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Division of Engineering Laboratories
Hydraulic Laboratory Branch

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES--WU-SHEH DAM TUNNEL SPILLWAY--WU-SHEH DAM PROJECT TAIWAN, CHINA

Hydraulic Laboratory Report No. Hyd-430

DIVISION OF ENGINEERING LABORATORIES

COMMISSIONER'S OFFICE
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FOREWORD

The model studies described in this report were performed in the Hydraulic Laboratory of the Bureau of Reclamation at Denver, Colorado, during the period January through May 1955.

The recommended structure was developed through the cooperation of the staffs of the Concrete Dams Section and the Hydraulic Laboratory.

During the course of the model studies Messrs. L. G. Puls, E. R. Dexter, Max Ford, A. T. Lewis, N. W. Cash, Abe Olshansky, and H. N. Cole of the Concrete Dams Section frequently visited the laboratory to observe the tests and to discuss the results.

The studies were conducted by T. J. Rhone and supervised by A. J. Peterka under the laboratory direction of H. M. Martin.

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Commissioner's Office--Denver
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HYDRAULIC MODEL STUDIES--WU-SHEH DAM TUNNEL SPILLWAY--WU-SHEH DAM PROJECT, TAIWAN, CHINA

SUMMARY

The model studies described herein were performed on a 1:41.25 scale model of the tunnel spillway, and a 1:25.28 scale model of the spillway over the dam. Figures 2, 2A, 3, and 21.

The investigations of the approach and entrance to the spillway showed that flow conditions at the left and center piers were satisfactory, but at the preliminary right pier the flow was very rough and a large water surface drawdown occurred, Figure 5. The resulting uneven flow distribution carried down into the elbow and almost filled the tunnel. An overhanging pier was developed, Figures 4 and 6, that produced smooth flow around the pier and down into the tapered shaft.

The discharge-capacity calibration of the tunnel spillway showed that the maximum discharge of 66,000 cfs could be attained at the design head, or reservoir elevation 3297.24, Figure 8. Piezometer measurements on the spillway crest and transition floor showed that no dangerous subatmospheric pressures occurred at any discharge.

Observations of the flow in the tapered shaft between the transition and the vertical bend showed that the design was adequate in every respect and that no changes were necessary. The flow in the 150-foot radius preliminary vertical bend or elbow had an unsatisfactory rough water surface caused by a too abrupt change in direction, Figure 12. The elbow was rebuilt and tested using a 250-foot radius. The water surface was much smoother; and although the flow depth in the conduit was about 0.85 of the diameter at the maximum discharge, performance was considered satisfactory, Figure 13. Flow in the horizontal conduit from the elbow to the flip bucket was satisfactory.

Performance of the preliminary flip bucket was not satisfactory. The curved floor of the bucket flipped the jet away from the structure, but the jet was directed away from the center of the river and consequently the impinging flow caused a considerable amount of

damage to the left riverbank, Figure 15A. A bucket was developed that incorporated a horizontal turn to the right and a superelevated floor, Figure 18. This bucket directed the flow toward the center of the river channel in a stable jet, Figure 19. Piezometer measurements on the walls of the bucket indicated that the left wall should be designed to withstand above atmospheric pressures as high as 90 feet of water. The pressures on the right wall were considerably lower but were above atmospheric for all discharges.

The structures recommended for prototype construction, on the basis of the model tests, are shown in Figures 4, 7, 9, 11, and 18.

Also included in this report, Figure 22, are curves showing the discharge-capacity calibration of the emergency overflow spillway located in the arch dam, Figure 2. These curves were obtained from a 1:25.28 scale sectional model of the emergency spillway, Figure 21.

INTRODUCTION

The Wu-Sheh Dam tunnel spillway is one feature of the Wu-Sheh Dam Project. The project is being constructed by the Taiwan Power Company to provide desperately needed electricity for Taiwan, China (Formosa). The project was designed by the Bureau of Reclamation through arrangements with the FOA.

The Wu-Sheh Project is a hydroelectric development located on the Wu-Sheh River in Central Taiwan, Figure 1. The project includes a concrete gravity arch dam approximately 300 feet high backing up the Wu-Sheh River; the Wan Ta Powerplant, an existing structure that is being enlarged; the tunnel spillway, located upstream from the dam; and an emergency overflow spillway in the center of the dam, Figure 2.

The emergency spillway is a 90-foot wide, radial-gate-controlled overflow spillway located in the center of the gravity arch dam. The maximum capacity of the emergency spillway is 30,000 cfs. Since this spillway discharges into the river upstream from the powerhouse, it will raise the water level in the powerhouse tailrace and reduce the power output; consequently, it will seldom be used.

The tunnel spillway, with a capacity of 66,000 cfs, is a fixed-wheel gate-controlled structure that will be used to pass the river runoff in excess of the quantity that can be stored in the reservoir, Figure 2A. The tunnel discharges into the river downstream from the powerhouse. Figure 2 shows the relationship of the various features of the Wu-Sheh Project.

The hydraulic model tests were concerned primarily with developing the hydraulic features of the tunnel spillway. The emergency spillway was not studied by extensive tests; however, a small sectional model of the crest section was built so that a discharge-capacity calibration could be obtained. The results of this calibration are included in this report as Figure 22.

THE MODEL

A model scale of 1:41.25 was selected so that the head box and tail box previously built for the model studies of Yellowtail Dam could be reused with a savings in both time and cost.

The model included the head box containing the portal of the tunnel spillway and the topography and approach area near the portal, the tail box containing the flip bucket and a part of the river downstream from the powerhouse, and approximately 20 feet of transparent plastic pipe representing the tunnel between the spillway and the flip bucket, Figure 3.

The head box was a galvanized-sheet-metal-lined wood structure about 12 feet square. The head box contained the approach channel and topography in the spillway vicinity, the spillway crest, and the tunnel portal. The topography was formed of rough concrete placed over expanded metal lath. The spillway shape was represented by very smooth concrete screeded to sheet metal templates. The two spillway gates were constructed from heavy-gage sheet metal. The piers were built of wood and waterproofed with a plastic paint.

The transition section between the spillway and the tapered tunnel, the tapered tunnel, the elbow, the 27-foot diameter circular conduit, and the transition section at the end of the circular conduit were modeled from 0.10-inch thick clear plastic sheet.

The downstream tail box was constructed of water-resistant plywood except for one wall which was built with waterproofed cement blocks. The riverbed and bank topography were formed in pea-gravel and covered with rough-finish concrete to provide a stable channel. The flip bucket was constructed of galvanized sheet metal with the curved surfaces formed in concrete screeded to sheet metal templates. All cuts in the topography and guide walls in the vicinity of the flip bucket were represented in smooth-finish concrete.

Water was furnished to the model from the permanent laboratory supply through a 12-inch centrifugal pump and was measured by a 4-, 6-, 8-, or 12-inch venturi meter, depending on the quantity. The flow, after entering the head box, passed through a 6-inch thick rock baffle before entering the modeled reservoir area, thus insuring a smooth and uniform approach flow. The water surface elevation in the head box was measured by a hook gage placed in a stilling well on the outside of the box; the piezometer was located in the floor near the center of the head box about midway between the rock baffle and the spillway.

The water surface elevation in the tail box was measured by staff gages located at several points. The tail water level was controlled by an adjustable gate at the downstream end of the box. Pressures on the crest, conduit walls and floor, and flip bucket were measured by piezometers connected to open-tube water manometers.

Details, physical dimensions, and other features of the model are shown on Figure 3.

THE INVESTIGATION

Approach Area

The approach to the tunnel spillway is a channel excavated in a hillside. On the left side of the channel the subgrade material was sound and could be cut to a steep slope. Thus it was possible to provide a straight approach to the tunnel portal, eliminating areas where eddies and objectionable current patterns could form. On the right side the subgrade material was poor, and a very large area had to be completely removed. In addition, the foundation was such that a large quantity of mass concrete would be required in the structure to anchor the spillway structure.

Right side approach pier. The preliminary design for the pier on the right side consisted of a large quantity of mass concrete in the form of a vertical column about 30 feet wide and 50 feet long in its greatest cross-section. The nose of the pier was streamlined with a short radius on the spillway side and a long radius on the opposite side, Figure 4A.

Flow passing around this pier produced an excessive contraction resulting in considerable drawdown of the water surface along the inside face of the pier. At the maximum discharge, 66,000 cfs, the drawdown at the crest line was about 15 feet, Figures 4A and 5. The normal drawdown, based on the assumption that the drawdown is equal to the velocity head at the crest axis, should be about 8.9 feet. Using this as a design criterion, the drawdown should be reduced by about 6 feet to produce ideal flow conditions. Another poor hydraulic feature resulting from the excessive drawdown was the rough and uneven water surface produced in the tunnel. The uneven water surface was apparent throughout the tapered shaft and contributed, to a large measure, to near filling of the tunnel in the vertical bend. Figure 5 shows the flow appearance at the right pier.

Overhanging pier, first change. Previous model studies have shown that a large-radius pier nose is often necessary to provide a gradual change in direction of flow at the pier nose without causing a rough or depressed water surface and that moving the pier nose upstream into a region of lower velocity flow as far as possible also increases the effective radius of a nose. It has also been established, however, that a large radius is not necessary for all discharges, the large radius being necessary for the larger discharges while a much smaller radius is adequate for the small flows.

To include these criteria in a single pier, the overhanging pier design was developed. The upper part of the pier has the nose farther upstream and may also have a larger radius than the lower part of the pier, Figure 4B, C, and D.

For the first revision the original pier shape and nose radius were retained up to elevation 3270.00. For the next 15 feet, to elevation 3285.00, the face of the pier was extended upstream on a 1:1 slope. From elevation 3285.00 to elevation 3302.00 the dimensions shown on Figure 4B were maintained.

With this pier in place the amount of drawdown was reduced from 15 to 10.5 feet at the crest axis. The roughness of the water surface in the tunnel shaft was also greatly reduced and presented a much smoother appearance.

At this stage in the testing program a change in the location of the prototype tunnel portal was made at the request of the designers. Field investigations had shown that the foundation conditions would be greatly improved if the tunnel entrance was moved 30 feet farther into the hillside. All topographic features in the model head box were therefore changed to represent the new entrance conditions.

The necessity for a large volume of mass concrete on the right side of the spillway was greatly reduced as a result of this change in spillway location. It therefore became desirable to determine the minimum size pier necessary for good hydraulic operation.

Recommended pier. The recommended pier shape was developed after observing the flow around the previous pier at the maximum discharge, determining the critical flow regions, and gradually modifying the pier until it was elliptical in form and had only a 6.3-foot overhang, Figure 4C. The water surface drawdown was about 10 feet at the crest axis compared to the theoretical value of 8.9 feet. The flow appearance around the pier was very good and the water surface entering the tunnel shaft was smooth, Figure 6.

Alternate recommended pier. A structural analysis of the recommended pier made by the designers indicated that the 16-foot thickness might possibly cause stresses that would twist the gate slots, causing the gate to bind and be hard to open or close. The analysis also showed that if the thickness was reduced to 12 feet the tendency for twisting would be entirely eliminated.

The right pier was redesigned, keeping the same general surface shape on the inside face from the crest axis to the nose, but reshaping the pier nose to conform to the reduced thickness. The amount of overhang was reduced to 5.8 feet, Figure 4D. The flow appearance with this pier was almost as good as for the first recommended pier, the only difference being a slight asymmetry of the water surface at the tunnel portal that became insignificant before the flow reached the vertical bend, Figure 6.

Preliminary left side approach. The initial layout of the approach on the left side of the spillway was similar to the right side, the difference being that an excavated bank on the left side acted as a flow boundary and guided the flow toward the tunnel entrance. The pier on the left side had the same shape on the inside surface as the right pier. On the side of the pier next to the bank, instead of the large-radius curved surface a cutoff wall set at a 30° angle with the inside face of the pier extended to the bank. The flow appearance with this pier was good, but at the maximum discharge the water surface drawdown at the crest axis was about 11 feet and there were some water surface irregularities that carried down into the tunnel shaft, Figure 5.

Recommended left pier. When the tunnel entrance was moved farther into the hillside the left side approach was completely altered. The excavated bank become much steeper and was closer to the left side of the spillway entrance. It was then possible to extend a cutoff wall at a right angle from the pier to the excavated bank, Figures 6 and 7. The flow with this pier face was very good, the water surface drawdown was not excessive, and there were no surface disturbances that carried down into the tunnel shaft, Figure 6.

Center pier. The streamlined center pier had the same pier nose radii as the original end piers, Figure 7. The flow at the center pier was very good; only very small disturbances to the water surface were evident. Since the flow entering the tunnel shaft was symmetrical and uniform, no changes were considered necessary. Figure 6 shows the flow at the center pier.

Spillway Crest Investigation

Calibration. The tunnel spillway was calibrated to determine the discharge capacity after the recommended piers had been installed. The relation between reservoir elevation and discharge was determined over the full range of reservoir elevations for regulated and free flow. The discharge capacity for regulated flow was measured for 5-foot gate-opening intervals with both gates equally open. The reservoir elevation refers to a point opposite the spillway about 250 feet upstream from the entrance, Figure 3. The piezometer was sufficiently upstream to practically eliminate the effects of the drawdown curve on the head measurement.

The results of the calibration are shown by the curves of Figure 8. The original data points obtained from the model were plotted and smooth curves drawn to connect the points. In the case of the partial gate opening data, the curves were cross-faired at several reservoir elevations to insure continuity and smoothness and then plotted in the form shown on Figure 8. The calibration showed that for uncontrolled flow the maximum discharge occurred at reservoir elevation 3297.24. In the equation

$$Q = CLH^{3/2}$$

where

C = coefficient of discharge
L = spillway width at crest axis, 53 feet
H = total head, 50 feet

the coefficient of discharge is 3.52, the same as used for design purposes.

Pressures on crest. Pressure measurements were obtained from 10 piezometers equally spaced along the center line of the left spillway bay, Figure 9. Measurements were obtained at the maximum reservoir elevation for each 5-foot increment in gate opening from 5 to 35 feet open. Measurements were also made at the maximum reservoir elevation for uncontrolled flow.

The pressures were above atmospheric at all piezometers for all gate openings, with the exception of the downstream piezometer. Piezometer 10 indicated slightly subatmospheric pressures for gate openings of 20 feet and less. The lowest pressure measured occurred at the 10-foot gate opening and was equivalent to 4.8 feet of water below atmospheric pressure. Since this value is well above the cavitation range, no alterations to the crest shape were recommended. The locations of the piezometers and the results of the pressure measurements are shown on Figure 9.

Water surface profiles. Water surface profiles for the maximum discharge were obtained both longitudinally and transversely. The longitudinal profile was taken along the right wall of the bay while the transverse profile was taken across both bays of the spillway along the axis of the crest. The profiles along the piers are shown in Figure 4. The water surface profile along the crest axis is shown in Figure 10. As a result of the transverse profile, it was recommended that the wide open position of the gate bottom be raised 2 feet in order to eliminate any danger of the water striking the gate at the maximum discharge and backing up the flow.

Flow in Tunnel

Entrance transition. The entrance transition extended from the tunnel portal at Station 1+42.75 to Station 1+95.00. The transition changed the shape of the tunnel from two 26.50-foot wide by 50.00-foot high rectangular passages to a single 41.00-foot diameter circular conduit. The center pier, which formed a common wall for the two rectangular passages, extended down into the transition to Station 1+90.00.

The flow in the entrance transition was not symmetrical during the initial testing; however, this was not caused by the transition shape but rather by the entrance conditions previously described. The entrance conditions had been improved before the following transition investigations were made.

The water surface in the transition was smooth and even except at the downstream end of the center pier. A small fin formed on the water surface at the junction of the flows from the two spillway bays. However, this fin was small and did not cause waves or other disturbances to form in the tunnel.

Six piezometers, Nos. 11-16 were spaced along the center line of the floor of the left half of the transition section, Figure 9. Pressures were obtained for eight discharges; and with one exception, the pressures were above atmospheric at all times. Piezometer 15, located near the end of the pier, indicated slightly subatmospheric pressures at all discharges. This was considered to be insignificant since the lowest reading was only 3.0 feet of water below atmospheric pressure. The pressures obtained are shown in the table on Figure 9.

At the intersection line between the end of the transition and the start of the tapered shaft there was a slight separation of the flow from the side wall of the conduit at large flows. The separation was about midway up the side wall and was more pronounced on the left wall than on the right wall. Two piezometers, Nos. 16A and 16B on Figure 9, were installed in the area of separation. Pressures at 66,000 cfs were about 2.0 to 3.5 feet of water below atmospheric pressure.

The separation seemed to be caused, in part, by the flowing water striking the curved portion of the arched roof and being deflected away from the wall. It seemed that if the line of intersection between the arched roof and the side wall could be raised so that the flowing water did not touch the arched roof, the tendency toward separation might be reduced. The designers stated that a modification of this type would present the problem of higher stresses in the arched roof and consequently the construction cost would be much greater. Since the separation occurred only at flows above 50,000 cfs and the pressures in the separation region were well above the cavitation range it was decided not to modify the arched roof of the transition.

Tapered shaft. The conduit between the end of the transition section and the start of the vertical bend tapered from 41.00-foot diameter at Station 1+95.00 to 27.00-foot diameter at Station 4+35.83, Figure 9. In this length the elevation of the invert of the pipe dropped from elevation 3199.74 to elevation 2958.91.

The flow in this section was very uniform. The water surface was smooth and uniform and flowed down the full length without vacillating from side to side. Eight piezometers spaced along the invert of the tapered shaft were used to obtain pressure measurements for eight discharges. Above-atmospheric pressures were obtained at all piezometers for all discharges; the pressures are tabulated on Figures 9 and 11.

Vertical bend or elbow. In the preliminary design the tunnel changed direction from the 1:1 slope of the tapered shaft to a 0.01 slope with a 150-foot radius vertical bend or elbow, Figure 11. The flow around the elbow was not satisfactory at discharges above 50,000 cfs. The redistribution of the velocity resulting from the flow passing around the bend caused irregularities in the water surface that almost completely filled the tunnel. This redistribution was also reflected by pressure readings along the invert of the elobw where pressures were 5 to 7 times as high as pressures just upstream from the elbow, Figure 11. Figure 12 shows the appearance of the flow in the elbow at 66,000 cfs.

Previous model studies of similar problems had shown that combinations of large discharges and high velocities in a confined area required a long-radius elbow to smoothly change the direction of the flow. On this basis, it was recommended that the radius of the elbow be increased to 250 feet, Figure 11. The longer-radius elbow in the model produced a much better flow pattern. The water surface still had a tendency to almost close over at the roof of the tunnel but only at the maximum discharge, Figure 13. The pressures on the invert of the elbow were up to 25 feet of water lower than for the shorter-radius elbow; the pressures for both elbows are tabulated on Figure 11.

In order for the invert at the end of the longer-radius elbow to be at the same elevation as the shorter-radius elbow, it would be necessary to redesign the entire tapered shaft. In the model, this would necessitate rebuilding all of the plastic sections between the crest and the elbow. Since this change would not have noticeable effect on the flow in the conduit and would cause only minor differences as far as studying erosion and other phenomena in the bucket and downstream river area, it was decided not to model the tapered section in its final form. The investigation of the flow in the horizontal tunnel was made with the 250-foot radius elbow, but the studies of the flip bucket were made with the 150-foot radius elbow.

Horizontal tunnel. The horizontal tunnel, actually on a slope of 0.01, extended from Station 5+40.37 to Station 9+00.37. With the 150-foot radius elbow in place, the flow had a tendency to fill the horizontal tunnel at discharges above 50,000 cfs, Figure 12. With the 250-foot radius elbow there was an open area along the crown of the tunnel at all discharges, Figure 13. The flow distribution was good for the full length of this section, and the appearance was satisfactory in every respect. No changes in the horizontal tunnel were recommended.

Downstream transition. The transition at the downstream end of the horizontal tunnel was 50 feet long and changed the 27-foot diameter circular tunnel to a horseshoe-shaped conduit 46 feet long. The horseshoe conduit discharged directly into the flip bucket. The transition performed its function very well; the change in shape was accomplished without any adverse effect on the flow distribution. No changes or alterations were recommended for the transition.

Seven piezometers were installed in the floor and side of the horseshoe conduit immediately downstream from the end of the transition, Figure 14. The piezometers were placed in regions most apt to produce subatmospheric pressures because of changes in flow pattern caused by the transition. The pressures at the piezometers were, for practical purposes, all atmospheric or above for the three representative discharges investigated. The pressure readings are tabulated in Figure 14.

Flip Bucket

Preliminary. The flip bucket was in a 50-foot long rectangular section at the end of the horseshoe section. The floor in the rectangular section curved upward from the flat bottom of the horseshoe section with a 300-foot radius arc to form the bottom of the flip bucket, Figure 14.

The bucket was adequate in flipping the jet away from the structure at all discharges. However, the alignment of the tunnel was such that the jet did not impinge near the center of the river channel but rather on the excavated berm along the left bank, Figure 15A. The flow struck the berm and subjected the open-cut face to extremely high velocities. In the model this did no damage since this section had been molded in concrete. However, considerable damage occurred farther downstream where the river made an abrupt turn to the right. In this area the river channel and banks were formed of loose pea-gravel. The pea-gravel on the far bank was pulled down into the river channel, and had this test run continued for a longer time, a change in the channel location would probably have occurred. In order to simplify the future tests, all topography included in the tail box was molded in concrete to represent the solid rock or slate as determined from the prototype investigations.

It was apparent from the severity of the erosive action evident in the initial tests that the jet would have to be turned so that it would enter the river near the center of the channel. Here the jet energy could be dissipated with a minimum amount of damage. At the maximum discharge of 66,000 cfs this energy was equivalent to about 2-1/2 million horsepower.

Three methods of turning the jet were discussed. These were:

1. Changing the alignment or direction of the tunnel at the entrance

2. Placing a horizontal curve as short as practicable in the downstream end of the tunnel

3. Placing a deflector or horizontal curve in the flip bucket

The first method was not model tested since construction of the prototype entrance structure had progressed to such a point that the tunnel alignment had already been established. The second and third methods were both investigated by the model.

Horizontal curve in pipe. In order to facilitate the model studies, a horizontal curve in the model tunnel was simulated by installing wedge-shaped pieces between short sections of the conduit, Figure 16. The first wedge was installed at Station 9+00, between the end of the circular conduit and the beginning of the transition. Only this one wedge was installed at first. The conduit at the outlet was thereby turned approximately 2-1/2° to the right, Figure 16A. Tests showed the jet to be deflected to the right, but it still was not close to the center of the channel. A second 2-1/2° wedge was installed between the transition section and the horseshoe conduit, Figure 16A. The jet was turned farther to the right but not sufficiently to place it near the middle of the river. A third 2-1/2° wedge placed between the horseshoe conduit and the flip bucket, Figure 16A, caused the jet to impinge near the right bank. With the full 7-1/2° turn to the right the flow conditions were greatly improved, but the right bank was subjected to extreme erosive forces. With the jet turned it appeared that the cut in the left bank could be eliminated and that a slight reduction of the 7-1/2° turn would result in good flow conditions. At the maximum discharge and with the downstream tail water at normal elevation 2938.0 the jet followed the right bank and produced a moderate amount of energy dissipation before the flow passed around the downstream bend in the river. When the tail water was raised slightly above normal the jet direction suddenly switched and followed the left bank, causing extreme turbulence and eddying action at the river bend. When the tail water was again lowered the jet did not return to the right side, Figure 15B.

Because of the problems and dangers inherent in turning high-velocity flow within a tunnel, it was decided that further tests should be made to determine the feasibility of accomplishing the full turning of the jet in the flip bucket.

Modifications to flip bucket. The first modification to the bucket consisted of placing a simple curve in the downstream 19 feet of the left wall. The curved section was the same height as the existing wall but reduced the exit width by 4.75 feet, Figure 16B. At the maximum discharge the wall turned part of the flow; but due to the thickness and velocity of the jet, the major portion of the jet was not turned, causing part of the jet to rise vertically in a fanlike pattern, Figure 15C.

In order to give more lift and turn to the jet and at the same time prevent it from fanning out, the curved left wall was sloped toward the outside and the floor of the bucket was superelevated, Figure 16B. The flow with this second modification was unsatisfactory since again only a small part of the jet was turned.

For the third modification, both walls of the 50-foot long bucket were curved to the right so that at the downstream end the center line of the bucket was approximately 17 feet to the right; the floor of the bucket was not superelevated, Figure 17A. Operation showed that the curvature of this bucket was too abrupt, and the jet along the left wall was deflected vertically into the air with such a large amount of spray that the design was unsatisfactory, Figure 15D.

When the amount of curvature was reduced to provide only a 10-foot deflection, the jet struck the riverbed a short distance to the left of the center of the river and the amount of spray and splash was reduced to a tolerable quantity.

A construction report received from the project office stated that some of the concrete had been placed at the downstream tunnel portal to prevent landslide damage during the rainy season. Because of this the outline and location of the flip bucket became established, but field reports indicated that it would be possible to extend the length of the bucket 10 feet downstream. Thus, there was allowed a total length of 40 feet in which the side walls could be curved and 60 feet in which the upward curve of the floor could be developed.

Starting with the above restrictions, the shape of the bucket was developed by "cut and try." That is, while running the model at the maximum discharge it was established, using sheet metal or wood inserts, that a specific poor flow condition might be corrected by changing some feature of the bucket. The model was shut down and the alteration made; the effect of the change was then evaluated, and at the same time the need for further changes ascertained.

As a result of this development the following critical dimensions were established. They are also shown in the fourth modification on Figure 17.

1. Left wall. At the top of the wall, elevation 2940.0, the wall was curved to the right in a 129.50-foot radius to provide a displacement of 7 feet. The wall was constructed with a batter that varied from vertical at the start of the curve to 1:10 at the downstream end. At the intersection of the wall with the floor a fillet was added having a 1-1/2:1 slope that varied in width from nothing at the start of the curve to 6 feet wide at the downstream end.

2. Right wall. The right wall was vertical and was curved to the right in a 92.5-foot radius to provide a 5-foot displacement at the downstream end. The point of curvature of the right wall was 15 feet downstream from the point of curvature of the left wall. No fillet was placed at the base of this wall.

3. Floor. The floor of the flip bucket was formed with a 300-foot radius curve; the point of curvature being 60 feet upstream from the end of the bucket.

The bucket was rebuilt for final testing with piezometers installed in both walls. In the left wall 10 piezometers were installed, 6 in a horizontal line at elevation 2920 and 4 in a vertical line 3 feet upstream from the end of the bucket. Fourteen piezometers were placed in the right wall, eleven in a horizontal line at elevation 2916 and three equally spaced along the curve 1 foot above the floor, Figure 17B.

The flow appearance with this bucket was good at discharges less than 20,000 cfs but for larger flows the jet was thrown too far to the right and appeared unstable with a large amount of splash and spray falling on the right bank.

Pressure measurements were made for five discharges, 10,000, 25,000, 40,000, 55,000, and 66,000 cfs. For discharges up to and including 40,000 cfs the pressures were near or above atmospheric with the exception of the area in the right wall at Piezometers 13 and 15 where subatmospheric pressures equivalent to 3 feet of water were measured. For the two larger discharges the pressures were above atmospheric except for the area at Piezometers 22 and 23. There the pressures were also equivalent to 3 to 4 feet of water below atmospheric pressure. In the left wall, which received the full impact of the flow in turning the jet, the pressures were all above atmospheric and reached a maximum of about 87 feet of water at the maximum discharge. This pressure occurred at Piezometer 1 and probably would have been much larger if the wall had not been battered. The complete list of pressures has been tabulated on Figure 17.

Recommended bucket. One change was made in the above bucket before it was recommended. This was to increase the radius of the right wall from 92.5 to 114.5 feet, Figure 18. The longer radius improved the bucket performance in that the reduced width at the end of the bucket stabilized the jet, reducing the tendency for spray and splash. In addition, the more gradual curve on the right side increased the subatmospheric pressures to above atmospheric.

When the bucket was rebuilt to incorporate this change for final testing, eight additional piezometers were installed in the left wall. One piezometer was placed in the formerly subatmospheric area of the right wall.

The operating tests for the recommended bucket showed that the jet was well controlled at all discharges and entered the river near the center of the channel, Figure 19. Pressure measurements indicated above atmospheric pressures in all areas. The maximum pressure was in the left wall in the same region as in the previous bucket but was equivalent to 90.9 feet of water. The area in the right wall that formerly showed subatmospheric pressures was above atmospheric, 26.8 feet of water at 66,000 cfs. The pressures are tabulated on Figure 18.

Water Surface Drawdown in River

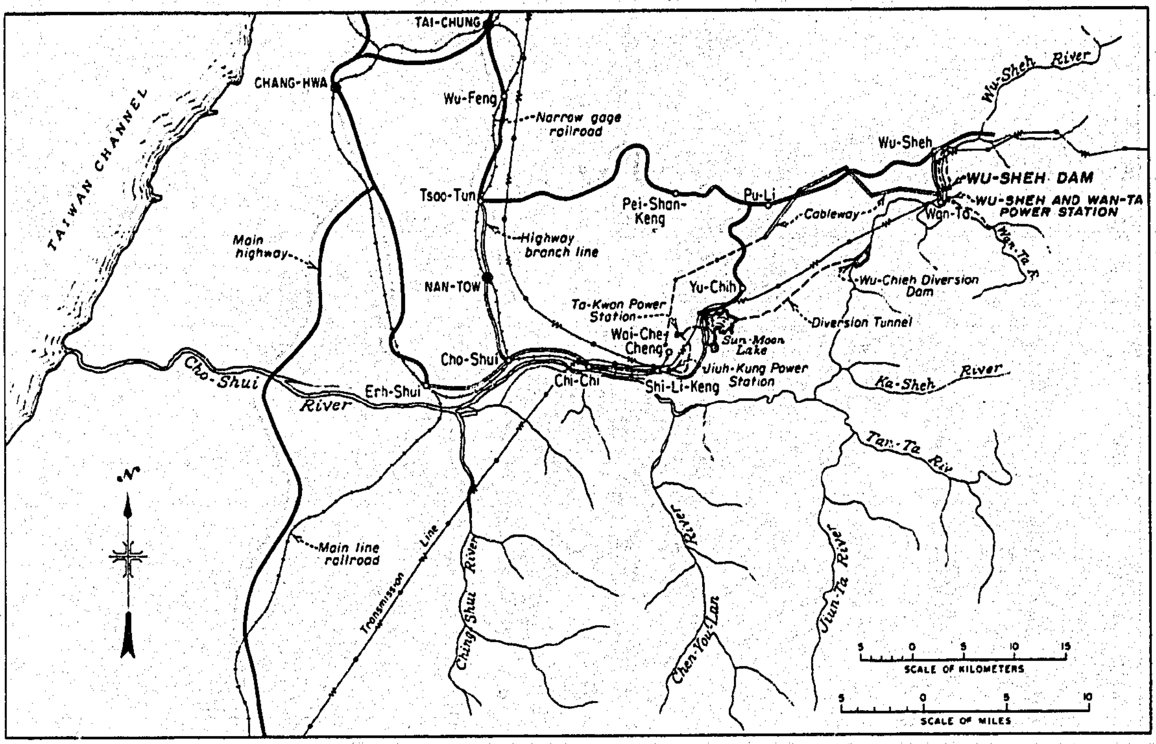
The ejector action of the jet from the flip bucket caused a drawdown in the river water surface upstream from the jet. Since the power house is located upstream from the tunnel outlet portal, the magnitude of the drawdown was determined so that, if necessary, corrective measures could be taken to insure that the tailrace of the power house would not be unwatered.

The magnitude of the drawdown was determined from the difference in tail water elevation between two measuring stations; the upstream station was in the river about 250 feet upstream from the flip bucket, the second station was located below the bend in the river about 1, 500 feet downstream. The method of determining the drawdown was to adjust the downstream water surface elevation with the tailgate so that the tail water corresponded to the design elevation. After the water level had stabilized, the elevation at the upstream station was determined. The two tail water curves are shown on Figure 20. The elevations shown are average readings; at the downstream station the water surface fluctuated a negligible amount at 10, 000 cfs and about 1 foot for 66, 000 cfs, at the upstream station the fluctuation was 1 foot for 10, 000 cfs and increased to 7 feet at 66, 000 cfs. The upstream fluctuations were not in the form of choppy or quick acting waves but had the appearance of long period swells. In the model the elapsed time between the maximum and minimum elevation was about 1 minute or equivalent to approximately 6 minutes in the prototype. This amount of time would be more than ample for the generator governors to compensate for the change in water surface. The difference in the average water surface elevation between the two measuring stations varied from 9 feet at 25, 000 cfs to 23 feet at 66, 000 cfs, Figure 20.

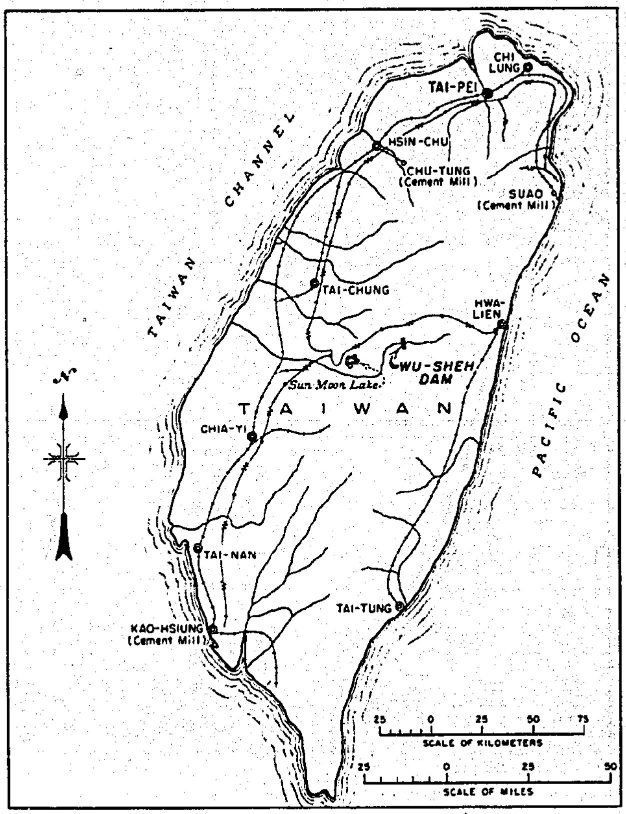
Calibration of Spillway Over Dam

The calibration of the spillway in the arch dam was performed with a 1:25. 28 scale sectional model. This scale ratio resulted when an existing model head box and crest station were modified to represent the Wu-Sheh spillway crest. The model, shown on Figure 21, provided good approach conditions and enough of the spillway face was included to insure that the flow characteristics were fully developed. A half pier, constructed from wood, was placed on each side of the crest as part of the wall, Figure 21. A section of one radial gate was constructed from sheet metal for use during the calibration of the regulated flow. Standard laboratory methods of determining the discharge and reservoir elevation were employed.

The crest was calibrated at 1-foot gate-opening increments and for uncontrolled flow. Since the sectional model represented only 19. 25 feet of the 45. 0-foot wide prototype bay, the prototype flow quantity was computed by dividing the discharge indicated by the model by 19. 25 and multiplying by 45. 0. The results of the calibration are shown on Figure 22. For the free flow condition a discharge of 15, 000 cfs was attained at the maximum reservoir elevation 3297. 24. This agreed with the quantity used in the spillway design.



LOCATION MAP



KEY MAP

NOTE

These maps were prepared from maps issued by the Taiwan Power Company.

CHINA
TAIWAN POWER COMPANY

WU-SHEH PROJECT-TAIWAN

WU-SHEH DAM
LOCATION AND KEY MAPS

TAIWAN POWER COMPANY	UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION	
EXAM'D.	DR. E.L.W. SUBM'D.	
REC'M'D.	TR. E.F.W. REC'M'D.	
APP'D.	CK. APP'D.	
TAIPEI, TAIWAN, CHINA	DENVER, COLO. 12-[illegible]-64	OA-16-261

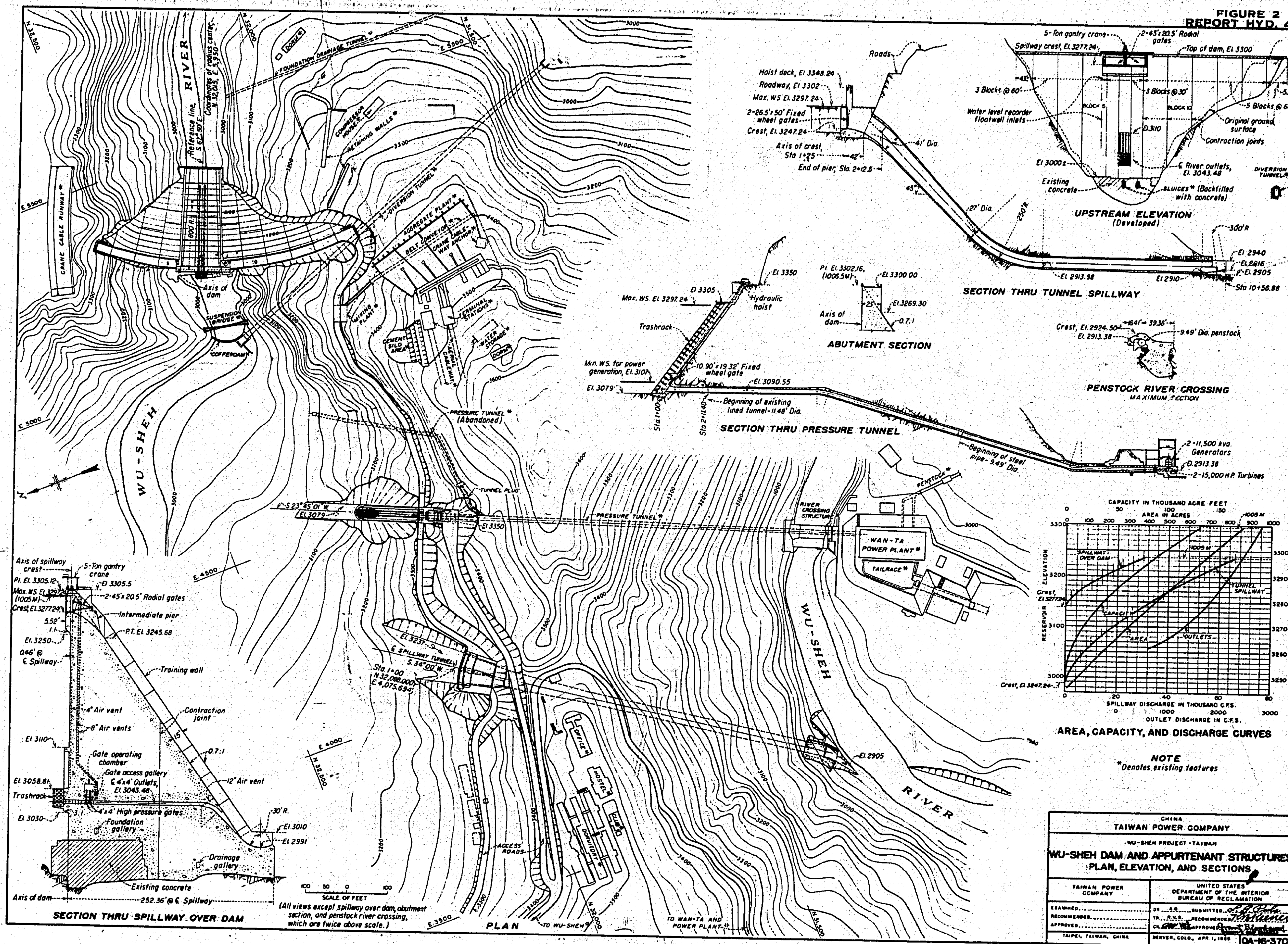
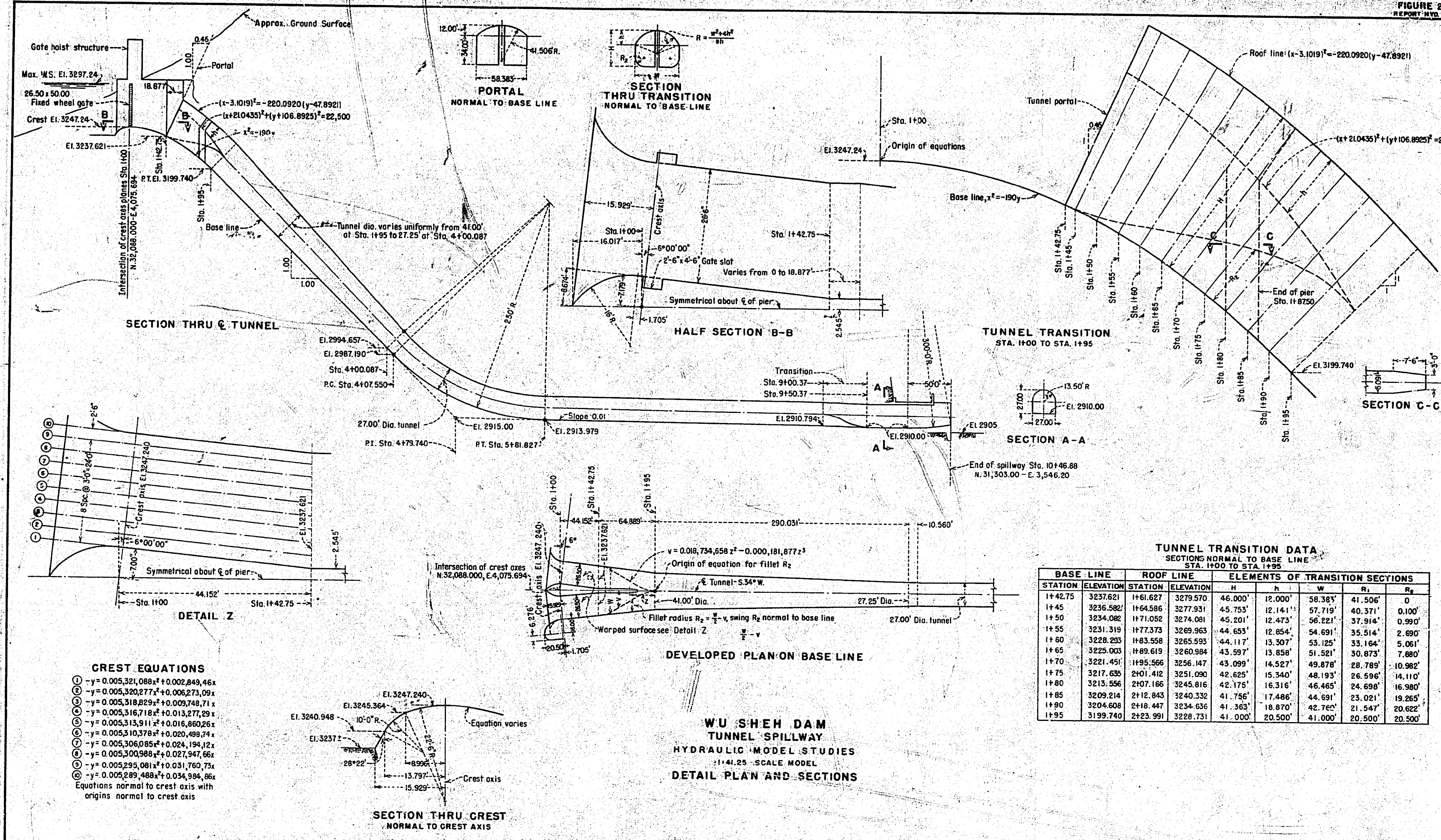


FIGURE 2
REPORT HYD. 430
SECTION THRU TUNNEL SPILLWAY
UPSTREAM ELEVATION
(Developed)
SECTION THRU PRESSURE TUNNEL
ABUTMENT SECTION
PENSTOCK RIVER CROSSING
MAXIMUM SECTION
AREA, CAPACITY, AND DISCHARGE CURVES
CAPACITY IN THOUSAND ACRE FEET
AREA IN ACRES
SPILLWAY DISCHARGE IN THOUSAND C.F.S.
OUTLET DISCHARGE IN C.F.S.
RESERVOIR ELEVATION
RESERVOIR ELEVATION - SPILLWAY DISCHARGE
NOTE
*Denotes existing features
SECTION THRU SPILLWAY OVER DAM
PLAN
SCALE OF FEET
(All views except spillway over dam, abutment section, and penstock river crossing, which are twice above scale.)
WU-SHEH
RIVER
WAN-TA POWER PLANT*
TO WU-SHEH
TO WAN-TA AND POWER PLANT
CHINA
TAIWAN POWER COMPANY
WU-SHEH PROJECT - TAIWAN
WU-SHEH DAM AND APPURTENANT STRUCTURES
PLAN, ELEVATION, AND SECTIONS
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TAIPEI, TAIWAN, CHINA
DENVER, COLO., APR. 1, 1955
OA-16-370



CREST EQUATIONS

(1) $-y = 0.005{,}321{,}088x^2 + 0.002{,}849{,}46x$
(2) $-y = 0.005{,}320{,}277x^2 + 0.006{,}273{,}09x$
(3) $-y = 0.005{,}318{,}829x^2 + 0.009{,}748{,}71x$
(4) $-y = 0.005{,}316{,}718x^2 + 0.013{,}277{,}29x$
(5) $-y = 0.005{,}313{,}911x^2 + 0.016{,}860{,}26x$
(6) $-y = 0.005{,}310{,}378x^2 + 0.020{,}498{,}74x$
(7) $-y = 0.005{,}306{,}085x^2 + 0.024{,}194{,}12x$
(8) $-y = 0.005{,}300{,}988x^2 + 0.027{,}947{,}66x$
(9) $-y = 0.005{,}295{,}081x^2 + 0.031{,}760{,}73x$
(10) $-y = 0.005{,}289{,}488x^2 + 0.034{,}984{,}86x$

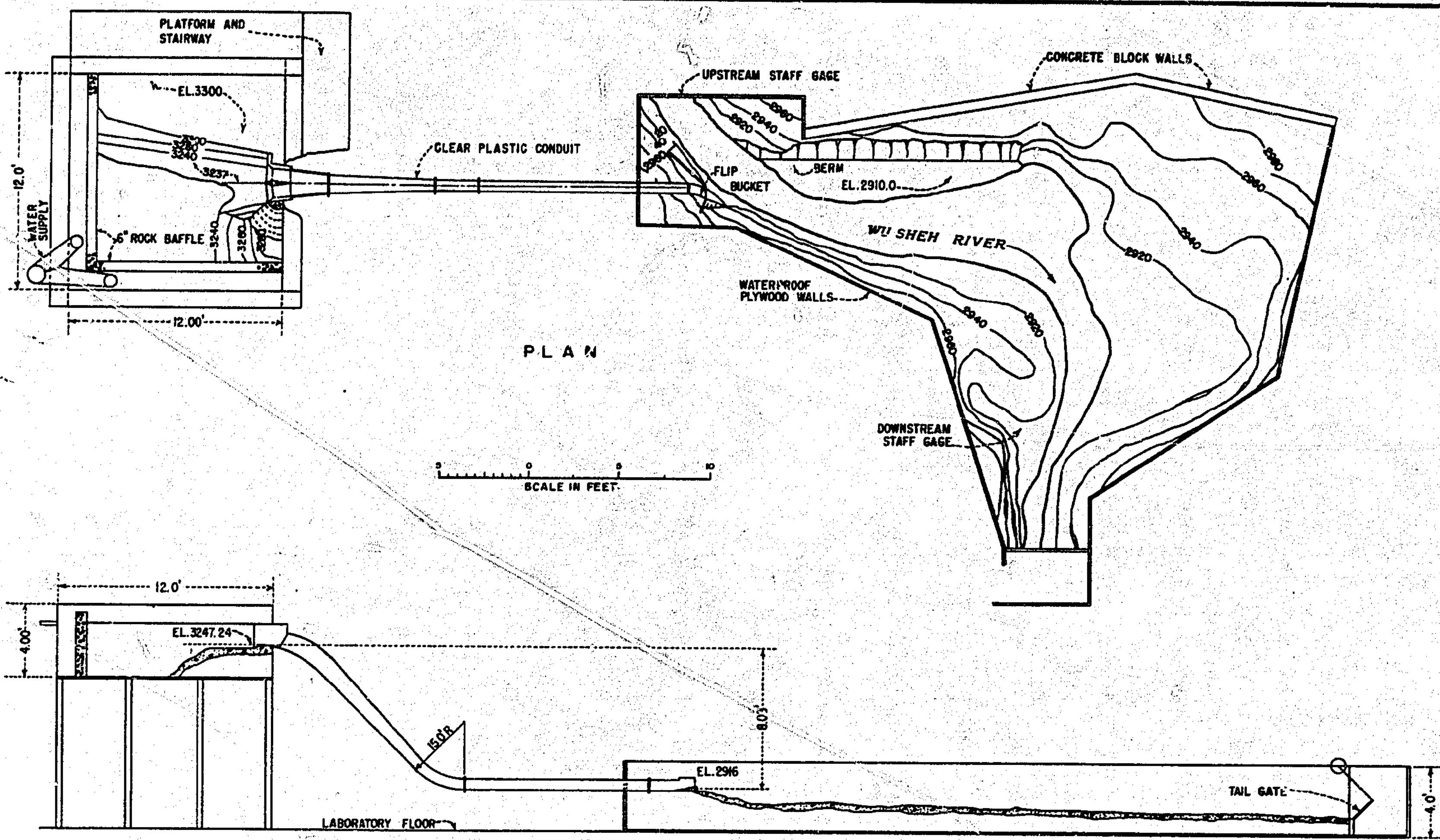
Equations normal to crest axis with origins normal to crest axis

TUNNEL TRANSITION DATA

SECTIONS NORMAL TO BASE LINE
STA. 1+00 TO STA. 1+95

BASE LINE		ROOF LINE		ELEMENTS OF TRANSITION SECTIONS				
STATION	ELEVATION	STATION	ELEVATION	H	h	W	R_1	R_2
1+42.75	3237.621	1+61.627	3279.570	46.000'	12.000'	58.383'	41.506'	0
1+45	3236.582	1+64.586	3277.931	45.753'	12.141'	57.719'	40.371'	0.100'
1+50	3234.082	1+71.052	3274.081	45.201'	12.473'	56.221'	37.914'	0.990'
1+55	3231.319	1+77.373	3269.963	44.653'	12.854'	54.691'	35.514'	2.690'
1+60	3228.293	1+83.558	3265.593	44.117'	13.307'	53.125'	33.164'	5.061'
1+65	3225.003	1+89.619	3260.984	43.597'	13.858'	51.521'	30.873'	7.880'
1+70	3221.451	1+95.566	3256.147	43.099'	14.527'	49.878'	28.789'	10.982'
1+75	3217.635	2+01.412	3251.090	42.625'	15.340'	48.193'	26.596'	14.110'
1+80	3213.556	2+07.166	3245.816	42.175'	16.316'	46.465'	24.698'	16.980'
1+85	3209.214	2+12.843	3240.332	41.756'	17.486'	44.691'	23.021'	19.265'
1+90	3204.608	2+18.447	3234.636	41.363'	18.870'	42.760'	21.547'	20.622'
1+95	3199.740	2+23.991	3228.731	41.000'	20.500'	41.000'	20.500'	20.500'

WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1:41.25 SCALE MODEL
DETAIL PLAN AND SECTIONS



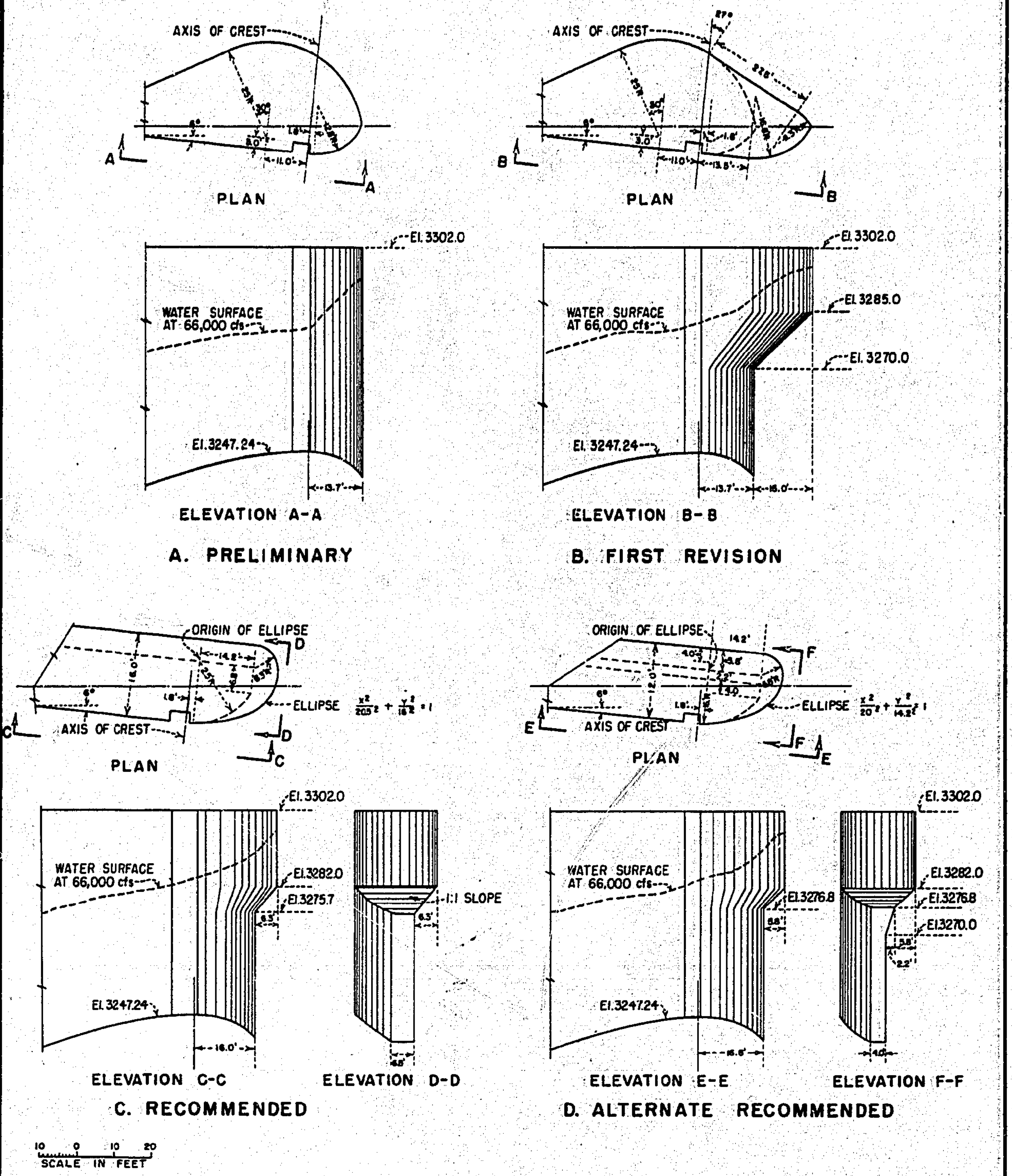
SECTION ALONG CENTERLINE OF CREST, CONDUIT, AND RIVER

NOTES
Details of platform and stairway not shown.
Construction details of boxes omitted

WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1:41.25 SCALE MODEL
MODEL LAYOUT

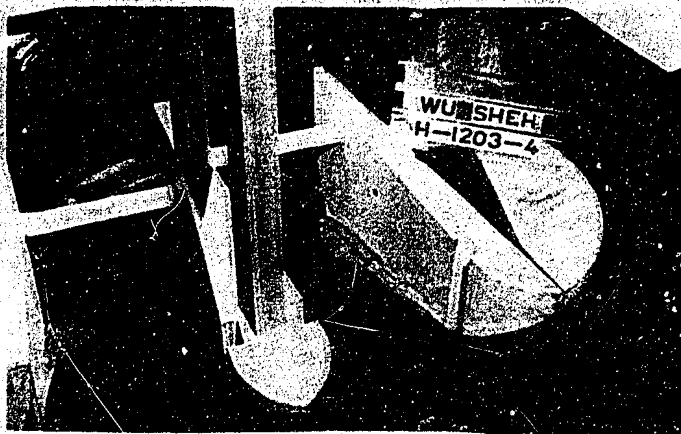
FIGURE 3
REPORT HYD. 430

FIGURE 4
REPORT HYD. 430

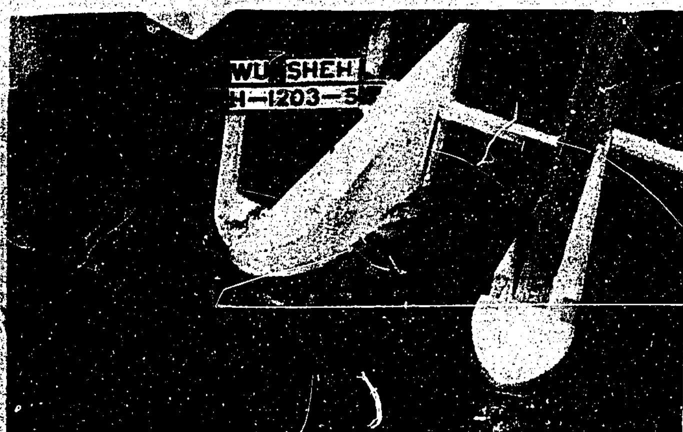


WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1:41.25 SCALE MODEL
RIGHT SIDE ENTRANCE PIERS

A. Preliminary Design



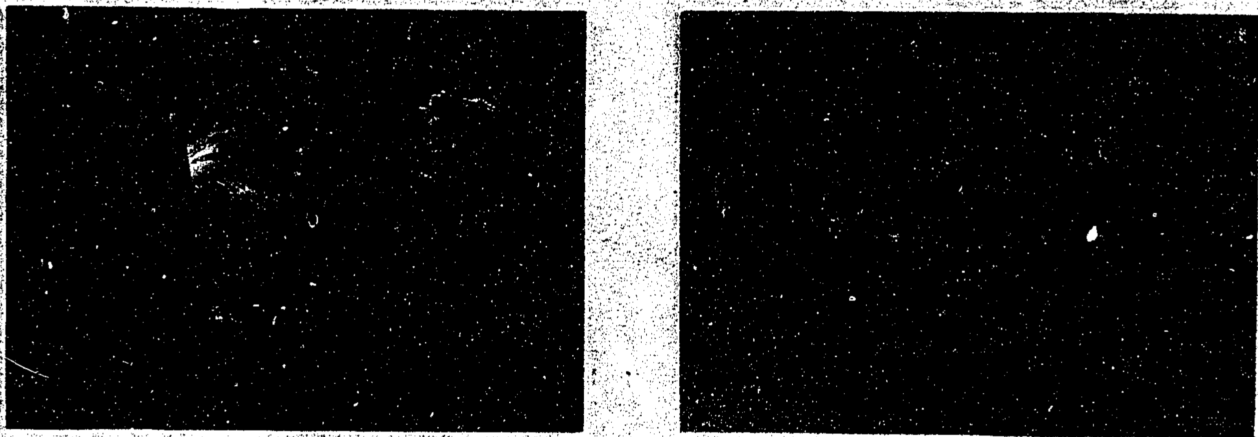
B. Flow Around Right Pier
66,000 cfs



C. Flow Around Left Pier
66,000 cfs

WU-SHEH DAM
TUNNEL SPILLWAY
Hydraulic Model Studies
1:41.25 Scale Model
PIER STUDIES

Q = 30,000 cfs

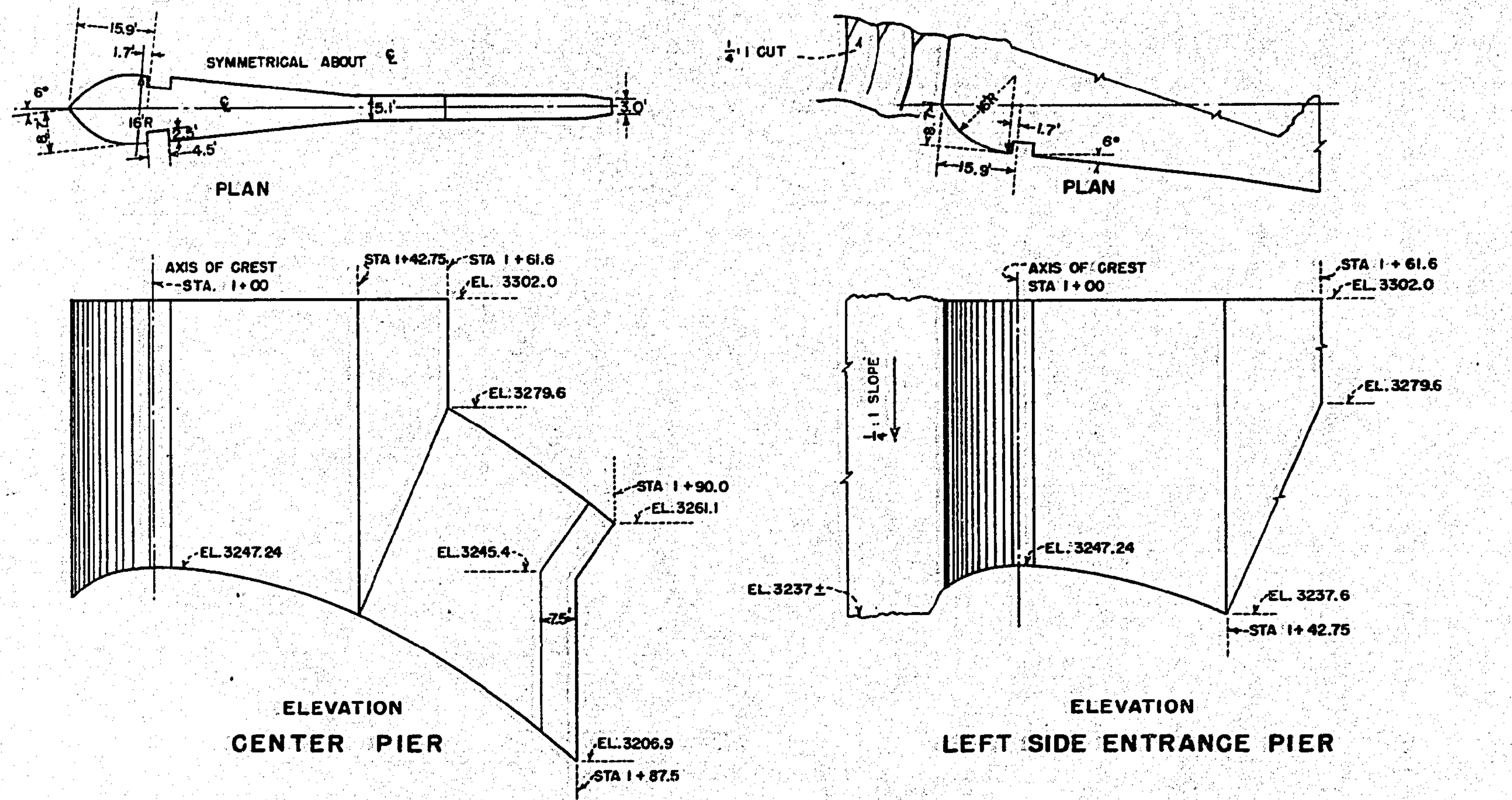


Q = 66,000 cfs

A. Recommended Piers

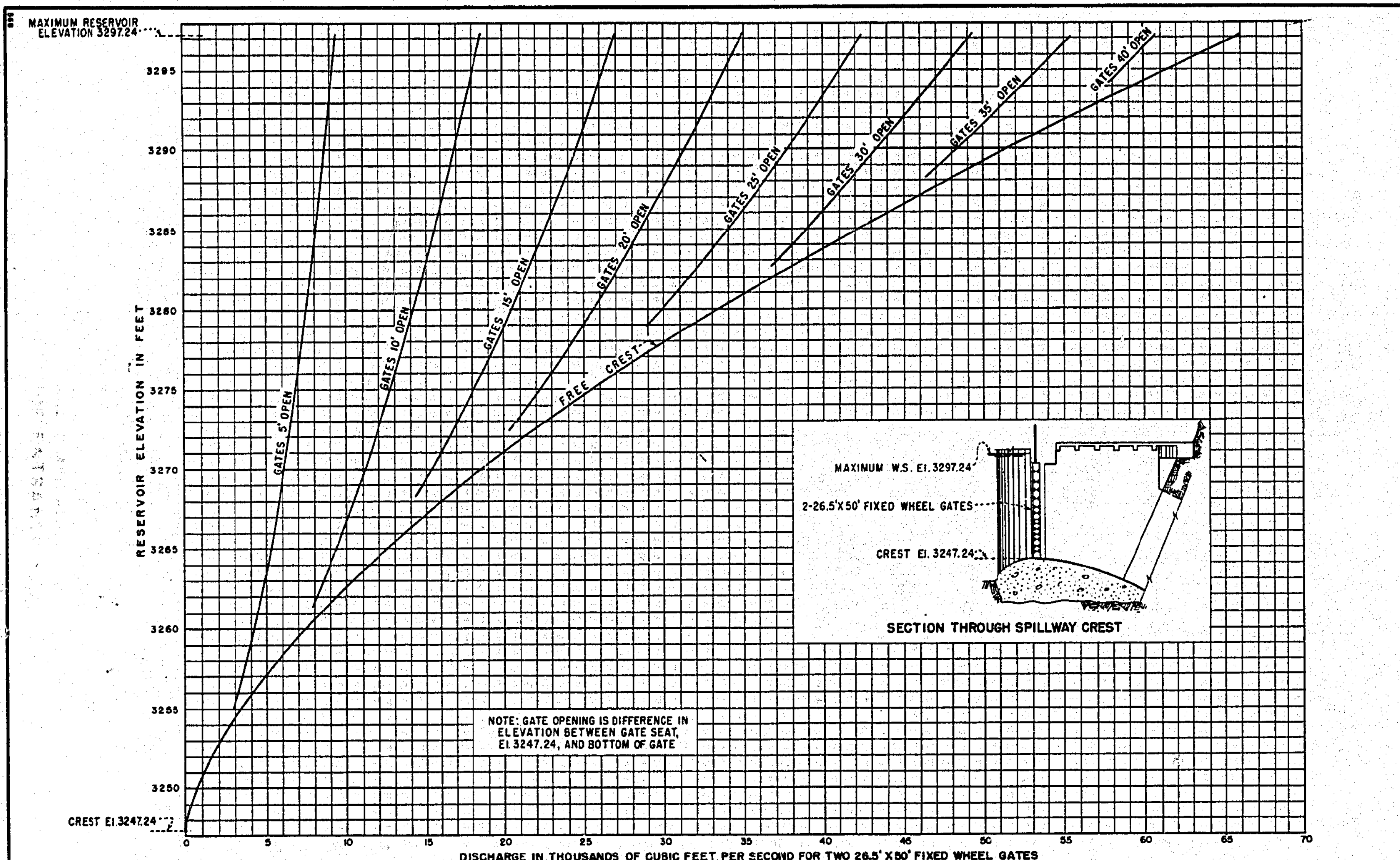
B. Alternate Recommended Piers

WU-SHEH DAM
TUNNEL SPILLWAY
Hydraulic Model Studies
1:41.25 Scale Model
PIER STUDIES

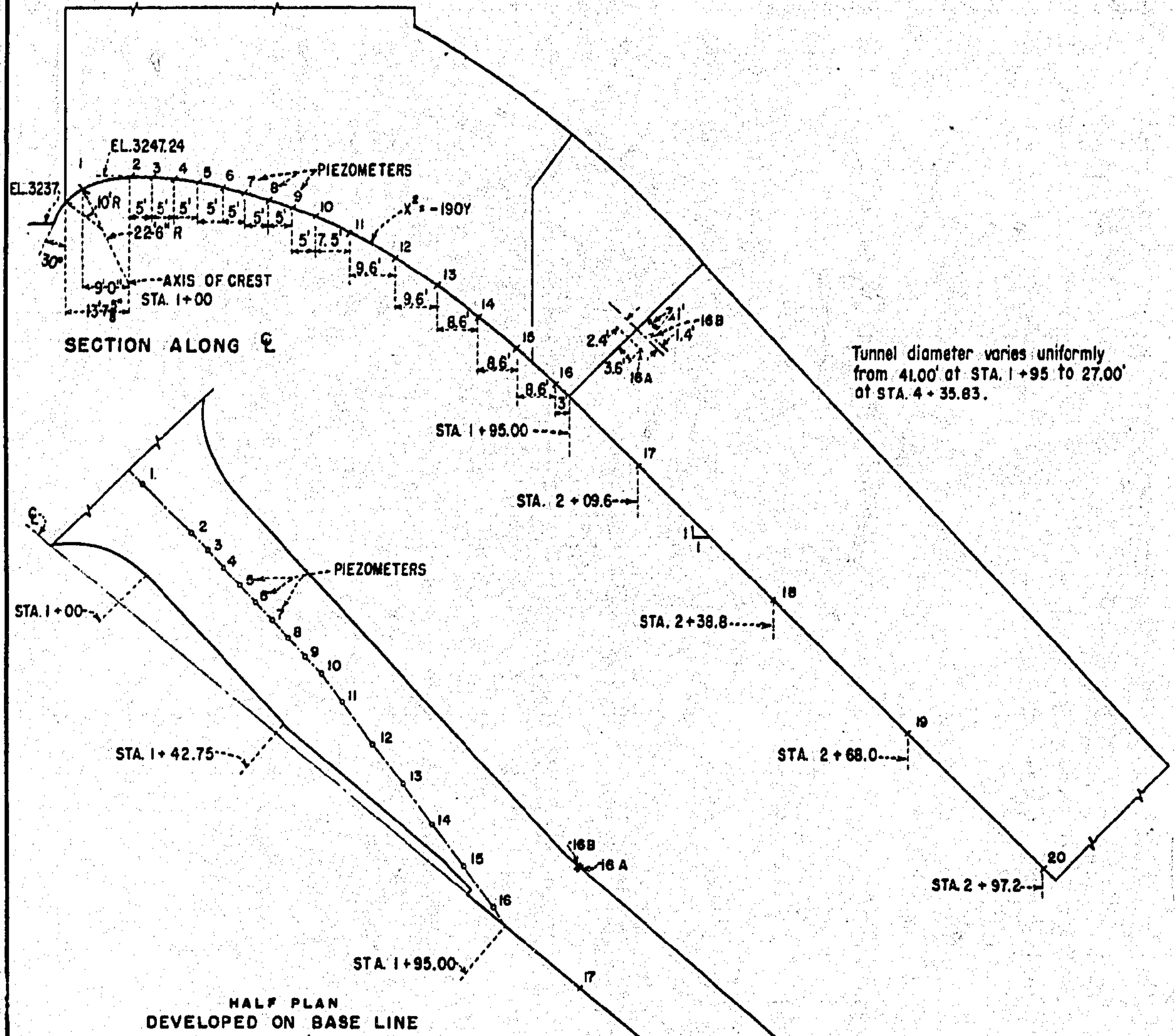


WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1:41.25 SCALE MODEL
PIER STUDIES

FIGURE 7
REPORT HYD 430



WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1:41.25 SCALE MODEL
DISCHARGE CURVES



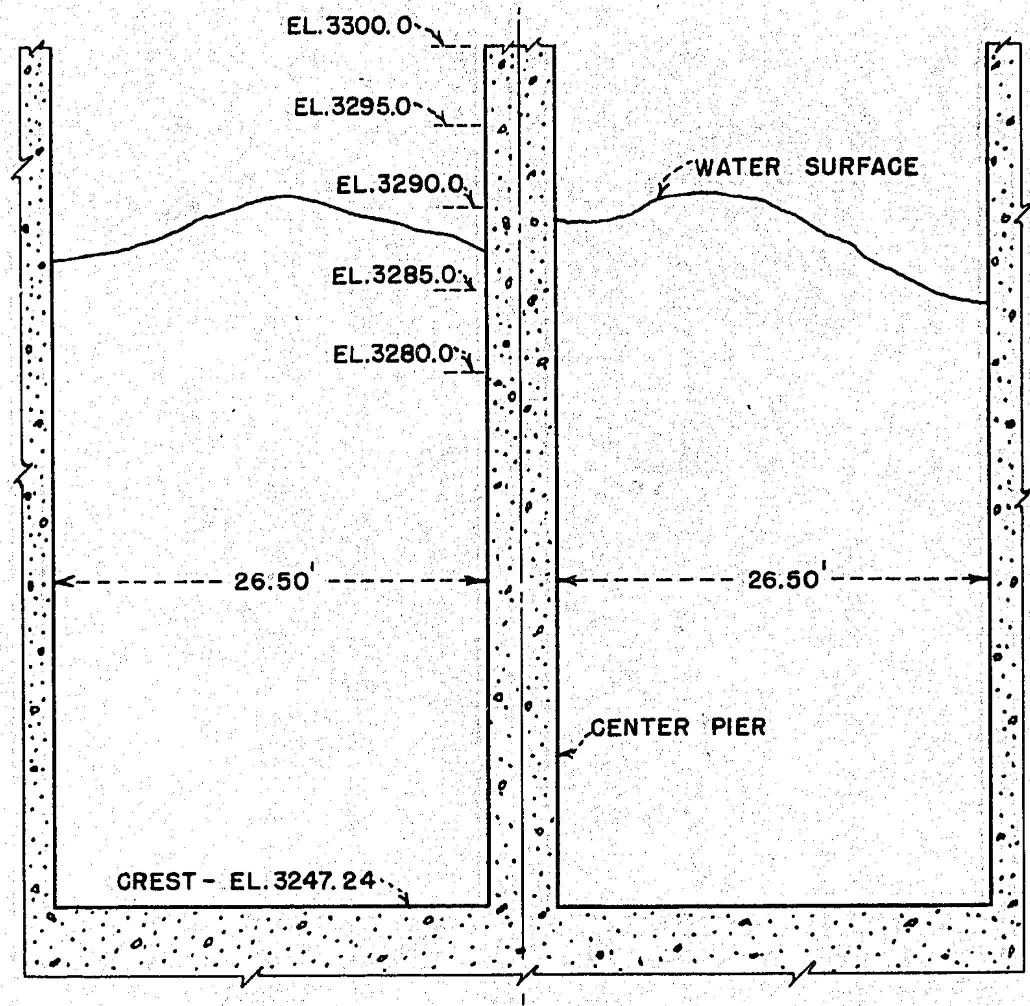
PRESSURES ON CREST AND IN TUNNEL

PIEZ. NO.	GATE OPENING IN FEET							
	5	10	15	20	25	30	35	FULL
1	50.0	47.4	43.4	40.3	39.3	38.6	38.0	36.5
2	22.8	24.5	23.8	23.3	23.6	23.7	24.1	24.1
3	3.5	11.6	15.5	19.4	19.1	20.5	21.7	23.1
4	0.5	4.5	9.2	12.3	15.1	17.4	19.5	22.2
5	0.9	2.4	5.5	8.6	11.7	14.5	16.9	20.6
6	-1.1	-0.7	1.3	4.2	6.9	9.9	12.6	17.4
7	0.3	0.6	1.9	4.1	6.5	9.3	12.3	17.6
8	1.2	1.6	2.4	4.1	6.1	8.8	11.6	16.8
9	0.6	1.0	1.7	3.4	4.9	7.2	9.8	15.1
10	-3.1	-4.8	-2.3	-0.3	1.1	3.4	6.0	11.3
11	1.0	1.5	2.7	6.4	6.3	8.2	10.5	14.9
12	0.8	2.1	3.8	5.1	7.4	9.0	10.8	14.4
13	1.4	2.8	4.6	5.6	7.2	8.2	9.4	12.0
14	1.3	1.8	2.1	2.1	2.9	3.5	3.9	5.5
15	-2.6	-2.9	-2.9	-3.0	-2.1	-2.0	-1.8	-2.1
16	0.9	0	0.1	-0.1	0	0.1	0.1	0.6
16A	-	-	-	-	-	-	-	-1.9
16B	-	-	-	-	-	-	-	-3.4
17	3.0	6.4	12.7	15.5	16.5	16.5	18.1	16.3
18	5.5	8.3	10.5	12.2	13.8	15.5	16.6	19.2
19	8.6	10.5	12.2	13.8	15.8	17.1	17.9	20.6
20	4.7	7.1	8.6	10.1	11.7	13.0	14.4	17.0

NOTES

1. Reservoir at EL.3297.24.
2. Both gates equally open.
3. Gate opening measured vertically above crest.
4. Pressures in feet of water.
5. Pressures are positive except where noted.

WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1:41.25 SCALE MODEL
PRESSURES ON CREST AND IN TRANSITION AND TUNNEL

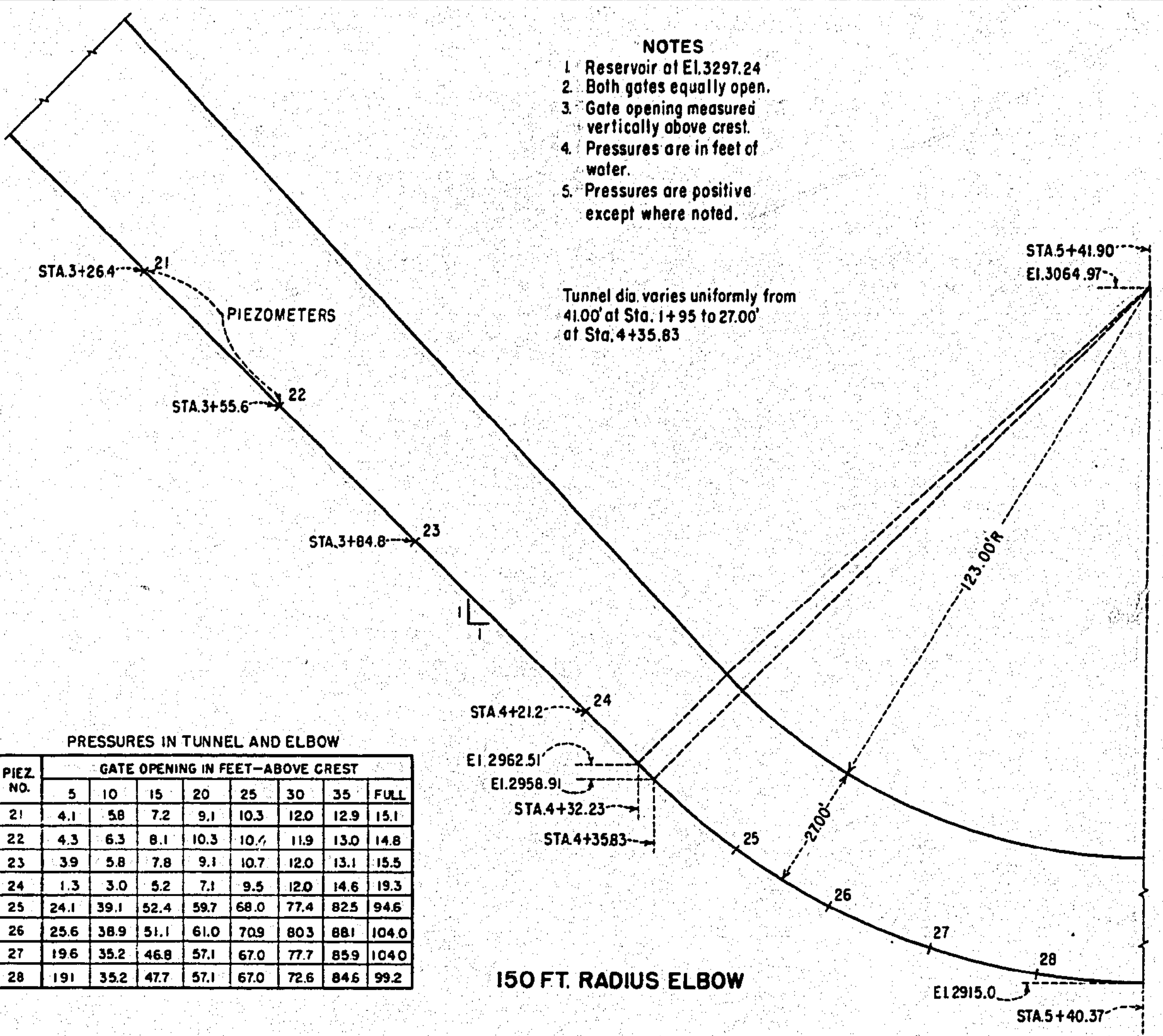


SECTION ALONG AXIS OF CREST
LOOKING DOWNSTREAM. GATE SLOTS NOT SHOWN

WATER SURFACE PROFILE FOR 66,000 cfs.

WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1:41.25 SCALE MODEL
WATER SURFACE PROFILES

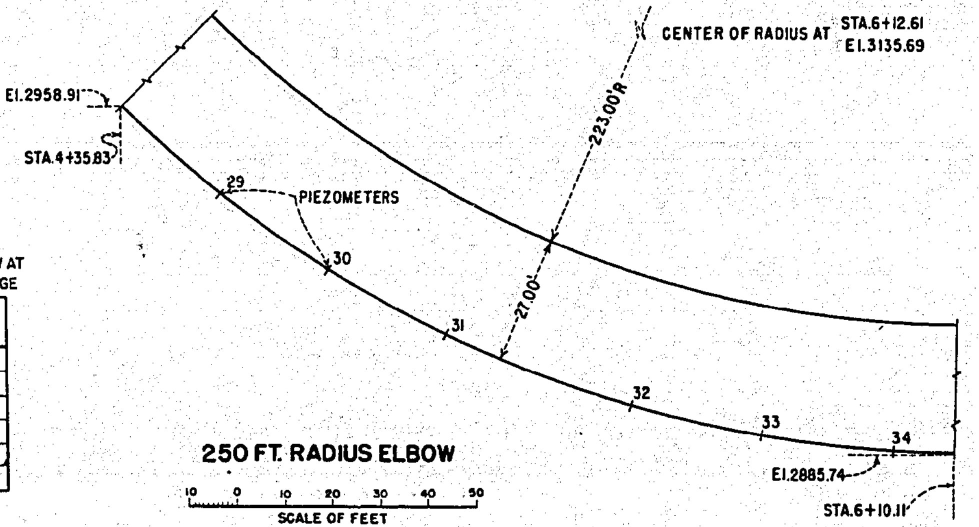
NOTES
1. Reservoir at El.3297.24
2. Both gates equally open.
3. Gate opening measured vertically above crest.
4. Pressures are in feet of water.
5. Pressures are positive except where noted.



150 FT. RADIUS ELBOW

PRESSURES IN TUNNEL AND ELBOW

PIEZ. NO.	GATE OPENING IN FEET—ABOVE CREST							
	5	10	15	20	25	30	35	FULL
21	4.1	5.8	7.2	9.1	10.3	12.0	12.9	15.1
22	4.3	6.3	8.1	10.3	10.4	11.9	13.0	14.8
23	3.9	5.8	7.8	9.1	10.7	12.0	13.1	15.5
24	1.3	3.0	5.2	7.1	9.5	12.0	14.6	19.3
25	24.1	39.1	52.4	59.7	68.0	77.4	82.5	94.6
26	25.6	38.9	51.1	61.0	70.9	803	881	104.0
27	19.6	35.2	46.8	57.1	67.0	77.7	85.9	1040
28	191	35.2	47.7	57.1	67.0	72.6	84.6	99.2



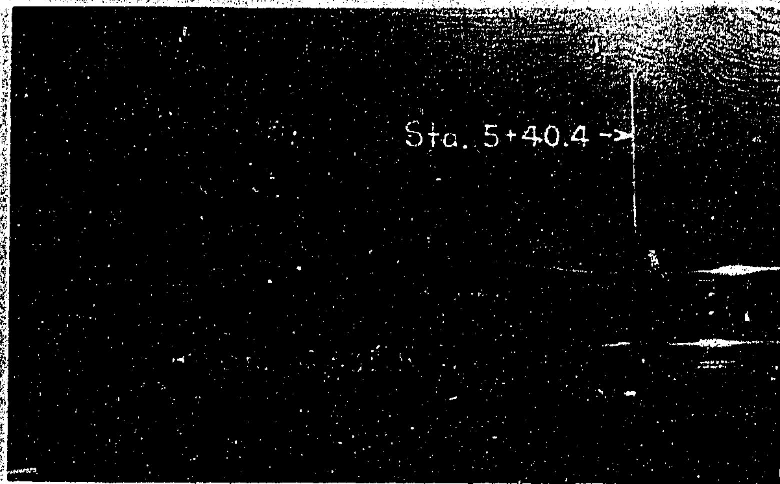
250 FT. RADIUS ELBOW

PRESSURES IN ELBOW AT MAXIMUM DISCHARGE

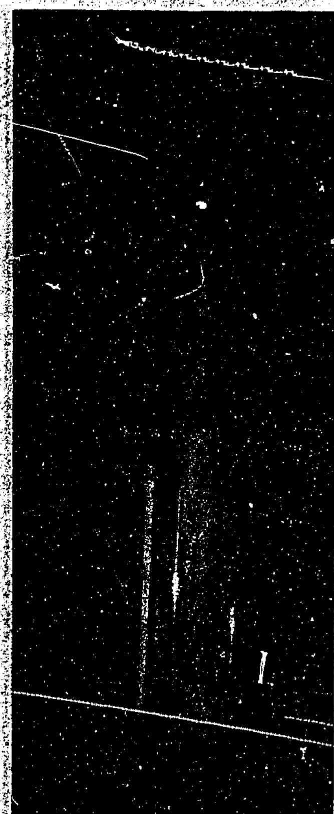
PIEZ. NO.	PRESSURE IN FEET OF WATER
29	65.8
30	71.4
31	77.4
32	74.8
33	78.7
34	70.5

WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1:41.25 SCALE MODEL
ELBOW STUDIES

Figure 12
Report Hyd-430

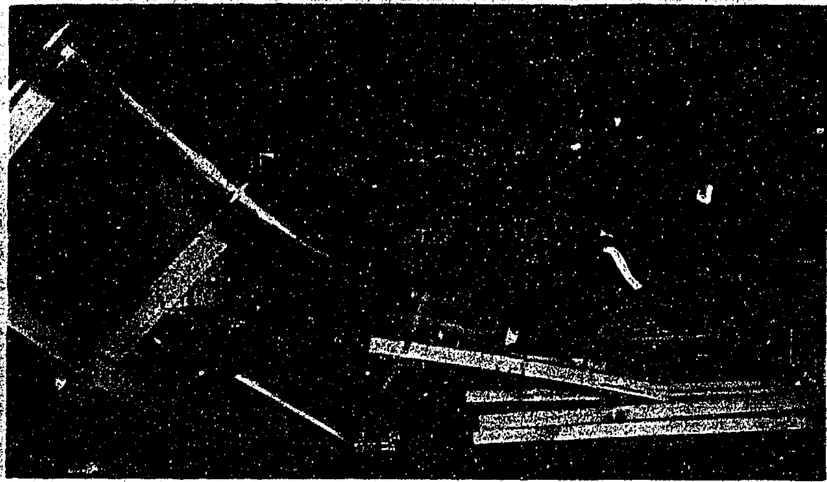


150-Foot Radius Elbow

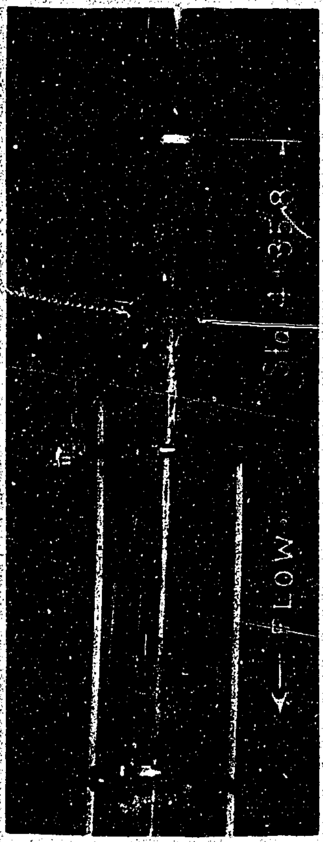


Overhead View of Flow in
Elbow at 66,000 cfs

WU-SHEH DAM
TUNNEL SPILLWAY
Hydraulic Model Studies
1:41.25 Scale Model
ELBOW STUDIES

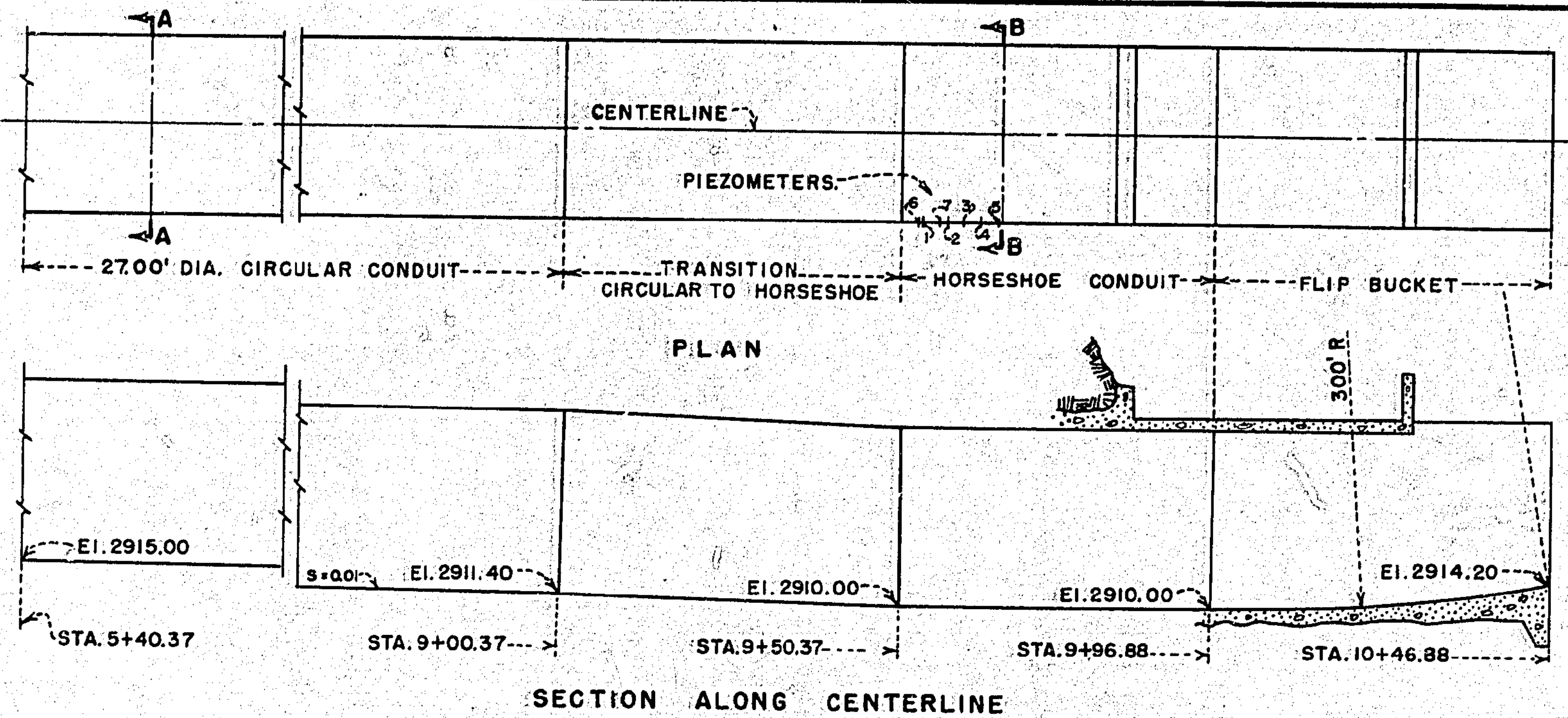


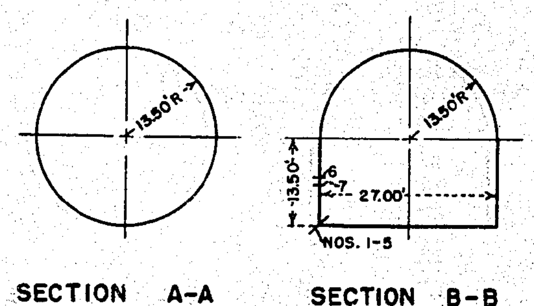
250-Foot Radius Elbow



Overhead View of Flow in Elbow at 66,000 cfs

WU-SHEH DAM
TUNNEL SPILLWAY
Hydraulic Model Studies
1:41.25 Scale Model
ELBOW STUDIES





WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1:41.25 SCALE MODEL
CIRCULAR CONDUIT, TRANSITION, AND PRELIMINARY FLIP BUCKET

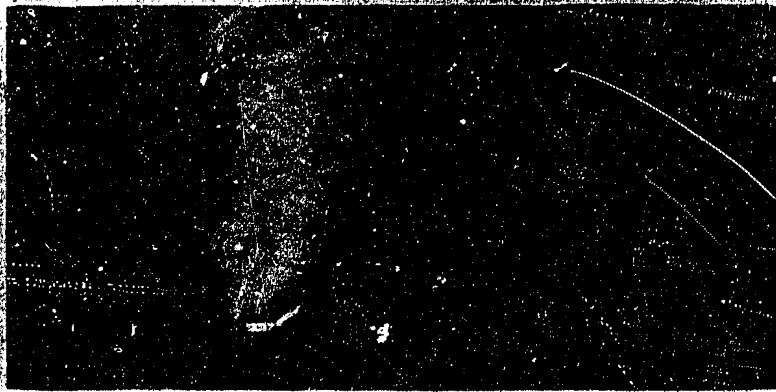
PRESSURES BELOW TRANSITION

PIEZ. NO.	STA.	ELEV.	PRESS.* AT 10,000 CFS	PRESS.* AT 40,000 CFS	PRESS.* AT 66,000 CFS
1	9+53.33	2910.00	4.6'	23.9'	38.6'
2	9+57.14	2910.00	5.8'	24.4'	38.0'
3	9+59.31	2910.00	4.2'	18.8'	31.4'
4	9+62.26	2910.00	4.1'	16.4'	28.9'
5	9+64.88	2910.00	4.0'	13.6'	25.6'
6	9+52.71	2917.15	-0.7'	9.6'	13.4'
7	9+55.88	2916.55	+0.2	13.4'	27.7'

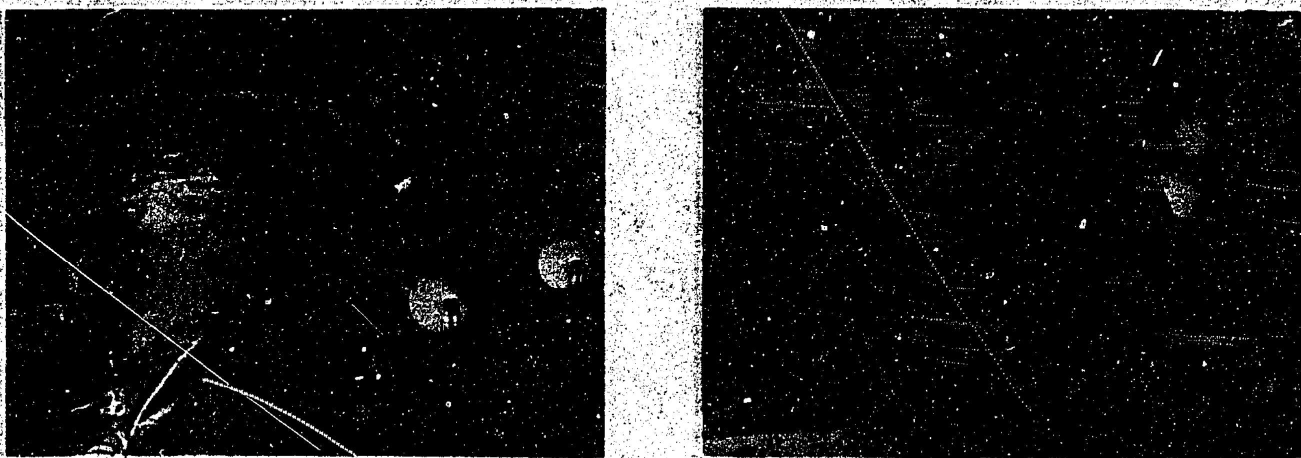
*PRESSURES ARE IN FEET OF WATER.

FIGURE 14
REPORT HYD 430

Figure 15
Report Hyd-430



A. Original Bucket. Q = 66,000 cfs

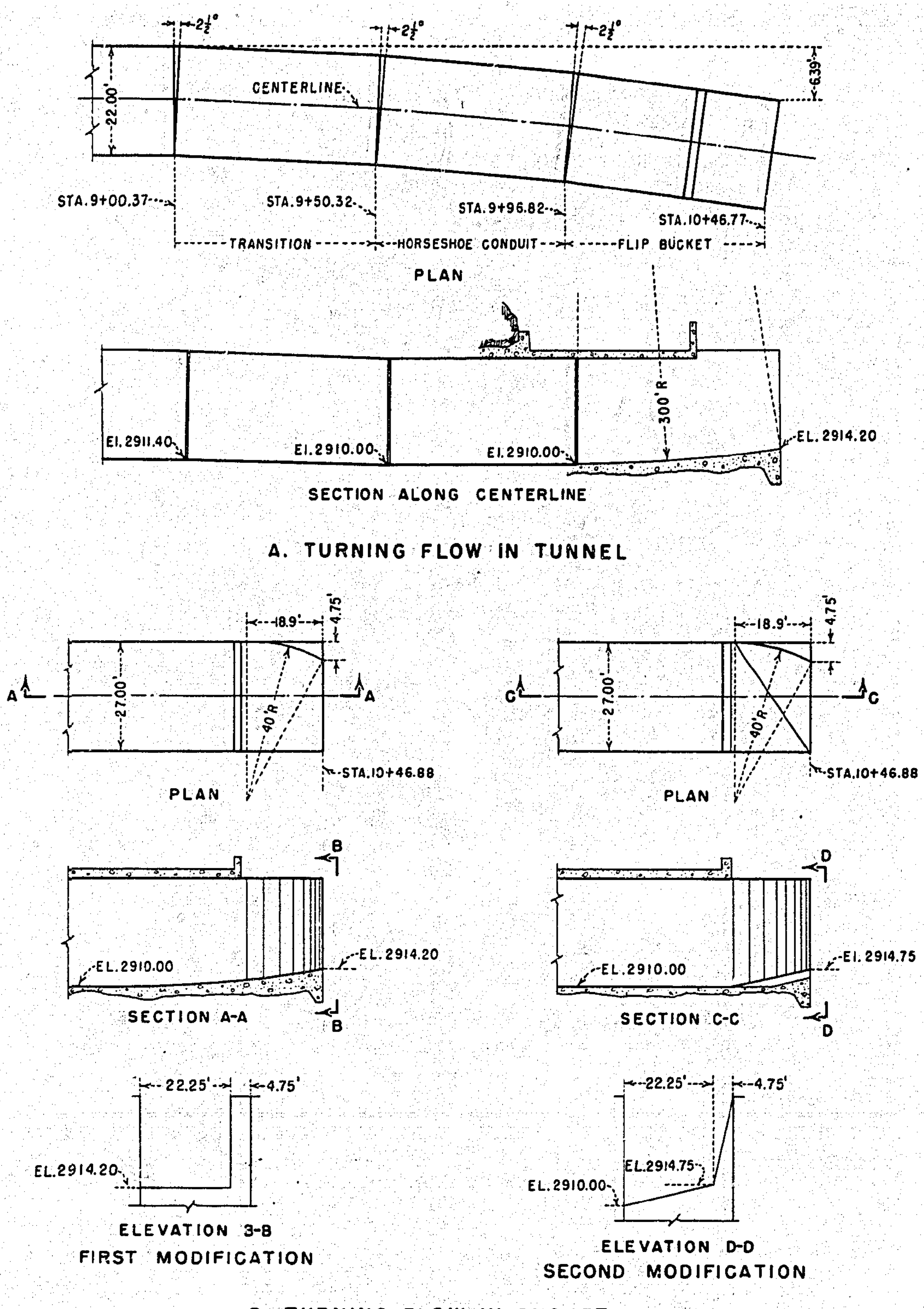


B. Turning Flow in Tunnel - Flow Switches from Left to Right in River, depending on Tail Water Depth. Q = 66,000 cfs.

C. First Modification. Q = 40,000 cfs

D. Third Modification. Q = 66,000 cfs

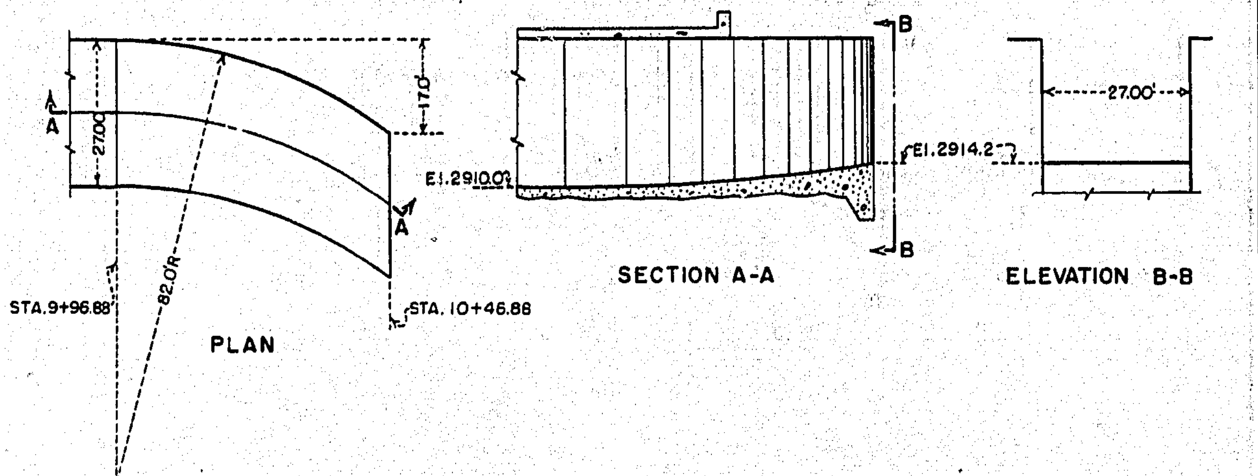
WU-SHEH DAM
TUNNEL SPILLWAY
Hydraulic Model Studies
1:41.25 Scale Model
BUCKET STUDIES



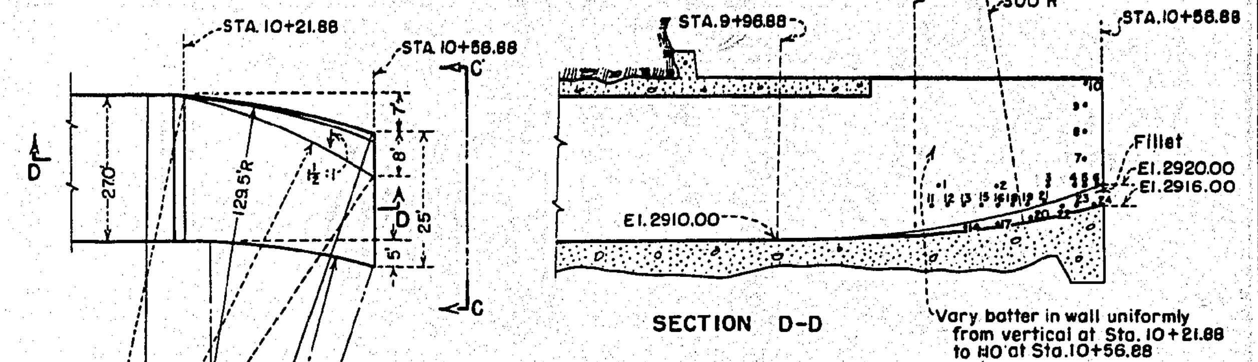
WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1:41.25 SCALE MODEL
BUCKET STUDIES

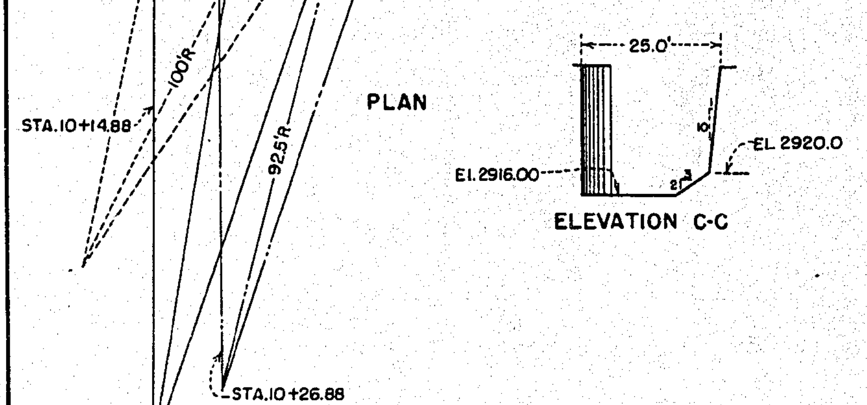
568

FIGURE 17
REPORT HYD 430



A. THIRD MODIFICATION





B. FOURTH MODIFICATION

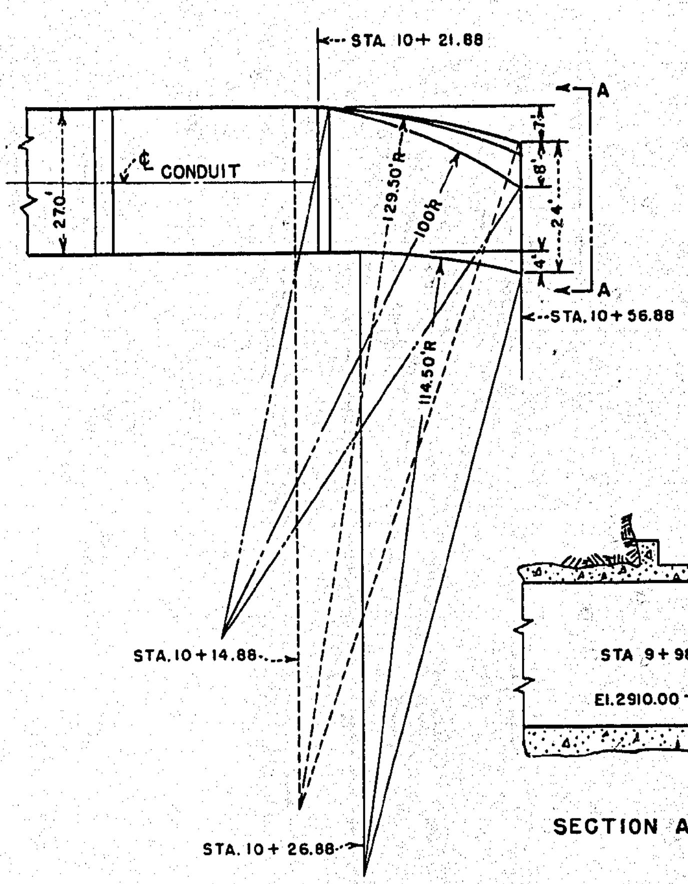
10 0 10 20 30
SCALE IN FEET

Nos. Refer to Piezometers in walls. Nos. 1-10 in left wall, Nos. 11-24 in right wall. See table below.

PRESSURES IN FEET OF WATER

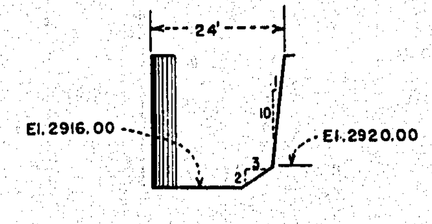
PIEZ No.	10,000 cfs	25,000 cfs	40,000 cfs	55,000 cfs	66,000 cfs
1	-0.3	+18.7	+48.8	+73.0	+87.1
2	+7.2	32.4	56.3	75.5	86.2
3	16.3	45.6	62.5	73.8	81.2
4	16.1	37.3	47.4	54.4	59.5
5	11.8	26.2	33.3	38.1	38.7
6	5.9	13.4	16.9	19.6	21.1
7	9.6	26.6	37.9	43.7	48.3
8	4.8	12.5	22.8	32.7	38.7
9	–	8.3	15.0	23.6	31.6
10	—	—	8.4	14.6	21.2
11	-0.5	-0.8	4.1	13.5	23.2
12	-0.9	-3.1	-0.8	8.0	15.2
13	-0.7	-3.4	-2.1	4.5	12.1
14	+1.5	+3.0	+7.6	16.0	24.4
15	-0.6	-3.5	-3.8	1.2	8.3
16	0	-0.8	-1.4	2.4	7.7
17	+2.5	+3.4	+6.2	11.2	17.0
18	-0.1	+0.8	-0.2	2.1	5.6
19	+0.5	1.6	-0.2	0.2	2.5
20	2.4	5.6	+5.9	1.2	10.9
21	1.4	4.0	2.4	0.4	2.5
22	0.6	2.9	0.4	-1.7	-0.8
23	0.4	1.6	-0.9	-4.0	-3.3
24	1.0	2.0	+0.4	-1.2	+0.5

WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1:41.25 SCALE MODEL
BUCKET STUDIES

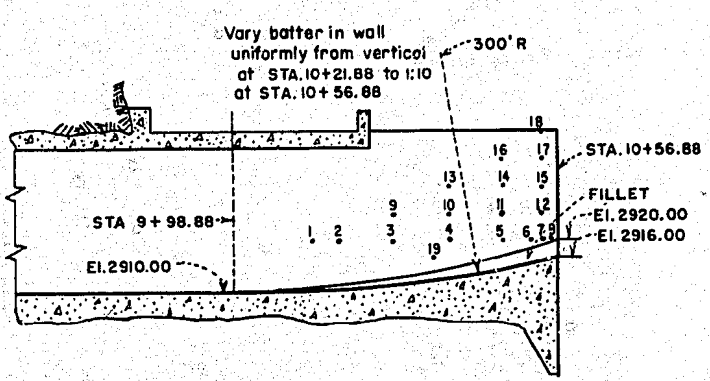


PLAN

10 0 30
SCALE OF FEET



ELEVATION A-A



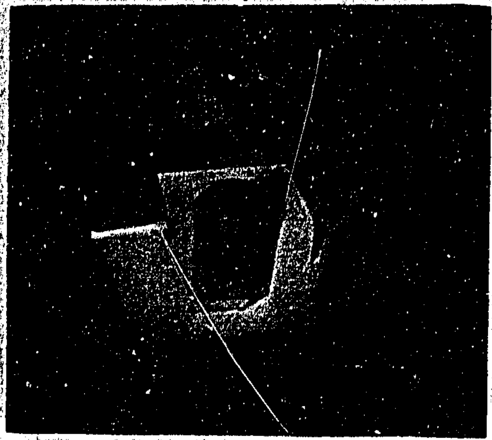
SECTION ALONG CENTERLINE

Nos. Refer to Piezometers
Nos. 1-18 in left wall.
No. 19 in right wall.
See table below.

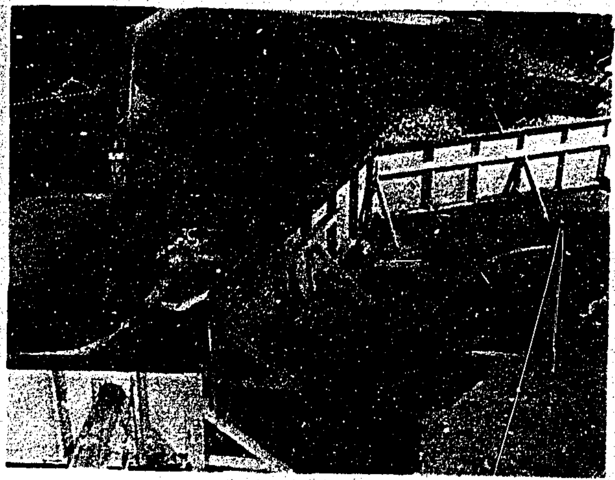
PRESSURES IN FEET OF WATER

PIEZ. No.	5,000 cfs	20,000 cfs.	35,000 cfs	50,000 cfs	66,000 cfs
1	–	–	+9.0	+28.4	+50.2
2	–	+0.2	20.0	43.6	66.6
3	0	12.1	43.2	70.3	90.9
4	+2.3	23.3	50.6	70.7	89.0
5	6.4	36.5	59.2	72.7	84.0
6	6.1	30.8	45.4	53.6	62.0
7	4.4	23.8	35.0	41.2	46.8
8	2.1	13.9	19.4	22.6	26.3
9	–0.7	0	11.7	38.8	65.6
10	0	6.4	26.8	51.2	74.1
11	+0.9	20.5	42.4	58.0	70.2
12	2.7	17.7	32.6	41.0	46.9
13	–	1.0	4.3	21.1	47.5
14	–	–2.1	+5.2	21.1	37.1
15	–	+8.4	17.6	28.2	36.2
16	–	–	0.3	8.0	24.7
17	–	2.6	8.6	16.1	26.8
18	–	–	2.5	4.0	9.0
19	+3.6	+13.8	19.5	22.5	26.8

WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1:41.25 SCALE MODEL
RECOMMENDED BUCKET

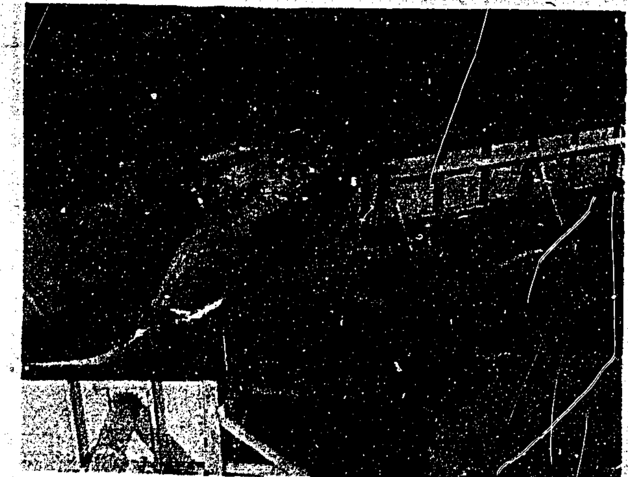


Recommended Bucket



Q = 5,000 cfs

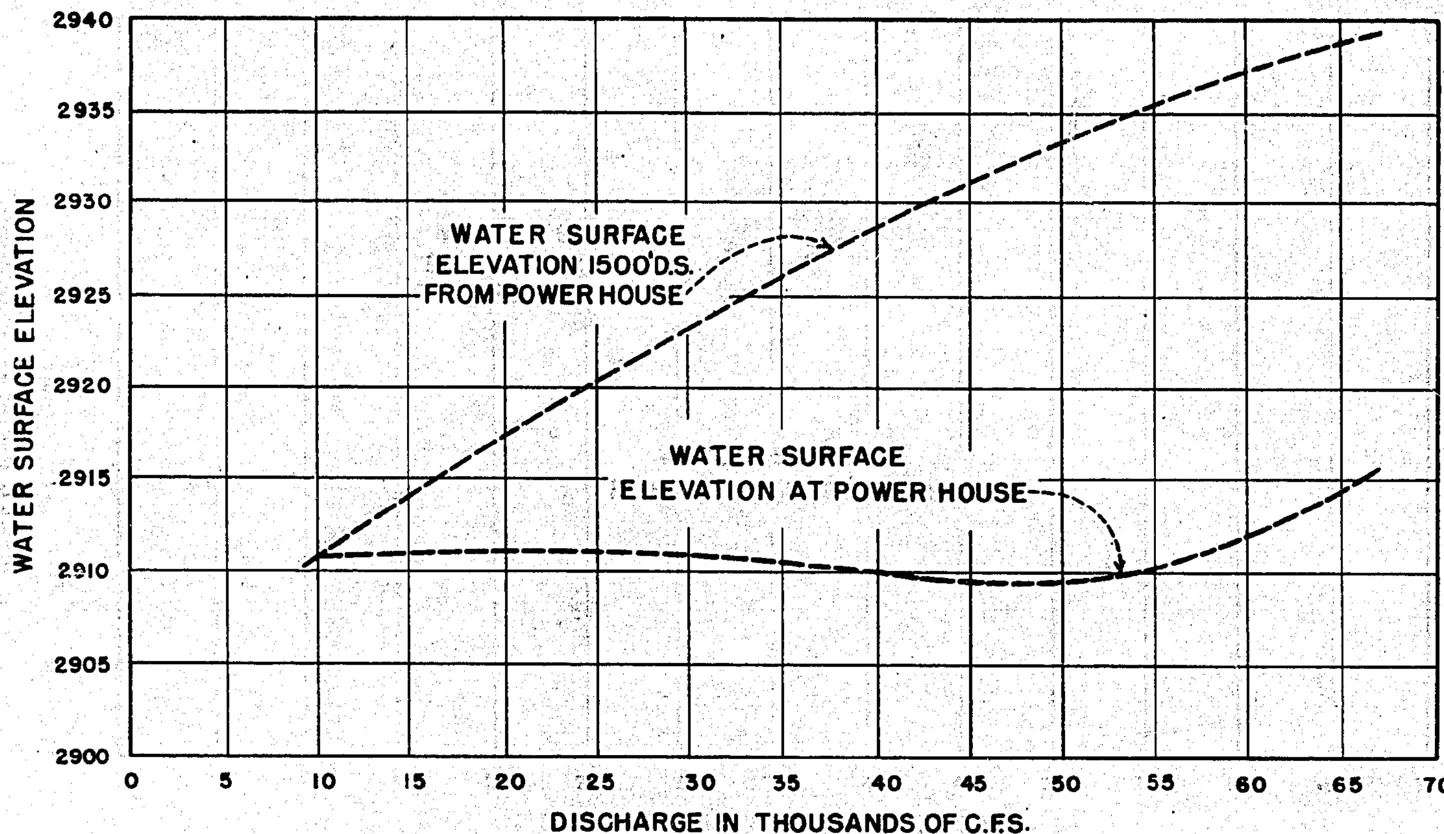
Q = 20,000 cfs



Q = 35,000 cfs

Q = 66,000 cfs

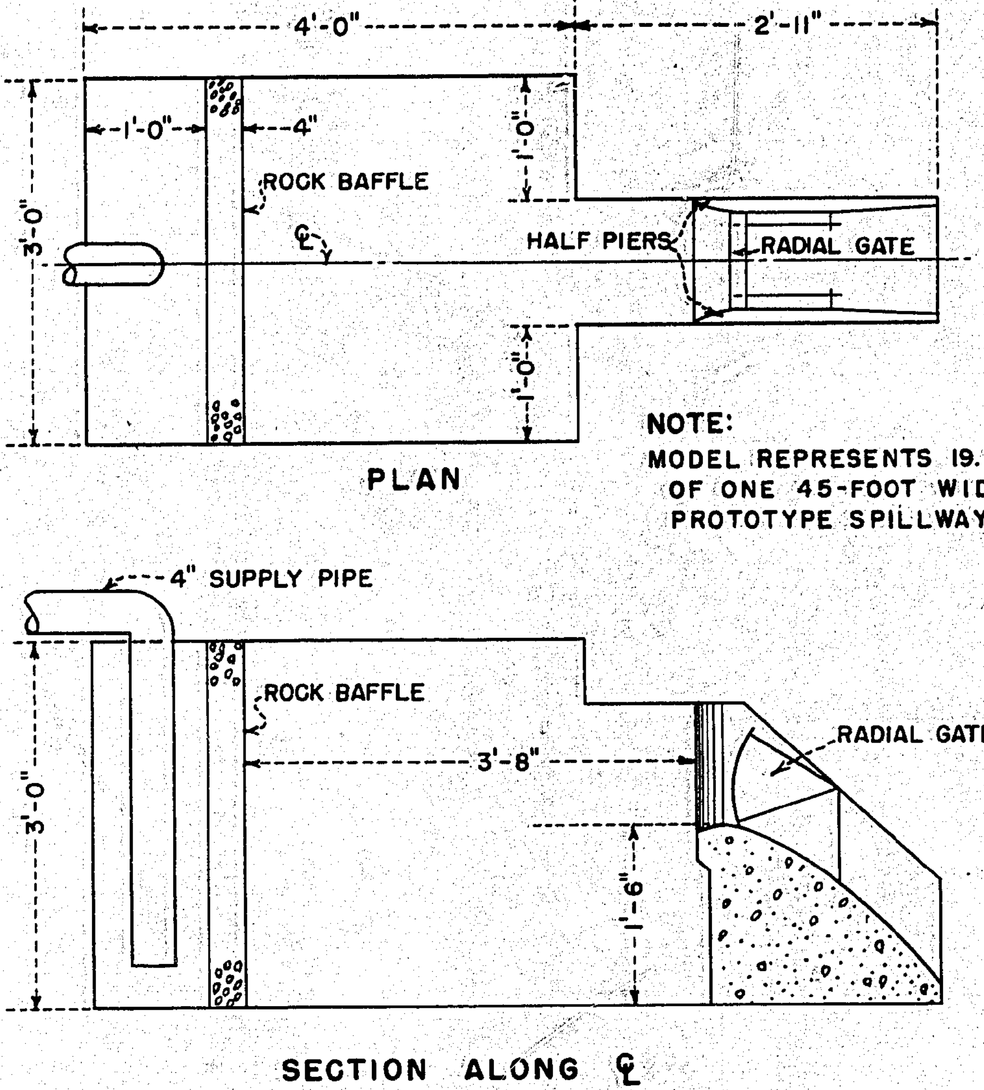
WU-SHEH DAM
TUNNEL SPILLWAY
Hydraulic Model Studies
1:41.25 Scale Model
BUCKET STUDIES
RECOMMENDED BUCKET



WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1:41.25 SCALE MODEL
WATER SURFACE DRAWDOWN

FIGURE 20
REPORT HYD. 430

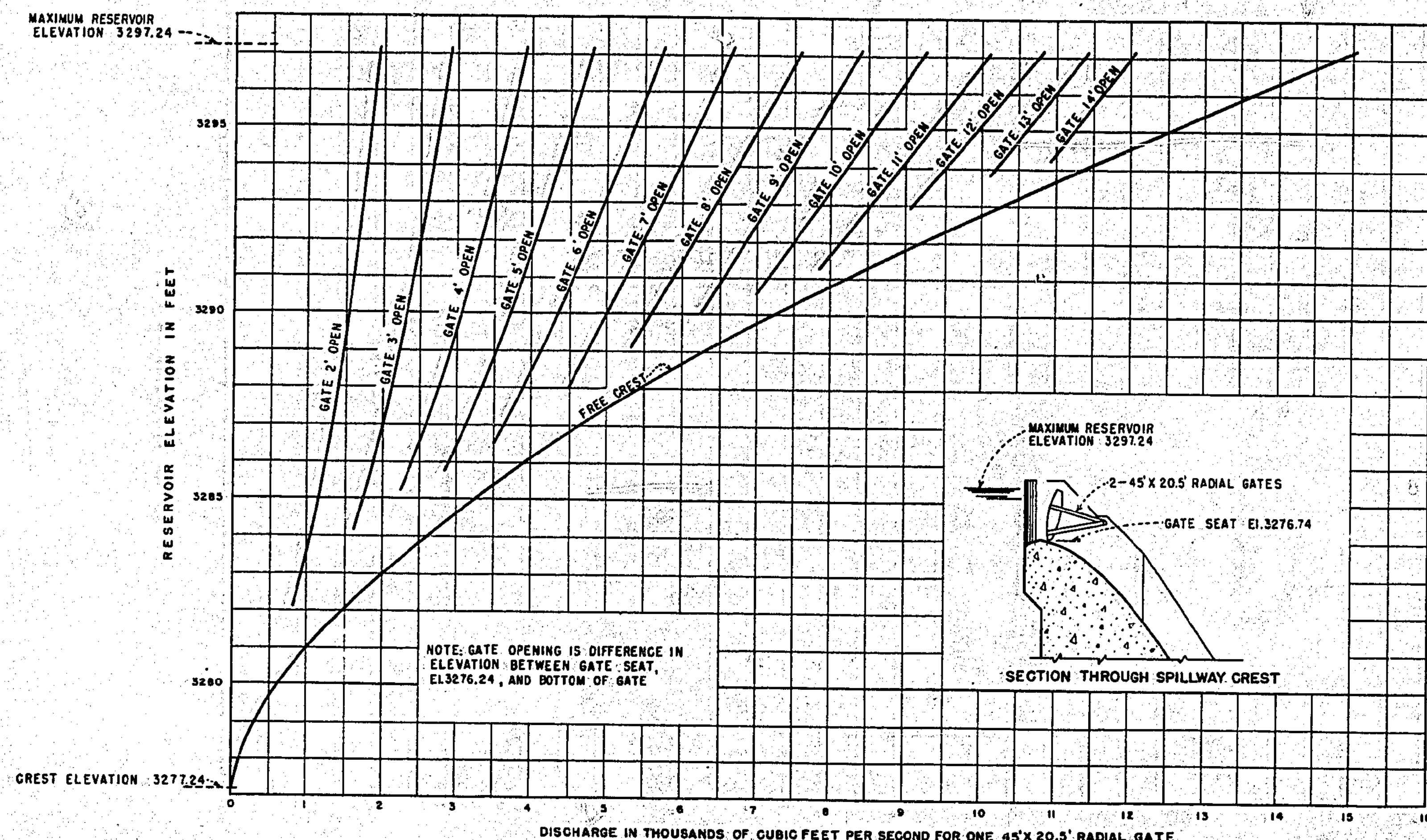
FIGURE 21
REPORT HYD. 430



NOTE:
MODEL REPRESENTS 19.5 FEET OF ONE 45-FOOT WIDE PROTOTYPE SPILLWAY BAY.

WU SHEH DAM
SPILLWAY OVER DAM
HYDRAULIC MODEL STUDIES
1: 25.28 SECTIONAL MODEL
MODEL LAYOUT

568



WU SHEH DAM
OVERFLOW SPILLWAY
HYDRAULIC MODEL STUDIES
1:25.28 SECTIONAL MODEL
DISCHARGE CURVES

FIGURE 22
REPORT HYD. 450

GPO 846529