30-8-1974. Cell 15 collapsed.

31-8-1974. Cell 14 collapsed.

13-9-1974. Damaged portion of tunnel 2 concrete lining visible for first time.

19-9-1974. Soundings indicated deep erosion of foundation rock (about 30′ below foundation level of structure) between intake structure of 1 and 2 and just downstream of wall B.

List of damages in hydraulic structures as finally noted above dumping the reservoir can be summarised as:—

1. Damage to the cells of the cellular coffer dam, for the protection of power house area (and acting as left protection wall of outlet area of tunnel 1 & 2).
2. The uncontrolled flow of water into the collapsed section of T_2 had eroded a large volume of surrounding rock to form a (500,000 cu. yd) crater like hole extending to the vicinity of T_1 & T_3 and about 80′ further D/S from the collapsed section.
3. Intake gates stuck-rails gone, rollers damaged—Gate bulked.
4. Piers in the intake gate chamber of tunnel 2 damaged (big hole through and through).
5. 250′ length of the right side of tunnel No. 2 downstream of the portal disappeared (some parts of the broken ribs and reinforcement bars on the left side were intact).
6. Unlined portion of tunnel 2 principally around the invert was also damaged (Pot holes). A considerable amount of hoop reinforcing had been exposed and pulled out.
7. Large number of sink holes found in the clay blankets.

E-1.1 Damage to Tunnel 1.

1. Gates of tunnel No. 1 was also damaged.

2. Pier damaged. The interior of ths U/S conduit section and intake were heavily damaged.

3. Unlined portion damaged with pot holes (to a lesser degree).

E-1.2 Damages to Tunnels 3 and 4.

1. Pitting action at bifurcation and transition-damage to concrete.

2. Steel liners in Tunnel 3 in the chamber downstream of the radial gates ripped-off.

3. Grating of primary airvent washed away.

4. Steel liner on sides ripped off in Tunnel 3.

5. Chute surface damaged downstream of gate slot of T_4.

6. Flow in the cistern was very unsatisfactory with severe turbulance and wave splashes all over the flank walls. The turbulance persisted way down the end of the cistern.

7. Heavy vibrations in the super-structure.

E-1.3 Damage to Stilling Basin.

1. Concrete floor of stilling basin with part of chute of Thannel 3 eroded with rock underneath scoured to a depth of 70 feet.

2. Side walls of stilling basin of T_3 undermined.
3. End Sill damaged.
4. Downstream rock bed scoured (not very heavy).

E-2.0 Departures From Model Recommendations,

A study of the regulation adopted (which is graphically illustrated) clearly shows that:—

1. Regulation of intake gates of Tunnel 1 and 2 were done on July, 13 by keeping the central gate open and closing the side gates to save cell 18. This was against all model recommendations for uniform gate opening. In fact the regulation was just the opposite of model recommendation of closing the central gate first and then the right and left.

2. Even for Tunnel 3 unsymmetric regulation of radial gates had now and then been adopted between 31st July to 15th August, again from 23rd August to 8th September both 3A, 3B and 4A, 4B had unsymmetric openings which were positively against the model recommendation. Without a central pier extension, it was pointed out from model studies that the entry of stones into the stilling basin shall damage the floor. Inspite of the fact that the divide wall length was reduced from the originally designed length, the unsymmetric regulation was permitted.

3. Unequal opening of radial gates had shown negative pressures at the crotch reaching cavitation range in case one gate is fully closed and the other fully open. The pitting noted at the bifurcation is the result of this unsymmetric gate opening.

4. Highly fluctuating pressures at the walls were noted in the model. However, as fluctuating pressures at the floor downstream of primary ventilation system were not observed at CSU, the anchorage of steel liners was not proper.

5. The primary airvent system was shown to be ineffective. Water was ejected from the primary system rather than drawing air. This was found to be the case in the prototype also.

6. The effect of stop log gate slots on pressures on chute was not tested.

7. Inspite of the fact that the depth of the stilling basin was considered bare minimum the increase in depth of stilling basin was ruled out without a trial on the model.

8. Long term ill-effects of continuous unsymmetric operation in distinction to sequence of openings was not stressed ftom the model. The very undesirable flow conditions generated on the model when central gates at the intake of T_1 and T_2 was open and side gates closed should have ruled out such a regulation. It appears the studies mainly related to the order of closure in regulation and, therefore, the results though

indicated by model did not receive due weight in long term regulation.

F-1.0 Post Damage Experiments at Irrigation Research Institute and CSU of Tunnel 3 and 2, 1974-1975.

After the failures and damages obserbed in tunnels 1, 2, 3 and 4 at different places as described above it was decided that further experiments may be carried out at Irrigation Research Institute and CSU Laboratories.

The Panel of Engineers for Tarbela Dam requested Irrigation Research Institute:—

(*a*) To model test the outlet structure of tunnel No. 3 for investigating the probable cause of dammage to gate chamber and chute leading to the stilling basin of Tunnel No. 3 and 4 and to evolve satisfactory aeration arrangement for the nappe of flow at the step in the floor of gate chamber by improving the geometric features of the transverse air trough for the primary air system.

(*b*) To model test the full length of tunnel No. 2 to investigate the possible hydraulic causes of collapse of Tunnel No. 2 by identifying the regions of low pressures and thus the cavitation potential areas and the flow conditions for the experienced operations of Tunnel No. 2.

The model of tunnel No. 2 was, however, utilized by TAMS to test the adequacy or otherwise of restorative works proposed to be carried out. The priority was given to the studies of restorative works of Tunnel No. 2 as it was necessary to make all project features operable and safe for the ensuing flood season.

E-3.1 THE MODEL.

The model of outlet control structure for tunnel No. 3 was constructed to a geometric scale of 1 : 36 and includes tunnel No. 3 from Sta. 32+83.02 to Sta. 33+30.06 of 43.5 ft. dia. tunnel, transition from 43.5' to 36 ft. dia pipe, bifurcation into two 24′ dia branch waterways, transition in a length of 120.0 ft. from 24.0′ circular, terminating into 16′ wide by 24′ high rectangular section at the radia' gates, open branch channels of the gate chamber, chute curve y= x /1800 and the stilling basin. The vertical slots in the chamber walls at the end of chamber for the installation of stop log (to hold back the tail water in case unwatering of the gate chamber is necessary) were represented in all details. The radius of the gates in gate passage 3A and the 3B which is 44′ from trunnion to gate face represented accurately to scale. The machining of the gates was completed satisfactorily despite initial problems. The gates were sealed drop tight. The trunnion was supported rigidy on the side wall in the chamber. The ends of trunnion were so machined that trunnion pin can be rotated independently to represent an upstream and downstream motion upto 0.8 inch on the prototype at any setting of the gate. The entire model upto the end of gate chamber including the gate was fabricated in plastic glass so that visualization of flow is possible. The side of chute were also in plastic glass. The seals were fabricated in polythine cloth. The primary and secondary ventilation system Fig. 376 was reproduced with the difference that the primary and secondary air feed pipe were not merged into 3 ft. horizontal common air pipe, to enable us to measure independently the quantum of air in each pipe. The collector dissipator basin to collect upward jet from the top seal was not represented on the model.

-3.2 Conclusions of Test Tunnel 3.

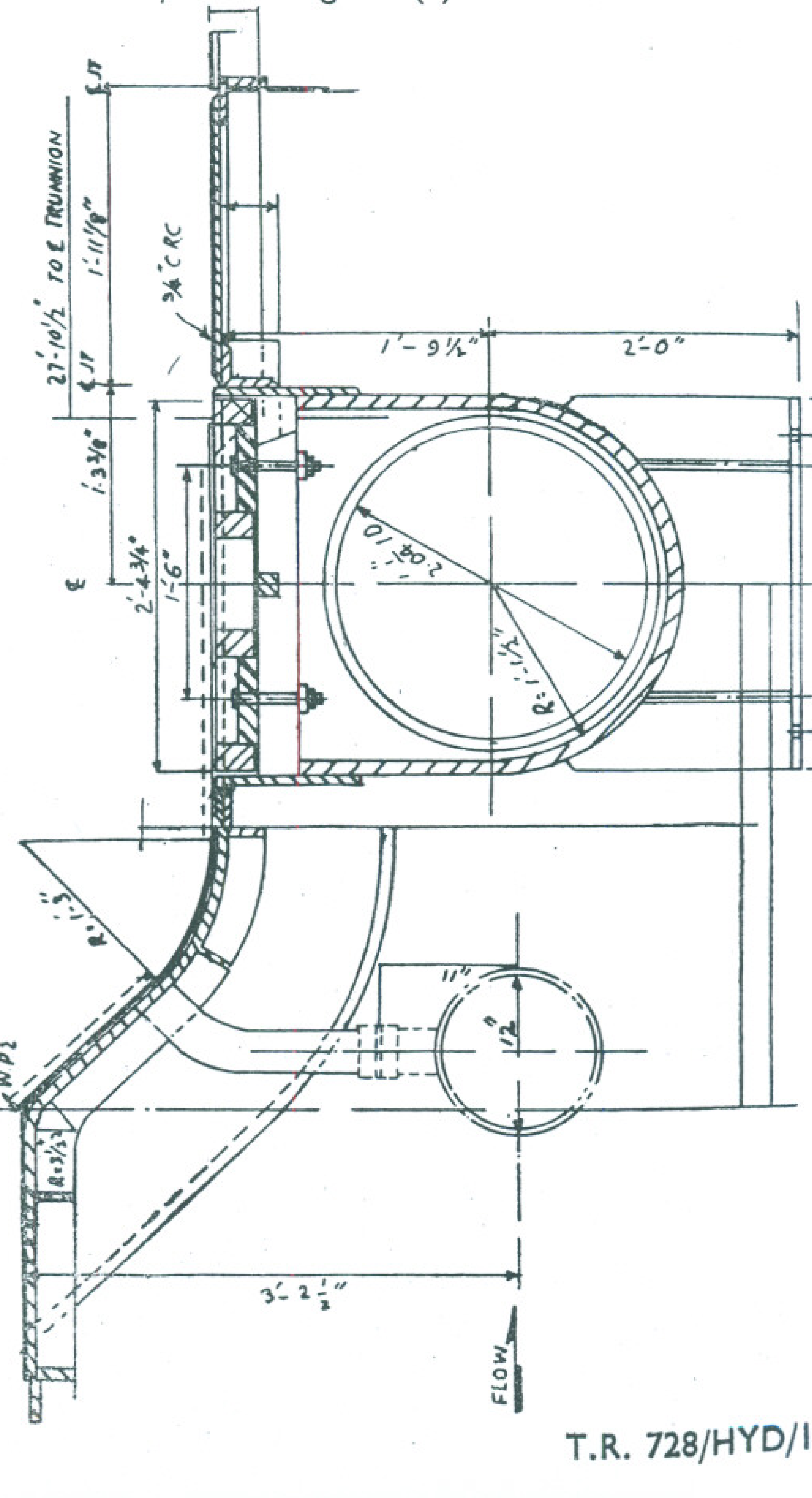
1. Downstream of the bifurcation splitter of Tunnel 3 (a) and 3 (b), negative pressures were observed at the crotch specially in the case when one gate is fully open and the other is fully closed and to a lesser degree for unsymmetrical gate openings. The cavitation resulting from this could be the cause of some pitting action noted on the prototype below the nose of the bifurcation. The conclusion from this observation is that unequal or unsymmetrical gate opening of 3 (a) and 3 (b) should positively be avoided in all future regulations.

2. The study of the primary and secondary ventilation systems located downstream of the toe of the step and in the step itself respectively, showed that :—

(a) Under all conditions of flow, secondary airventilation system located in the step is more efficient than the

DRG. 7-75 72/3 Fig. 37 (b)

TARBELA DAM PROJECT
OUTLET CONTROL STRUCTURE AERATION SYSTEMS
Primary and Secondary Original Design



SECTION THROUGH AERATION TROUGH

primary airvent system located below the toe of the floor of the chamber.

(b) The primary airvent system downstream of the toe of the step showed no air demand with full gate opening up to reservoir El : of about 1400, whereas the secondary system started having air demand at about El : 1350.

(c) In case of partial gate opening excepting 6.0 feet gate opening the component of flow created by the curved face of the gate prevents the jet from springing clear of the primary airvent and consequently very little air demand is created in the trough of the primary airventilation system. In fact the trough was partly filled with turbulent water.

(d) Slightly negative pressures at high reservoir elevation above 1400 were observed at the toe of the step in the floor of gate chamber at full gate opening and 6.0 gate opening and the value of maximum negative pressure at 6 feet gate opening and reservoir elevation of 1500 was - 8.0 feet. The primary air system immediately D/S of the step starts drawing of air as soon as some negative pressure starts developing in the region of primary air system.

3. To improve the working of the primary air ventilation ystem, in one of the series of tests the primary airvent trough vas provided with 3 slits, placed in parallel across the flow, nstead of the grating slits which were in the direction of flow. rom the tests it was found that :—

(a) All the three slits functioning collectively were as effective as the uppermost slit working alone.

(b) The slit nearest to the toe of the step is more effective than the original design in supplying air through the airvent.

The test, therefore, indicated that airvent slit should be laced as near the toe of the step as possible. The shape of ie replacement cover of the trough now decided by the consuants to be installed as a results of tests at CU and Irrigation

Research Institute, Laboratories provides a slit four inches downstream of the upstream edge of the air trough. The air intake slit as now modified at site is at the position indicated by the test made at Irrigation Research Institute.

4. Highly fluctuating positive pressures were noted in the chamber downstream of the radial gate at the position corresponding to the location where the steel liner plates have been ripped off in the prototype. The maximum pressure fluctuations at reservoir EL of 1400 were of the order of 47′, 48′, 36′, 43′, 76′ and 72 feet head of the water at the respective gate opening of 24′, 12′, 8′, 6′, 4′ and 2 feet. At an El. of 1500 the maximum fluctuation of pressure on the floor of the gate chamber were of the order of 85′, 68′, 44′, 49′, 62′ and 111′ head of water for corresponding gates openings of 24′, 12′, 8′, 6′, 4′ and 2 feet. The very high fluctuating pressures, though all positive, specially occurred at low gate openings. Thus loading and unloading from high fluctuating pressure causing viberations in the steel liner panels could have dislocated weeker welded joints of the 4′×8′ steel liner plates. This can explain quantitatively the cause of the damage to the steel liners in the chamber downstream of the gate.

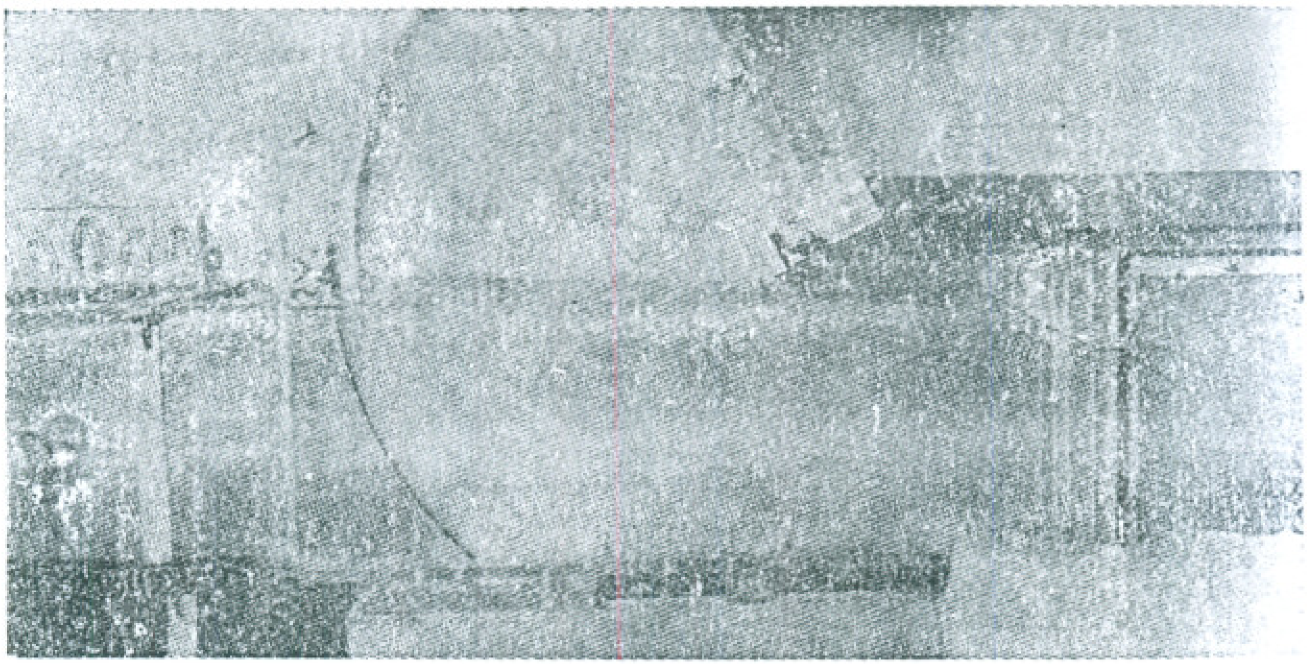
5. In the 13″ recess of the two walls to ensure the sealing of the radial gate, a 9″ thick filling of the recess having 36′ length at the top and 24′ at the bottom with upstream and concentric with arc of the gate, leaving 4′ recess for the gate, were found effective in eliminating the vortex along the off-set walls of the chamber Fig. **38** and reducing the pressure fluctuations over the floor of the chamber by about 50%. From practical considerations of construction at this stage, this has not been adopted.

6. The negative pressures in cavitation range were observed just downstream of the slots for the stop log gates indicating a cavitation potential downstream of the end of the gate chamber. Filling of the stop log gate slot nearly eliminated the negative pressures. The filling of the gate slots were recommended for eliminating a damage to the side walls and floor downstream of the stop log gate position.

7. Negative pressures upto-12 feet head of water were experienced on the chute profile of the stilling basin near the

TARBELA DAM, TUNNEL No. 3

Model Scale = 1 : 36



Vortex formation along the offset wall of Gate Chamber

U/S pond level = R. L. 1400

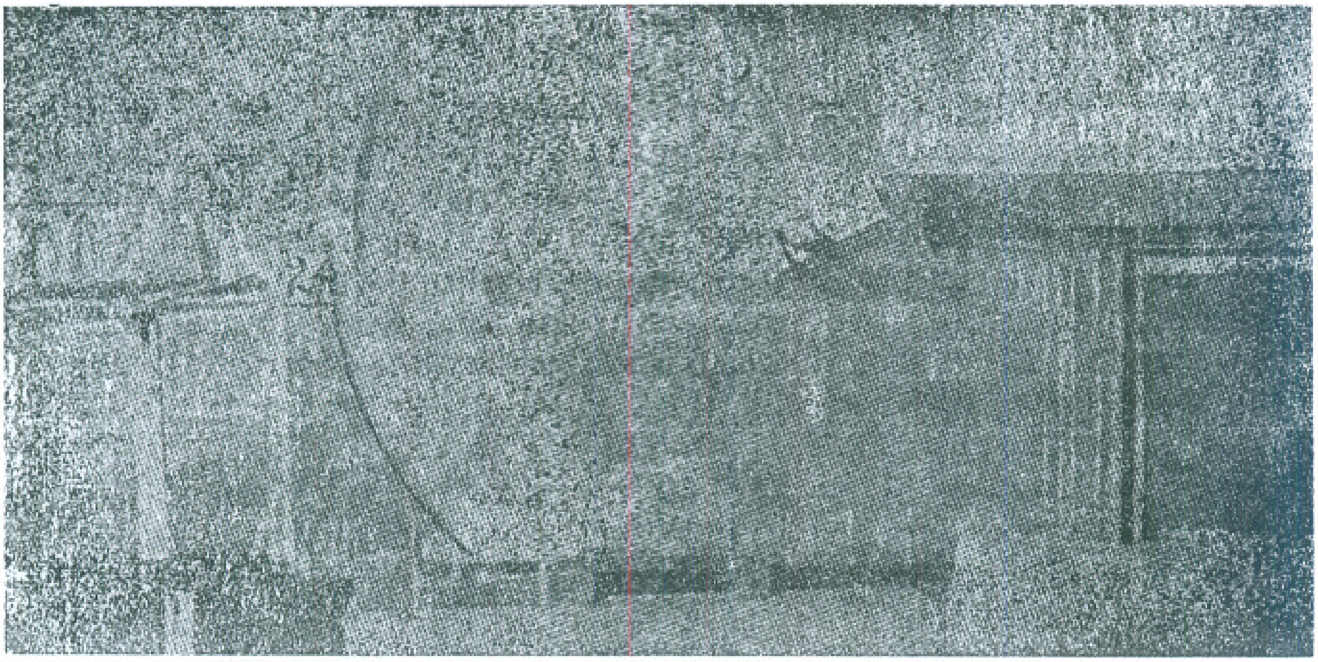
3-A = Closed .

3-B = 24' open

Fig. 38

TARBELA DAM, TUNNEL No. 3

Model Scale = 1 : 36



Vortex formation along the offset wall of Gate Chamber

U/S pond level = R. L. 1400

3-A = Closed

3-B = 24′ open

Fig. 38

downstream end of the pier. From experiments it was found that if the pier is extended by another 40′—50′, on a continued slope, the negative pressures get eliminated. The recommendation for the extension of the pier was, therefore, made in the interest of the safety of the chute. An extension of pier by 40′ has now been agreed to.

8. Splashing and spilling of water over the sides walls (specially for unequal gate openings in 3(a) and 3(b) were clearly indicated on the model. Raising of the walls (not in the first case made to a level recommended by Irrigation Research Institute) and provision of the wave arrestors were more than justified. On the prototype raising of the walls by 6′ and construction of wave and splash arrestor has been done without a further check on the model.

9. In case T_{3a} is closed and T_{3b} is in operation the entire flow after leaving the chute converges in a narrow width along the stilling basin wall on the side of T_{3b} and a very high velocity back eddy (with vertical axis) forms in the stilling basin. This return eddy presses the forward, high velocity, highly turbulent jetting flow in a very narrow width along the wall of the stilling basin. The high velocity return roller brings down stone from the river bed and hurls it against the floor of stilling basin. This stone is then handed over to the jetting forward flow which rubs it against concrete at a velocity more than 100 fps. The entire concrete in the bed of the model stilling basin was removed by flow in about 16 hours running of model at reservoir elevation of 1500 when T_{3a} was closed and T_{3b} was fully open.

Thus interlocking of outlet radial gates of T_3 and T_4 is a must.

10. In case of symmetrical operations of radial gates at reservoir elevation of 1500 and two gates fully open the jet flips off the stilling basin at tail water elevation of 1104 and the flow in stilling basin is highly turbulent and heaving for tail water level variation from R. L. 1108-1104.

11. In case the stilling basin with floor elevation 1025′ is extended from 263 feet to about 450 feet and new cill 14 feet in height with U/S slope of 1:2 is added at the end of extended floor and the existing cill is raised by 16 feet, the stilling in stilling basin improves significantly for normal and retrogressed tail level conditions. The extended floor with two cills as described above can cater for future retrogression of tail levels by about 20 feet.

E-4.0 Post damage Experiments at CSU on outlet gates of tunnel 3 and 4 (1974-1975)

The 1/12 scale model was renovated for further studies now required. The principle modification was to reconstruct the wall off-set of the gate chambers to 13 inches prototype dimensions from 16 inches that was last tested in 1969-70, with other changes in air supply pipes to conform to the prototype conditions. The other modification was the extension of D/S chute from a length of 68′ to 192′. It may be pointed out that the bifurcation and U/S tunnel, and D/S stilling basin could not be constructed on such a larger scale model.

E-4.1 Scope of model studies:

The scope of additional tests to be made in 1974 were :—

1. Measure magnitudes of pressures and pressure fluctuations on the floor and wall of the radial gate chamber at gate openings of 2, 4, 6, 8, 10, 12 and 24 feet and reservoir levels of 1300, 1425 and 1550 feet.
2. Perform tests of various cover designs over the aeration trough located immediately downstream from the step in the floor.
3. Identify areas of low pressure in the gate chamber.
4. Measure pressures on portions of the floor and wall of the chute leading to the stilling basin.

E-4.2 Model tests and Results.

In the prototype the grating over the aeration trough was uprooted and carried down by the force of water jet. Several schemes shown in Figure 39 for the design modification of the covering of the aeration trough to achieve adequate safety for both the impact of water jet and for getting air flow to the region below the nappe of the jet were tested.

E-4.3 Results and Conclusions.

The principal results from the hydraulic model tests at C. S. U. are :—

1. The flow does not separate sharply at top of the step in the floor, but "clings" to the radius ($3\frac{1}{2}''$) and subatmospheric upto cavitation range pressures occur at the top of the step.

2. On the basis of the test data there was little difference in performance of aeration trough cover schemes 2 and 4 Fig. 39 as regards 'observed' air flow through the aeration trough.

3. The jet impacts close to the downstream edge of the aeration trough. Maximum mean pressure heads of about 150 feet occurred on the cover of Scheme 2, 120 feet on the cover of Scheme 4, 75 feet on the floor of the gate chamber and 80 feet on the wall. Minimum mean pressure heads to – 10 feet were measured on the gate chamber floor near the stoplog slots. Pressure fluctuations give higher and lower instantaneous values of heads from the maximum and minimum mean heads.

4. There were negative pressure heads measured on the chute floor at about 50 and 135 feet downstream from the P. C. of the vertical curve of chute. A minimum mean head of – 8 feet was measured on the 50 feet region and – 17 feet beyond 135 ft.

E-5.0 Model Recommendations as Accepted for Repair work.

1. Appoxy repair to gate chamber of T_3 (accepted as temporary repair only) as replacement to steel lining.

2. Change made in the design of primary airvent trough as per scheme 4 of Figure 39.

TARBELA DAM PROJECT
HYDRAULIC MODEL STUDIES

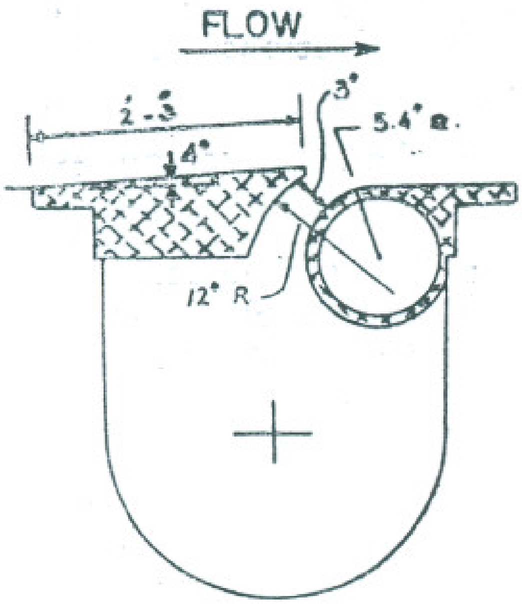


FIGURE 2_9 SCHEME 1 COVER

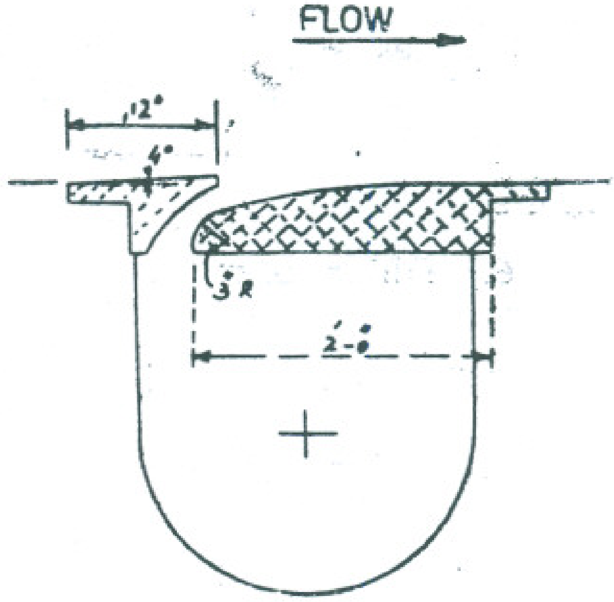


FIGURE 2_10 SCHEME 2 COVER

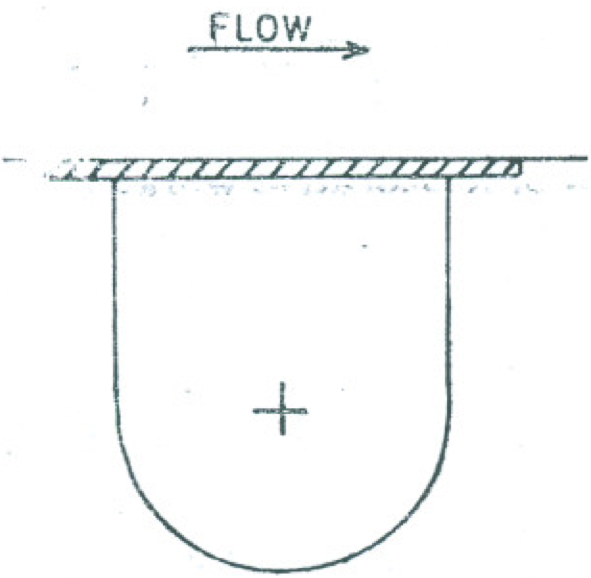


FIGURE 2 11a SCHEME 3 COVER

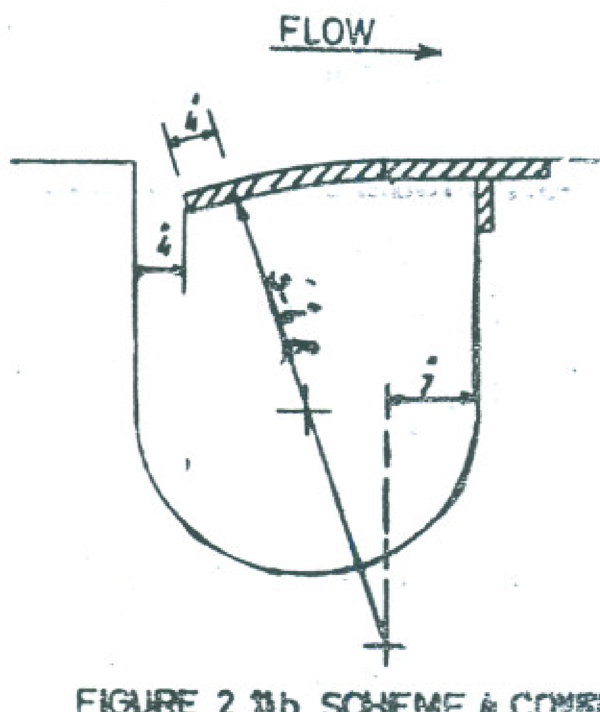


FIGURE 2 11b SCHEME 4 COVER

Fig. 39

3. Separation of the air intake duct for 3A and 3B and 4A and 4B incorporated.

4. Increase in number and length of anchor bars for steel liner plates were provided to counter the vibrations due to fluctuating pressure resulting from the jet impact.

5. Better grouting for cavitations for the same reason as in 4 above.

6. Filling of gate slot could improve flow and iessen the chances of damage on sides and on the chute (This point has to be established further due to difference in irrigation Research Institute and CSU results).

7. Holes and cavities in the chute are to be repaired to avoid cavitation.

8. Extension of central pier (by 40′) as suggested by Irrigation Research Institute,

E-6.0 Post Damage model Experiments on Tunnel 2 at Irrigation Research Institute (1975).

As stated earlier two series of experiments:

(a) For helping in the design of restorative works, and

(b) to study the causes of damages to tunnel 2 were to be carried.

E-6.1 Model studies for the Design of Restorative works in Tunnel 2.

The Model :

A complete model of Tunnel No. 2 was constructed on 1/50 natural scale at the request of Pakistan Panel of Engineers for Tarbela Dam.

The perspex model simulated in shape and alignment the full length of Tunnel No. 2. The model also incorporated a 36′ diameter sleeve as proposed with a sag in the tunnel. The downstream cone, the intake gates, the central gates, the true geometary of the passage ways at the intake and service gates were represented. The power shaft was also represented on the model. It was closed at El. 1225. The three 6" dia air-vents on the periphery of the power shaft were duly represented on the model. Air ducts opening in the crown of intake behind three gates in simulation to prototype were provided.

E-6.2 Main Features of Repair

The principal features of repairs to the Tunnel No. 2 are :—

(i) The central sluice way at the intake to be plugged with cencrete and the two side closures gates to be repaired for future operation. (as the central gate was damaged beyond repairs).

(ii) Pier extension downstream of the plug in the centre sluiceway by extending sides of piers in outside gate passages to terminate in rounded pier nose. Pier height is 10′ at downstream end and sloped upward to 22.5′ at the ends of the existing piers.

(iii) Replacement of the tunnel section from station 5+43.66 to station 10+70.54 by a sleeve of 36 ft. dia with a sharp bell-mouth at the intake (Fig. 40) and a 90 feet long transition from 36′ dia to 45′ dia at the tail end. The equations of the ellipse of bell mouth is :

$$\frac{X^2}{.5\,(D)^2} + \frac{Y^2}{.15\,(D)^2} = 1 \quad (D = 36)$$

(iv) Raising of the invert level from station 11+60.54 to station 16+62.37 by 1.5′ with a uniform flat surface at the top of the raised invert.

(v) To provide sag in the tunnel (in the scoured portion of tunnel) from station 6+87.66 to station 9+08.78. See Fig. 41.

(vi) To construct central pier at the service gate shaft and to install service gates.

(vii) To provide reducer cone from 43.5′ to 36′ in a length of 30′ at the end of Tunnel. (See Fig. 41) to throttle the flow and to avoid formation of hydraulic jump in the tunnel.

(viii) To reconstruct the fill above the intake area.

In 250 feet length of the tunnel D/S of the conduit (because the rib portion at the invert and sides were missing,) it was decided to fill the bottom with concrete upto R. L. 1089 and after raising a wall on the right side to complete it as a shell of 45 feet diameter. Outside this shell roll concrete has been filled in the gap between tunnels 2 and 3 to build the portion of hill that washed into tunnel No. 2. From an access shaft, the sleeve wall be made by concreting inside roll concrete shell.

E-6.3 Operation of Intake Diversion or Service Gates.

Tests clearly showed that uniform gate operation (right and left gates) is a must. When one gate is closed, very severe spiral flow occurs for a long distance downstream from gates creating severe hydraulic instability. Tunnel 2 will be operated in the flood season of 1975, only in the event of an emergency requiring draw-down of the reservoir for such an operation the intake gates shall be opened fully while the service gates are in a closed position. In this manner, the hydraulic head on the upstream and downstream sides of the intake gates will be exactly the same. Flow through the tunnel will then be started by opening both service gates concurrently to 45 feet.

Various gate opening rates under various heads were tested with the purpose of determining areas of low pressures and the magnitude of fluctuating pressures with pace transducers, as well as velocities generated in the Tunnel downstream of service gates. The conclusions are summarised below :—

E-6.4 Conclusions.

1. Velocities varying from 160 feet per second to 80 feet per second in the concrete transition (from rectangular to circular section) downstream of the service gates were recorded on the model during operation of the service gates in the upward direction. Such high velocities can cause damage to the 90′ unlined transition leading to erosion of concrete at the start of the steel liner which could then be ripped up with the force of very high velocities.

2. As the velocity order is very high, the time taken for

lifting up the gates should be minimum possible to minimise the chances of erosion of the concrete in the unlined transition and the pier of service gates. Upto a gate opening of 30′ the tunnel flows free at Res. elevation upto 1500. The rate of opening of gates from gate opening of 30′ to 40′ should be at the optimum rate of one ft. per one and half minute to one foot per miuute as at this rate the uplift pressure at the bottom of service gates at the time of arrival of bore wave from the end cone will be minimum (Bore wave is the standing wave moving upstream from the end cone towards the service gates before the tunnel starts running full bore).

3. During operation of the service gates there will be negative pressure of the order of −20′ on the sides of the tunnel d/s of the service gates.

   The flow in the bore wave creates fluctuations of pressures in bore wave as high as 70 feet head of water.

4. Under the above serious conditions steel lining of unlined transition of 90′ length between the service gates and the start of steel liner is considered essential. As steel lining of 90′ transition was not possible at site it was expoxy painted.

5. Uniform gate operation (right and left gate) is a must, while one gate is closed and the other open, very severe, spiral flow occurs for a long distance downstream of the gates, creating severe hydraulic instability. It was agreed to by TAMS that unequal gate opening is definitely not permissible.

6. A severe vortex forms in the service gate slots due to sudden turning of flow downstream of the upstream sealing edge under the botom of the gate. This vortex creates a swirl velocity of 145′ in the section and pressure fluctuations equal to 140′ head of water at Res. El. 1500 and gate opening of 20 feet. This severe vortex may cause cavitation of slots and lateral movements of gates causing damage to wheel flanges and guide rails. The gates may also get stuck-up under such a situation.

was negative at the time of blow out upstream of the cone and was of the order of 12 feet.

(*b*) Negative pressures on the crown of sleeve tunnel at and blow Res. El. 1200 were indicated by manometer readings. These negative pressures were recorded with transducers at different points at reservoir elevation of 1200, 1180, 1160, 1140 and 1137 and the pressure – head and time traces were made. The lowest negative pressure were recorded at one point were – 14, – 16, – 10, – 18, and – 13′ at reservoir elevation of 1200, 1180, 1160, 1140 and 1137 respectively. The negative pressure at other points does not exceed – 12 feet, which was recorded at reservoir elevation of 1140 and 1137.

(*c*) At Res. El. 1145, the 45′ dia tunnel becomes free, whereas the sleeve is till running full bore and the air bubbles in the flow, discharge in the air space in the 45′ dia tunnel near the crown. At Res. El. below 1140 and above El. 1130, two jumps form in the upstream portion of the sleeve near the sag portion. The two jumps are separated by a hump touching the crown. The hump gets eliminated periodically and the two jumps merge in a single jump. The fluctuations of pressures at the hump were recorded with transducer which indicate negative pressures of the order of – 10 feet.

E-6.5 Model Studies on Tunnel 2 to Investigate the cause of Damage.

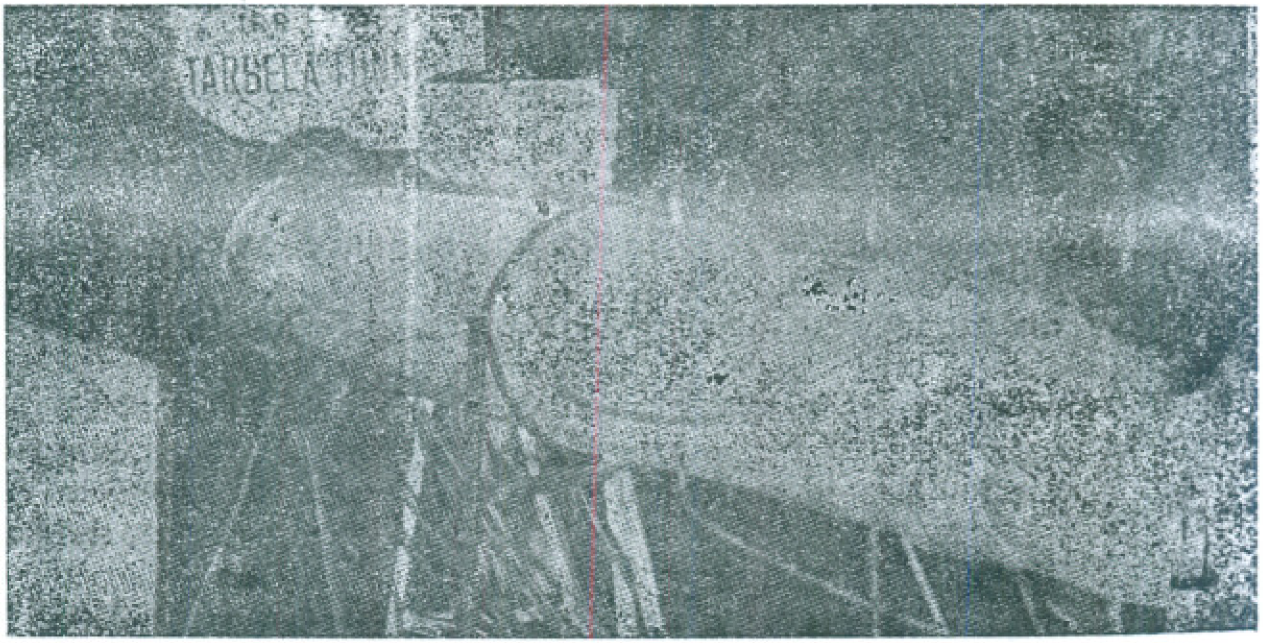
The experiments to investigate the hydraulic causes of damage to the 250′ long section downstream of the conduit portion, the damage to the piers at the intake, etc. were started more recently and are in final stages of completion.

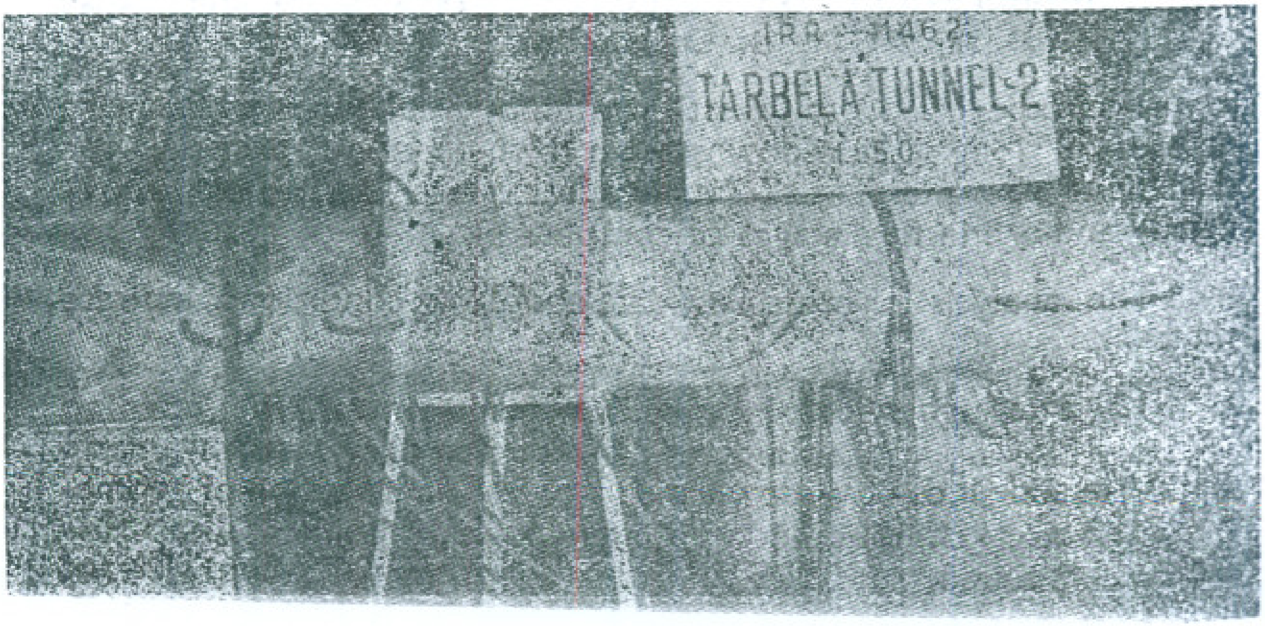
The analysis of pressures data and flow conditions indicate that:—

1. The high velocity jetting flow from central gate (opening 45-27') with side gates closed being fully aerated on top and sides dives down and impinges on a section of geometrical transition at an oblique angle, rebounds after dispersion, and rides the sides spiralling to other side of

TARBELA DAM, TUNNEL-2
ORIGINAL DESIGN
1 : 50
Flow Conditions through 45′ DIA Tunnel
From STA 5+43.54
Res. EL. 1426.5
Gates
L 0
C 45
R 0

Fig. 42 (a, b)



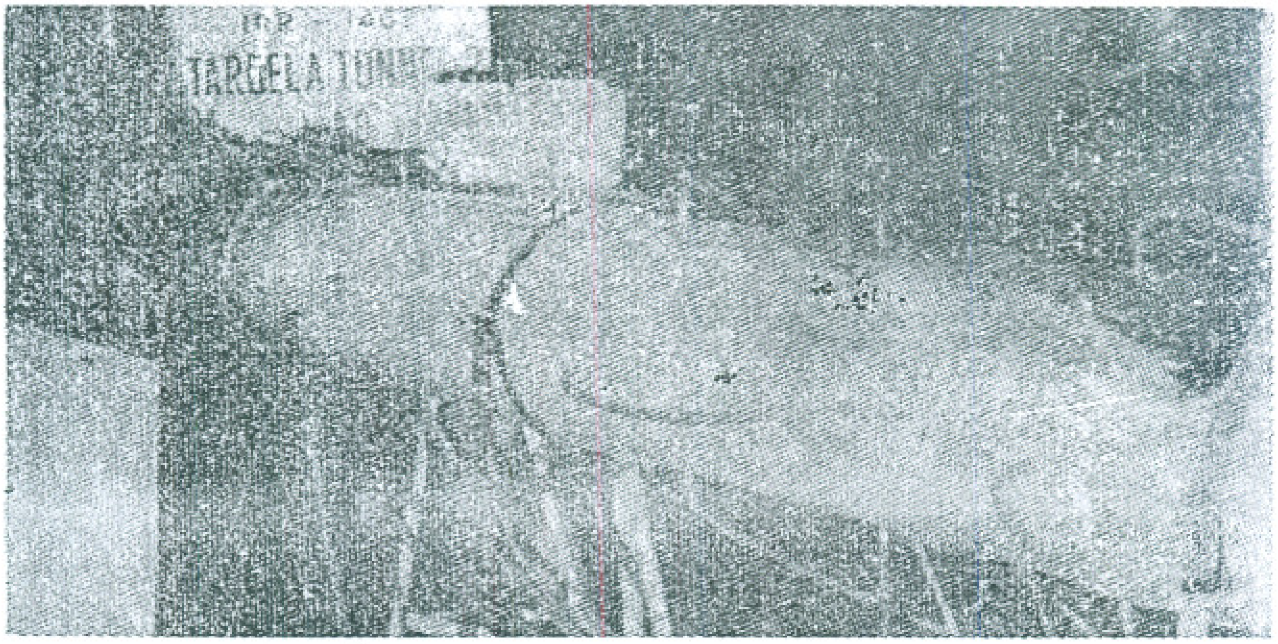


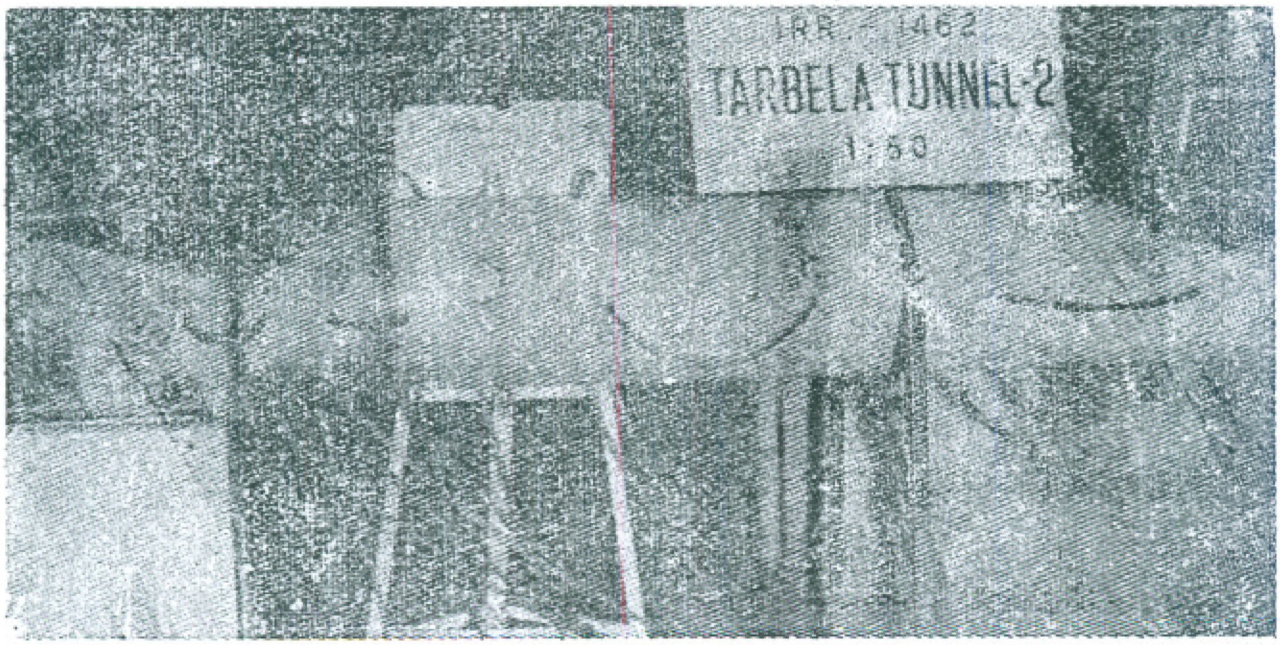
TARBELA DAM, TUNNEL-2
ORIGINAL DESIGN
1 : 50
Flow Conditions through 45' DIA Tunnel
From STA 5+43.54
Res. EL. 1426.5

Gates

Fig. 42 (a, b)

L 0
C 45
R 0





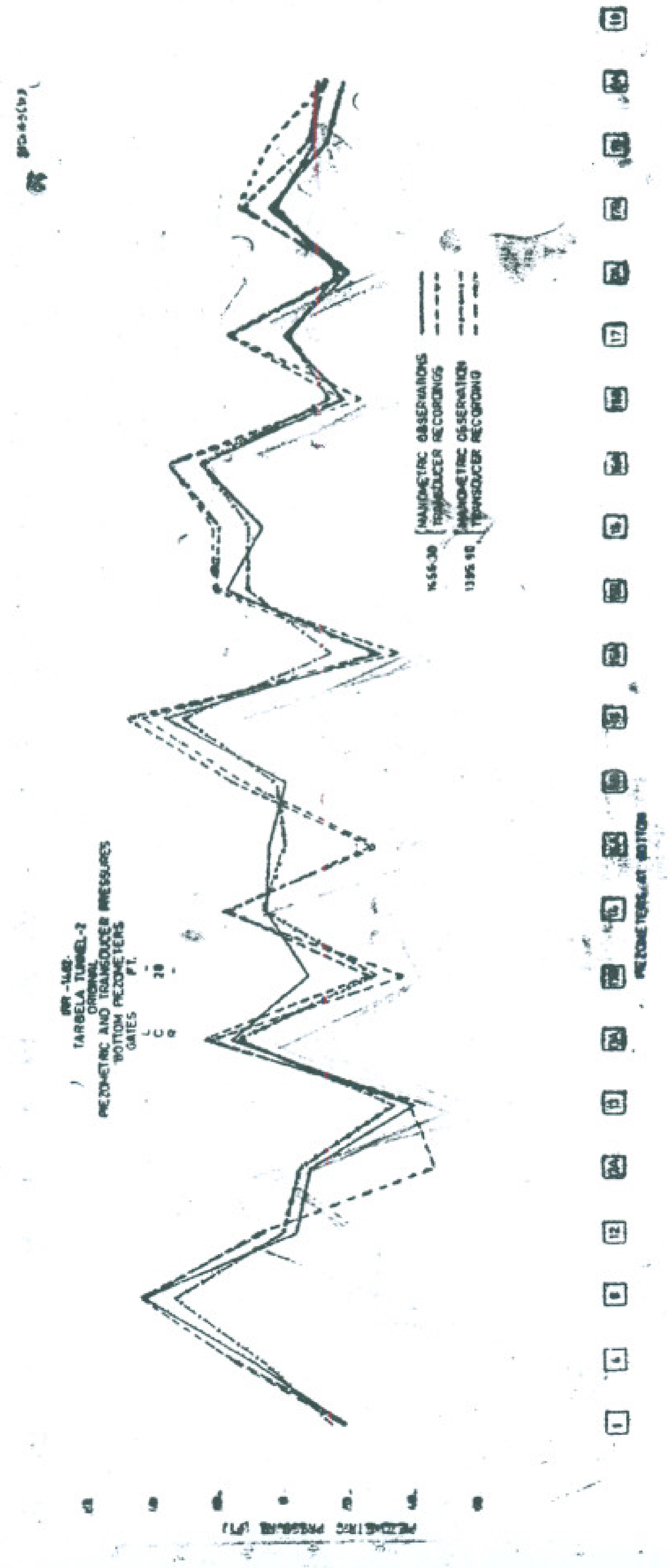

TARBELA TUNNEL-2
ORIGINAL
PIEZOMETRIC AND TRANSDUCER PRESSURES
BOTTOM PIEZOMETERS
MANOMETRIC OBSERVATIONS
TRANSDUCER RECORDINGS
MANOMETRIC OBSERVATION
TRANSDUCER RECORDING
PIEZOMETERS AT BOTTOM

Fig. 43-b

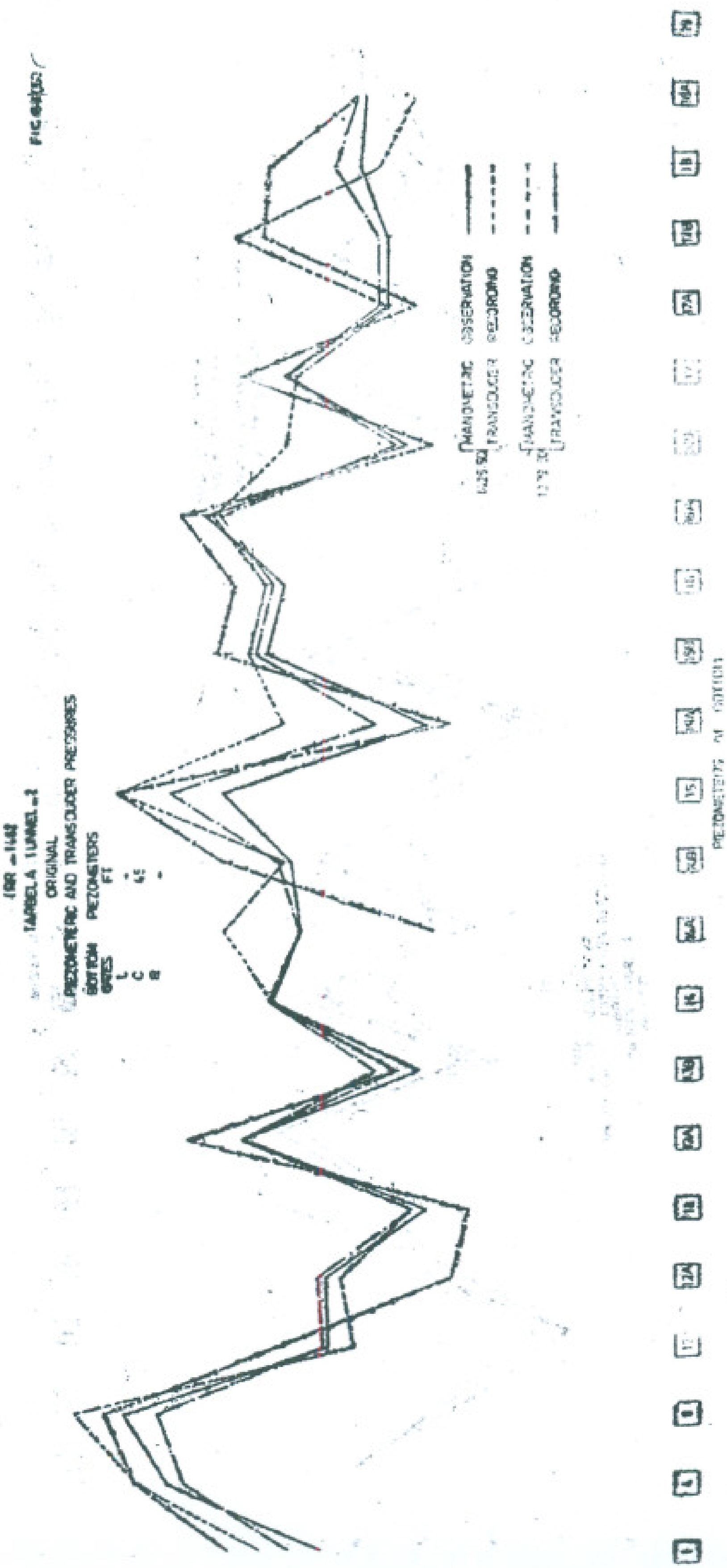


Fig. 43-C

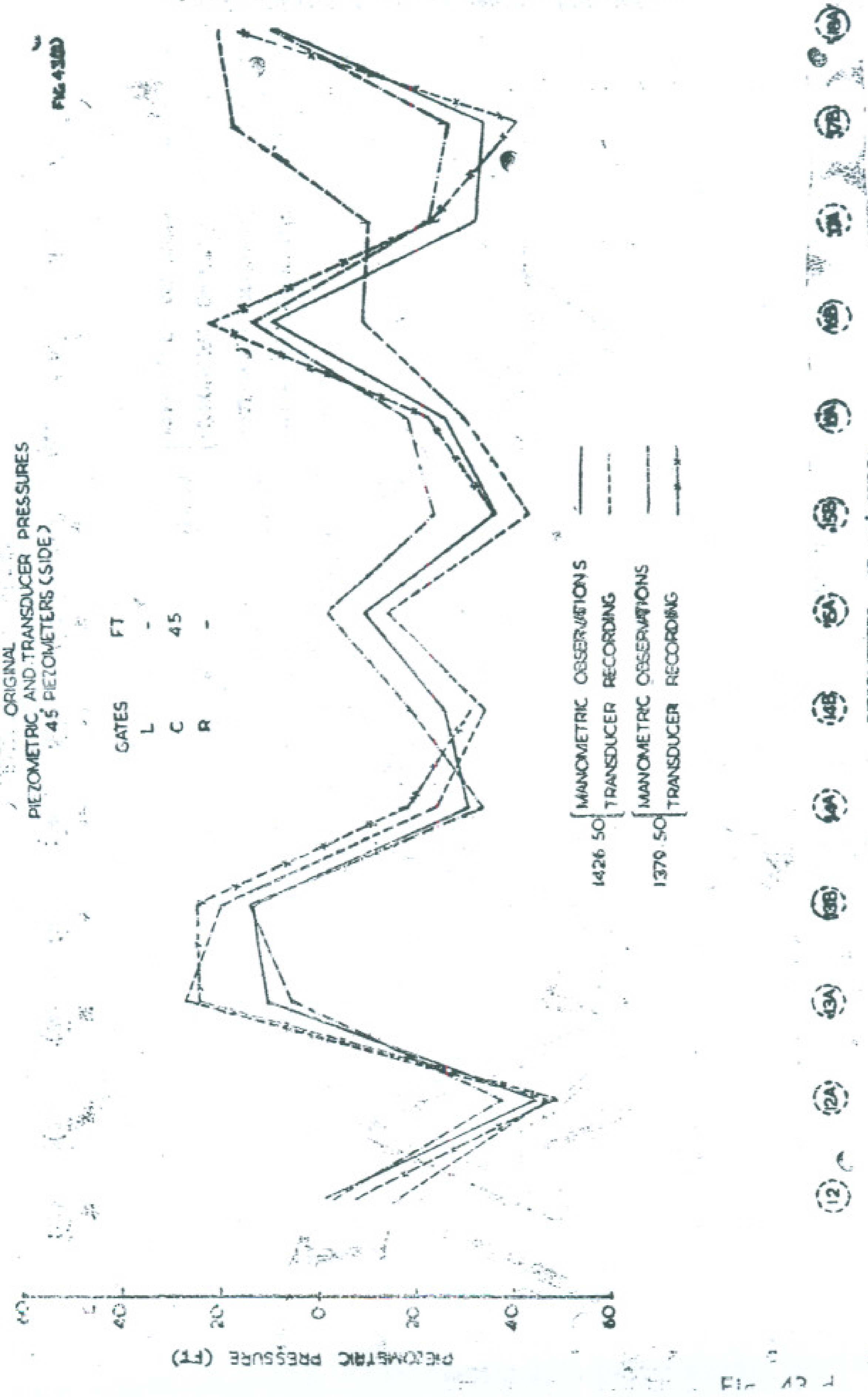
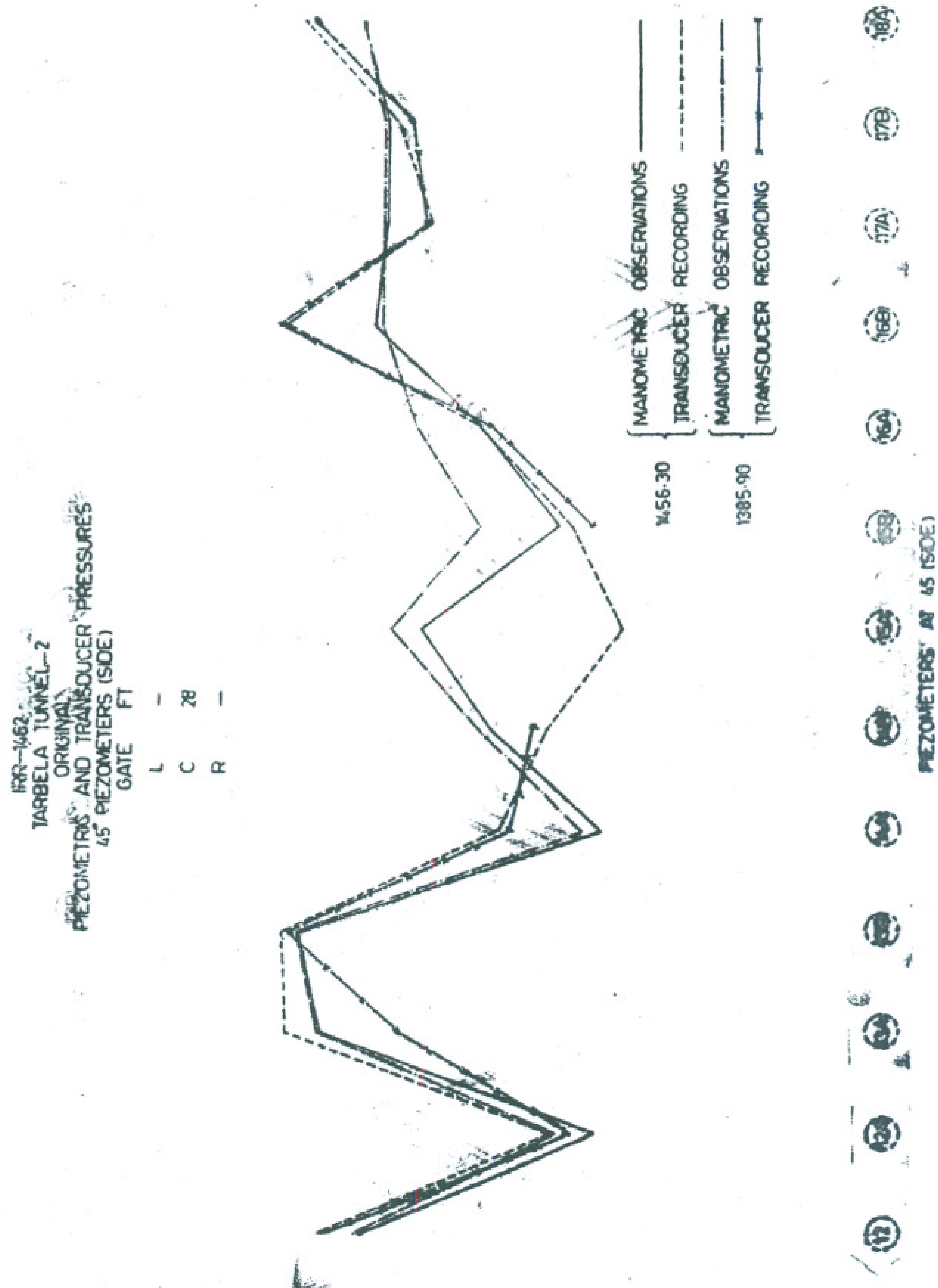


FIG. 43(b)
IRR-1462
TARBELA TUNNEL-2
ORIGINAL
PIEZOMETRIC AND TRANSDUCER PRESSURES
45° PIEZOMETERS (SIDE)
GATES FT
L -
C 45
R -
1426 SO MANOMETRIC OBSERVATIONS
TRANSDUCER RECORDING
1370 SO MANOMETRIC OBSERVATIONS
TRANSDUCER RECORDING
PIEZOMETRIC PRESSURE (FT)
40
20
0
20
40
60
12
12A
13A
13B
14A
14B
15A
15B
16A
16B
17A
17B
18A
PIEZOMETERS AT 45° (SIDE)


IRR-1462
TARBELA TUNNEL-2
ORIGINAL
PIEZOMETRIC AND TRANSDUCER PRESSURES
45° PIEZOMETERS (SIDE)
GATE FT
L —
C 28
R —
1456.30
1385.90
MANOMETRIC OBSERVATIONS
TRANSDUCER RECORDING
MANOMETRIC OBSERVATIONS
TRANSDUCER RECORDING
17B
17A
16B
16A
PIEZOMETERS AT 45 (SIDE)

transition. The two spiralling flows interact and form a single white jetting flow that follows the bottom section of tunnels. (See Fig. 42). The peizometeric and transducer pressures along invert and 45° angle in the damaged portion of tunnel are plotted in Fig. 43 a-e which indicate negative pressures up to 40′ feet during operational condition of 1974. The negative pressure could result in a damage by cavitation. Abrasion due to high velocity of the order of 160 ft. per/sec. can also occur. It is to be noted that material round this tunnel being Sugary lime is highly pervious. Inside negative pressure combined with outside positive pressure built up due to seepage may have caused the tunnel failure. Once the tunnel wall got damaged and punctured in the initial stages the hill consisting of sugary lime stone could be intermittently sucked into the tunnel, explaining the turning of the river water emerging from tunnel 2 intermittently brownish red for a few minutes. The damage progressively increased once the tunnel gave away at one place.

2. The unequal gate openings create highly supercritical flow with shock waves in the tunnel.

3. The model shows a negative pressure of the order of —28 feet at the piers immediately D/S of the gates and high unbalanced cross pressure on the piers U/S of the gates. The intake piers were damaged due to cavitation in the areas which were not ventilated to air.

In the unlined portion, the damage to the concrete in the form of pot holes resulted from the hill rock that passed hurtling through the tunnel. The lined portion remained without a major damage.

GENERAL SUMMARY AND CONCLUSIONS

As the present paper presents an exhaustive resume of results of model tests conducted in respect of vexed problems relating to different elements of Tarbala Dam carried out during 1963 to 1975 at Irrigation research Institute, in Pakistan, at CSU, and Alden Laboratories in U. S. A. conclusion of each study at the end of the paper is rather difficult. The method adopted is to present each study with brief back ground, scope of study, some details of experimental set up followed up with the summary and conclusions. Thereafter the recommendations as used by the designers and the performance of the works on the prototype has also been briefly discussed for each of the following distinct stages of construction and components of the project.

1. Stage I Diversion.

STUDIES ON Indus river bridge and Cofferdams constructtion.

2. Stage II Conversion.

Studies on River Diversion through the diversion channel, and buttress structure, location and protection of upstream and downstream coffer-dams for the main Dam.

3. Stage III Diversion.

Studies on power cum diversion tunnels with their intakes and out fall structures-studies on Irrigation tunnels, their Intakes and then outlet structures including the stilling basins.

4. Studies on Hydraulic design features of the service spillway auxiliary spillway and Daldarra channel.

In the final summary and conclusions it is proposed to recount only the important points of model results reflecting on hydraulic simlitude or divergence and departures from the model results, that have caused the damages that were experienced.

The General Conclusions are:

1. The hydraulic models have faithfully represented the prototype conditions and solved most of the problems in the first and second stages of diversion very satisfactorily.

2. For the third stage diversion it appears that the complete model of tunnel No. 1 on 1/69-6 natural scale was not fully utilized for highlighting the possible zones of danger under high heads during dumping of reservoir as it was originally envisaged that during diversion stages the reservoir elevation will not rise above elevation

1300. The complete model tunnel No. 1 has not been fully used. The 1/80 natural scale model at Nandipur was utilized for testing outlet areas of tunnels 1 & 2 for high head operations during 1971. The actual flow conditions in the outlet area of Tunnel 1 & 2 were very similar to that observed on model as is evident from flow current in outlet area of Tunnel 1 and 2 at Reservoir elevation of 1225 on model and prototype. (Fig. 44). The cell No. 18 of cellular cofferdam was the first to collapse on the prototype. The 1/80 scale model at Nandipur had clearly indicated that there will be quite head across at cell No. 18, a high velocity flow (Velocity of the order of 31 fps) will brush past this cell and there will be heavy scour at the outer periphery of this cell. The cells were not taken into the rock and adequate protection against scour was not provided and so the cells of cellular cofferdam from 18-14 collapsed due to heavy scour.

3. All the assential features of prototype that can reflect on the flow pattern or pressure distribution in a supercritical flow should be essentially reproduced on the model and the model scale should be as large as possible to commensurate with the cost of project feature and existing (or even en-hanced) facilities available in a Laboratory.

4. The testing of intake gate operation at C. S. U. on 1/69.6 model and the sequence of gate operation showed that the opening of the central gate is not at all desirable. It should be the last gate to be opened and that the symmetrical gate operation is a must.

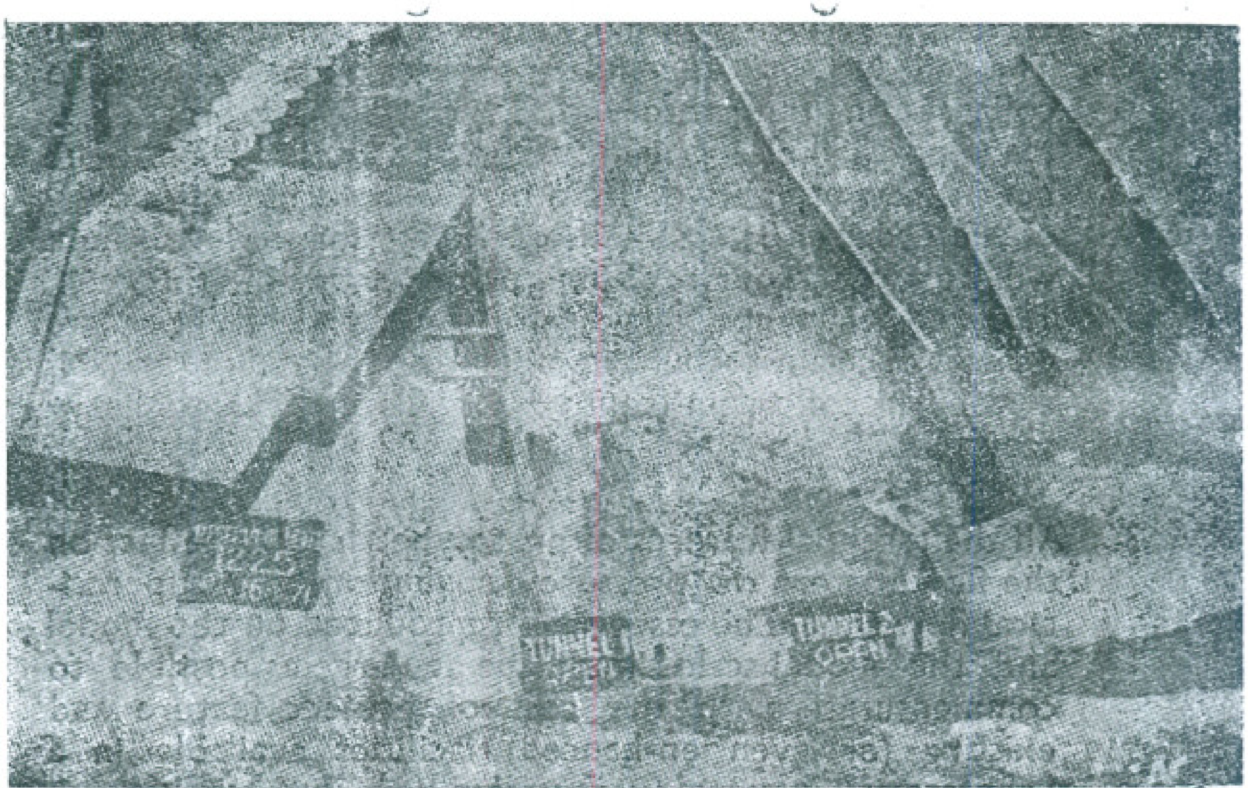
5. Opening of central gate on any pretext (whether it is from the fact that, the side gates when open, the outlet capacity is 45% of the full capacity and if the central gate is open it has 90% of the full capacity, or to save the cells downstream) WAS BASICALLY IN CONTRAVENTION OF REGULATION PATTERN indicated from model studies. This is one of the major causes of damage to Tunnel 2.

Fig. 44 (a)

TARBELA DAM PROJECT

RIVER DIVERSION MODEL

Scale = 1 : 80



Flow Conditions in Tunnels outlet area with Tunnels 1 & 2 open

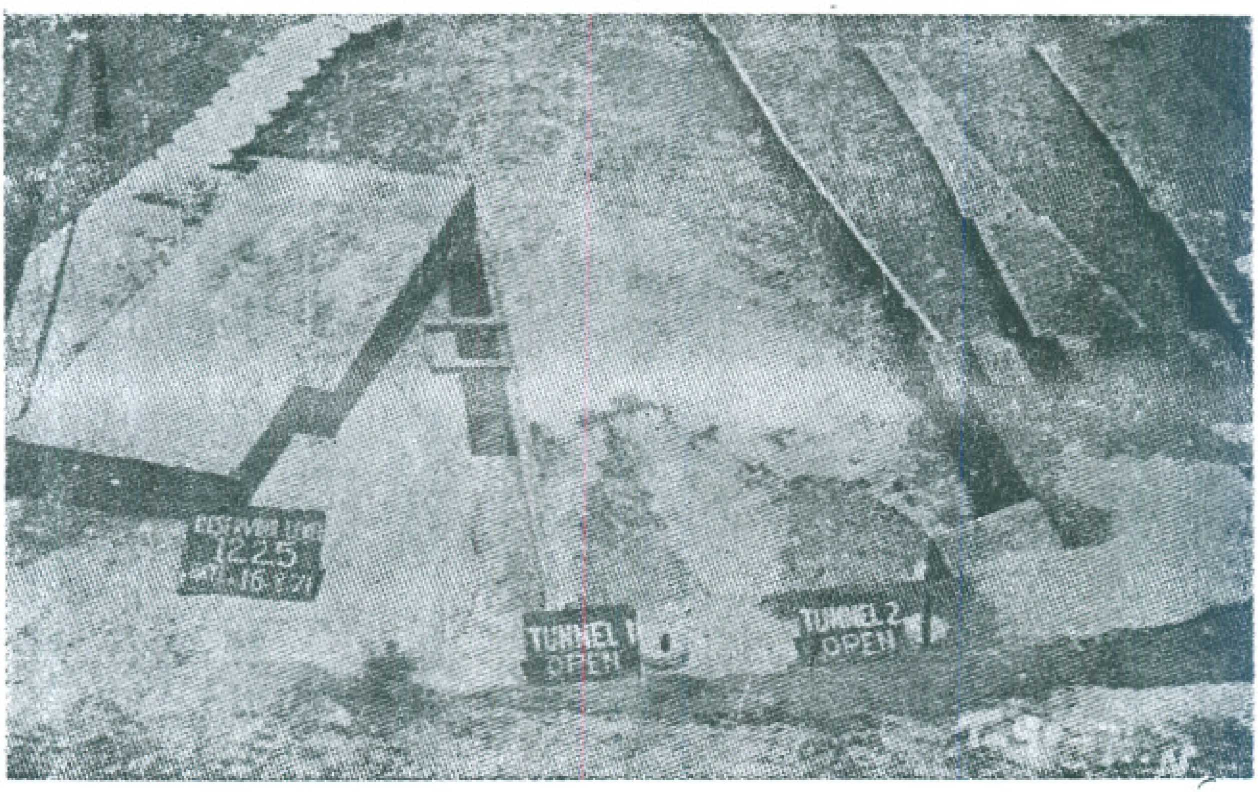
Res, EL. 1225

Fig. 44 (a)

TARBELA DAM PROJECT

RIVER DIVERSION MODEL

Scale = 1 : 80

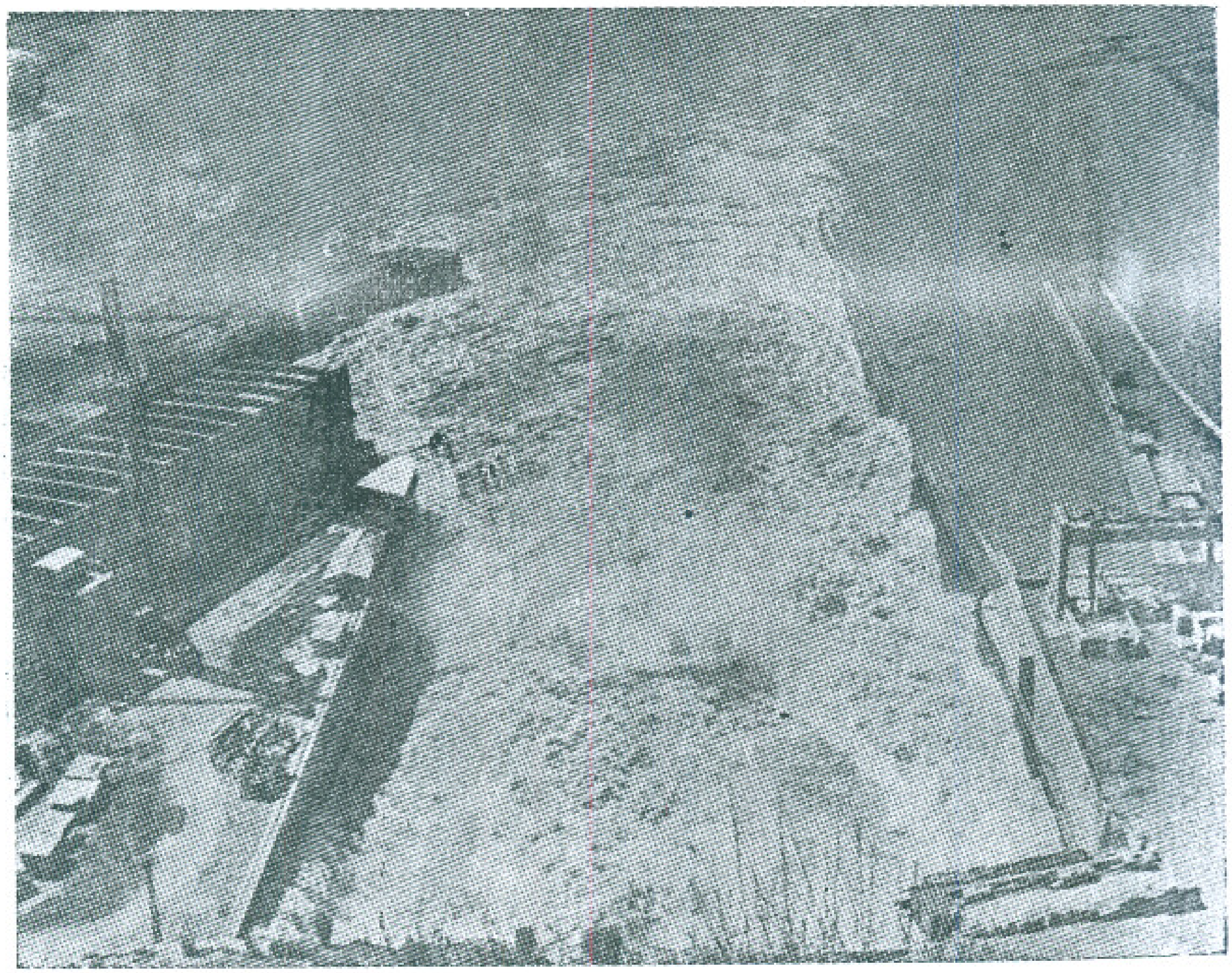


Flow Conditions in Tunnels outlet area with Tunnels 1 & 2 open

Res, EL. 1225

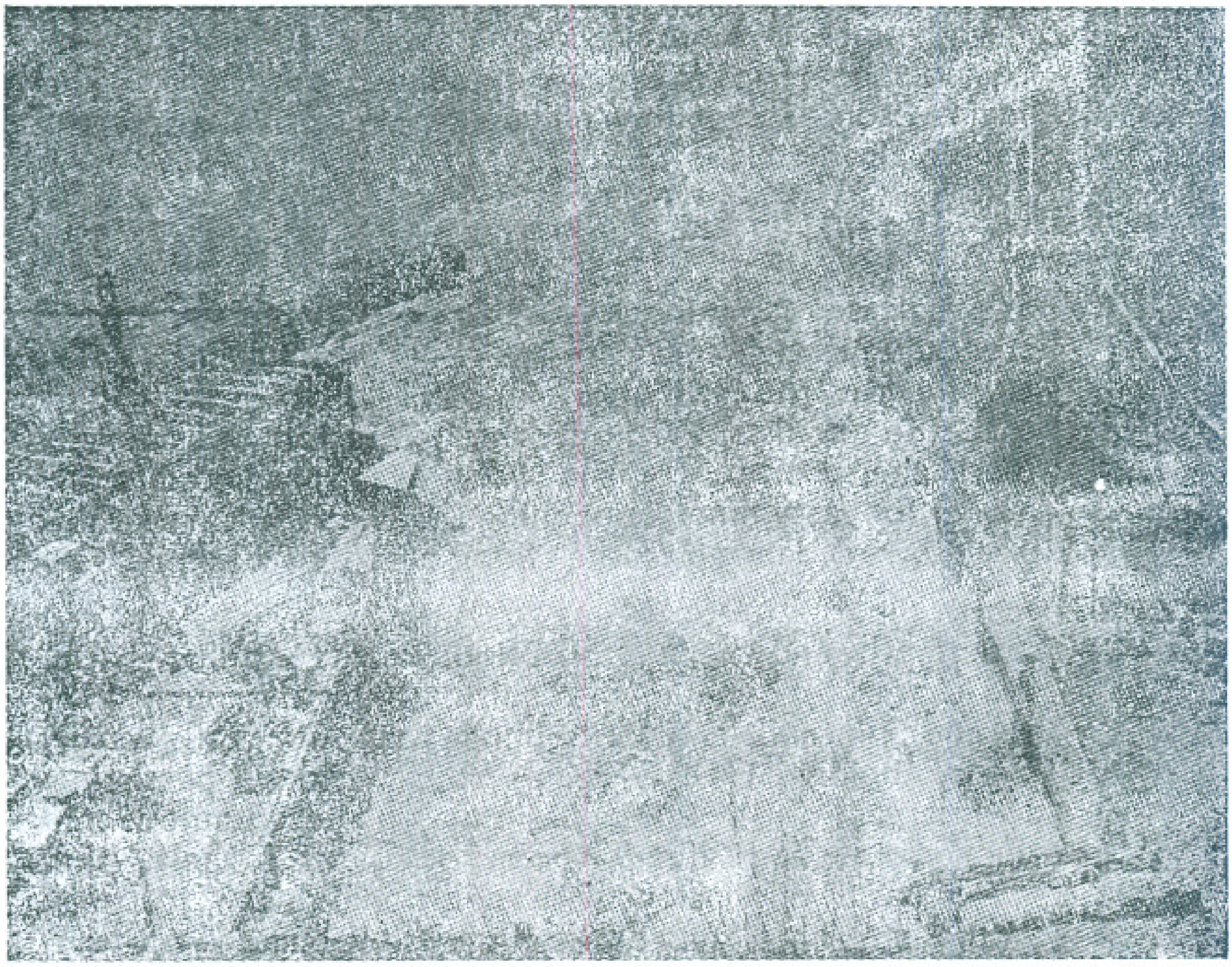
Fig. 44 (b)

TARBELA DAM PROJECT
STAGE III RIVER DIVERSION
PROTOTYPE



Flow Conditions in Tunnels outlet area with Tunnels I & 2 open

Fig. 44 (b)
TARBELA DAM PROJECT
STAGE III RIVER DIVERSION PROTOTYPE
PRQTOTYPE



Flow Conditions in Tunnels outlet area with Tunnels I & 2 open

6. The high order of fluctuating pressures on the steel liner plates of gate chamber downstream of the primary air-vent combined with inadequate anchor bars for the steel liner plate, hollows under neath and inadequate fixing of the grid cover of the primary air system to withstand the jet impact and the vibrations could have caused the uprooting of the grid and ripping up of plates. The negative pressures downstream of toe of the step in gate chamber being sought in the model was not there. The main lesson is that high fluctuating positive pressures should also be given full regard in design.

7. The assymtric opening of T_{3A}, T_{3B} and T_{4A}, T_{4B} as predicated from model studies caused back entry of stones, which when caught in highly turbulent high velocity flow in the stilling basin damaged the floor and the side walls. Without the extension of central piers the damage to the floor was predicted from model for assymmetric openings. The result naturally was severe damage to the floor of stilling basin of T_3.

8. The depth and the length of floor of the stilling basin was kept at bare minimum resulting in highly rough flow at end cill and D/S of stilling basin for tail water levels below 1114. Such rough flow persisting downstream of the stilling basin may be acceptable when the rock is good, but for poor rock as at Tarbela, the stilling basin dimensions with additional safety margins should have been more thoroughly investigated on the model. The heights of the walls were also kept lower than recommended by model studies resulting in heavy splashing over the walls.

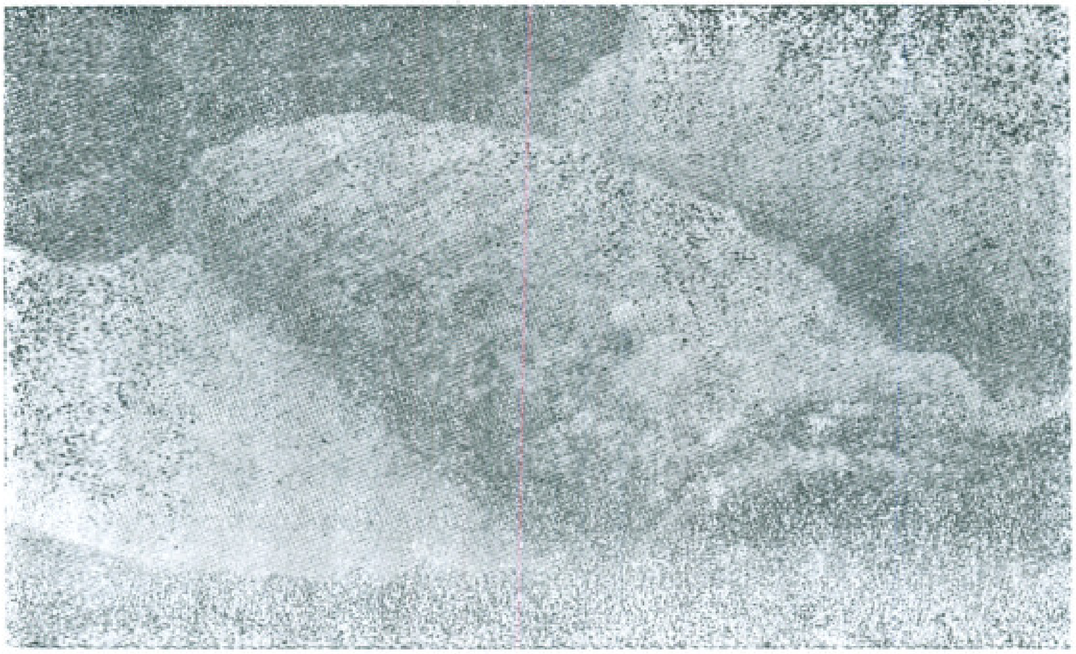
9. The heavy erosion of the left bank of Daldarra near its outfall into the main river was predicted from the model. Earlier action for the safety of electric towers would have avoided the necessity of re-running Tunnel No. 3. It is known that the more damage to the floor of stilling basin occurred during this period of re-running of Tunnel 3 and 4.

10. The general performance of the spillways was according to model predictions and was very satisfactory. The side actions are also according to model predictions. The rock downstream of stilling basins and spillways (excepting for igneous black sound rock) is erodible and plunge pool scours as predicted from the model have formed away from the flip bucket Fig. 45 (a, b) & 46 (a, b).

11. The exact simulation of rock erosion on model is not possible as model material is yet to be evolved that represents the characteristics of non-homogenous and layered rock on the prototype.

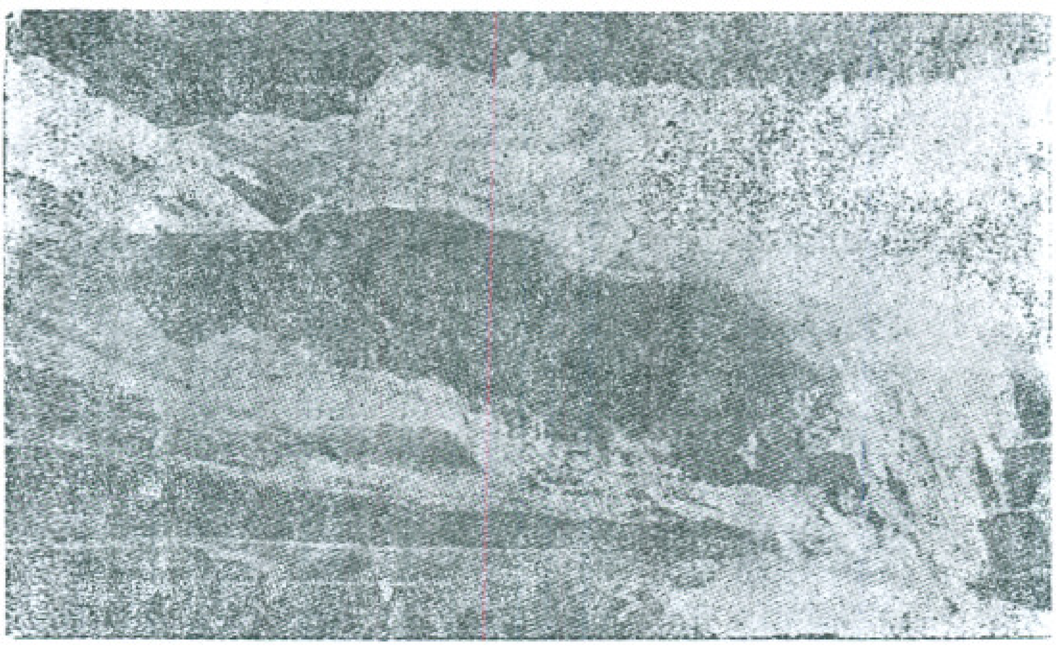
TARBELA DAM
SERVICE SPILLWAY

Fig. 45 (a)



The plung pool has formed some distance away from the flip bucket.

Fig. 45 (b)



B

Notice erosion on both flanks in the plunge pool area in the tail face of service spillway. The basic rock 'B' is still holding where as the poor rock upstream of it has gone.

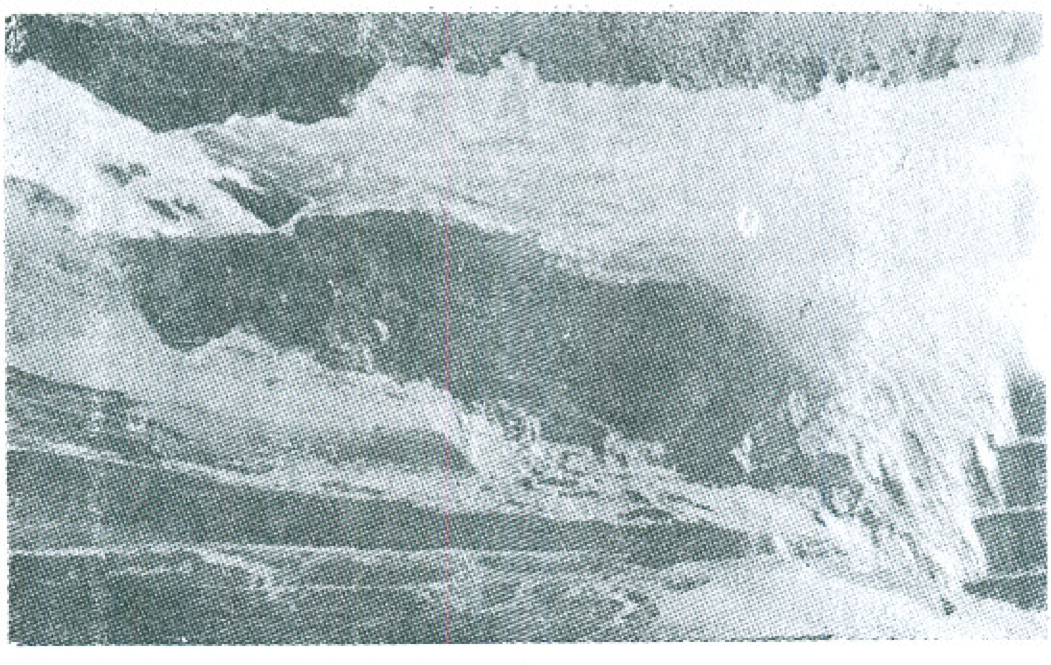
TARBELA DAM
SERVICE SPILLWAY

Fig. 45 (a)

The plung pool has formed some distancea way from the flip bucket.

Fig. 45 (b)

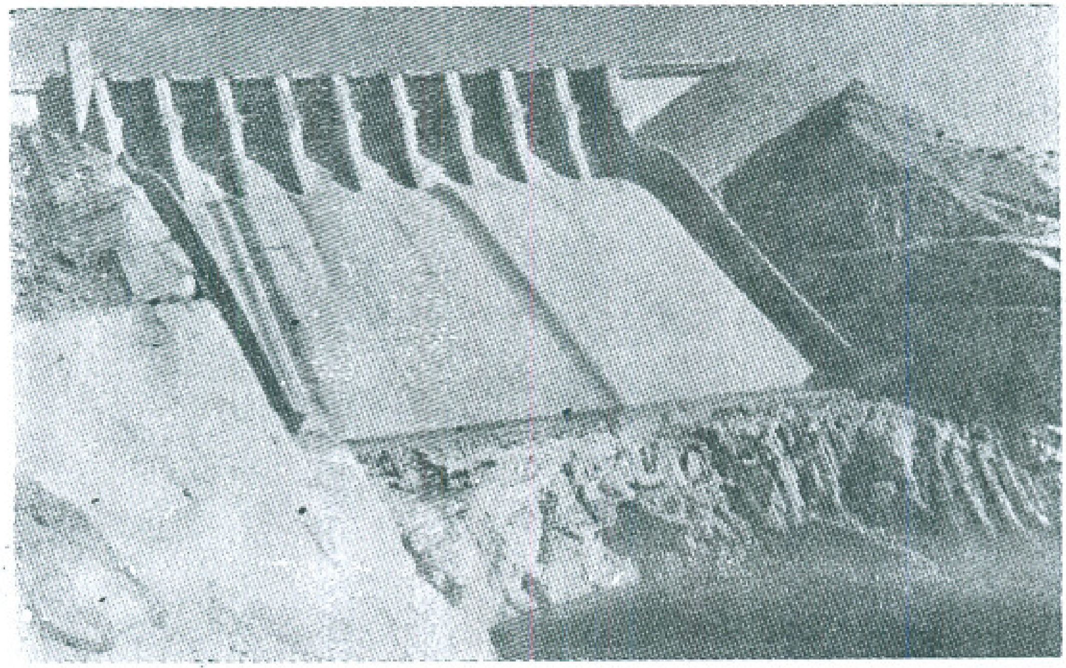
B



Notice erosion on both flanks in the plunge pool area in the tail face of service spillway. The basic rock 'B' is still holding where s the poor rock upstream of it has gone.

TARBELA DAM PROJECT
AUXILIARY SPILLWAY

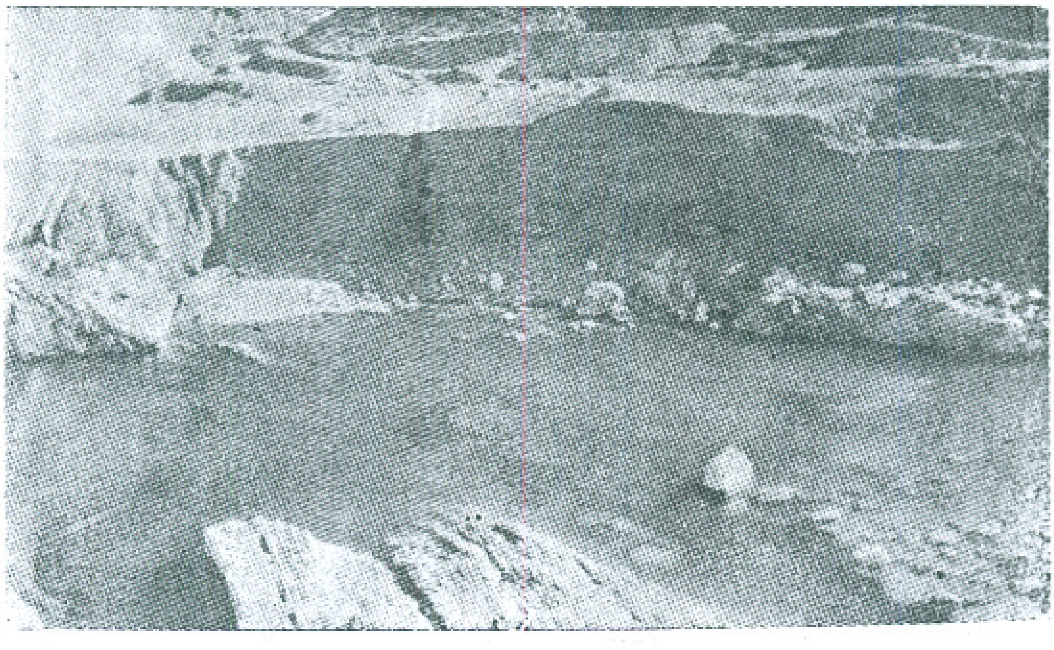
Fig. 46 (a)



The plunge pool has formed some distance away from the flip bucket.

Fig. 46 (b)

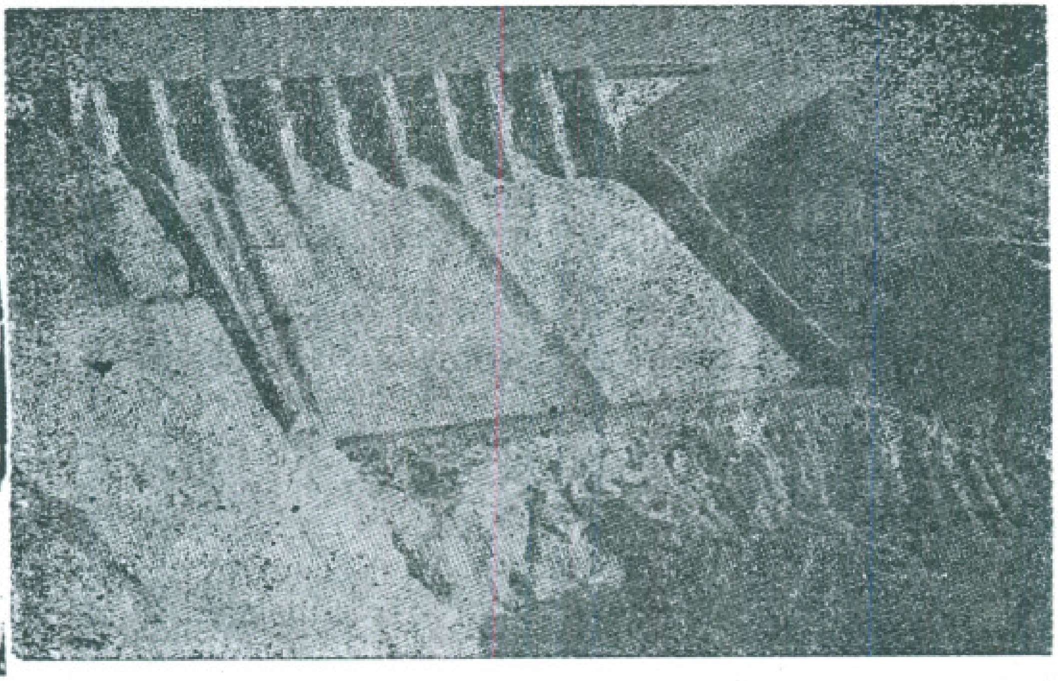
A



Notice erosion on left bank immediately D/S of the flip bucket. The flip bucket at its left end is protected by rock at 'A'

TARBELA DAM PROJECT Fig. 46 (a)

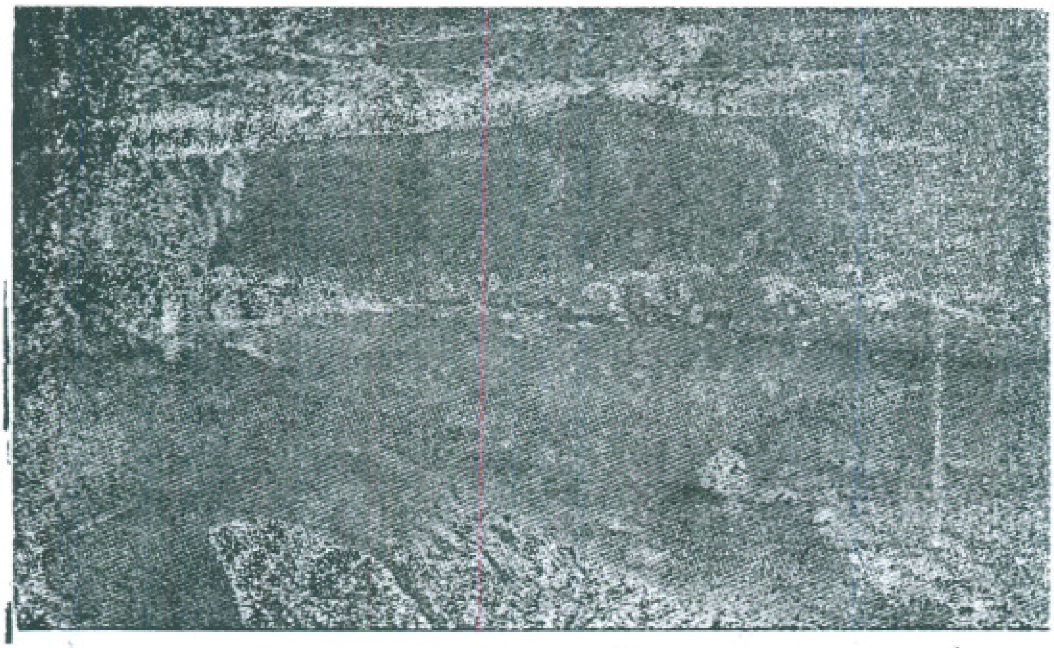
AUXILIARY SPILLWAY



The plunge pool has formed some distance away from the flip bucket.

Fig. 46 (b)

A



Notice erosion on left bank immediately D/S of the flip bucket. The flip bucket at its left end is protected by rock at 'A'

Acknowledgement :

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