972).

(iii) Post damage model studies were carried out at Irrigation Research Institute, Lahore and CSU Laboratories Fort Collins (1974-1975). The results of tests carried out at Irrigation Research Institute (1964-1965) and CSU (1964-1972) will be summarised and then the prototype behaviour and damages will be briefly discussed high lighting the similarities and damages arising as a result of ignoring the model recommendations. The results of post damage model studies will then be discussed indicating possible causes of the damages.

C-2.1 Model studies at Irrigation Research Institute for flow conditions at the Intake and outlet of Irrigation and Diversion Tunnels—Stage III Diversion

The studies carried out at Irrigation Research Institute, on stage III Diversion (Tunnel 1 to 4) relate to predetail design period and involved the following features only :—

(a) Finding a workable solution to the problem of conveying the uncontrolled high velocity flow from Tunnels 1 and 2 to the river downstream without entailing work stoppage or damage to existing structures.

(b) Observation of flow to the tunnel intake structures.

(c) Hydraulic performence of stilling Basin for tunnel No. 3 and 4.

C-2.2 The Models.

Outlet Model.

Two models, the tunnel outlet Model (Scale 1:60) and the Geometric River Model (Scale 1:84) were utilized for the studies performed at the Nandipur Hydraulic Research Station. The first, with a scale ratio of 1:60, was constructed specifically to study the effluent problem downstream from Tunnels 1 and 2.

The 1:80 scale geometrical river model was used to observe the approach flow to the four tunnel intakes as well as to study the manner in which the discharge from the four tunnels re-entered the river for all likely conditions of operation.

C-2.3 Model Investigation and Results.

The preliminary tests showed that the exit tunnel velocities as experienced from the model can not withstand much high velocities for any length of time since the rock in the prototype between the power house and tunnel 3 stilling basin being highly fractured. With velocity of this order irregularities in surface could be a source of cavitation by which the process of erosion would be accelerated. In the model with head water elevation at 1250 even after a short period of running, practically the whole area was scoured extending down to as deep as EL. 930. It was, therefore, decided early that surface in contact with the high velocity flow be paved with good concrete.

At pre-design stage, it was considered unlikely that the headwater would exceed Elevation 1300 during Stage III Diversion, with all four tunnels operating. Later a tunnel operating and closing procedure designed to produce a full or partly full reservoir at the completion of stage III diversion came under consideration. It was suggested that if such a procedure is employed the reservoir level during stage III diversion may exceed 1420 feet with four tunnels open. Further model studies were carried out to assess the effects of these higher headwaters on flow conditions in the tunnel outlet area.

The testing of tunnels 1 and 2 outlet area involved numerous schemes and modifications. The final arrangements tested for outlet area is shown in Fig. 14. The area in between power house well, left bank of stilling basin of T_3 was paved upto end of stilling basin of Tunnel No. 3 & 4. The floor level of outlet area of T_1 & T_2 near the outlet end was at R. L. 1078 in a length of 77 feet and then lowered to R. L. 1071 in a slope of 1:12. The length of level floor of 1071 is 253 feet which is again lowered in a slope of 1:12 to RL 1064 which was a length of 470. The loose material in diagonal draw was taken out and filled with concrete. The natural table work downstream was removed to EL. 1080. A part of curved cellular coffer dam forming the right bank of diversion channel stage II was retained up to cell No. 18, and thus the back current (which unbalances the main flow) in the wake of 4 unit power house was confined to minimum possible area-

The flow emerging from tunnels 1 and 2 impinges at an angle to the left stilling basin walls of tunnel No. 3 resulting in overtopping of flow into the stilling basin of tunnel 3. The detailed fluctuating water levels were observed on the two sides of the stilling basin wall of tunnel No. 3 for different reservoir levels and different comibinations of flows from tunnels 1 to 4 to work out the probable maximum head across at different points along this wall. The average and surging pressures were observed at different points on the face of left stilling basin wall of tunnel 3, along the power house wall and bed of the outlet area of tunnels 1 and 2 with different operation conditions of reservoir and tunnels. The maximum fluctuation in intransient pressure was of the order of 62′ head of water.

The maximum head across the left wall of stilling basin for tunnel No. 3 as worked out from water levels was quite more than worked from transient pressures. The detailed velocities were observed along the bed of paved area below tunnel 1 and 2 to design the slab free of cavitation.

C-3.0 Model Studies for Approach to Tunnel Intakes (1:80 Geometric River Model).

Approach flow to tunnel intakes was observed for head water elevation ranging from 1090 to 1490 with no control on any of the four tunnels. The flow as visually observed appeared satisfactory for the full range of operation, however, a vortex was occasionally observed near the intake of tunnel 1 and 2 for head water elevation between 1160 and 1280. Above Elevation 1280 the vortex vanished if all the four tunnels were in operation. No vortex appeared at the intake of tunnels 3 and 4.

C-3.1 Model Studies on Stilling Basins of Tunnels 3 and 4 at Irrigation Research Institute, Nandipur (1964).

The model studies relating to pre-detailed design period were carried out at Nandipur on the performance of tunnels 3 and 4 on two models to a scale of 1:60 and 1:80 described earlier.

C-3.2 The Model Results and Conclusions :

The observations and conclusions from the preliminary tests are recorded below :—

(*i*) There was essentially no difference in the operation of the stilling basins with vertical of 1:12 battered walls. In both cases the jump was rough for the higher head water elevations. The velocity entering the basins was about 135 fps and the Froude number was 8.5 for the maximum headwater.

(*ii*) The structures appeared short, but the jump, although quite rough, remained within the basin for the tailwater range (1105 to 1120) tested. The jump was repelled off the stilling basin at RL 1104.

(*iii*) It appears that scour downstream of the structure will pose no particular problem.

(*iv*) A matter of some importance noted from these tests was that bed material carried into the basin could not entirely escape due to the high vertical end cill. The entrapped material circulated about the basin and in the prototype could in time subject the concrete surface to abrasion. Velocities near the bottom were in an upstream direction about one third of the time which provided an opportunity for bed material to enter the basins intermittently. One way to keep loose material out of the basins entirely may be to remove it from the vicinity of the basins during construction. It was considered that a sloping end-cill should have a beneficial effect.

(*v*) For asymmetrical operation within one basin it was found that the pier as designed, (207 ft long) was not of sufficient length to be effective. By operating the basin No. 4 with left gate open and right gate closed, "a strong horizontal circulation results which persists from surface to the floor of the basin. The movement of bed material into and out of the basin could be quite destructive", To remedy the situation the center pier in stilling basin No. 4 was lengthened by 80 ft. This gave acceptable performance for asymmetrical gate operation. The centre pier should be at least as long as 287 feet or be eliminated entirely depending on the decision as to whether or not it is advisable to design for asymmetrical gate settings.

(*vi*) The optimum geometry and shape of stilling basin (including the divide walls and end cill) for tunnels 3 and 4 was determined as a result of detailed model study. Static and transient pressures for the recommended design were observed on the Chute, in the floor of the stilling basin and the side walls. With proposed divide wall 287'.0 in length, no negative pressures were observed in the Chute or the floor or the side walls of stilling basin.

(*vii*) From hydraulic stand-point the stilling basin with sides at 1:12 from RL 1025 to 1080 and at 1:1 from RL 1080 to 1132 was quite acceptable. The design with sloping walls could not be adopted as excavation for foundations and sides had been done at site.

C-4.0 Hydraulic Model Studies for Tunnel 1-4 for Stage-III at C. S. U. (1964-1965).

C-4.1 THE MODEL.

The actual scale selected for complete models of tunnels 1, 3 and 4 was 1:69.6. This scale was determined by the available size of plastic pipe. The diversion intake, gates, transitions and tunnels were constructed with clear plexiglass to facilitate visual flow observations. The piers, gates and all straight sections were machined from solid pieces of plexiglass stock. To compensate for greater wall roughness in the plastic pipe the model slope was increased. Two slopes were utilized in the tests of the diversion place of tunnel 1 to reproduce flows for a discharge range from 110,000 to 140,000 cfs (slope .0114) and from 10,000 to 40,000 cfs (slope .003). No slope adjustment on tunnel length adjustments were made for tunnels 3 and 4.

The geometrical details of the intake of tunnels 1, 2, 3 and 4 were same as in the original design and are given in Figs. 15 (a) and 15 (b). The intake structure of tunnel No. 1, with two piers and three vertical fixed wheel gates with not openings of 13′-6″ wide by 45 feet high, together with a vertical shaft of 36 feet in diameter, and sealed by a hemispherical bulk head at EL. 1225 (which formed the permanent intake for power) was represented on the model.

The details of central gate chambers which were duly represented on the model are shown in Fig. 16. The central gate chambers house two vertical lift service and two emergency bulk head gates in all the tunnels. The gates and centre pier in tunnels 1 and 2 will be constructed after the diversion is completed and conversion is made to power-tunnels. All service gates and bulkheads have net openings 13′-6″ wide by 45′ high. The service gates are designed to close against the flow, while the bulkhead will close only under balanced pressure conditions. The gate slots were represented on the model of tunnel No. 1.

The gate chamber for tunnels 3 and 4 Chute and the stilling basin were also represented on the complete model of T_3. The

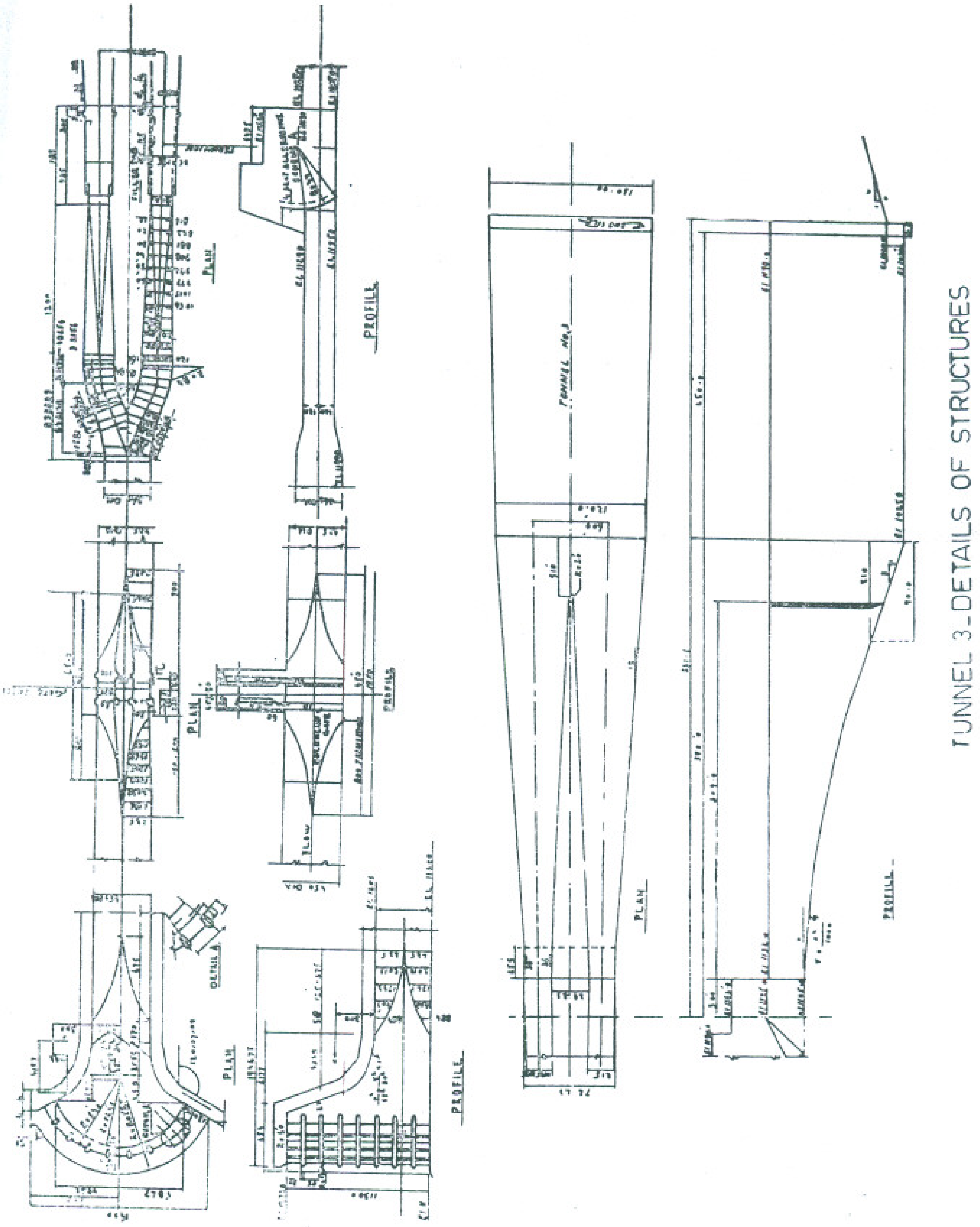


Fig. 16

radial gates at the gate chamber for tunnels 3 and 4 were shaped accurately.

It was originally contemplated to provide outlet constrictions for tunnels 1 and 2 to prevent negative pressures along the crown of the tunnel and deflectors were considered essential to deflect the flow, especially at tunnel No. 2, to protect the base of the stilling basin wall of tunnel No. 3 from scour. The outlet constriction and the deflector were provided on the model of tunnel No. 1 but were deleted in the final design of tunnels 1 and 2.

C-4.2 Scope of the Model Study.

The purpose of the model studies was to investigate the hydraulic conditions within the tunnels for the entire range of possible discharges and velocities. The objectives for the different tunnels are listed separately below :—

A. Tunnel No. 1 (tunnel No. 2 similar).

1. Diversion (with flow ceilIng below the vertical shaft).

(*a*) Determine through visual observation, photographs, and pressure data the flow characteristics throughout the tunnel for the range of expected discharges.

(*b*) Measure low pressures if any, on the boundaries.

(c) Determine the most suitable closure sequence of the diversion intake gates.

(d) Measure the air demand in the model at the intake gate shafts.

(e) Determine the magnitude of the unbalanced pressures on the piers within the intake structure.

(*f*) Calculate the form loss coefficients at the entrance and at changes in cross section of the tunnel.

(g) Study the effect of outlet constriction at the tunnel portal and the deflector.

2. Diversion (with flow ceiling removed) evaluate the flow conditions at the junction of the vertical shaft with the horizontal tunnel during diversion and gate closure.

3. (*a*) Investigate possible low pressure areas on the walls of the vertical shaft and transition to the horizontal tunnel.

   (*b*) Study hydraulic conditions at all tunnel sections and in particular at the central gate structure during normal operation of the powerhouse, during failure of a penstock, and during emergency service gate closure.

   (*c*) Determine the magnitude of pressure differences across the pier in the central chamber during unequal service gate closure.

B. Tunnel No. 3.

Irrigation

(*a*) Observe flow conditions and measure pressures throughout the tunnel for the full range of expected discharges.

(*b*) Measure low pressures within the intake structure, transitions, and sections involving changes in geometry.

(*c*) Determine magnitude of differential hydraulic pressures at the pier in the central gate structure.

(*d*) Determine form loss coefficients.

(*e*) Measure pressures in the bifurcation with equal, and various combinations of unequal flow.

(*f*) Determine which of the two alternatives proposed for the walls beyond the radial gates will be most satisfactory.

(*g*) Investigate performance of the chute and stilling basin.

(c) Tunnel No. 4

Irrigation

Determine possible difference in flow and pressure conditions at the bifurcation due to change in diameter of the tunnel from 43.5 ft in Tunnel 3 to 36′ in Tunnel 4.

C-4.3 Model Results and Conclusions.

The conclusions and recommendations from model studies are summarised below:—

1. Tunnel 1—Diversion.

1. At reservoir levels greater than 1150 the tunnel flowed full. Tunnel 1 for diversion with and without the flow ceiling, performed equally satisfactorily. It is recommended that the ceiling be eliminated. Pressures at the boundaries throughout the length of the tunnel were observed to be positive for the range of reservoir level from 1090 to 1460, Low pressures were measured (fig. 17 a and b) downstream of the gate. Extreme values occured at reservoir Elevation 1411 and gate 10 % open uniformly.

2. Closure of the three intake gates should be effected in unison to avoid large unbalanced forces on the piers upstream of the gates. The maximum differential head just u/s of the gates was at reservoir elevation of 1462 when central gate was closed and right gate was fully opened and this was of the order of 292′ at base of pier and 312′ at axis line of intake. Airvents will be required during gate closure, and should be supplied to all the three gate shafts. Test results of various gate closure sequence was to first close the centre gate then either the right or the left gate and finally the remaining gate.

3. All transitions in the tunnel were designed adequately from the hydraulic view point. Tunnel curvature was satisfactory with regard to flow conditions and boundary pressures. Flow through the central gate section was satisfactory for the entire range of discharges which could occur. No modification was required.

4. The deflector at outlet (for Tunnel 2 as well as Tunnel 1) should be modified slightly to prevent the outflow jet from impacting too closely to the base of the stilling basin wall of Tunnel 3. It is suggested that the right wall increased in height by approximately 15 feet in a length of 40′ beginning from the portal, and the wall should be projected further into the outflow beginning at a point midway between the portal and the end of the wall. Fig. 18.

TARBELA DAM PROJECT—HYDRAULIC MODEL STUDIES
TUNNEL I-DIVERSION LOW PRESSURES
On intake wall at 70% uniform Gate opening.

Res. Elev. 1460
Discharge Qp. = 142000 cfs.
Gate % Open
Right 70, Center 70, Left 70

On Pier at 70 % uniform Gate opening
Res. Elev. 1460
Discharge Q = 142,000 cfs.
Gate % Open
Right 70, Center 70, Left 70

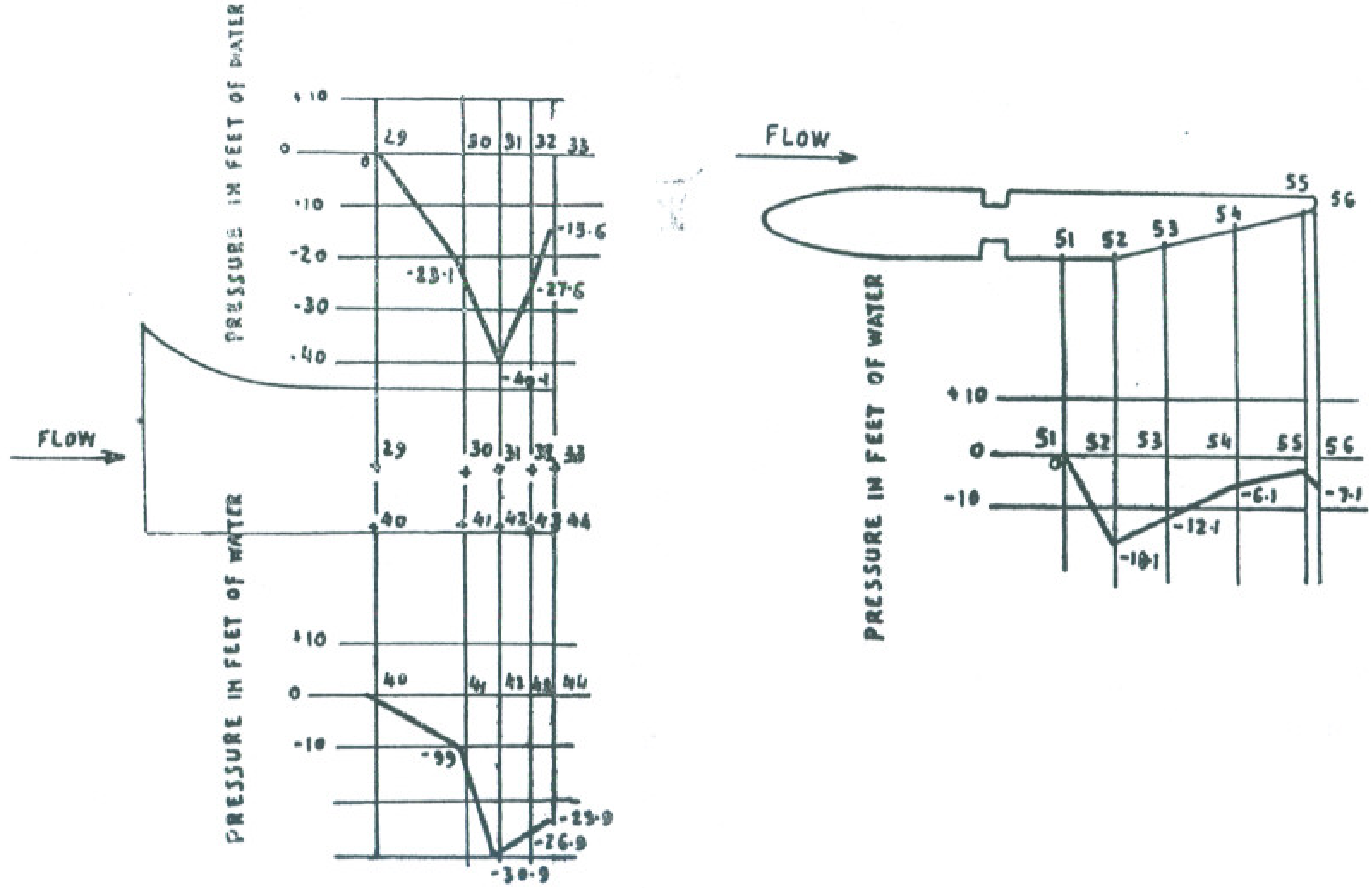


Fig. 17

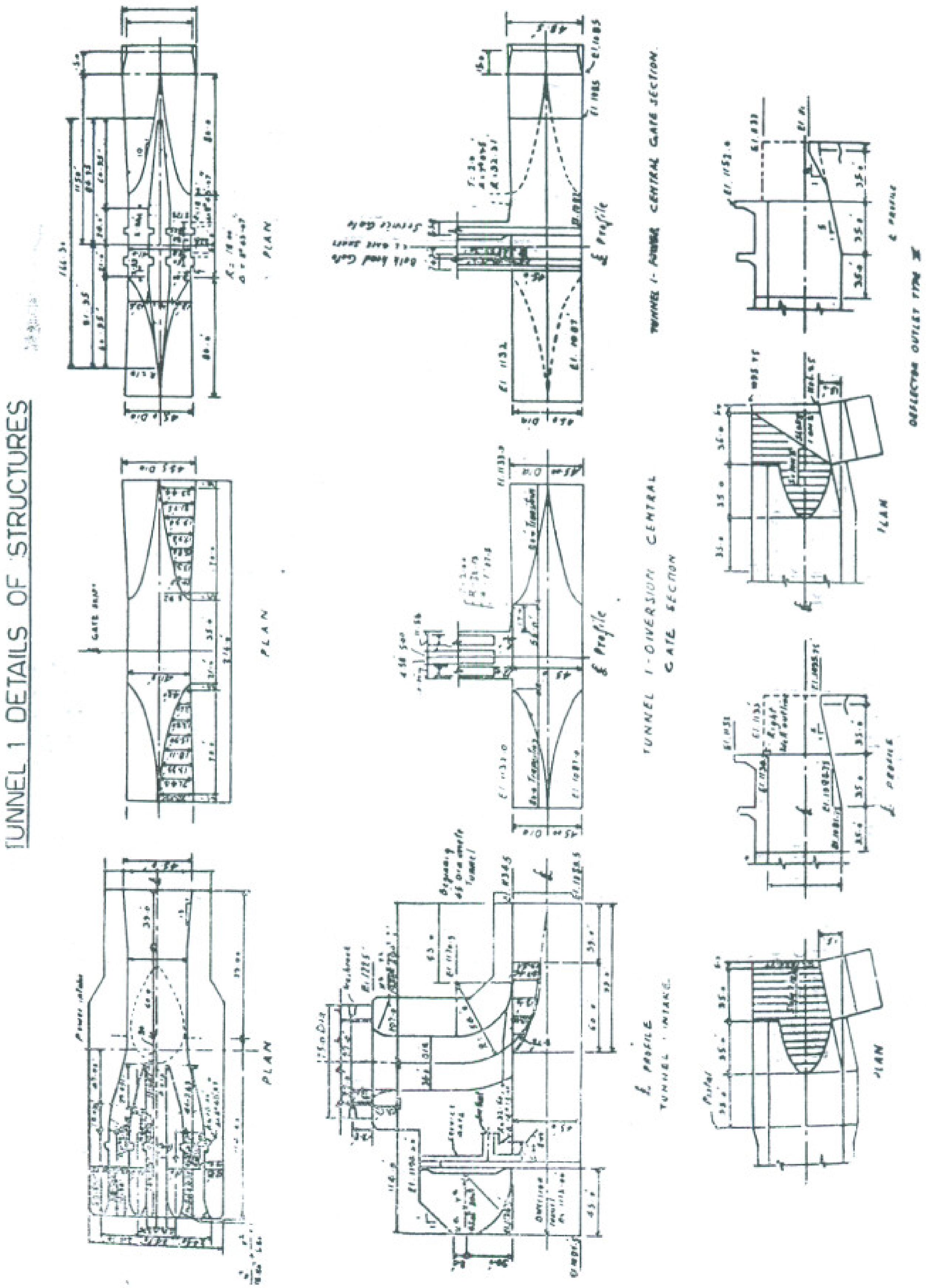

TUNNEL 1 DETAILS OF STRUCTURES
PLAN
PLAN
PLAN
TUNNEL 1-POWER CENTRAL GATE SECTION
TUNNEL 1-DIVERSION CENTRAL GATE SECTION
℄ PROFILE
TUNNEL 1 INTAKE
PLAN
℄ PROFILE
PLAN
℄ PROFILE
DEFLECTOR OUTLET TYPE

Fig. 18

5. The form loss coefficients were worked from energy gradient through the tunnel for two slopes of tunnel No. 1.

Tunnel 1—Power.

1. Tunnel 1 for power plant operation should be entirely satisfactory as designed. The service gates in the central gates chamber could be used to stop the flow in case of emergency, but the gates should be closed simultaneously to avoid large unbalanced forces on the pier at a section upstream of the gates.

Tunnel 3.

The flow in Tunnel 3 above the minimum operating reservoir level of 1300 was satisfactory. The intake structure was adequately designed and no low pressures occurred at the boundaries. Pockets of air were formed in the tunnel when the reservoir was between 1205 and 1225 but it is anticipated that the reservoir level will not be lower than 1300 during normal operation.

2. Normal flow conditions through the central gate chamber were satisfactory. If the service gate were closed with the radial gates completely open as it might be necessary in case of emergency, low pressure (upto prototype vapour pressures) were developed on the wall of the pier near the base downstream from the gates. The gates should be closed simultaneously to avoid large unbalanced forces on the wall upstream of the gates.

3. If the radial gates are not opened symmetrically, negative pressures could develop in an area immediately downstream of the crotch (up to 63 feet head of water at Reservoir elevation 1550) along the horizontal plane through the centreline. Although the low pressure area was small, vapour pressures will be reached when one gate is fully open and the other nearly closed. If the radial gates are at nearly equal opening low pressures will not develop in the bifurcation.

4. The wall of the radial gate chamber should begin as closely as possible in line with the walls of the tunnel. With an offset, the high velocity diverging flow impinged against the wall created a high fin of water to develop considerable spray into the radial gate chamber and over the chute walls.

5. No change is suggested for the vertical curvature of the chute. Although it was not tested in the model, warped and battered wall of the chute from vertical to 1:12 should not materially affect pressures on the chute floor. The divider wall may be reduced in length or eliminated completely, provided the radial gates are operated uniformly. With no divider wall, a fin formed on the chute but it will not create any problems. Negative pressure of =3.2 was recorded near mid chute (at R. L. 1088.18) at Maximum Reservoir elevation of 1550 and gate opening of 25%.

6. The stilling basin performed satisfactorily for the entire range of discharges and tailwater variations even though the jump was not entirely contained at maximum discharge and tailwater level below 1104. This statement is based upon the fact that scour downstream was not too severe. Increasing the length of the basin by 100 feet or increasing the height of the end-cill did not lead to total confinement of the jump. At reservoir levels below 1425 the hydraulic jump was completely contained in the stilling basin for the entire range of tailwater levels from 1097 to 1114.

C-5.1 Model studies at CSU on the outlet gates for Tunnel 3 and 4 (1970 Report).

Hydraulic model studies of the radial gate and appurtenent works for tunnels 3 and 4 of the Tarbela Project were performed at Colorado State University.

There are two radial gates each at the ends of tunnels 3 and 4 to regulate irrigation releases. The gates may be set at any required opening between closed and nominally half-open position. The heads against the gate may very from 183 feet at low reservoir level to 433 feet at full reservoir. With these large heads, standard seals for radial gates located on the edge of the skin plate would be inadequate; therefore; caisson-type seals were placed on the face of the skin plate in the form of a picture frame. This arrangement of the seal made it necessary to provide a seating surface at the tunnel portal. The seating surface along the bottom was made possible by a step in the floor; the seating surface along the sides necessitated recesses in the side walls; and the seating surface for the top seal was cylindrical in shape, extending from elevations 1129 to 1146. The off-sets in side walls and step in the floor to provide bottom seating surface created problem concerning cavitation of the gate structure, gate chamber d/s of recesses, spray and splash in gate chamber, Ventilation of the jet at the step in floor and fluctuations in pressure in the gate chamber.

Specific questions which the model studies had to answer are listed as follows:—

(*a*) With the large-velocity flow past the seals, are the provisions adequate to prevent cavitation?

(*b*) Will the deflector hood at elevation 1158 intercept the jet issuing from the top seal and direct it into the collector-dissipator basin in case of failure of the jet arrestors to function as intended, and is the basin adequate in size and arrangement to dissipate the energy?

(c) Are pressures in the vicinity of the side wall recesses above the cavitation range under all conditions of operation?

(*d*) What provisions need to be made to contain the jets from the side seals within the waterway channel ?

(e) In the event that the original design of the gate recesses requires modification, what arrangement is satisfactory in terms of the hydraulics or the flow?

(*f*) Are corrections necessary in the assumed pressure distribution used for computing the hydraulic load on the gate?

(*g*) What is the coefficient of discharge at each gate opening?

(*h*) What ventilation arrangement is needed at the step in the floor?

C-5.2 The model.

Three models of different scales were utilized to answer these questions. One was a part model with a geometric scale of 1:4, model to prototype, designed to investigate details of the flow past the top seal and into the collector-dissipator basin.

This model was also used to observe the flow past the side seals and side wall recesses. The model was a two dimensional representation of a 36-in. prototype length of the gate chamber and skin plate and is referred herein as the sectional model. The second was a geometric scale model of 1:12 which included the entire transition from 24′ circular to 16×24 rectangular d/s of the 'Y' branch to the gate structure and gate chamber Fig. 19 and a portion of the chute to the stilling basin. The model did not represent the bifurcation. The third was a 69.6 scale model of the transition and gate structure which was used to provide qualitative answers relating to the effectiveness of changes in geometry of the side wall recesses.

C-5.3 Conclusions and recommendations.

The major questions which were outlined above under the head scope of the model study were answered in the same order as below:—

(*a*) The large velocity flow past the scale, which occurs when the gate is retracted prior to movement, will not cause cavitation of the seal clamp bars or the gate seat. There were conditions encountered in the 1:4 scale model when ventilation was inadequate, but these conditions are generally outside the expected range of operation for the prototype. Although ventilation slots on the clamp bats were tested in the model, they are not recommended for the prototype.

(*b*) In the event that the jet arrestor does not function as intended, the deflector hood at elevation 1158 will adequately deflect, the upward jet when pressure heads measured at the mid point of the conduit transition are greater than 200 feet.

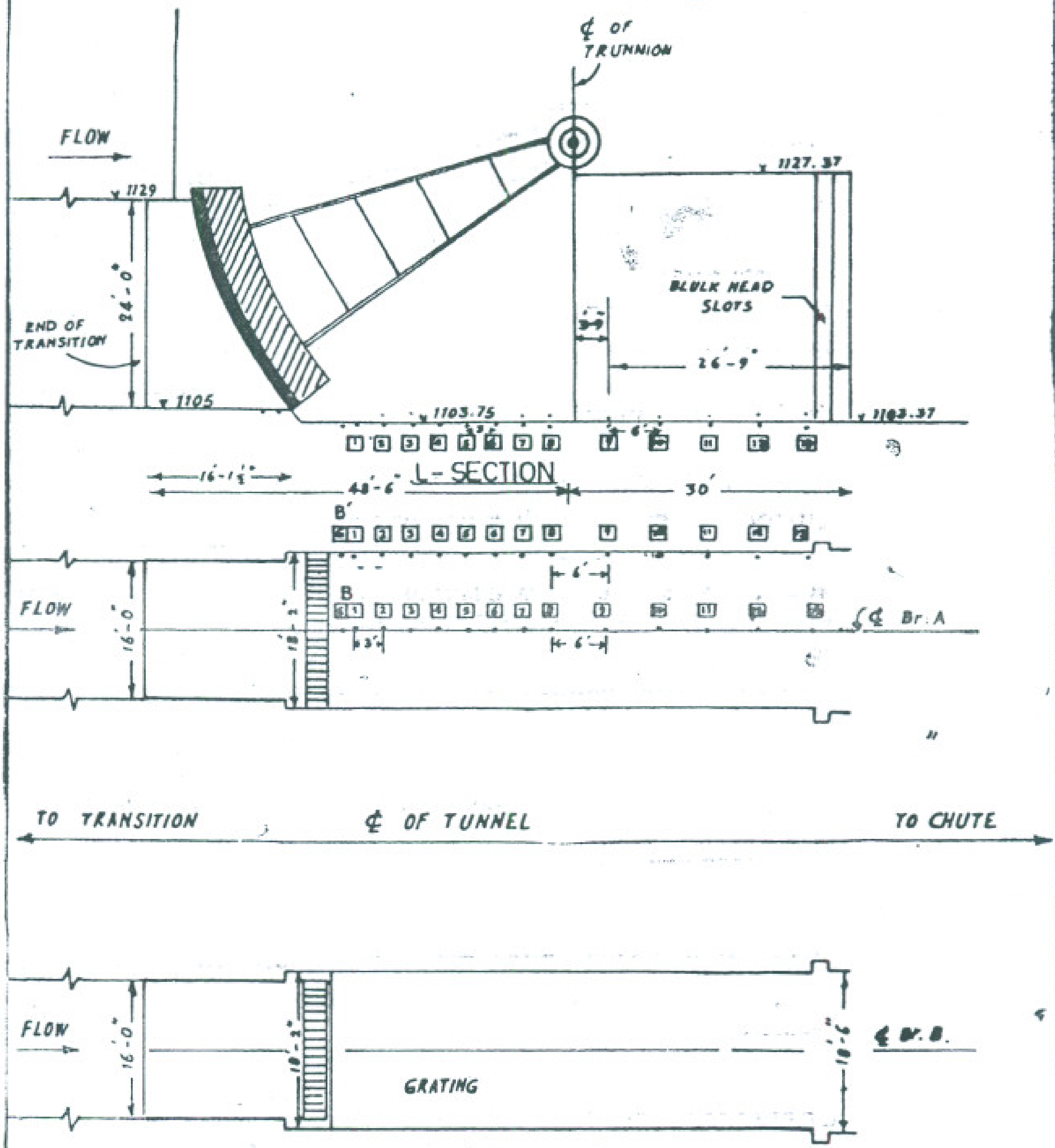
(*c*) With the triangular recess geometry, negative (subatmospheric) heads to equivalent prototype vapour pressure were measured in the model for all gate openings, indicating that a change in the geometry of the side Walls was necessary to avoid cavitation.

(*d*) This side wall jets, created when the gate is retracted, were directed by the triangular recess geometry directly

TARBELA DAM PROJECT

GATE CHAMBER OF TUNNELS 3 & 4

FIG. 19



into the structural member of the radial gate arms. The jets spread out and created objectionable spray. It appeared that a change in the wall recess geometry offered the best prospect for improving the jet action.

(e) Side walls which were offset 16 inches from the transition walls to provide seating surfaces for seals were found to be hydraulically satisfactory for all normal and retracted gate position for all reservoir elevation between 1300-1500. Side wall deflectors were found to be necessary to keep the jets of water contained within the channel. Some spray in the form of droplets land outside the lateral boundaries of the chute walls for normal gate position. The spray was most noticeable for a gate opening of 8 feet with maximum head. If it is desired to contain this spray within the channel, a roof above the deflectors in the gate structure and downstream of the stoplog slot on the channel will be required. Pressure fluctuations at the walls were largest in a small region downstream from the offsets near the floor. The mean pressures of these fluctuations were greater than atmospheric pressures although a small percentage of the fluctuations were found to be subatmospheric. The region of the expected subatmospheric pressures is within the area of the wall where steel lining is planned. There was marked improvement over the effect of the original triangular recess geometry.

(f) The forces on the gate were determined in the model by integrating (summing) the measured pressure force over the gate surface. The model forces (Fig-20) were larger than those predicted in design and seemed to be independent of gate position (upto half open position).

(g) The discharge coefficients for the gate at various openings were determined by the model. They were not materially different from those used in design (Fig-21).

(h) The ventilation system originally planned, which vents downstream of the toe of the step (referred to as the primary ventilation system) was not materially effective. Water was ejected from the system during each test rather than drawing air. Adequate amounts of air drawn into the region below the nappe from the offset space. The sceondary ventilation system which vents to the face of the sloping step in the floor should be provided. The vents must be located below the seating surface of the bottom seal when the gate is closed.

The piezometers points spaced and identified in Fig. 22 were installed in the wall of the gate chamber. Pressure fluctuations were recorded with transducer.

There is a small region of the wall containing piezometers No. 18, 25, 26, 30 & 34 which is subject to low pressures. There is cavitation potential along the wall near the floor of the gate structure just downstream from the off set.

The cavitation potential area is cross-hatched in Fig. 22. It may be pointed out that the detailed pressures at the face of 16″ step and the floor of the chamber were not observed on the model.

C-6.0 Model Studies at C.S.U. in Service Gates for T_3 and T_4.

Normal regulations of flows through T_3 and T_4 is provided by radial gates at the tunnel outlets. However, in the event of emergency the service gates may be required to close off the flow or open up for irrigation releases if the radial gates are disabled. Downpull is defined as the hydrodynamic force acting on the service gates in the direction of gate travel and is designated as positive downward (adding to the total load of gate hoisting equipment) and negative upward (uplift). The downpull force is a consequent of non-uniform pressure and velocity distribution in the complex three dimensional flow field in the region of gate which is not amicable to mathematical solution and so was determined in 1/24 natural scale hydrodynomical model which will create flow field similar to prototype.

C-61 Model.

The service gate assembly is shown in Fig. 23a and is identical for all the tunnels. In a general way the gates are 16 feet

TARBELA DAM PROJECT
HYDRAULIC MODEL STUDIES
Hydraulic Forces on the Radial Gate.

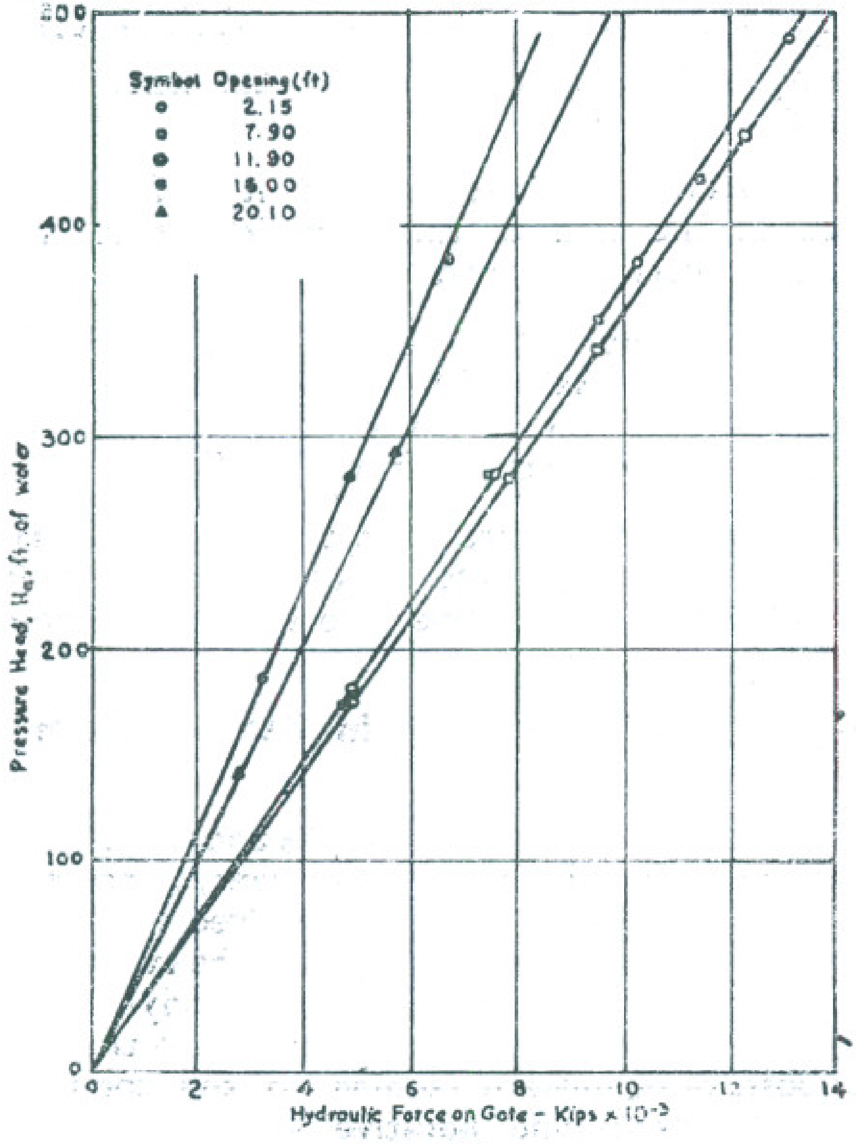
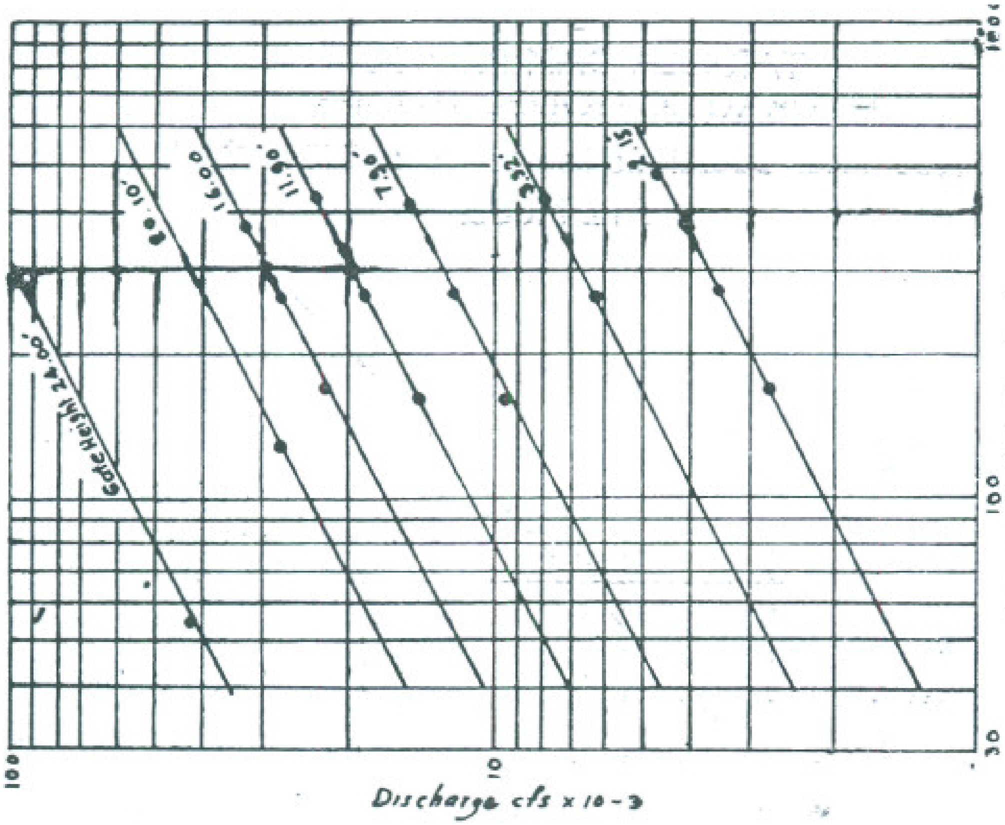
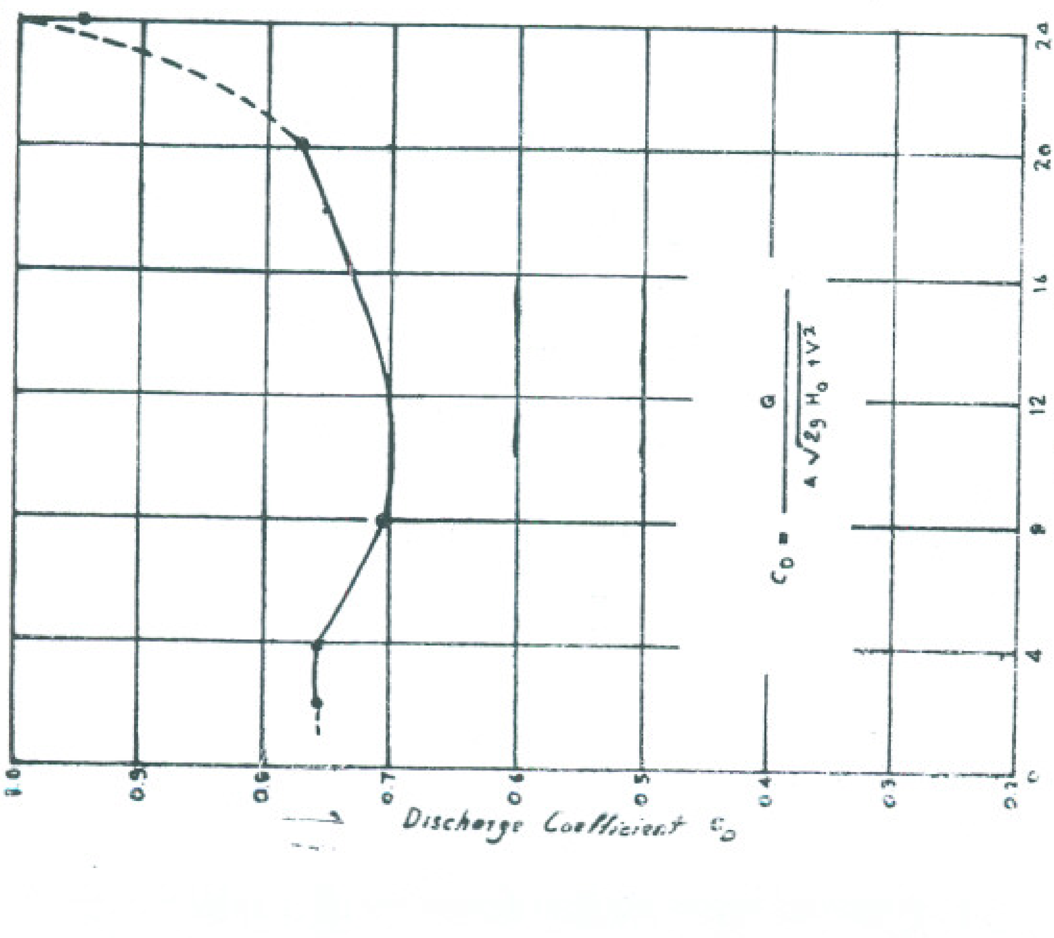


Fig. 20

TARBELA DAM PROJECT
HYDRAULIC MODEL STUDIES



Discharge rating curve for the radial gate



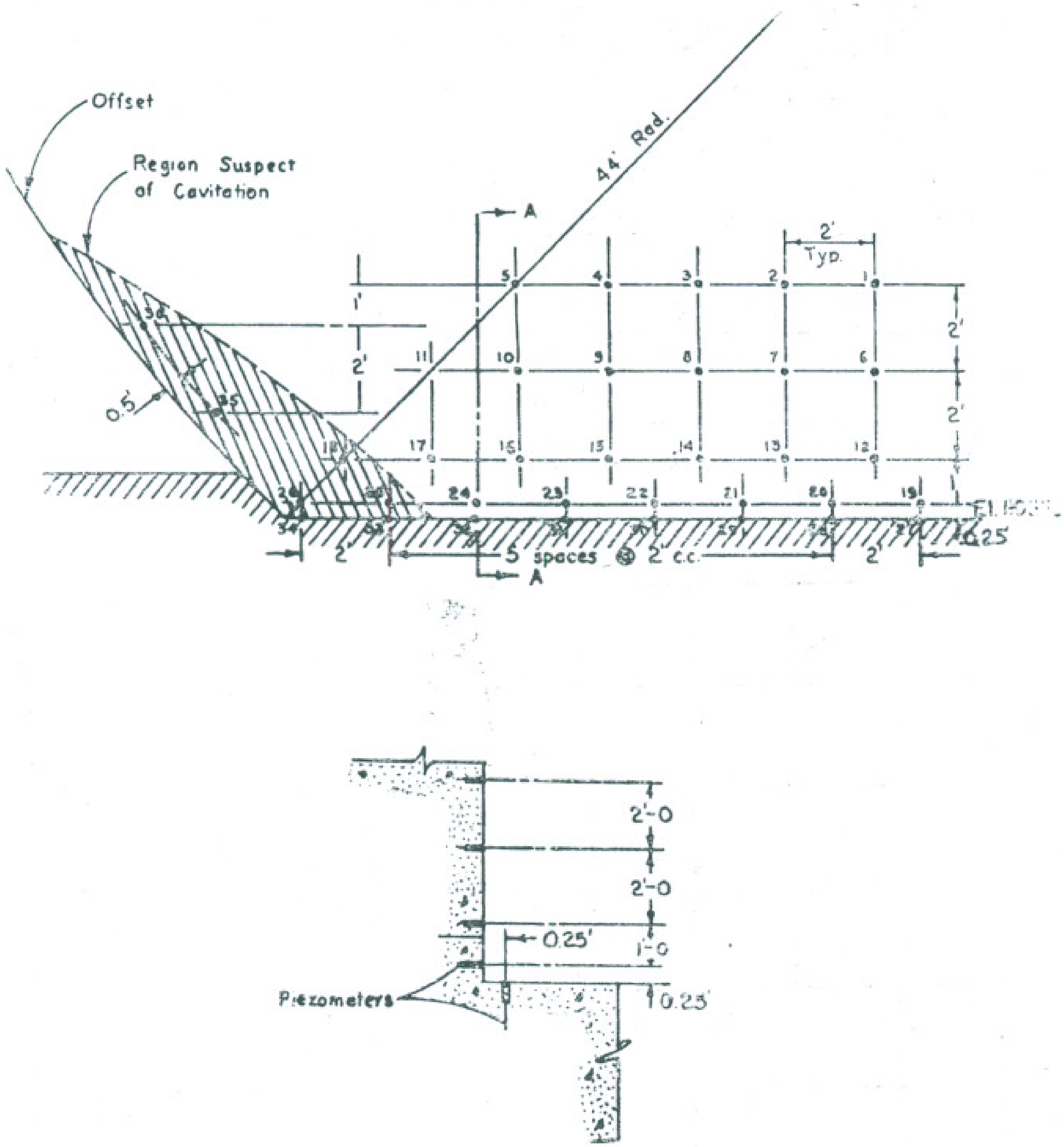
Computed discharge coefficients for the radial gate

Fig. 22

TARBELA DAM PROJECT
HYDRAULIC MODEL STUDIES

Piezometer Locations in the 16-in Offset Wall

1 : 12 Model



Section A-A

Fig. 23 (a)

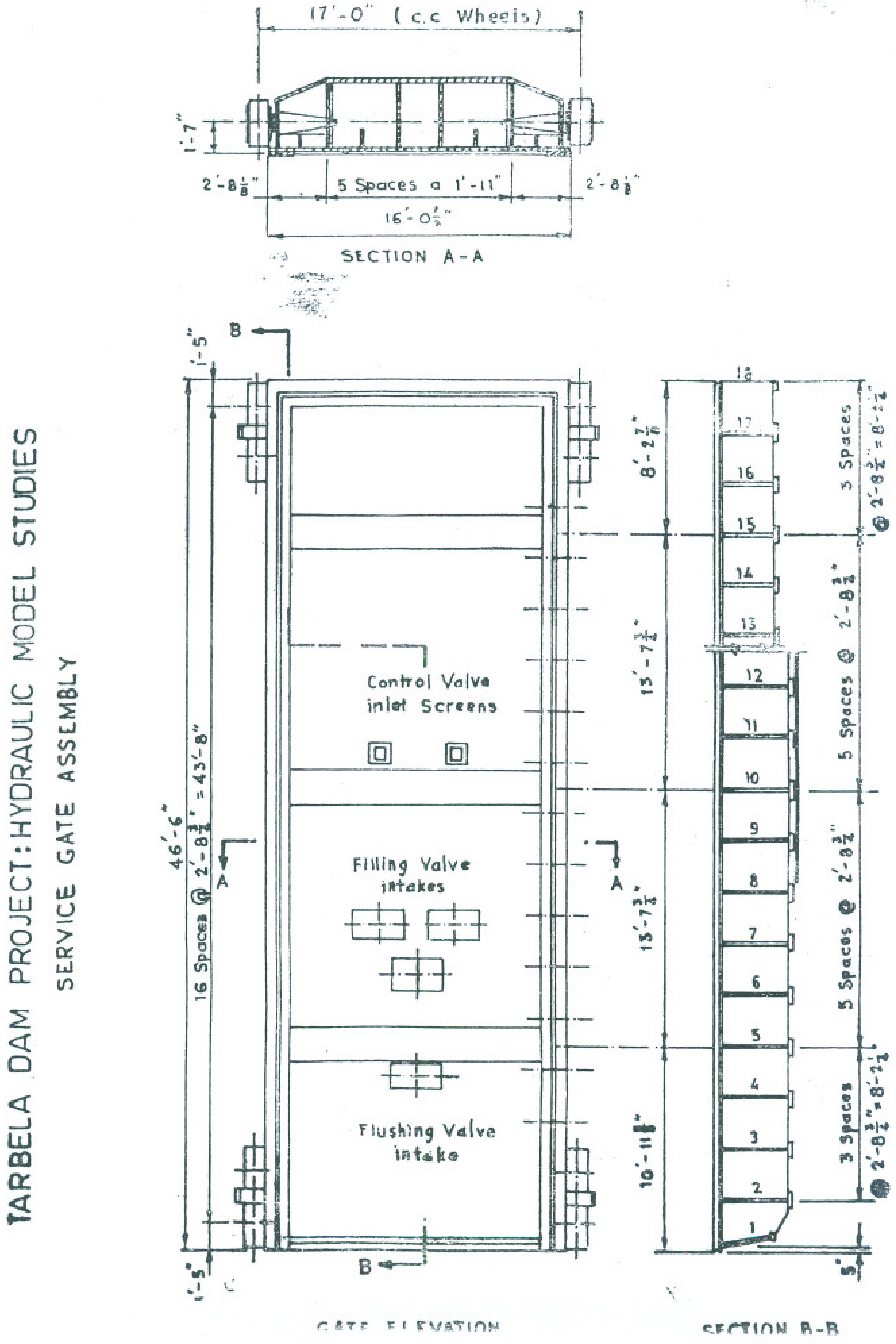

TARBELA DAM PROJECT: HYDRAULIC MODEL STUDIES
SERVICE GATE ASSEMBLY
17'-0" (c.c Wheels)
1'-7"
2'-8 1/8"
5 Spaces a 1'-11"
2'-8 1/8"
16'-0 1/2"
SECTION A-A
1'-5"
46'-6"
16 Spaces @ 2'-8 3/4" = 43'-8"
1'-5"
Control Valve
inlet Screens
Filling Valve
intakes
Flushing Valve
intake
8'-2 7/8"
13'-7 3/4"
13'-7 3/4"
10'-11 5/8"
3 Spaces
@ 2'-8 3/4" = 8'-2 1/4"
5 Spaces @ 2'-8 3/4"
5 Spaces @ 2'-8 3/4"
3 Spaces
@ 2'-8 3/4" = 8'-2 1/4"
5"
GATE ELEVATION
SECTION B-B

Fig. 23 (b)

TARBELA DAM PROJECT
HYDRAULIC MODEL STUDIES
GATE SHAFT GEOMETRY