UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF TWIN BUTTES DAM OUTLET WORKS

SAN ANGELO PROJECT, TEXAS

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CONTENTS

CONTENTS--Continued

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SUMMARY

Hydraulic model studies of the Twin Buttes Dam outlet works, Figure 3, described herein were performed on a 1:30 scale model, Figure 4. The model included a section of the outlet conduits through the dam, the gate control structure containing the fixed-wheel emergency gates and the topseal radial gates, the horseshoe conduits downstream from the gate structure, the outlet works stilling basin, and a section of the channel downstream from the stilling basin, Figure 5.

The preliminary design, with minor modifications, was found to be satisfactory in all respects. Pressure measurements in the gate structure, Figure 8, and discharge capacity tests, Figure 26, showed that the upstream transition, gate chamber, and topseal radial gates were adequate. Observations and photographs, Figure 9, showed that flow in the horseshoe conduits was smooth and well distributed.

Flow emerging from the three conduits spread evenly across the stilling basin chute. However, at the point where the flow from adjacent tunnels met, a large surface fin formed, Figure 11. Six piers with different lengths and shapes were investigated to reduce the fins, Figures 12, 13, 14, and 15. A tapered pier was developed which prevented the tunnel flows from converging too rapidly, eliminated the fins, and produced a smooth water surface. The piers had little, if any, adverse effect on unsymmetrical tunnel operation. The flow spread across the chute and was well distributed on entering the stilling basin for all combinations of closed and operating conduits, Figure 16.

Three stilling basins were investigated, the preliminary basin, the preliminary basin with the chute blocks removed, and a basin 50 feet longer than the preliminary. Erosion tests, wave measurements, and general flow observations were made with all basins. With the preliminary basin, there was good energy dissipation and very little bed erosion for all flows at both low and high tail water conditions, Figures 18, 19, and 20. The stilling action was adequate with the chute blocks removed from the preliminary basin. However, the surface action of the hydraulic jump was considerably rougher, causing more severe channel bank erosion, Figures 21A and 22, and it was recommended that the chute blocks be retained.

The longer basin provided slight improvement in bed erosion and greater improvement in wave action, bank erosion, and general appearance, but the overall performance was not improved sufficiently to warrant the extra cost.

Dynamic pressure measurements were obtained at critical points in the upstream transition, gate chamber, piers, and stilling basin sidewalls. These measurements revealed no cavitation pressures; however, they indicated the frequency and magnitude of the pressure fluctuations, Figure 25, and were of particular value in the structural design of the stilling basin training walls.

INTRODUCTION

Twin Buttes Dam is an earthfill structure located on the Middle Concho River about 9 miles southwest of San Angelo, Texas, Figure 1. The dam embankment will be approximately 8 miles long at the crest and will rise about 134 feet above the riverbed.

The principal hydraulic features are the service spillway and the outlet works, both located near the left abutment of the dam. The spillway, designed for a maximum discharge of 48,000 cfs, will have an uncontrolled overfall crest 200 feet wide with a chute about 300 feet long and a stilling basin, Figure 2.

The outlet works, Figure 3, designed for a maximum discharge of 34,000 cfs, includes an intake structure, three 15.5-foot-diameter conduits from the intake structure to the gate chamber, three 12- by 15-foot emergency fixed-wheel slide gates located just upstream from the three 12- by 15-foot topseal radial regulating gates, three 17-foot-diameter horseshoe conduits from the gate chamber to the stilling basin chute, the chute, a stilling basin, and an excavated channel extending from the stilling basin to the river, Figure 2.

The model studies described herein were concerned with the outlet works from the downstream end of the intake structure to the end of the excavated channel. The studies were made to investigate flow conditions in the transition to the gate chamber and in the gate chamber, the capacity of the radial gates, the flow distribution in the horseshoe conduits and stilling basin chute, the effectiveness of the stilling basin, and the flow in the excavated channel.

The intake structure was studied in a separate and specially constructed model which used low velocity air as the test fluid; these results are discussed in a separate report.* Some of the unique features of the emergency fixed-wheel gates could not be adequately represented in the 1:30 scale model, and it is expected that these features will be studied in a larger model.

*Report Hyd-470 "Twin Buttes Dam Outlet Works Intake Structure" by D. Colgate.

THE MODEL

The model, built to a geometrical scale of 1:30, included the three 15.5-foot-diameter steel conduits, the gate chamber and radial gates, the horseshoe conduits, the stilling basin, and the excavated channel, Figure 4.

Since the intake structure was not studied in this model, the water was distributed to the three circular conduits through a baffled manifold connected directly to the laboratory water supply system. In order to assure smooth flow in the conduits, 4-vane flow straighteners, 1 foot long, were placed at the upstream end of each conduit.

The circular conduits were represented in the model by 6.2-inch-diameter sheet metal pipes. The gate chamber and horseshoe conduits were fabricated from transparent plastic. The radial gates were made of galvanized sheet metal. The gate chamber piers, the stilling basin, and the stilling basin chute blocks and dentated end sill were made of wood treated to resist swelling. The downstream channel was formed in river sand to facilitate scour testing, Figure 5A.

Discharges in the model were measured using calibrated Venturi meters permanently installed in the laboratory. Pressure heads in the conduits were measured by means of piezometers placed in each conduit and located 1 conduit diameter upstream from the gate chamber transition. For each outside conduit, four piezometers were equally spaced around the periphery and connected to a common lead. The center conduit piezometers were placed on the crown and invert and connected to a common lead. The leads from each of the conduits were connected to a separate open-tube glass manometer. Tail water elevations were controlled by an adjustable tailgate at the downstream end of the model; elevation was measured on a staff gage located in the center of the channel 3 feet upstream from the tailgate.

Pressure measurements were made in critical flow areas throughout the structure by means of piezometers connected to open-tube glass manometers. Any piezometer showing subatmospheric pressures, or greatly fluctuating pressures, was connected to a pressure cell and recording oscillograph and a continuous instantaneous dynamic pressure curve obtained.

The model did not include the reservoir area or the entrance structure for the three circular conduits; therefore, to represent a given reservoir water surface elevation in the model, it was necessary to know the corresponding pressure head in the conduits. The pressure head at the piezometer rings in the conduits was determined by computing the hydraulic losses from the reservoir water surface to the pressure-measuring station 1 diameter upstream from the gate chamber transition. The hydraulic losses included were the entrance loss at the intake structure and the friction loss between the gate chamber and the piezometer ring.

The entrance loss (h_e) was determined from $h_e = K \frac{V^2}{2g}$ and K = 0.1, using the velocity based on the discharge and area of the 15.5-foot-diameter conduit. A relatively high K value was used because of the gate slots in the bellmouth entrance. The friction loss h_f was computed from

$$h_f = \frac{1}{d} \frac{V^2}{2g}$$

where

f = 0.010 and 0.014 (to bracket the extremes of roughness)
l = 178 feet
d = 15.5 feet
$V = \frac{Q}{A}$ for the 15.5-foot-diameter conduit (Q = discharge in cfs)
(A = area in sq ft)

The two f values used indicate the probable losses in new and old pipe, respectively; however, for most of the tests, an average loss based on f = 0.012 was used.

THE INVESTIGATION

During the investigation, test discharges were based on 3 reservoir elevations and 2 tail water elevation curves. The 3 reservoir elevations were the maximum reservoir elevation 1985, maximum flood control pool elevation 1969.1, and maximum conservation pool elevation 1940.2.

For discharges at maximum reservoir elevation, the high tail water curve was used, Figure 6. At maximum reservoir elevation, the spillway discharges 48,000 cfs. Since the flow from the outlet works discharges into the same channel as the spillway, the tail water elevation for the outlet works is governed by the spillway flow plus the outlet works flow. At the two lower reservoir elevations, the tail water elevation is dependent only on the outlet works flow, and the lower tail water curve of Figure 6 was used.

The Gate Chamber

In this report, the gate chamber will consist of (a) the transition between the circular conduit and the gate control structure, (b) the gate control structure, and (c) the transition between the gate control structure and the horseshoe conduits.

Upstream transition. The upstream transition was 20 feet long and changed the 15.5-foot-diameter circular tunnel to a 12-foot-wide by 15-foot-high rectangular passage, Figure 7. The change in shape was accomplished by increasing the radii of the surfaces forming the top, bottom, and sides of the transition. While maintaining the 0.02 slope of the tunnel invert, the crown was sloped to attain the 15-foot height, and both sides were equally converged to the 12-foot width. A table showing the rate at which the cross section of the transition was changed and the dimensions at each of four sections is given in Figure 7.

The tunnels upstream from the gates will be pressure tunnels; therefore, of primary concern was whether there would be any local cavitation pressures in the transitions. Three rows of four piezometers were installed in one transition, one row along the intersection of the roof arc and side arc, a second row along the intersection of the invert arc and the side arc, and the third row along the side midway between the invert and the roof, Figure 7.

The piezometers indicated that there were no subatmospheric pressures in the transition. The lowest pressure readings were obtained for a discharge of 34,400 cfs through the 3 conduits with the reservoir at elevation 1969.1. At this operating condition, the lowest pressure was near the downstream end of the transition and corresponded to elevation 1896.9, equivalent to 1.4 feet of water above atmospheric. With a discharge of 36,000 cfs at reservoir elevation 1985.0, the lowest piezometric pressure head elevation was 1897.4, equivalent to 1.9 feet of water above atmospheric. For a discharge of 25,000 cfs at reservoir elevation 1940.2, the lowest piezometric pressure head elevation was 1899.8, equivalent to 4.3 feet of water above atmospheric. The pressures at all piezometers for the three discharge conditions are tabulated on Figure 7. These operating conditions were based on the three basic reservoir elevations previously described with the radial control gates fully open. With the gates partially closed, the piezometric pressures were higher in all instances.

Gate control structure. The gate control structure in each of the 3 conduits consists of a 12- by 15-foot rectangular passage. Each passage contains a fixed-wheel slide gate followed by a topseal radial gate, Figure 8. The radial gates will be used for flow regulation, and the fixed-wheel gates will be used for emergency regulation if the radial gates need repair. The fixed-wheel gates were not tested, but the gate slots were reproduced in the model. Piezometers were installed in critical areas in the right side and roof of the right passage, Figure 8. Two rows of piezometers were placed in the gate slot and downstream from the slot along the right wall. One row containing 3 piezometers was 4 inches above the floor, and the second row containing 5 piezometers was 5 feet above the floor. Two rows of piezometers were also located in the roof of the passage in and downstream from the gate slot. One row containing 5 piezometers was placed along the center line and one row containing 4 piezometers 4 inches from the right wall.

These piezometers were used to determine whether satisfactory pressure conditions existed during operation with the emergency gates fully open. The tests showed that the pressures were at or above atmospheric for all flows. The lowest pressures found were in the roof on the upstream side of the gate slot. These pressures were about 1/2 foot of water below atmospheric when the radial gates were fully open and discharging 34,400 cfs. The pressures were above atmospheric for the other discharges tested. With the radial gates partially closed, the pressures were consistently higher.

The downstream edge of the gate slot was offset 2.25 inches away from the flow. The return to the original alinement was by means of a long radius curve in a length of 29.31 inches, Figure 8. Piezometers at the offset and at the tangent point downstream indicated above-atmospheric pressures for all flows. The pressures obtained from these piezometers are tabulated in Figure 8.

Downstream transition. Downstream from the radial gates, the sidewalls in each passage diverged to increase the channel width to 17 feet. The roof of the gate chamber was also transitional to form the semicircular crown of the downstream horseshoe conduits, Figure 8.

Flow was observed to pass through this section in a satisfactory pattern at all times. When the radial gates were partially closed, the water surface was smooth and theflow was well distributed at the start of the horseshoe conduits, Figure 9. When the gates were fully open, the pattern was similar and the gate trunnions were above the water surface at all times. Figure 10 shows the water surface profile for the maximum discharge. Piezometers placed along the right side of the right tunnel indicated above-atmospheric pressures for all discharges. The pressure readings are given in the table in Figure 8.

Horseshoe Conduits

The flow passages between the end of the gate chamber, Station 22+86.00, and the start of the stilling basin, Station 24+68.50, were horseshoe conduits 17 feet in diameter. The spacing from center line to center line of adjacent conduits was 20 feet, Figure 3.

Flow in the horseshoe conduits was satisfactory at all discharges. There were some undulations in the water surface, but they were minor in nature and never extended more than 2 feet above the springline of the semicircular crown. Figure 9 shows the flow in the horseshoe conduits for maximum discharge.

Exit Portal of the Horseshoe Conduits

The water jets began to spread as they emerged from the three conduits. However, along the contact line, where the flow from adjacent tunnels met, a large surface fin of water formed, Figure 11. These fins extended down into the stilling basin, and although they did not cause extensive adverse flow conditions, it was decided to eliminate the fins (a source of spray) and improve the appearance of the flow.

To prevent the tunnel flows of adjacent tunnels from converging too rapidly, tapered piers were installed between the tunnels at the tunnel portals. Four different pier shapes were investigated during this study, Figures 12 and 13. All of the piers were 12.5 feet high and 3.0 feet thick at the portal, but each pier differed in length, taper, and end or nose shape. The first pier was 14.75 feet long; in the downstream 5 feet the pier was tapered to a sharp edge. The tapered portion was joined to the parallel portion with a 9.08-foot-radius curve, Figure 12A. This pier reduced the fin height but did not eliminate it. The second pier tested was increased to 16.25 feet long, and the downstream 7.5 feet was tapered to a sharp edge. The two portions were joined with a 19.50-foot radius, Figure 12B. This pier practically eliminated the fin for discharges up to 20,000 cfs, but for larger discharges, an objectionable fin was formed. The third pier tested was 14.75 feet long, the downstream 6 feet was tapered with a 21.00-foot-radius curve and terminated in a blunt nose 1.25 feet thick, Figure 12C. This pier was very effective at all flows with only a small fin appearing for the larger discharges. The fourth pier tested was 18.75 feet long; the downstream 8.75 feet was tapered to terminate in a blunt nose 1.25 feet thick, Figure 13. No radius was used. This pier reduced the fin to negligible size, resulting in a very smooth water surface entering the stilling basin.

Three piezometers were installed on the side of the fourth pier to determine whether adverse subatmospheric pressures were formed at the sharp break at the start of the taper. The piezometers were in a vertical line 0.16 foot downstream from the break point; one piezometer was 0.63 foot above the floor; the second was 2.5 feet above the floor; and the third was 5.0 feet above the floor. Pressure measurements were made for 3 discharges, 12,000, 25,000, and 34,400 cfs (all flows are for 3 conduits operating). For the first 2 discharges, reservoir elevations of 1985.0, 1969.1 and 1940.2 were used. For the maximum discharge, only the first 2 reservoir elevations were used, making a total of 8 test runs. Subatmospheric pressures were found in all tests; the lowest pressure occurred at the piezometer nearest the water surface. No tests were made with the flow depth less than 2.5 feet. The lowest pressure measured was 8.6 feet of water below atmospheric and occurred at a discharge of 25,000 cfs and reservoir elevation 1985.0; for the other discharge-reservoir elevation combinations, the pressures were higher. A complete tabulation of the pressures is shown on Figure 13.

Although the lowest pressure was above the cavitation range, it was considered desirable to raise the low pressure by rounding the sharp intersection at the start of the pier taper. The sharp break was replaced successively with arcs, tangent to the plane surfaces, described by radii of 12.5, 25, and 37.5 feet. Pressure measurements were made at the discharge which previously had indicated the lowest pressure, 25,000 cfs at reservoir elevation 1985.0. The piezometer pressures were plotted versus the radius of the arc, and curves connecting these points were drawn, Figure 14. The curves showed that pressure increased with an increase in radius. Extrapolation of the curves indicated that a 100-foot-radius arc would be required to provide atmospheric pressure conditions on the pier.

The piers were rebuilt using a 100-foot-radius arc to streamline the converging section, and 2 rows of 4 piezometers were installed along the curved portion; 1 row was 0.63 foot above the floor, and the second row was 5 feet above the floor, Figure 15. Pressure measurements were made at discharges of 12,000, 25,000, 34,400, and 36,000 cfs, with the maximum reservoir elevation 1985.0 and maximum storage pool elevation 1969.1. The lowest pressures occurred at a discharge of 25,000 cfs and maximum reservoir elevation. The pressure was 2.76 feet of water below atmospheric and was in the same area where low pressures had occurred in the previous tests. Since pressures of this magnitude are not conducive to cavitation and since the tests had indicated that further streamlining would not materially improve the pressure, this pier was recommended for prototype installation. The results of the pressure studies are tabulated on Figure 15.

Flow appearance with the recommended piers was very good. The flow spread evenly across the chute and was uniformly distributed on entering the stilling basin, Figure 11. The piers did not greatly hinder the lateral spreading of the individual jets when the tunnels were operated unsymmetrically. When 1 or 2 tunnels were discharging, the flow spread across the chute and was distributed sufficiently to produce an acceptable hydraulic jump for emergency operation, Figure 16. Best operation, as might be expected, occurred when the tunnels were operated symmetrically.

One piezometer was installed in the right wall downstream from the right tunnel exit portal to determine whether the intersection of the tunnel wall with the stilling basin training wall should be streamlined to prevent excessive subatmospheric pressures, Detail A, Figure 17. The lowest pressure was 4.74 feet of water below atmospheric when the discharge was 25,000 cfs at reservoir elevation 1985.0. When the intersection was streamlined with successive radii of 12.5, 25.0, and 37.5 feet, the pressure was increased at the piezometer to -4.69, -2.04, and -1.44 feet of water, respectively. The streamlining was accomplished in the model by placing half of the curve on the straight wall of the conduit and half on the diverging wall of the chute, Detail A, Figure 17. Based on the results of the tests, it was decided to use a 100-foot-radius arc to streamline the intersection. But in order to avoid the added expense of special concrete forms at the prototype conduit exit, the point of curvature of the arc was placed at the exit portal and the angle of divergence was increased slightly so that the chute walls would be tangent to the 100-foot-radius arc. Since this change was negligible when reduced to model dimensions, the model was not modified.

Stilling Basin Chute

The chute to the stilling basin was a rectangular open channel diverging from 57 feet wide at the tunnel portals to 80 feet wide at the stilling basin. The channel bottom had a 0.01 slope for the first 47.82 feet, a vertical curve for the next 100 feet, and a 2:1 slope for the final 32.18 feet, Figures 3 and 17.

It is important that flow in the chute be well distributed laterally (the flow must diverge rapidly to follow the diverging walls) and that the velocity be more or less uniform from side to side. Tests showed that the performance of the chute was excellent in all respects during symmetrical tunnel operation. This was due, in large part, to the almost horizontal area upstream from the vertical curve which allowed the flow to begin to spread before passing over the vertical curve. The flow was uniformly distributed across the chute and entered the stilling basin smoothly and evenly.

With all combinations of 1 or 2 tunnels operating, the flow distribution was satisfactory on entering the stilling basin. Although the flow did not spread uniformly across the full width of the chute, the distribution was sufficiently good that no adverse eddies formed in the stilling basin. Figure 16 shows the flow conditions in the chute and stilling basin for various combinations of closed and operating conduits.

Stilling Basin Studies

The stilling basin studies were concerned with developing an effective stilling basin that would provide good energy dissipation with a minimum amount of bank and channel bed scour for all of the expected combinations of discharge and tail water elevation. Before the stilling basin studies are described, the design will be discussed in terms of the theoretical considerations.

The stilling basin is a Type II basin as defined in Engineering Monograph No. 25, "Hydraulic Design of Stilling Basins and Bucket Energy Dissipators." The critical flow conditions on which the design was based are:

Discharge = 34,200 cfs
Reservoir elevation 1969.1
Tail water elevation 1877.1

Using the Manning formula and a skin friction loss coefficient of $n = 0.008$, the depth and velocity of the flow entering the basin are computed to be:

$D_1 = 5.03$ feet and $V_1 = 85$ feet per second

The Froude number of the entering flow is $F = \dfrac{V}{\sqrt{gD}} = 6.67$

The basin dimensions derived from the hydraulic design curves of the monograph are:

$D_2 = 8.8D_1 = 44.3$ feet (D_2 is the sequent or conjugate depth)
Elevation of stilling basin floor = 1877.7 - 44.3 = 1833.4
Min TW = $8.5D_1 = 42.8$ feet (This is the minimum depth possible)
$L_{II} = 4.1D_2 = 181.7$ feet (L_{II} is the basin length required)
Chute block height, width, and spacing = $D_1 = 5.03$ feet
Height of dentils on end sill = $0.2D_2 = 8.9$ feet
Width and spacing of dentils = $0.15D_2 = 6.6$ feet

D_1 and V_1 at the upstream end of the chute blocks, determined from depth measurements made on the model, were 6.18 feet and 73.8 feet per second, respectively, giving a Froude number $F = 5.23$. Using these values, the basin dimensions are:

$D_2 = 7D_1 = 43.3$
Elevation of stilling basin floor = 1877.7 - 43.3 = 1834.4
Min TW = $6.7D_1 = 41.4$
$L_{II} = 3.9D_2 = 169$ feet
Chute block height, width, and spacing = $D_1 = 6.18$ feet
Height of dentils on end sill = $0.2D_2 = 8.7$ feet
Width and spacing of dentils = $0.15D_2 = 6.5$ feet

Past experience has shown that the length of Type II basins can be reduced to about $3D_2$ when model tests are made. This can be done safely because in a model study it is possible to improve entrance flow conditions and other factors which adversely affect the performance of a stilling basin. Since it was expected that this basin would be tested, the length of the preliminary basin was set at 135 feet rather than the 182-foot length specified by the monograph methods.

Table 1

STILLING BASIN COMPARISON

Basin	Data source	Design basis	D_1	V_1	F	D_2	El of basin floor	Minimum tail water Depth	Minimum tail water El
(1)Theoretical	Computed V_1, D_1	Mono 25	5.03	85.0	6.67	44.3	1833.4	42.8	1875.8
(2)Theoretical	Measured V_1, D_1	Mono 25	6.18	73.8	5.23	43.3	1834.4	41.4	1874.4
(3)Preliminary & recommended	Computed	Modified Mono 25	5.03	85.0	6.67	45.2	1833.0	42.8	1875.8

Basin	L_{II}	Chute block height and width	Dentil height	Dentil width
(1)	181.7	5.03	8.9	6.6
(2)	169.0	6.18	8.7	6.5
(3)	135.0	5.25	9.0	5.25

Tail water elevation at the outlet works stilling basin is governed not only by the discharge from the outlet works but also by the flow from the emergency spillway since the efflux from both the spillway and the outlet works eventually enters Middle Concho River channel. Whenever the reservoir elevation is higher than the service spillway crest elevation 1869.1, the tail water elevation is determined by the combined flows of the spillway and the outlet works; when the reservoir elevation is below 1869.1, the tail water elevation is determined by the outlet works discharge only.

Channel conditions in the Middle Concho River also have a bearing on tail water elevations. Two tail water elevation curves are shown in Figure 2; the upper curve for existing channel conditions, the lower curve for a degraded channel. Exploratory model tests showed that the most severe operating conditions occurred for low tail water elevations; consequently, the tail water curves of Figure 6, derived from the lower curve of Figure 2, were used in the model tests.

Preliminary Stilling Basin--Recommended

The preliminary stilling basin was 135 feet long and 80 feet wide. The floor was at elevation 1833.0, and the top of the training walls was at elevation 1896.0. Chute blocks were used at the upstream end of the basin, and a dentated sill was placed at the downstream end, Figure 17. The seven chute blocks, equally spaced across the basin at the toe of the chute, were 5.25 feet high and 5.25 feet wide; the top edges of the chute blocks were streamlined with elliptical curves. The 20-foot-long dentated end sill had a 2:1 slope on the upstream and downstream faces. Six dentils, equally spaced on the upstream face of the sill, were 9 feet high and 5.25 feet wide. The two dentils adjacent to the wall on either side were 5 feet 10-1/2 inches wide. The upstream edges of each dentil were streamlined with a 12-inch-radius quarter-circle, Figure 17.

Downstream from the stilling basin, the riprapped channel bed sloped upward on a 5:1 slope to elevation 1862.0. The bottom width diverged from 80 feet at the stilling basin to 200 feet at the top of the slope. The sides of the channel were formed on a 2:1 slope. At the top of the slope, the channel curved to the right in a 400-foot-radius curve with a 26.5° central angle, Figure 4. Although the prototype specifications called for riprap cover on the channel bed, the model channel was formed in river sand to facilitate the scour studies, Figure 5A.

The effectiveness of the stilling basin was evaluated for two maximum flow conditions. The first operating condition was: discharge 34,400 cfs, reservoir elevation 1985.0, tail water elevation 1890.8. The second operating condition was: discharge 34,400 cfs, reservoir elevation 1969.1, tail water elevation 1878.4. The criteria used to evaluate the stilling basin performance were (1) the general appearance of the hydraulic jump, (2) the magnitude of the wave action in the channel downstream from the basin, and (3) the amount and extent of bank erosion and channel bed scour after a 45-minute model test.

For the first operating condition, flow appearance was very good. The surface of the hydraulic jump was rough with considerable surging but was well contained within the confines of the basin, Figure 18. At the toe of the jump, there was considerable splashing, and some water overtopped the training walls; however, these actions did not extend downstream. The wave heights, measured in the center of the downstream channel halfway around the curve, were about 2.1 feet maximum with the average height being about 1.5 feet. There was some channel bank erosion during the 45-minute run, but it was not severe. The channel bed scour was very mild, Figure 19B. The apron was not undercut, and the upward slope of the bed was not damaged. In the prototype, riprap will be used on the banks and channel bottom; erosion and scour are, therefore, expected to be negligible.

For the second operating condition, the channel bed was remolded as shown on Figure 19A, and the model was operated for 45 minutes. With the lower tail water elevation, the jump action in the basin was considerably rougher although the training walls were not overtopped, Figure 20. The boil at the end of the jump extended about 30 to 50 feet beyond the end of the basin. This resulted in some swirling action on the surface with a clockwise eddy forming on the right side of the basin and a counterclockwise eddy on the left side. The swirling action caused rapid destruction of the channel banks, but the bed scour was still negligible, Figure 19C. Wave heights in the channel were about 2.8 feet maximum and averaged 2.0 to 2.5 feet.

The values for the minimum tail water depth given on Table 1 indicate that with the stilling basin floor at elevation 1833 the minimum tail water elevation for the design discharge would be 1875.8 for Basins (1) and (3) and 1874.4 for Basin (2). If the tail water elevation dropped below the theoretical elevation, the jump could be expected to start to sweep out.

In order to establish a safety factor for the stilling basin, a sweep out test was made to determine the tailwater elevation at which the toe of the hydraulic jump moved downstream onto the horizontal floor. The sweep out test was made at a discharge of 34,400 cfs and reservoir elevation 1985.0. At the maximum tail-water elevation, 1890.8, the toe of the jump was about 30 feet upstream from the chute blocks; for tail water elevation 1873.5, the toe of the jump was directly over the chute blocks; at tail water elevation 1871, the toe of the jump fluctuated from the downstream end of the chute blocks to about 10 feet downstream. For all practical purposes, this was considered to be the sweep out point. The tail water could thus be lowered over 7 feet below the theoretical design elevation, or 3 to 5 feet below the theoretical minimum tail water elevation, before the jump was in a position to be swept out of the basin. The higher-than-expected factor of safety is probably due to the stilling basin training walls extending into the channel with a relatively large body of water on each side of the stilling basin. These areas stand at a higher level than the high velocity water leaving the basin; consequently, there is flow from them toward the basin. This tends to increase the tail water elevation in a local area at the end of the basin and delays the sweep out.

First Modification

The chute blocks in the preliminary basin were removed to determine whether the basin would still operate satisfactorily. The other features of the basin were not changed. The same tests that had been used to evaluate the preliminary basin were also used for the investigation of the modified basin.

At a discharge of 34,400 cfs at reservoir elevations 1985.0 and 1969.1 and with tail water elevations 1890.8 and 1878.4, the hydraulic jump was much rougher. Surges and boils at the end of the basin were more severe than they had been for similar operating conditions in the preliminary basin, Figures 21A and 22. Wave heights in the center of the channel averaged about 2.5 feet with a maximum height of 3.0 feet during high tail water operation; for low tail water operation, the average wave height was about 3.4 feet, attaining a maximum height of 3.6 feet.

A scour test was run at maximum discharge and low tail water elevation since this had proved to be the most severe operating condition for the preliminary basin. Bank damage was very severe, much worse than for the previous tests, but the bed scour was negligible, Figure 22. A bar composed of sand, which had eroded from the banks, formed on the right side of the channel at the start of the curve but did not cause any adverse flow conditions.

Jump sweep out tests showed that the toe of the jump moved onto the horizontal floor of the basin when the tail water elevation was 1875.0, reducing the factor of safety from about 7 feet to about 3 feet.

Because of the poor flow appearance, severe bank erosion, and lower margin of safety against jump sweep out, it was concluded that the chute blocks should be retained in the stilling basin.

Second Modification

For the second modification, the basin length was increased to 180 feet, the approximate theoretical length determined from Engineering Monograph No. 25, Basin (1), Table 1. The preliminary chute blocks were retained in position, but the end sill was placed at the end of the basin extension. The same criteria that had been used to evaluate the previous basins were used here.

For a discharge of 34,400 cfs at reservoir elevations 1985.0 and 1869.1 and tail water elevations 1890.8 and 1878.4, the flow appearance was better than in the previous basins, Figures 21B and 23. The hydraulic jump was fully contained within the basin, and the surging and splashing were greatly reduced. The eddy currents beyond the end of the basin were almost negligible, and the bank erosion was consequently reduced. For the low tail water elevation, the average wave height was about 1.7 feet and the maximum wave height was about 2.2 feet. For high tail water, the average wave height was about 2.5 feet and the maximum wave height was 3.0 feet. The sweep out tests indicated that the toe of the jump moved onto the horizontal floor when the tail water elevation was 1872.5, giving a factor of safety of about 6 feet.

A scour test made with a discharge of 34,400 cfs, reservoir elevation 1869.1 and tail water elevation 1878.4, showed that after a 45-minute run, both the bank erosion and channel bed scour were negligible, Figure 23.

The tests indicated that the longer stilling basin performed better than the preliminary basin. However, since the preliminary basin was satisfactory in all respects in that riprap will be used to protect the riverbanks, it was recommended that the shorter preliminary basin be constructed rather than the more expensive long basin.

Pressure Investigations

Because the stilling basin is constructed without wingwalls, the end of the stilling basin projects out into the tail water pool. The water behind the training walls is not in motion and stands at about the elevation of the downstream tail water, producing a more or less constant force on the outside of the training walls. During outlet works operation, the water surface profile inside the basin is below the tail water elevation, producing a differential pressure on the walls. In addition, dynamic forces produced by the hydraulic jump action create intermittent pressures on the inside face of the walls that vary above and below the average pressures. To aid in the structural design of the training walls, these forces were evaluated in the model. Pressure measurements were made on the training walls of the stilling basin to determine the magnitude of the pressures on each side of the wall, the pressure differential on the wall, and the amount of pressure fluctuation.

Piezometers were installed along the inside surface of the right wall at Stations 27+20, 27+60, and 27+80; at each station, piezometers were placed at elevations 1855, 1865, 1875, and 1885. One piezometer was also installed at Station 26+51.75, elevation 1839, Figure 17.

The piezometers were connected to pressure cells sensitive to instantaneous pressure fluctuations. Pressure fluctuations and magnitude were converted in an electronic circuit to signals which activated a direct writing oscillograph. The trace produced on the oscillograph chart thus became a measurement of the frequency and amplitude of the dynamic pressure at the piezometer. Pressure readings of the four piezometers at each station were recorded simultaneously along with the water surface elevation on the outside of the wall. In this manner, the difference in pressure on the two sides of the wall, the instantaneous dynamic pressure on the inside of the wall, and the amplitude and frequency of the pressure fluctuations could be obtained. These data were obtained for a discharge of 34,400 cfs at reservoir elevation 1985.0 with tail water elevation 1890.8 and a discharge of 34,400 cfs at reservoir elevation 1969.1 with tail water elevation 1878.4. Water surface profiles along the stilling basin training walls were measured by mechanical means during the pressure tests to aid in analyzing and interpreting the pressure measurements. These profiles are shown in Figure 24. Typical oscillograph records of some of the piezometers are reproduced in Figure 25. A tabulation of representative pressures obtained from the oscillograph records is given in Figure 17.

An examination of the oscillograph records showed that the greatest pressure fluctuations and the lowest subatmospheric pressures occurred on the wall near the toe of the chute for the 34,400-cfs discharge at reservoir elevation 1969.1 and tail water elevation 1878.4. For these operating conditions, the water surface on the outside of the wall fluctuated between elevations 1876.5 and 1880.4, the average frequency between peaks being about 3 to 5 prototype seconds. In this test, the piezometers at elevation 1885.0 were above the water surface.

At Station 26+51.75, the piezometer at elevation 1839.0 indicated a pressure variation from elevation 1837.0 to 1902.0; on the outside of the wall the earthfill extends approximately to Station 26+65 so the tail water fluctuation would probably not have a direct effect on the wall. On the inside of the wall, the water surface profile indicates that the maximum water surface elevation would be about 1863.0 or 24 feet of water above the piezometer opening or datum. The dynamic or instantaneous pressure obtained from the piezometer indicated an impact pressure equivalent to about 63 feet of water above the datum and a subatmospheric pressure of about 2 feet of water below the datum, a total fluctuation of about 65 feet of water. As indicated on Figure 25, a maximum and a minimum pressure usually occurred a fraction of a prototype second apart followed by a period of several seconds during which there were lesser fluctuations before another major fluctuation occurred.

At Station 27+20, the maximum water surface elevation was at about 1874.0 or 9 feet of water above the piezometer located at elevation 1865.0 and 19 feet of water above the piezometer at elevation 1855. The dynamic pressure measurement from the piezometer at elevation 1865.0 indicated a maximum pressure of 13.5 feet of water above the datum point and a minimum pressure equivalent to atmospheric pressure. The period between a maximum and a minimum pressure was about 1 second, with about 5 seconds elapsing before a subsequent pressure surge. At the lower piezometer, elevation 1855.0, the dynamic pressure measurements indicated a maximum pressure of 29.5 feet of water and a minimum pressure of 3.2 feet of water. The maximum and minimum pressures occurred within an interval of less than 1 second followed by a period of several seconds during which there were lesser fluctuations before another maximum-minimum surge occurred.

At Station 27+60, the water surface profile had a maximum elevation of about 1882.0. The piezometer at elevation 1875.0 showed a maximum pressure of 6 feet of water, and the minimum pressure was equivalent to atmospheric pressure. As shown on Figure 25, no abrupt change in pressure was indicated, but rather a slow surge from minimum to maximum and back to the minimum value. The piezometer at elevation 1865.0 measured a maximum pressure of about 16.6 feet of water and a minimum pressure of 5.4 feet of water. There were no abrupt pressure fluctuations indicated, but the interval between the maximum and a minimum was less than at the higher piezometer. The lower piezometer, at elvation 1855.0, showed a maximum pressure of 29.1 feet of water and a minimum pressure of 8.6 feet of water. Pressure fluctuations were more frequent at this piezometer with a cycle of maximum to minimum pressure occurring every 1 to 2 seconds.

At Station 27+80, near the end of the basin, the maximum water surface was at elevation 1882.0. The dynamic pressure indicated by the piezometer at elevation 1875 indicated a maximum pressure of 10.6 feet of water and a minimum pressure 1.4 feet of water below atmospheric. Generally, the minimum pressure was followed by a surge to the peak pressure, then the pressure dropped off to about 8 feet of water and remained fairly constant for about 6 to 8 seconds when another minimum-maximum surge would occur. The piezometer at elevation 1865.0 measured a maximum pressure of 14.3 feet of water and a minimum pressure of 3.0 feet of water. The variation at this piezometer was similar to that shown at the upper piezometer; a low reading would be followed by a sudden increase, the pressure would then fall to about 10 feet of water and remain fairly constant for several seconds when another fluctuation would occur. The piezometer at elevation 1855.0 registered a maximum pressure of 32.0 feet of water and a minimum pressure of 12.4 feet of water. The pressure fluctuated between maximum and minimum about once a second for several seconds, then smaller fluctuations occurred at the same frequency for several seconds followed by a series of maximum fluctuations.

Pressures on Chute Blocks

Fourteen piezometers were installed in one of the chute blocks in the stilling basin, Figure 17. The piezometers were located in areas that past experience had indicated to be most liable to cavitation damage. Pressure measurements were made for the maximum discharge 34,400 cfs at reservoir elevation 1985.0 and tail water elevation 1890.2 and at reservoir elevation 1969.1 and tail water elevation 1878.4. All of the piezometers registered above-atmospheric pressures for both operating conditions. For the high tail water condition the pressures were 30 feet of water or above at all piezometers. For the low tail water condition, the pressures were equivalent to 8 feet of water or above at all piezometers.

Pressure measurements were also made with the tail water lowered to elevation 1870. With this low tail water, the tow of the jump was downstream from the chute blocks. The piezometers on the downstream face of the chute block indicated subatmospheric pressures equivalent to 4 to 10 feet of water below atmospheric pressure. Piezometers 20 and 21, located about two-thirds of the way around the elliptical surface, Figure 17, and Piezometer 19, located at the top of the chute block near the downstream end, also showed subatmospheric pressures of between 4 and 8 feet of water. Since these subatmospheric pressures were above the cavitation range and occurred only when the hydraulic jump was near the sweep out point, no changes in the curved surfaces of the chute block were necessary.

Discharge Capacity

The discharge capacity of the structure with the flow controlled by the radial gates was obtained as a part of the model studies. The measurements were made with the 3 gates equally opened in 2-foot increments, commencing with the gates raised 1 foot. The calibration procedure was to set carefully the gate opening at the desired increment, increase the discharge through the model until the pressure head in the circular conduits 1 diameter upstream from the transition was equivalent to elevation 1905.0, and measure the quantity of flow. This procedure was repeated for 10-foot increments in in pressure head up to elevation 1985.0. Discharges were then plotted versus pressure heads for each gate opening, and curves were drawn as shown in Figure 26. To use these curves for the determination of prototype release quantities, it will be necessary to install a pressure-measuring piezometer in each conduit of the prototype structure. Piezometers should be installed in the same relative location described for the model, and a gage suitable for use by an operator should be provided. After the relationship between headwater elevation and piezometer pressure head has been established as a result of prototype operation, the ordinate of Figure 26 may be changed to show the relationship of discharge to headwater elevations.

FIGURE 1
REPORT HYD. 463

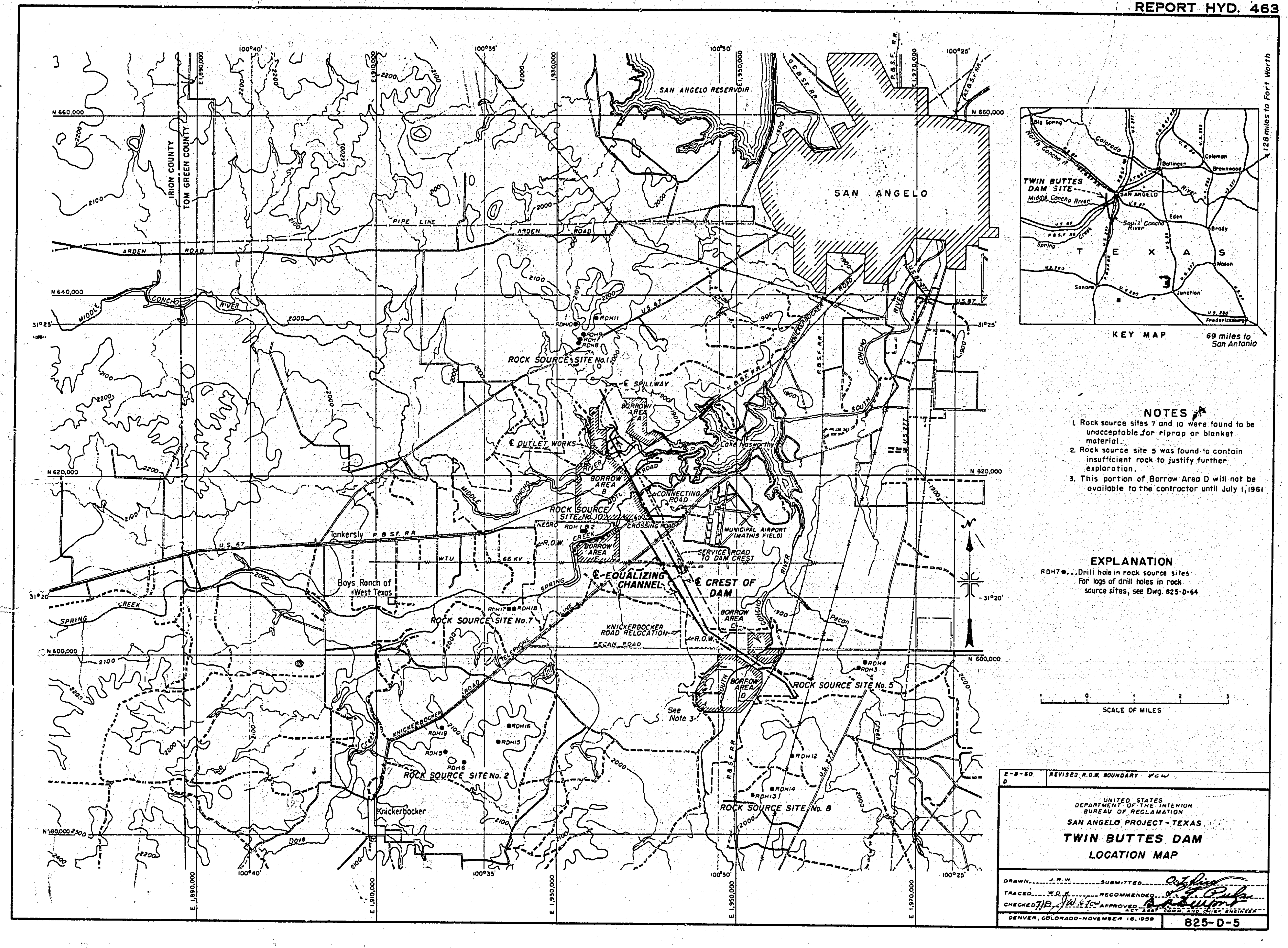


FIGURE 2
REPORT HYD. 463

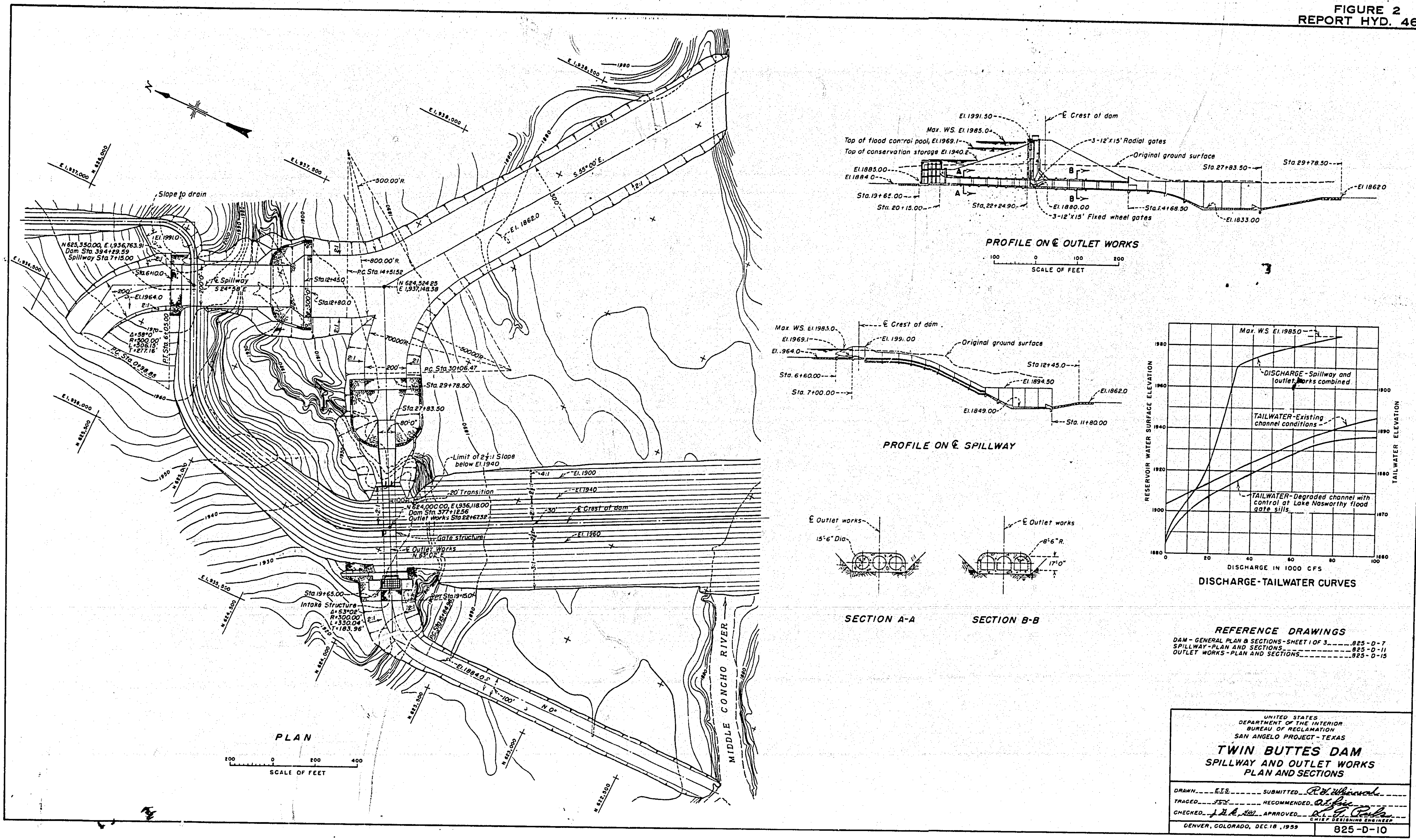
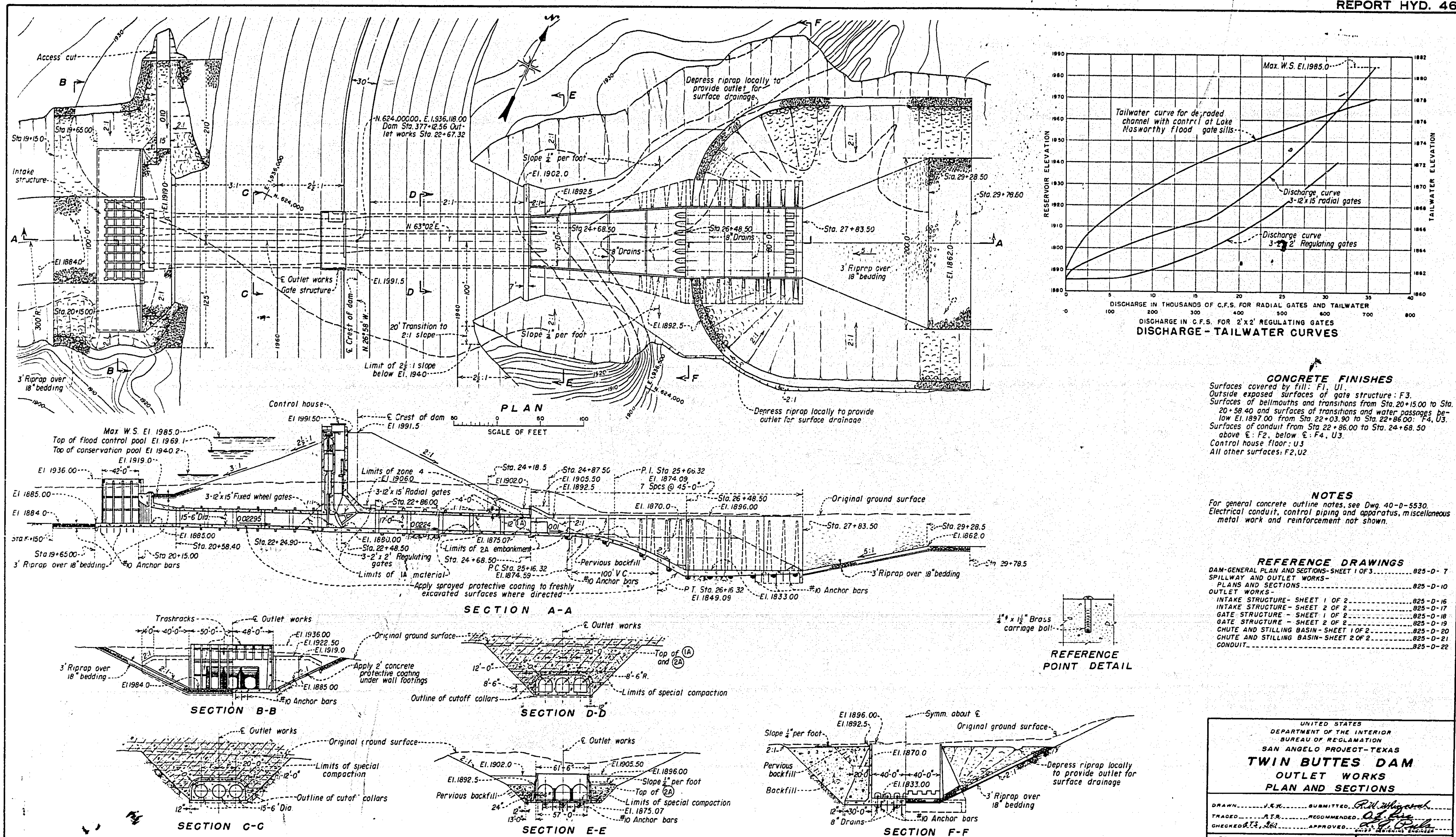
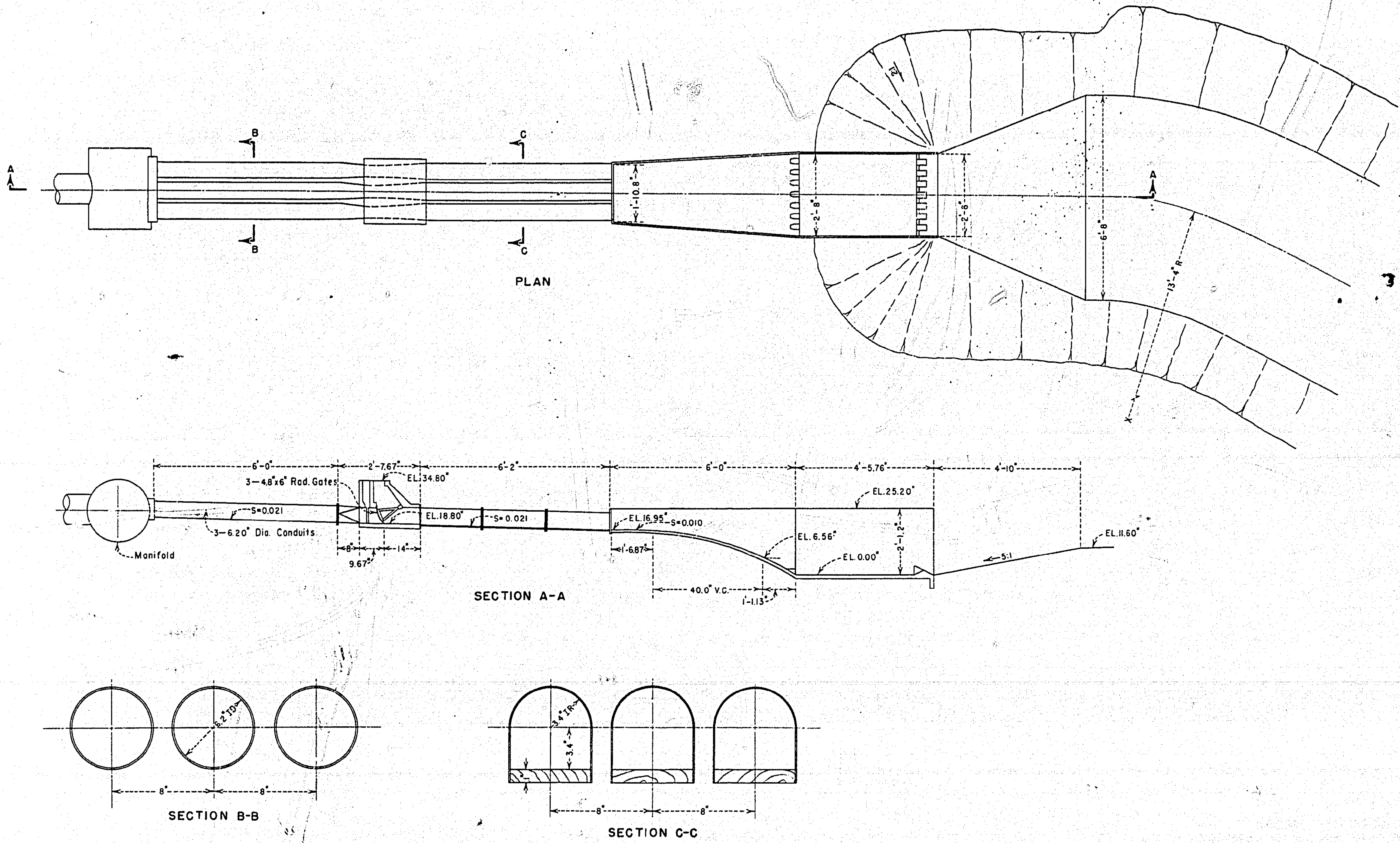
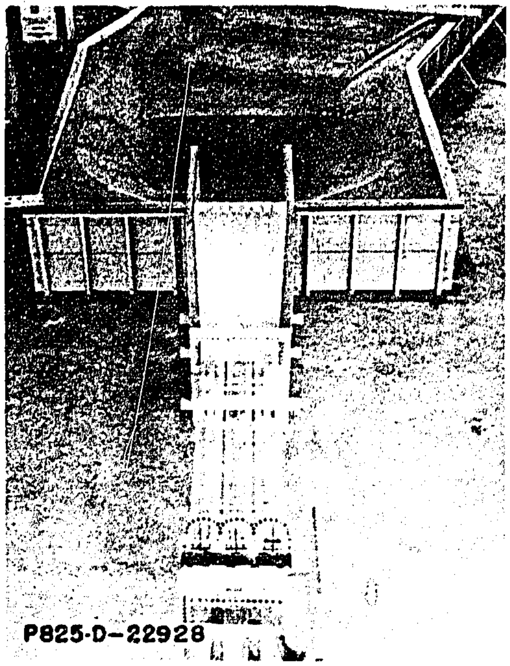


FIGURE 3
REPORT HYD. 463

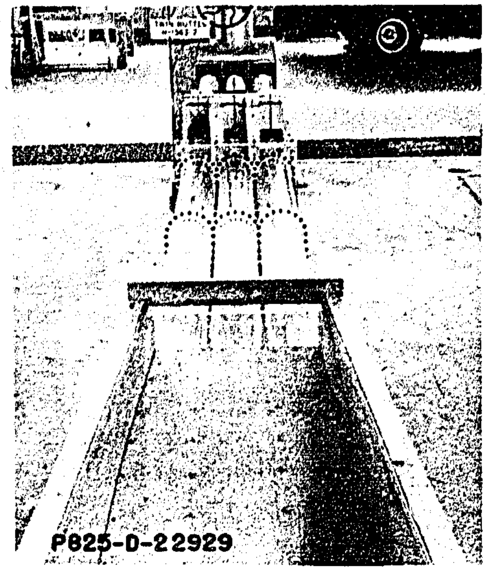




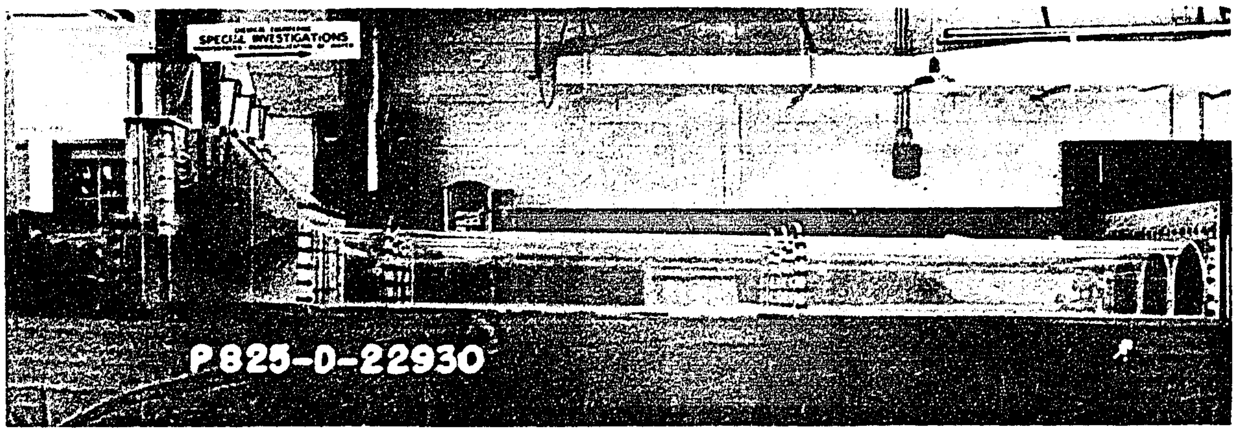
1:30 SCALE HYDRAULIC MODEL STUDIES
TWIN BUTTES DAM OUTLET WORKS
MODEL ARRANGEMENT



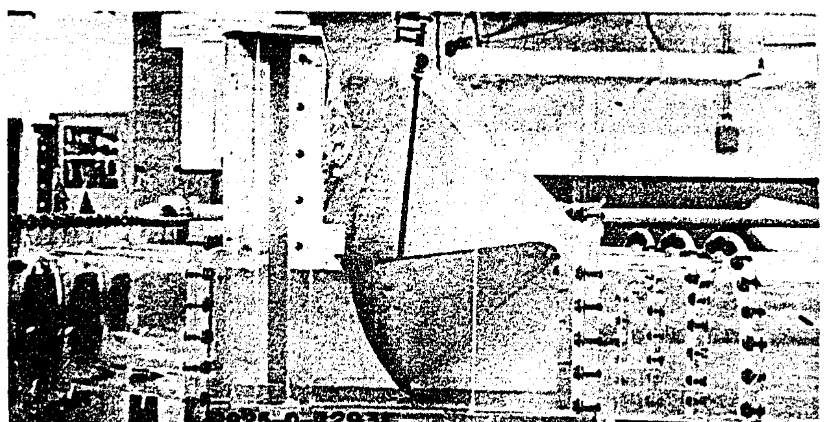
A. Outlet Works Model



B. Exits of downstream tunnels

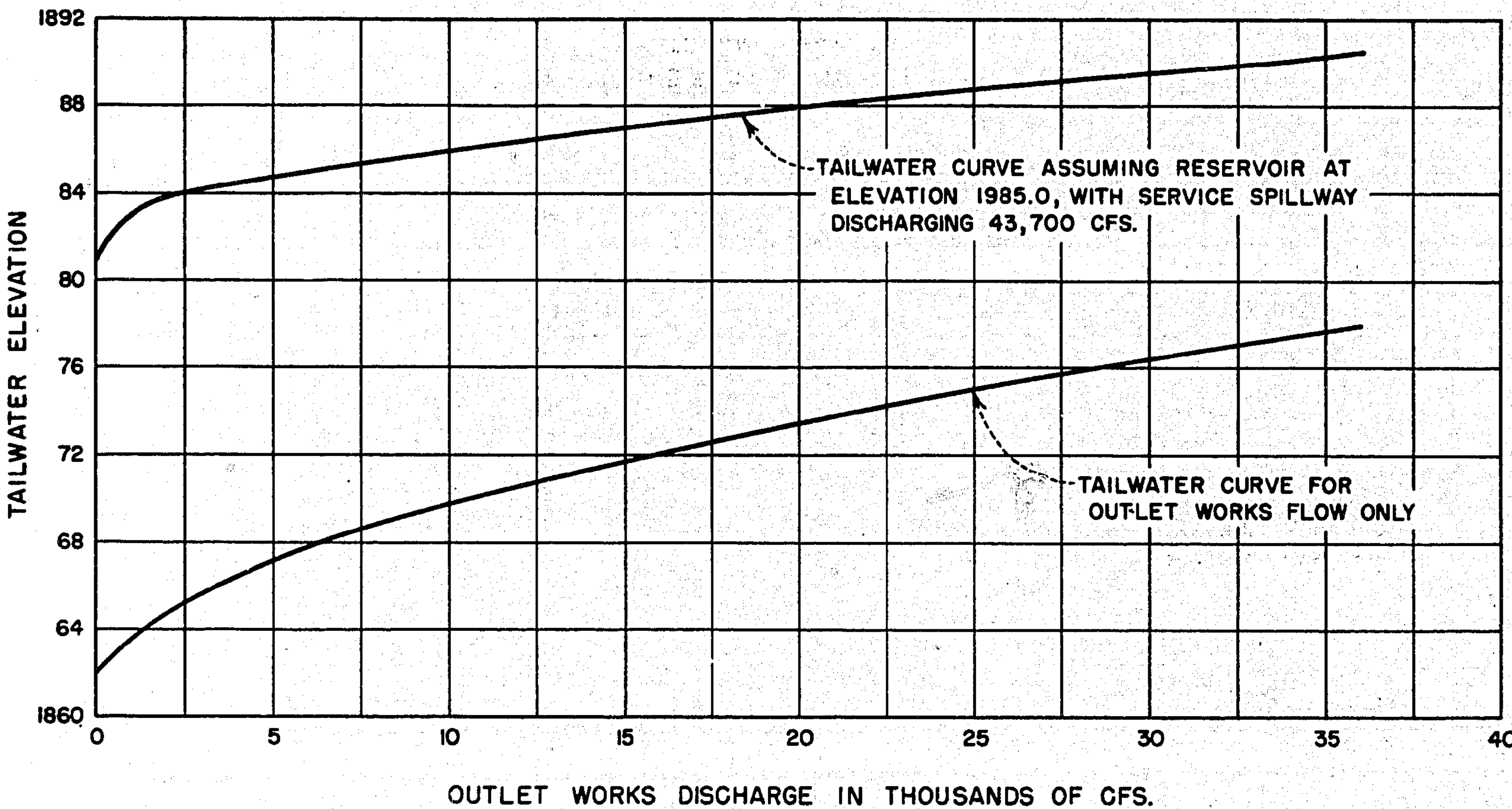


C. Gates and downstream tunnels



D. Control Section

1:30 SCALE HYDRAULIC MODEL STUDIES
TWIN BUTTES DAM OUTLET WORKS
Completed Model



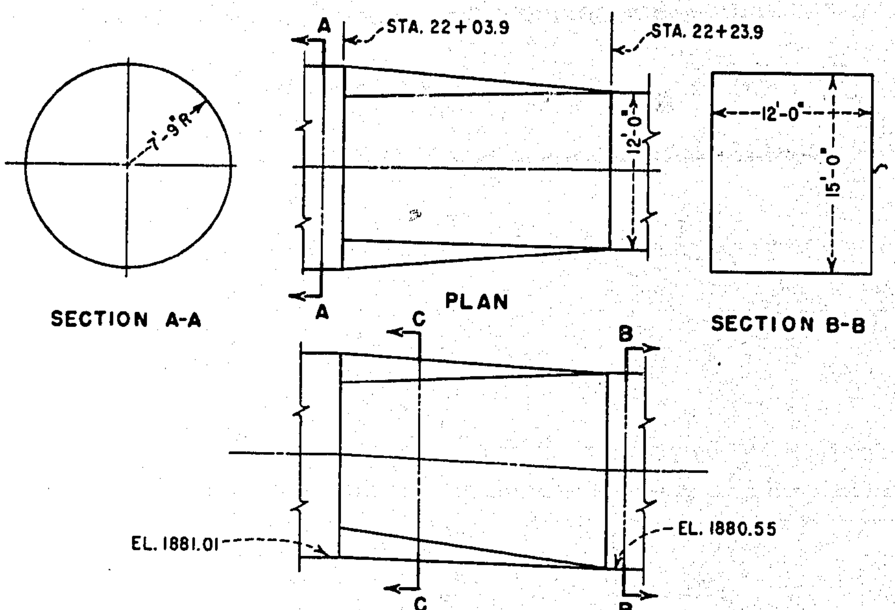
1:30 SCALE HYDRAULIC MODEL STUDIES

TWIN BUTTES DAM OUTLET WORKS

TAILWATER ELEVATION CURVES

618

FIGURE 6
REPORT HYD. 463

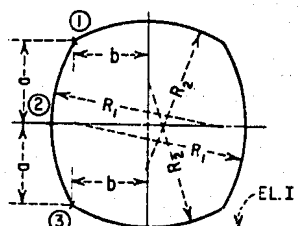


SECTION A-A

PLAN

SECTION B-B

ELEVATION



SECTION C-C
STA. 22+09.9

ELEMENTS OF THE TRANSITION STA. 22+03.9 TO STA. 22+23.9

STA.	EL. I	a	b	Hgt.	Width	R_1	R_2
22+03.9	1881.01	5.48	5.48	15.50	15.50	7.75	7.75
22+09.9	1880.87	6.09	5.64	15.35	14.45	12.49	10.83
22+17.9	1880.65	6.89	5.84	15.15	13.05	34.99	25.24
22+23.9	1880.55	7.50	6.00	15.00	12.00	—	—

① Indicates piezometers.
Piezometers also located at stations 22+04.9, 22+17.19, and 22+22.9, in same relative positions to those shown in section C-C.

PRESSURES ON TRANSITION SIDEWALL IN FEET OF WATER

Q=36,000 CFS-RES. EL. 1985.0		
STATION	PIEZ. #	PRESSURE
22+04.9	1	9.7
	2	19.0
	3	18.0
22+09.9	1	8.0
	2	14.9
	3	19.1
22+17.9	1	5.4
	2	12.1
	3	20.1
22+22.9	1	9.8
	2	1.9
	3	20.5

Q=34,400 CFS-RES. EL. 1969.1		
STATION	PIEZ. #	PRESSURE
22+04.9	1	8.4
	2	17.1
	3	17.2
22+09.9	1	7.0
	2	13.6
	3	18.3
22+17.9	1	4.6
	2	11.3
	3	19.2
22+22.9	1	1.4
	2	9.5
	3	19.6

Q=24,000 CFS-RES. EL. 1940.2		
STATION	PIEZ. #	PRESSURE
22+04.9	1	8.4
	2	16.4
	3	18.9
22+09.9	1	7.9
	2	14.0
	3	19.7
22+17.9	1	6.1
	2	12.8
	3	20.5
22+22.9	1	4.3
	2	12.1
	3	20.9

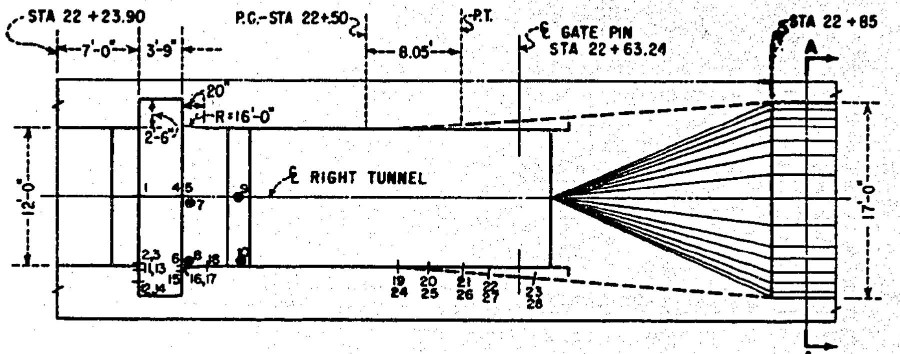
Note: Only right transition shown, other transitions are identical.

1:30 SCALE HYDRAULIC MODEL STUDIES

TWIN BUTTES DAM OUTLET WORKS

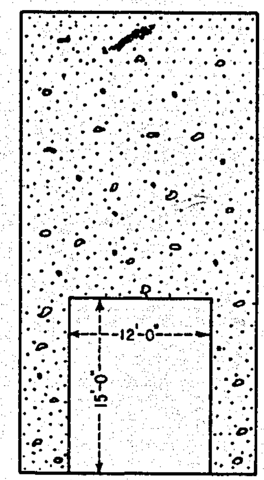
UPSTREAM TRANSITION

618

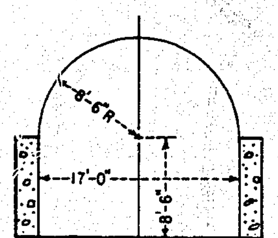


PLAN

Numbers shown piezometers in roof and along right wall.

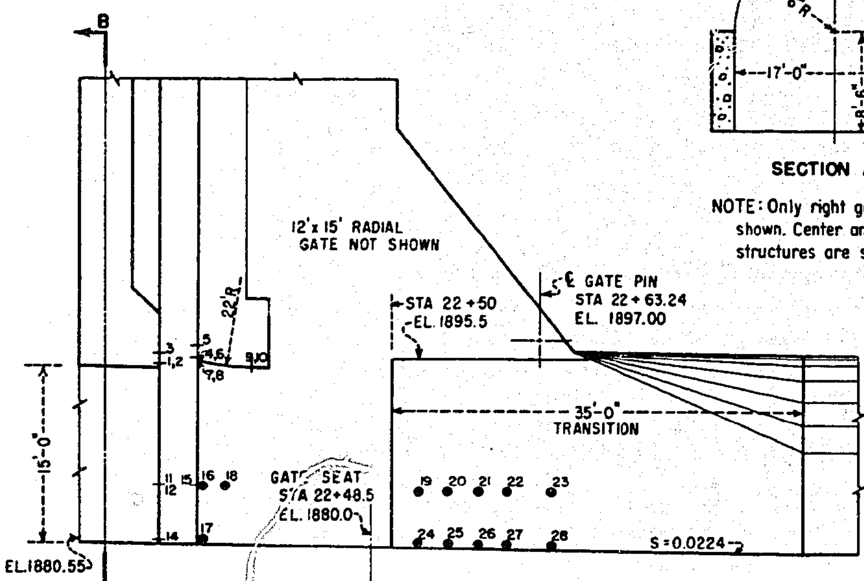


SECTION B-B



SECTION A-A

NOTE: Only right gate structure shown. Center and left structures are similar



SECTION ALONG ℄

PRESSURES IN GATE CHAMBER
IN FEET OF WATER

Piez No.	Q=36,000 cfs Res. El. 1985	Q=34,400 cfs Res. El. 1969.1	Q=25,000 cfs Res. El. 1985
1	1.6	1.3	4.5
2	0.6	0.6	4.0
3	0	-0.4	3.3
4	8.4	7.7	8.8
5	1.2	0.8	3.8
6	24.4	21.8	12.8
7	15.6	13.7	12.5
8	15.0	13.3	12.2
9	0.3	0.2	5.6
10	0.1	0.1	5.5
11	13.3	12.7	15.2
12	12.4	11.8	14.9
13	18.4	17.8	19.7
14	18.4	17.8	19.7
15	25.5	23.2	18.9
16	14.6	13.6	14.3
17	19.8	19.1	19.4
18	2.8	3.6	9.7

PRESSURES IN DOWNSTREAM TRANSITION
IN FEET OF WATER

Piez No.	Q=34,400 cfs Res. El. 1985	Q=25,000 cfs Res. El. 1985	Q=12,000 cfs Res. El. 1985	Q=6,000 cfs Res. El. 1985
19	0.9	—	—	—
20	3.6	2.8	0.2	—
21	3.5	1.7	—	—
22	5.0	2.3	—	—
23	5.9	2.8	—	—
24	7.2	9.2	2.6	0.8
25	5.2	9.0	4.3	2.8
26	9.3	9.2	4.9	3.4
27	8.9	7.7	3.7	1.3
28	9.2	6.7	2.5	-0.6

DISCHARGES ARE FOR 3 TUNNELS OPERATING

1:30 SCALE HYDRAULIC MODEL STUDIES

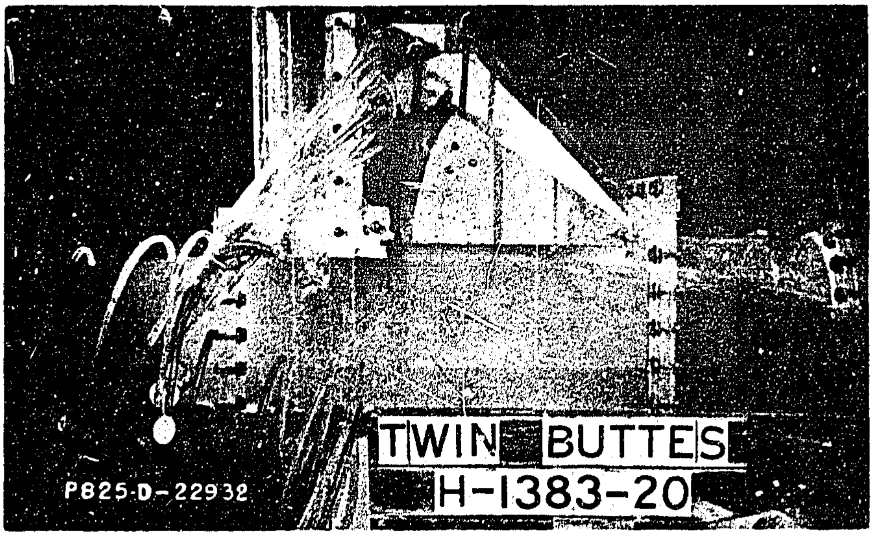
TWIN BUTTES DAM OUTLET WORKS

GATE STRUCTURE AND DOWNSTREAM TRANSITION

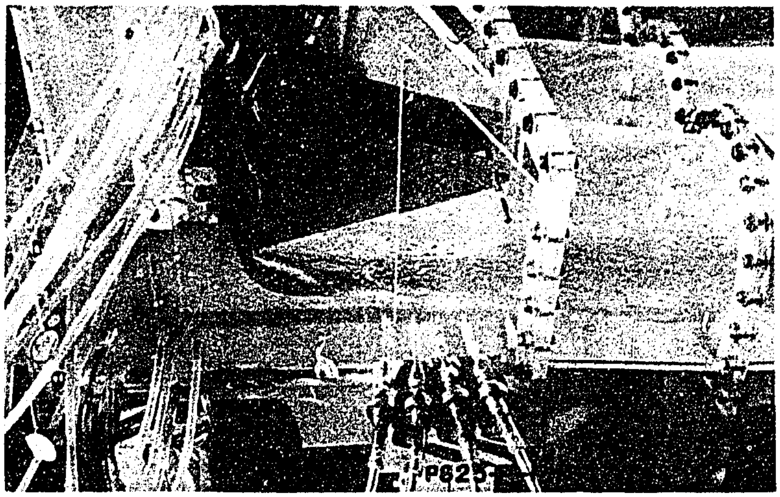
PIEZOMETER LOCATIONS AND PRESSURES

618

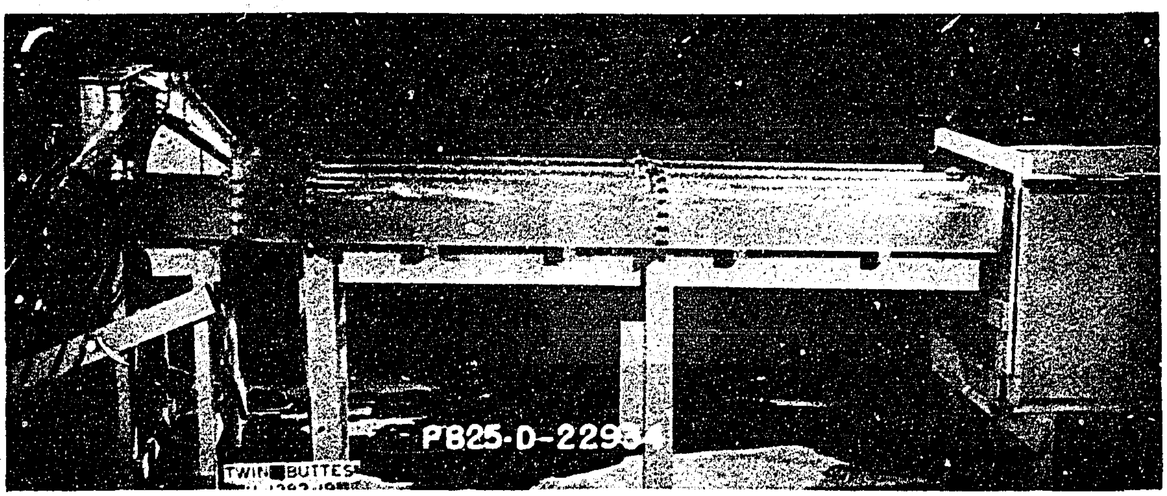
FIGURE 8
REPORT HYD. 463



A. Gate Fully Open - Discharge = 11,470 cfs

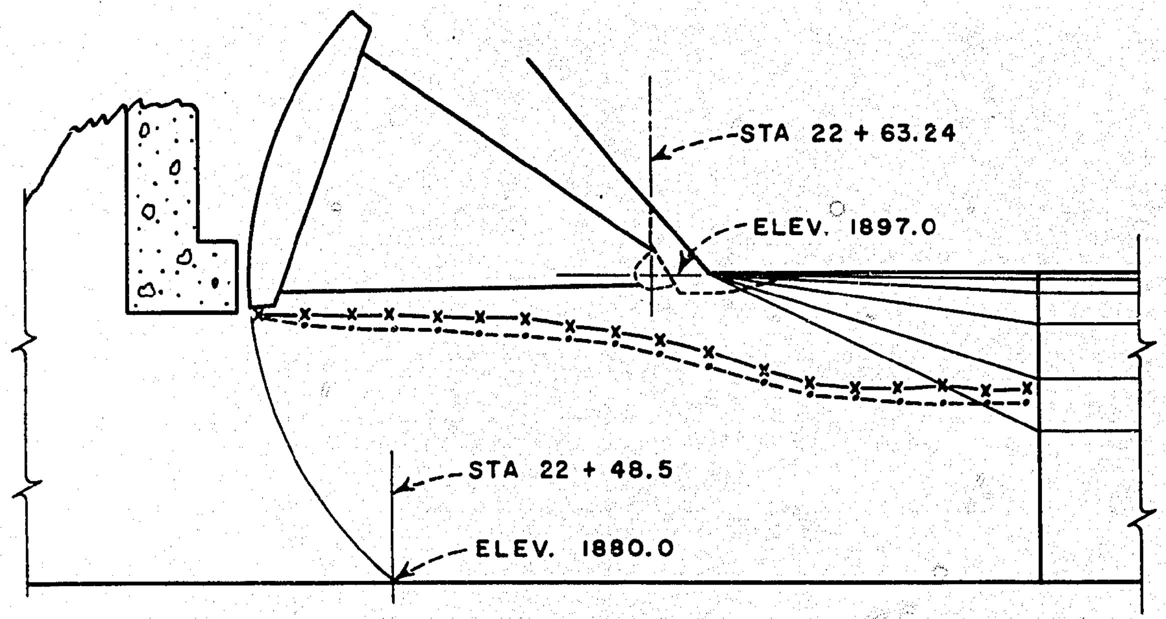


B. Gate partially closed - Discharge = 8,333 cfs
Note pressure cells attached to piezometers in downstream transition

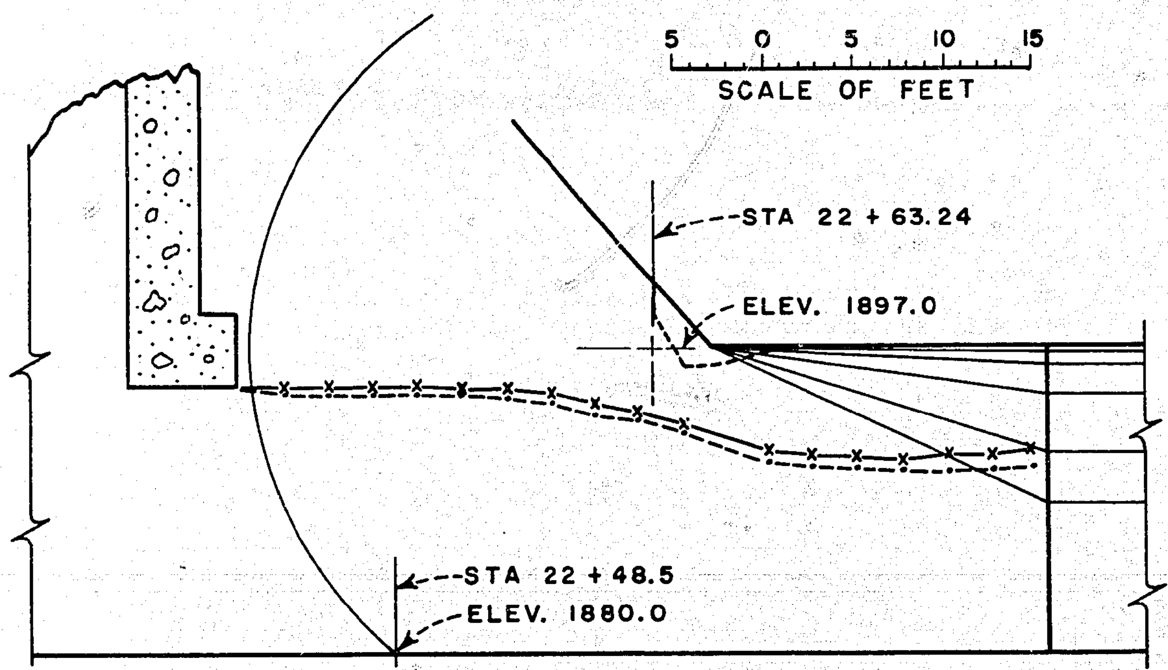


C. Flow in horseshoe conduits - Discharge = 34,400 cfs
Reservoir Elev. 1985.0, Tailwater Elev. 1890.8

1:30 SCALE HYDRAULIC MODEL STUDIES
TWIN BUTTES DAM OUTLET WORKS
FLOW CONDITIONS
PRELIMINARY STILLING BASIN (RECOMMENDED)



RIGHT SIDE OF RIGHT GATE CHAMBER



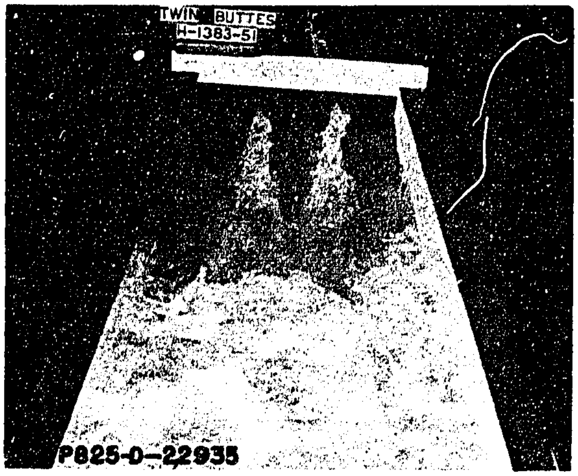
LEFT SIDE OF LEFT GATE CHAMBER

—·—·— Q = 34,400 CFS (3 GATES) RES. ELEV. 1985.0
—x—x Q = 34,400 CFS (3 GATES) RES. ELEV. 1969.1

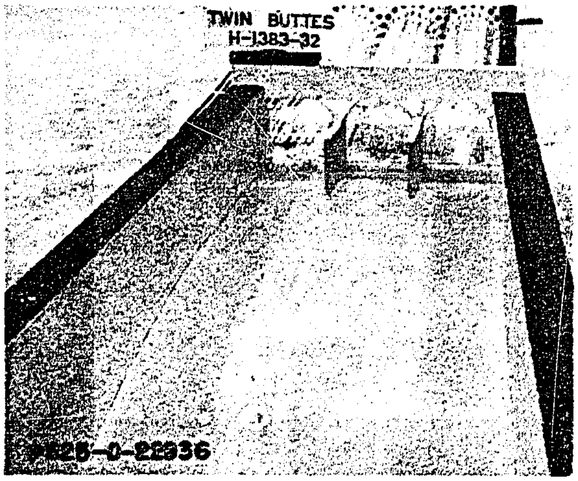
1:30 SCALE HYDRAULIC MODEL STUDIES

TWIN BUTTES DAM OUTLET WORKS

WATER SURFACE PROFILES IN GATE CHAMBERS



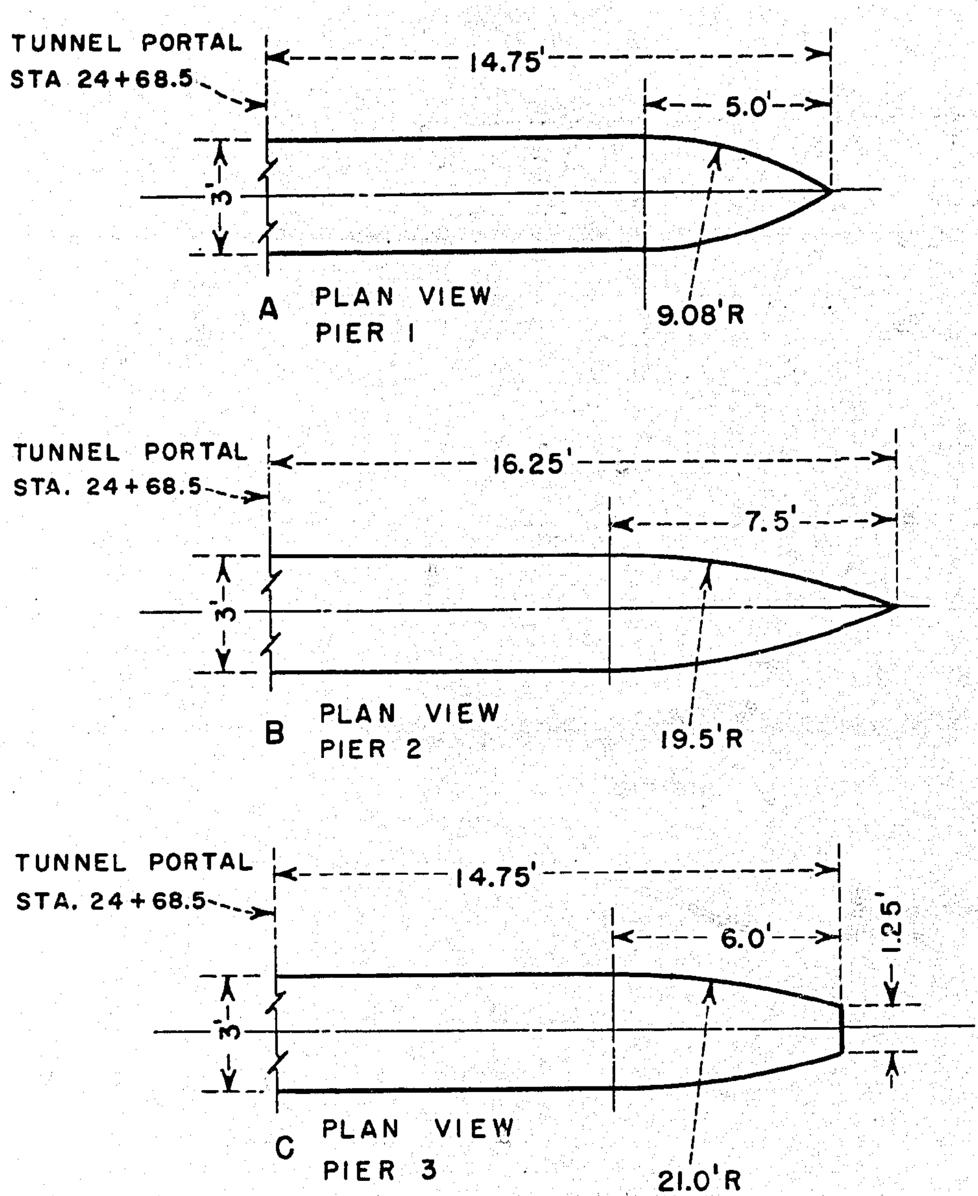
Water fins form at tunnel portals;
no piers used.



Recommended piers installed

Water fins are greatly reduced by the
recommended tunnel portal piers

1:30 SCALE HYDRAULIC MODEL STUDIES
TWIN BUTTES DAM OUTLET WORKS
Flow Conditions at Tunnel Portals
Discharge 34,400 second-feet

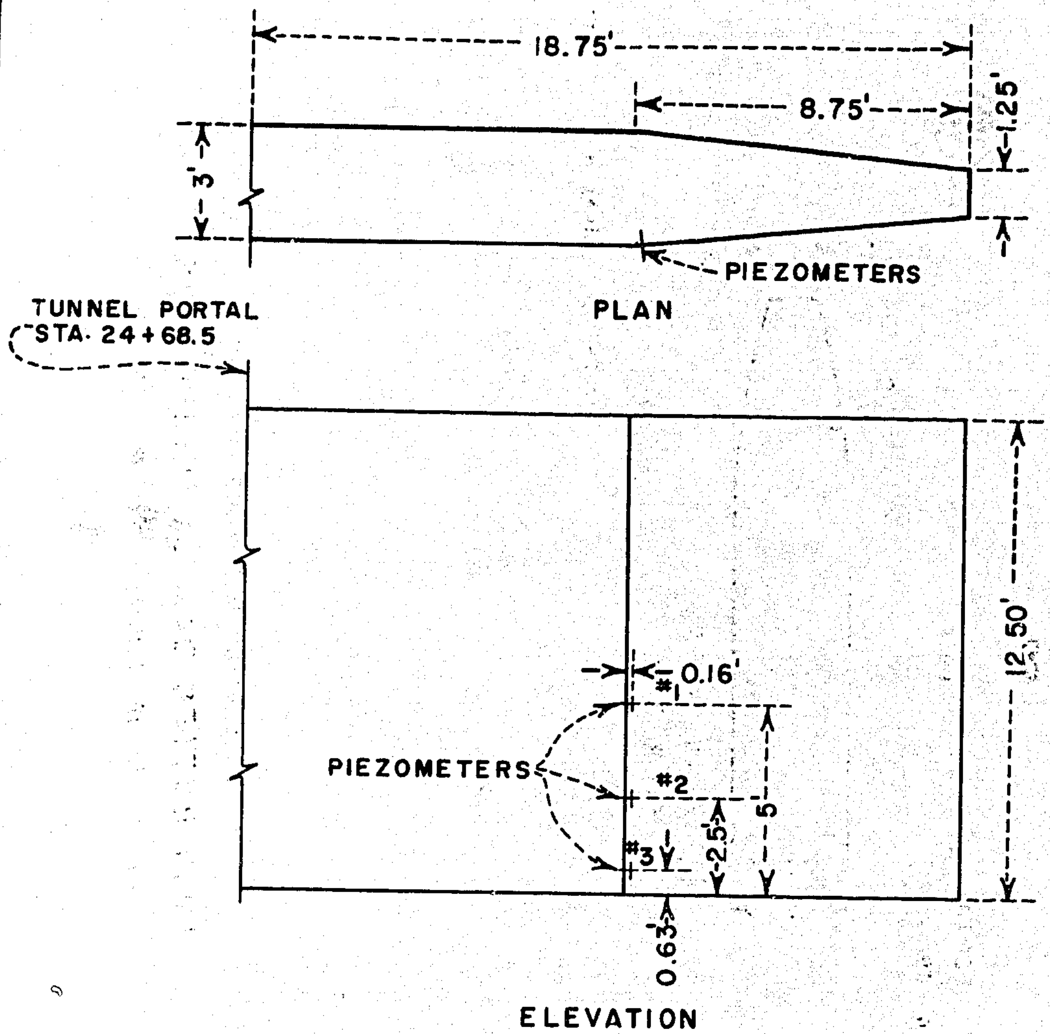


NOTE: All piers are 12' high.

1:30 SCALE HYDRAULIC MODEL STUDIES

TWIN BUTTES DAM OUTLET WORKS

DETAILS OF TUNNEL PORTAL PIERS

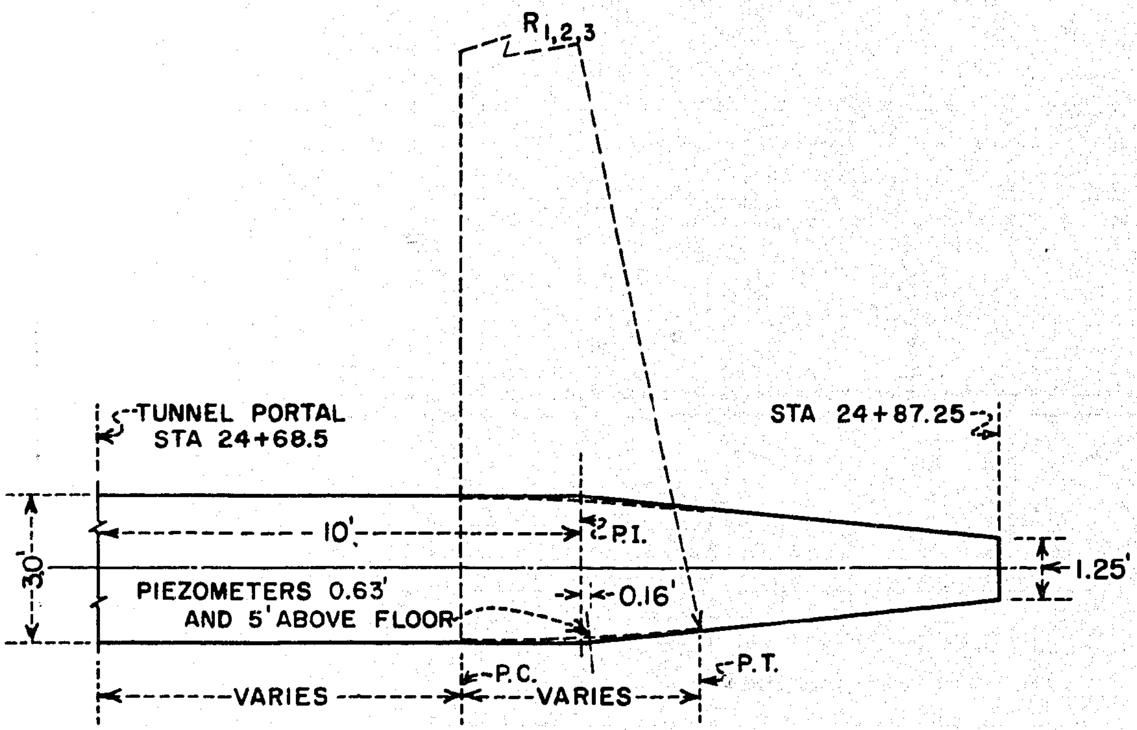


PRESSURE IN FEET OF WATER			
PIEZ	1	2	3
Q= 34,400 CFS			
RES. EL. 1985.0	-6.90	-5.40	-2.49
1969.1	-5.10	-3.00	-0.09
Q= 25,000 CFS			
RES. EL. 1985.0	-8.55	-8.34	-5.34
1969.1	-6.54	-5.40	-2.49
1940.2	-2.04	0	+2.67
Q= 12,000 CFS			
RES. EL. 1985.0	—	-6.21	-5.49
1969.1	—	-5.67	-4.65
1940.2	—	-3.15	-1.65

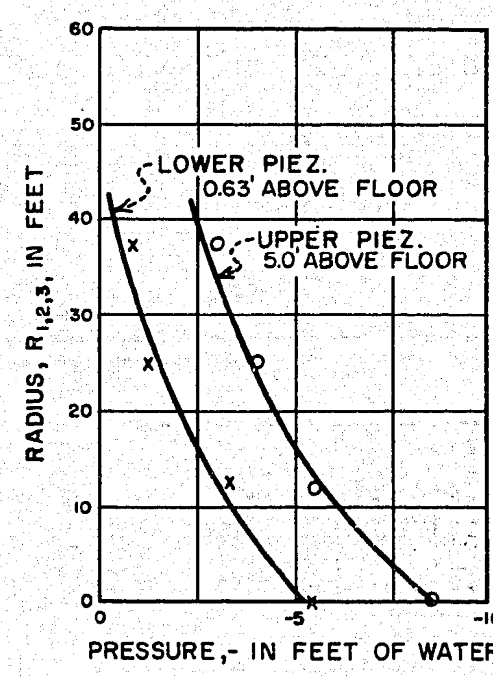
1:30 SCALE HYDRAULIC MODEL STUDIES

TWIN BUTTES DAM OUTLET WORKS

TUNNEL PORTAL PIER NO. 4.
PIEZOMETER LOCATIONS AND PRESSURES



PLAN



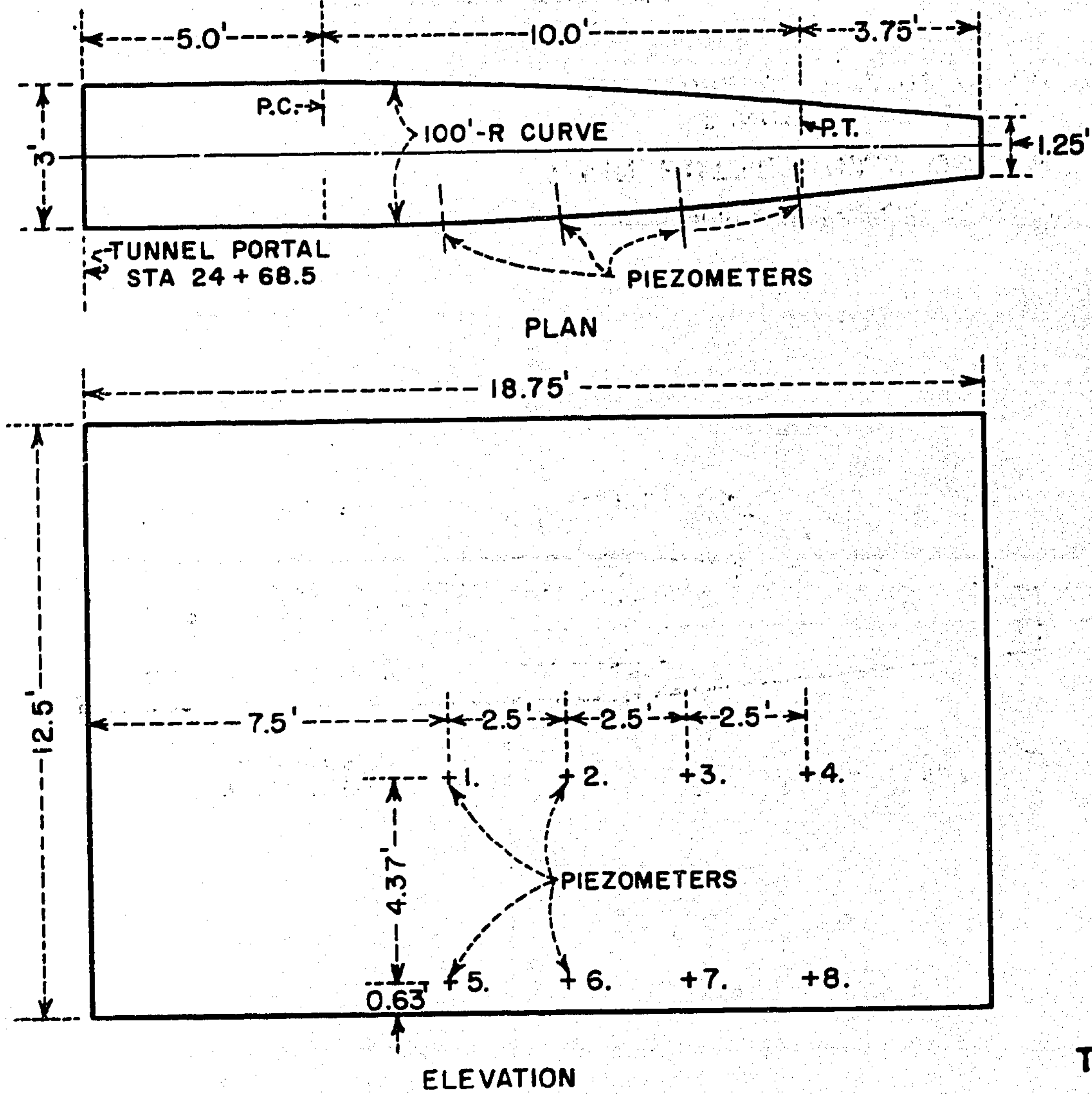
NOTE: Pressures measured at Q=25,000 cfs, Reservoir elevation 1985.0

1:30 SCALE HYDRAULIC MODEL STUDIES

TWIN BUTTES DAM OUTLET WORKS

TUNNEL PORTAL PIERS
PRESSURE VS. RADIUS OF CURVATURE

FIGURE 14
REPORT HYD. 463



PRESSURE IN FEET OF WATER

PIEZ NO.	RES. EL. 1985.0			RES. EL. 1969.1		
	Q 34,400	Q 25,000	Q 12,000	Q 34,400	Q 25,000	Q 12,000
1	-1.50	-2.46	–	+0.24	-1.05	–
2	-2.04	-2.76	–	-0.36	-1.32	–
3	-0.15	-0.75	–	+0.90	+0.09	–
4	+3.00	+1.26	–	-3.66	+1.71	–
5	+4.86	+3.18	+1.98	+0.33	+4.53	+2.43
6	+2.97	+1.32	+0.24	+4.56	+2.97	+0.91
7	+3.72	+2.01	+0.72	+5.23	+3.42	+1.32
8	-6.63	+4.83	+2.88	+7.56	+5.58	+3.18

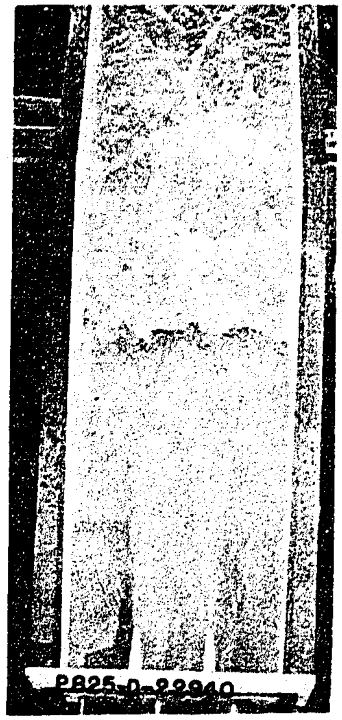
1:30 SCALE HYDRAULIC MODEL STUDIES

TWIN BUTTES DAM OUTLET WORKS

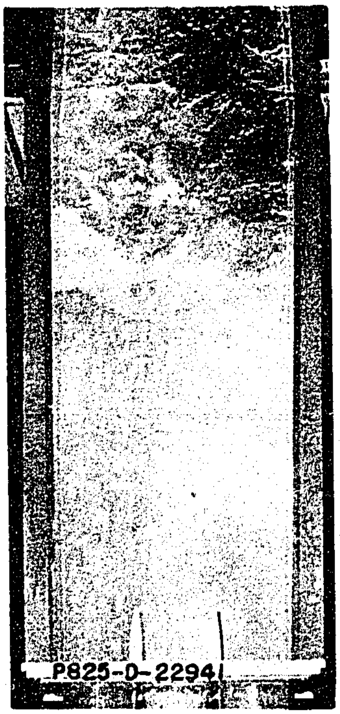
RECOMMENDED TUNNEL PORTAL PIERS
PIEZOMETER LOCATIONS AND PRESSURES

Center & Left Tunnel gates open

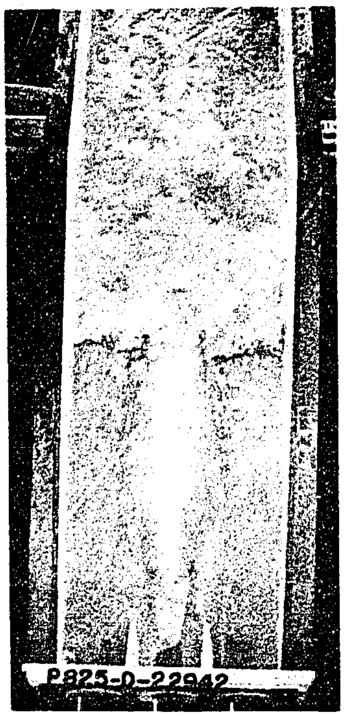
Center tunnel, gate open



Center & Right Tunnel gates open



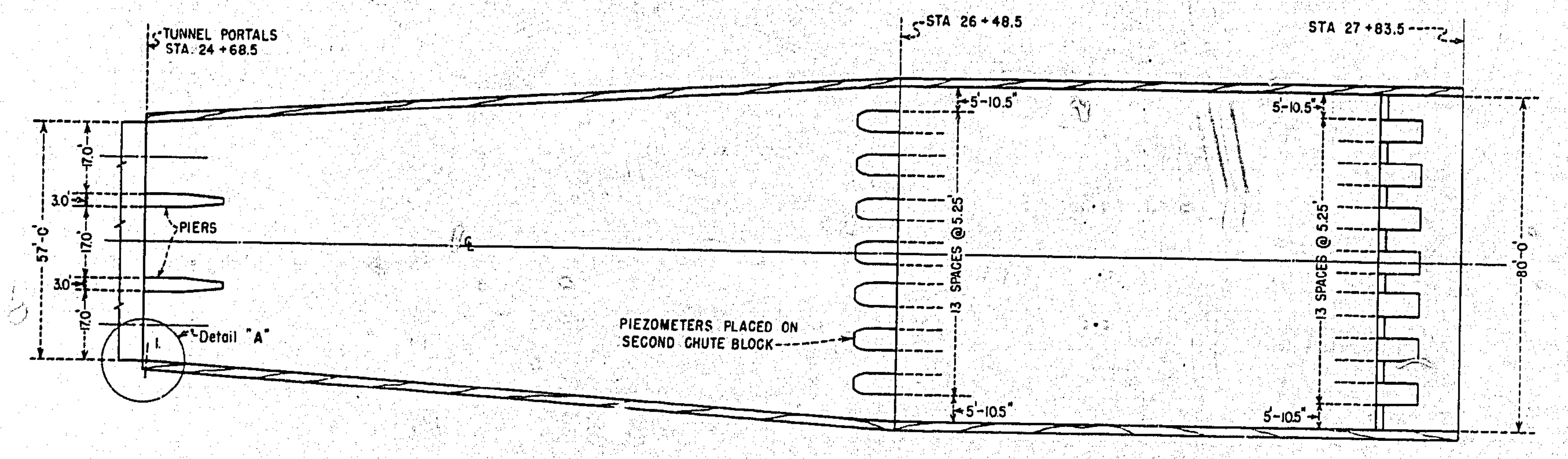
Left Tunnel gate open



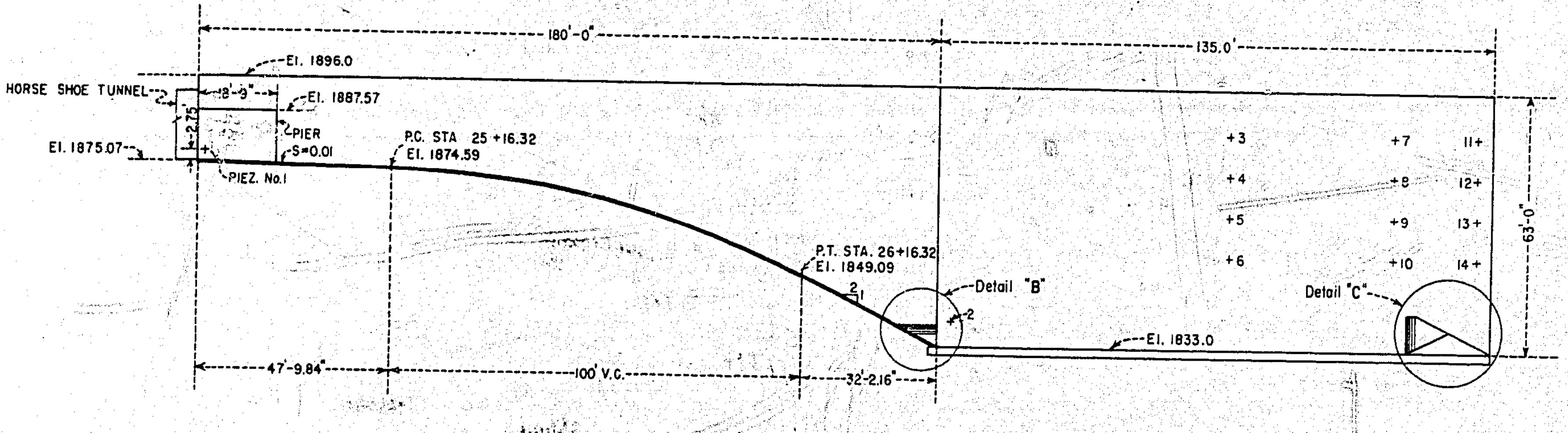
Outside Tunnel gates open

Right Tunnel gate open

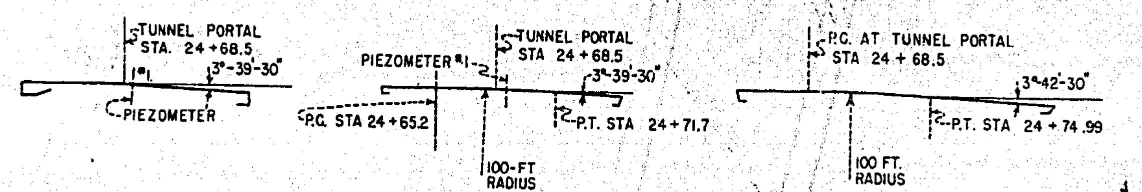
1:30 SCALE HYDRAULIC MODEL STUDIES
TWIN BUTTES DAM OUTLET WORKS
Unsymmetrical Gate Operation, Recommended Design



PLAN



SECTION ALONG ℄



PRELIMINARY INTERSECTION

MODEL INTERPRETATION

PROTOTYPE INTERPRETATION

RECOMMENDED INTERSECTION

DETAIL A

NOTES
Numbers 1-14 indicate piezometers located in right wall of stilling basin, numbers 15-28 indicate piezometers on chute block

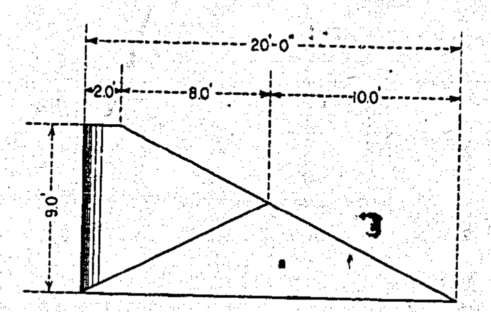
PRESSURE ON CHUTE BLOCK
Q=34,400 CFS.

PIEZ. NO.	T.W. ELEV. 1890.0	T.W. ELEV. 1877.6	T.W. ELEV. 1870.0*
15	+44.45	+35.75	+31.40
16	40.65	24.15	15.55
17	38.25	21.75	12.47
18	36.45	16.50	3.90
19	35.80	12.10	-5.53
20	34.70	11.40	-5.90
21	36.65	11.23	-7.44
22	39.40	16.60	-0.06
23	40.05	17.05	+0.30
24	50.70	34.05	+21.60
25	37.20	13.05	-4.58
26	37.45	12.30	-6.06
27	38.90	12.30	-6.92
28	38.95	12.00	-8.76

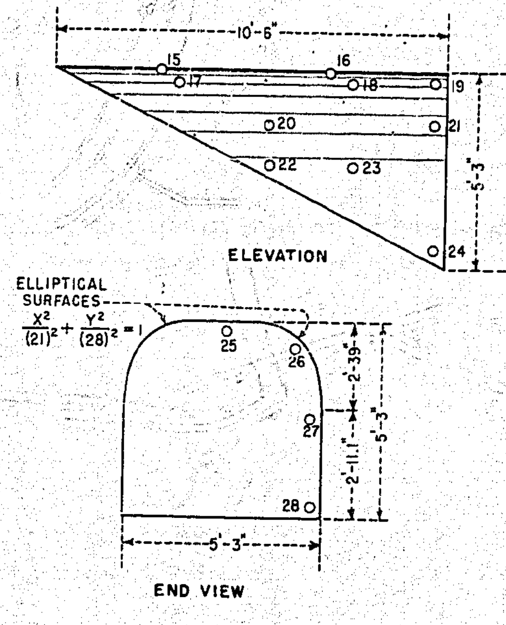
* At this T.W. elevation toe of jump was downstream from blocks.

PRESSURES ON STILLING BASIN SIDEWALL

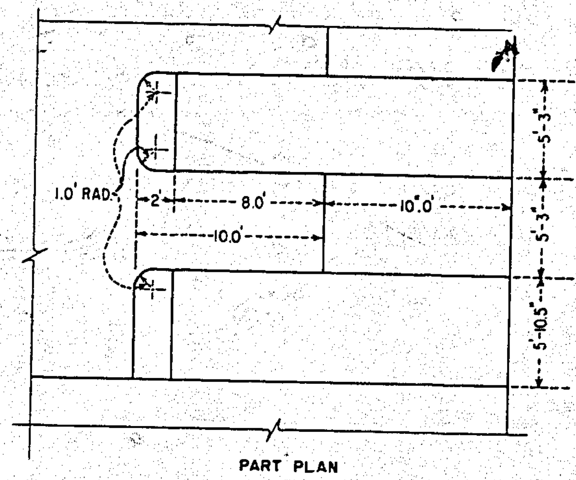
PIEZ NO.		1	2	3	4	5	6	7	8	9	10	11	12	13	14
Q=34,400 RES. El. 1985.0	MAX.	+1.5	+63.0	0	+16.4	+24.7	+34.6	+6.6	+15.0	+30.2	+40.1	+9.0	+16.7	+27.0	+42.1
	MIN.	-2.3	+27.0	0	+2.9	+12.0	+18.2	0	+3.0	+12.9	+17.6	0	+4.7	+13.6	+18.8
Q=34,400 RES. El. 1969.1	MAX.	+3.0	+63.0	0	+3.7	+13.4	+21.4	0	+6.0	+16.6	+29.1	0	+10.6	+14.3	+32.0
	MIN.	-1.0	-9.0	0	0	0	-3.2	0	0	+5.4	+8.6	0	-1.4	+3.0	+12.4
Q=24,000 RES. El. 1985.0	MAX.	+0.8	+45.0												
	MIN.	-2.5	0												
Q=24,000 RES. El. 1969.1	MAX.	+4.5	-47.5												
	MIN.	+3.0	+15.0												



ELEVATION

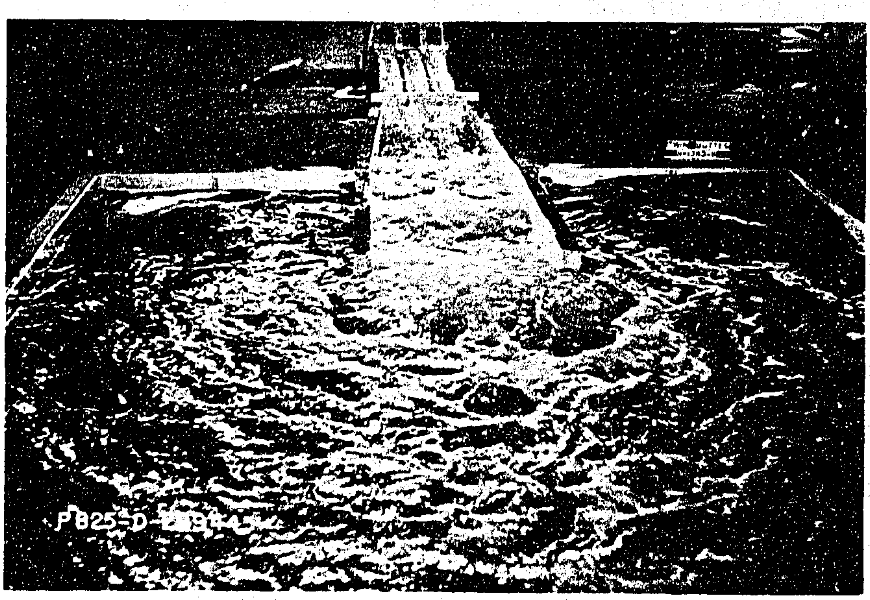


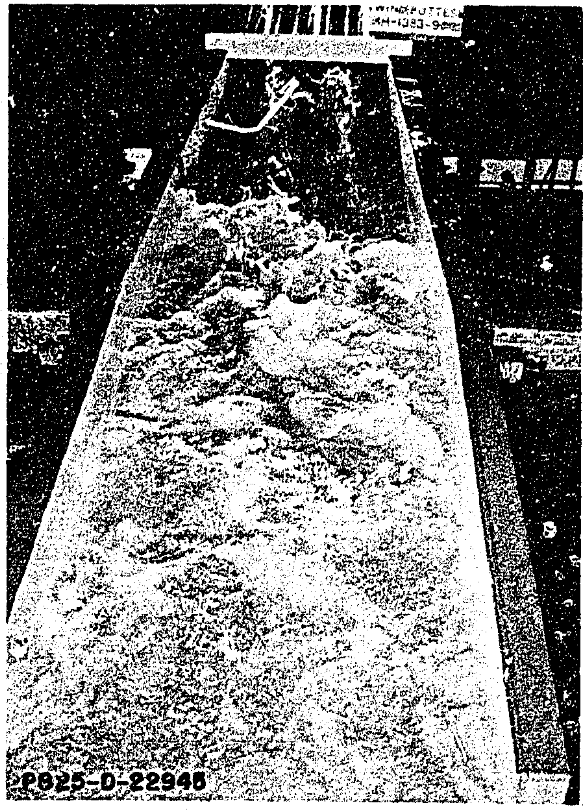
DETAIL B
CHUTE BLOCK



DETAIL C
END SILL

1:30 SCALE HYDRAULIC MODEL STUDIES
TWIN BUTTES DAM OUTLET WORKS
DETAILS-PRELIMINARY STILLING BASIN-(RECOMMENDED)

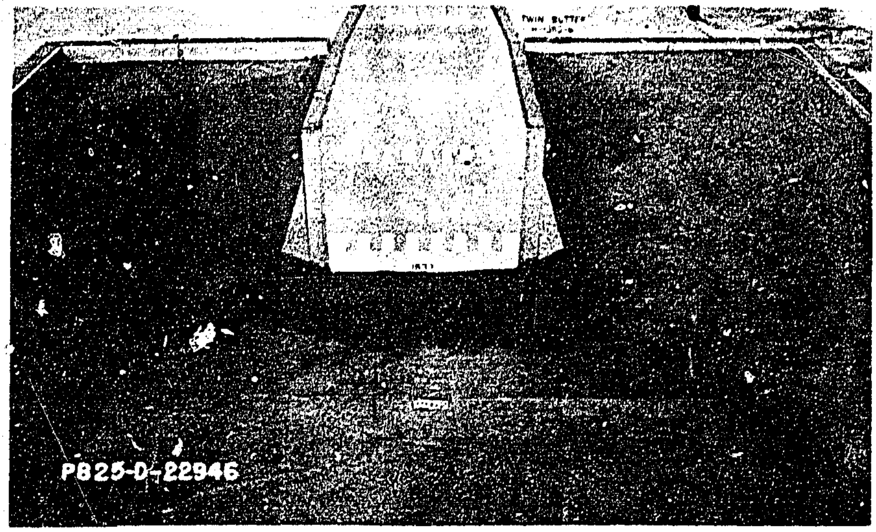




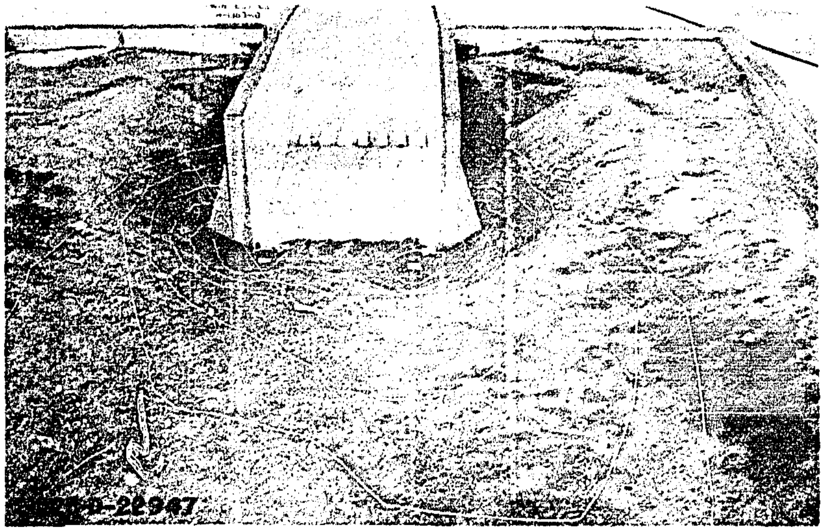
1:30 SCALE HYDRAULIC MODEL STUDIES
TWIN BUTTES DAM OUTLET WORKS
FLOW CONDITIONS
PRELIMINARY STILLING BASIN (RECOMMENDED)

DISCHARGE = 34,400 cfs
Res. Elev. = 1985.0
T.W. Elev. = 1890.8

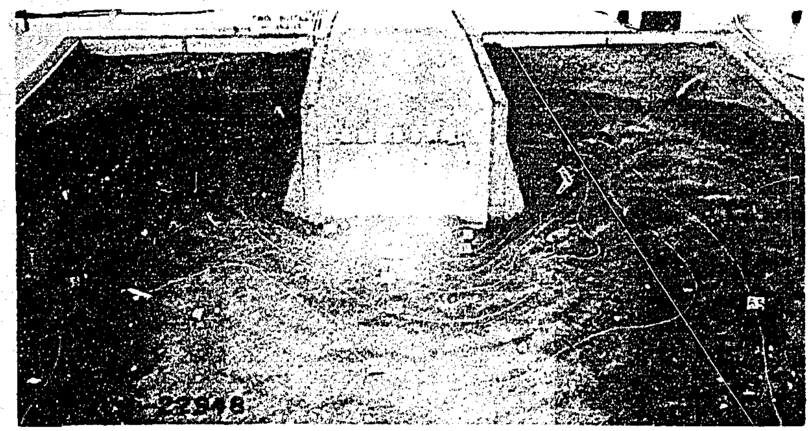
Figure 19



A. River Channel before operation

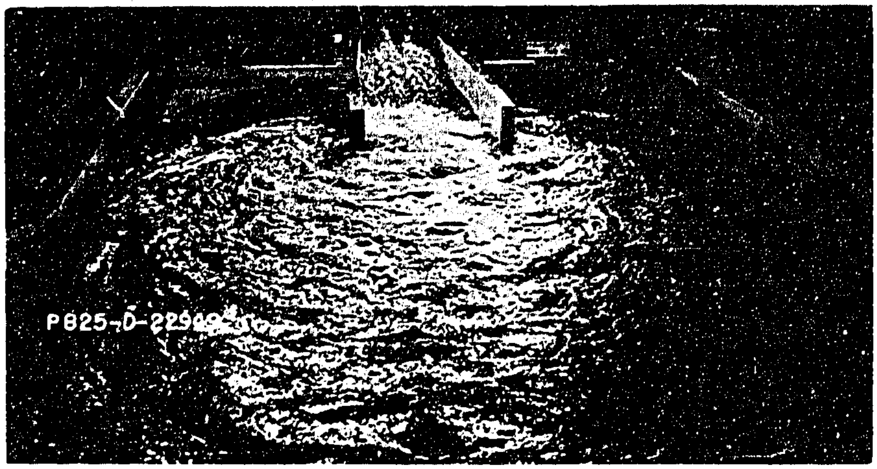


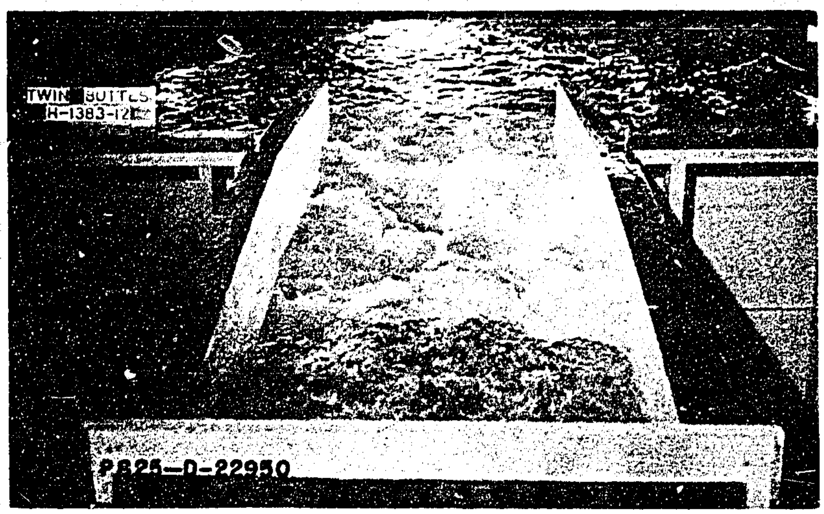
B. Scour after 45 minutes of operation - Q = 34,400 cfs
Reservoir Elevation 1985.0, Tailwater elev. 1890.8



C. Scour after 45 minutes of operation - Q = 34,400 cfs
Reservoir Elevation 1969.1, Tailwater Elev. 1878.2

1:30 SCALE HYDRAULIC MODEL STUDIES
TWIN BUTTES DAM OUTLET WORKS
SCOUR IN DOWNSTREAM CHANNEL
PRELIMINARY STILLING BASIN (RECOMMENDED)

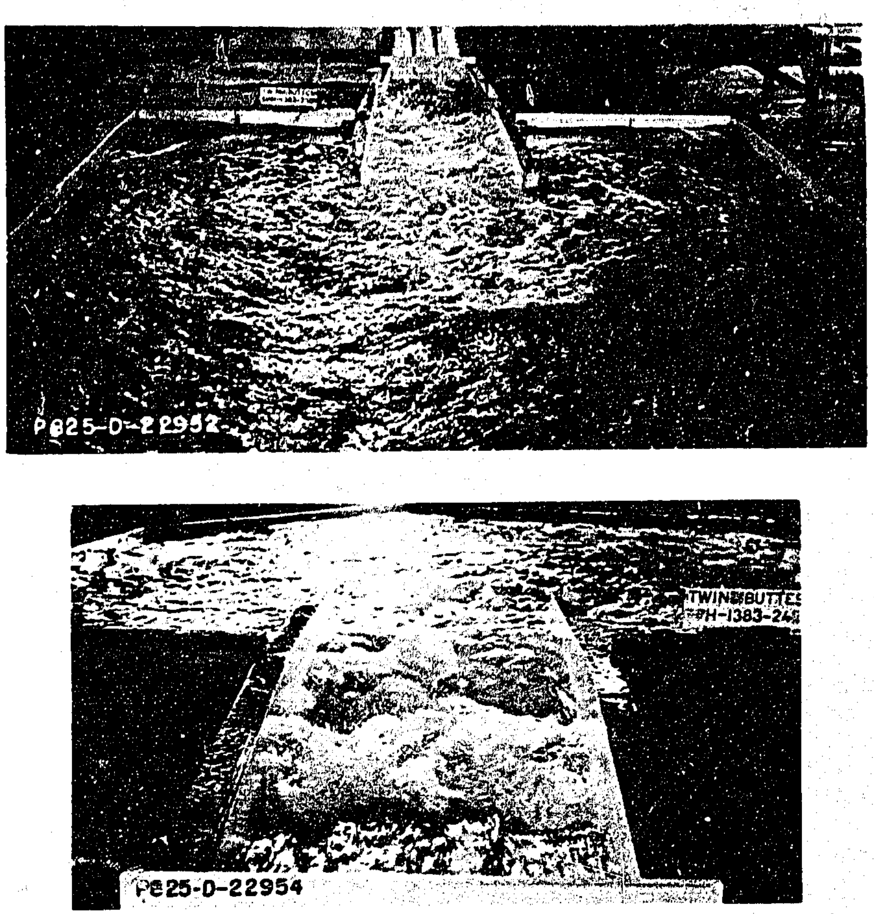




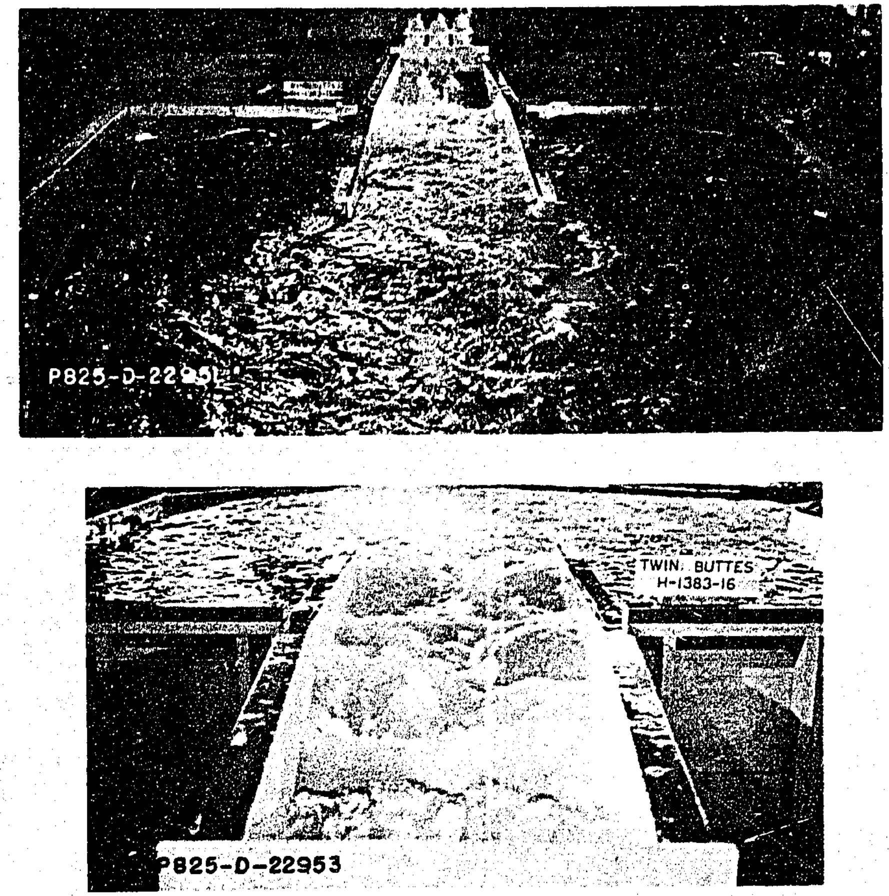
1:30 SCALE HYDRAULIC MODEL STUDIES
TWIN BUTTES DAM OUTLET WORKS
FLOW CONDITIONS
PRELIMINARY STILLING BASIN (RECOMMENDED)

Discharge = 34,400 cfs
Reservoir Elev. 1969.1
Tailwater Elev. 1878.4

Figure 21



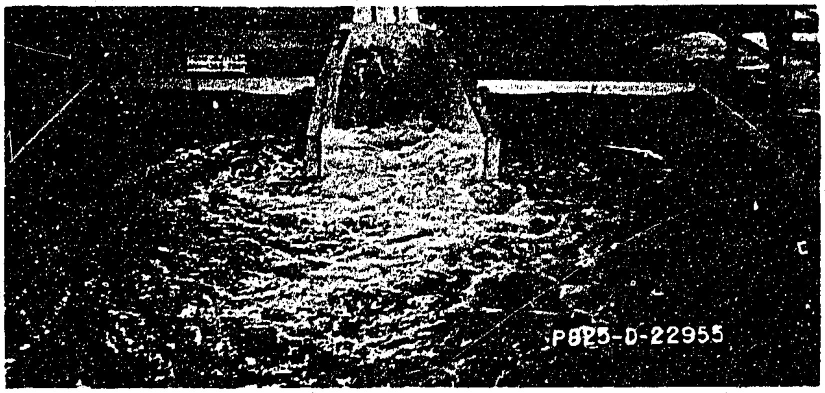
A. Preliminary Basin Without Blocks

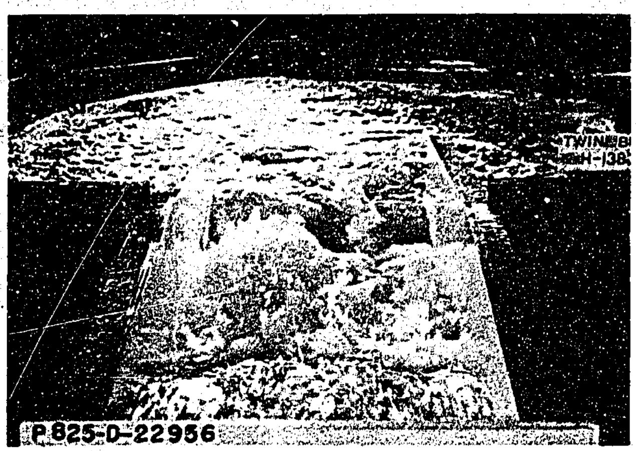


B. Basin With 50-foot Extension

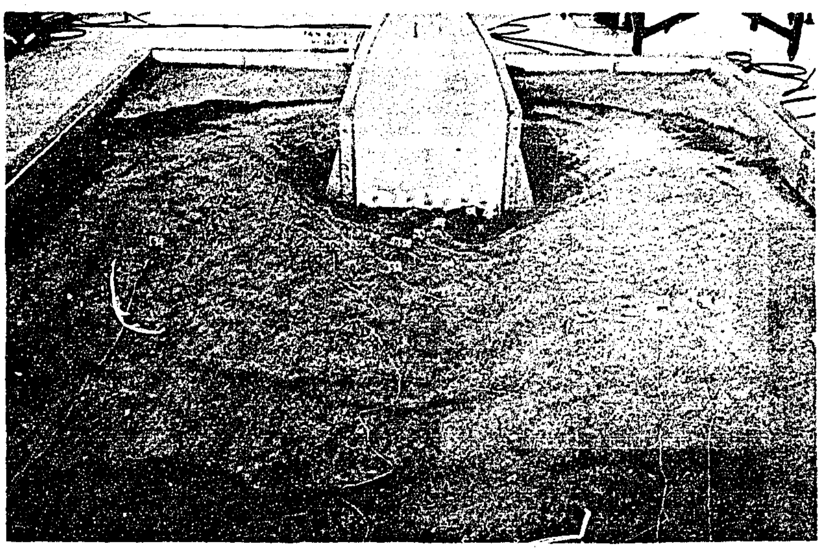
1:30 SCALE HYDRAULIC MODEL STUDIES
TWIN BUTTES DAM OUTLET WORKS
FLOW CONDITIONS FOR
STILLING BASIN MODIFICATIONS

Q = 34,400 cfs
Reservoir Elev. 1985.0
Tailwater Elev. 1890.8





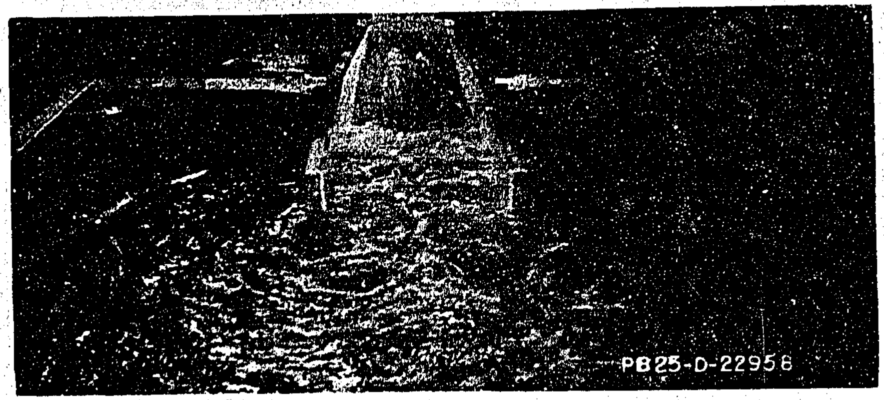
Discharge = 34,400 cfs
Reservoir Elev. 1969.1
Tailwater Elev. 1878.4

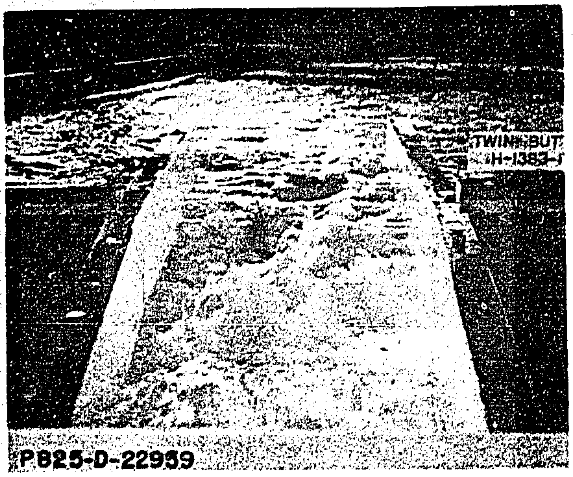


Scour after 45 minutes of operation

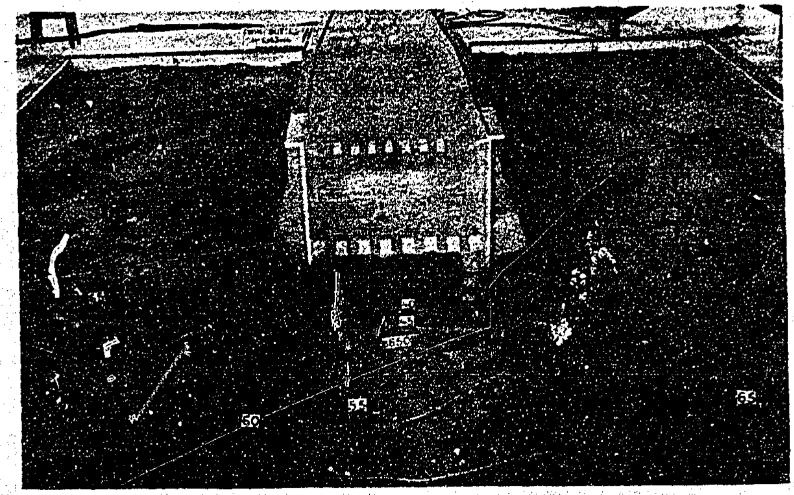
1:30 SCALE HYDRAULIC MODEL STUDIES
TWIN BUTTES DAM OUTLET WORKS
FLOW CONDITIONS AND SCOUR
PRELIMINARY STILLING BASIN
WITHOUT CHUTE BLOCKS

Figure 23



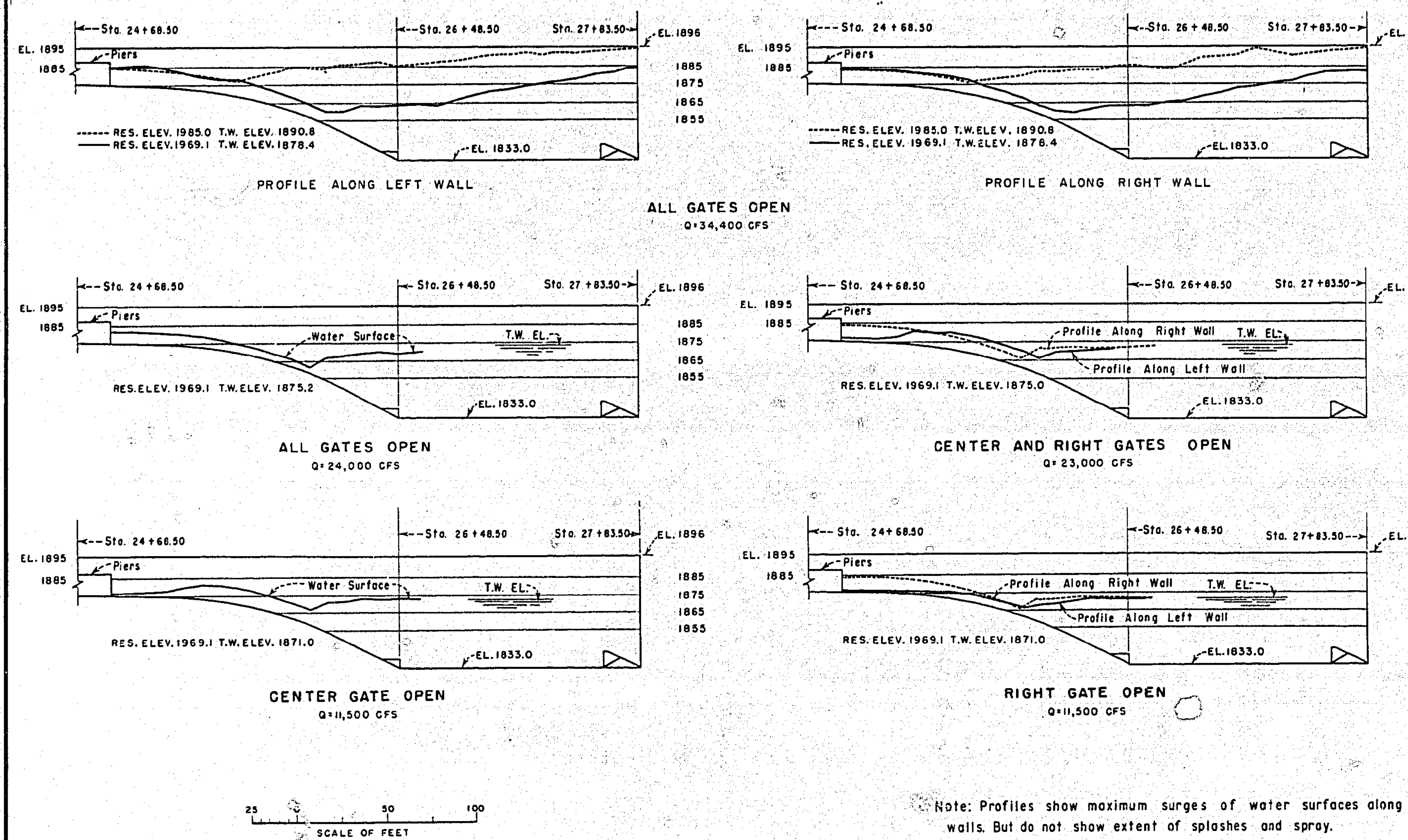


Discharge = 34,400 cfs
Reservoir Elev. 1969.1
Tailwater Elev. 1878.4



Scour after 45 minutes operation

1:30 SCALE HYDRAULIC MODEL STUDIES
TWIN BUTTES DAM OUTLET WORKS
FLOW CONDITIONS AND SCOUR
PRELIMINARY STILLING BASIN
EXTENDED 50-FEET

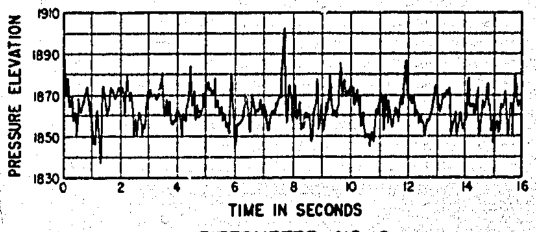


1:30 SCALE HYDRAULIC MODEL STUDIES

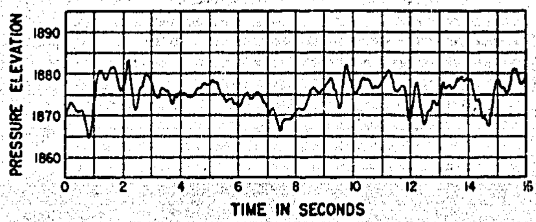
TWIN BUTTES DAM OUTLET WORKS

WATER SURFACE PROFILES IN RECOMMENDED STILLING BASIN

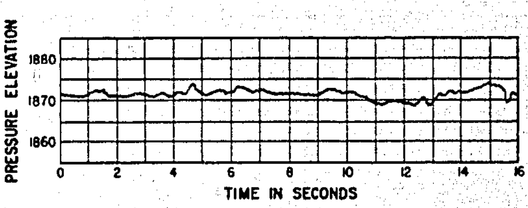
FIGURE 24
REPORT HYD. 463



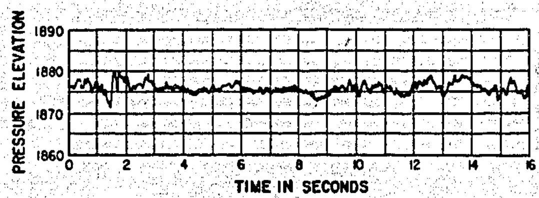
PIEZOMETER NO. 2



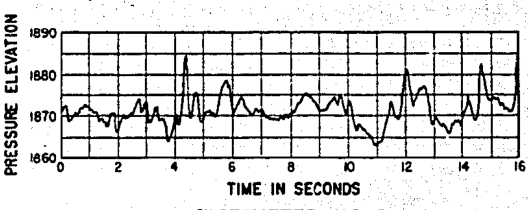
PIEZOMETER NO. 10



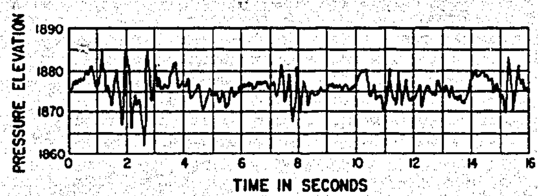
PIEZOMETER NO. 5



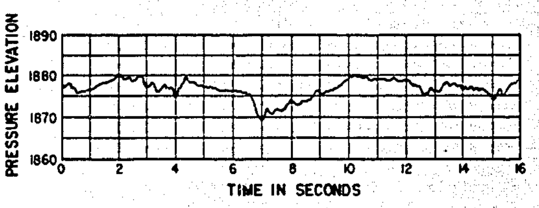
PIEZOMETER NO. 13



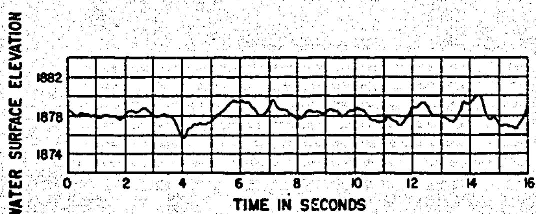
PIEZOMETER NO. 6



PIEZOMETER NO. 14



PIEZOMETER NO. 9



WATER SURFACE LEVEL ON OUTSIDE OF STILLING BASIN

NOTES

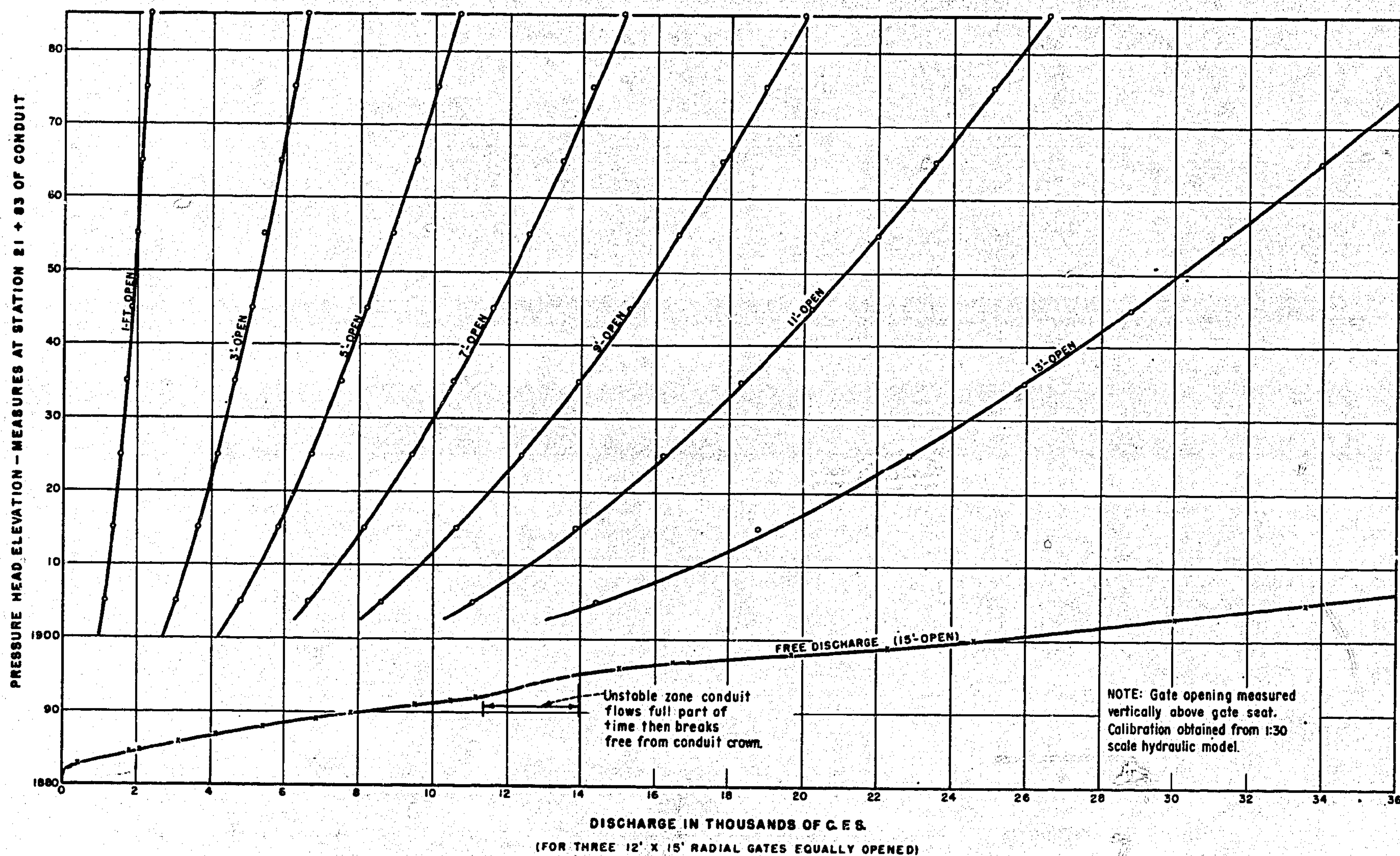
For location of piezometers see Figure 17
Discharge = 34,400 cfs, Reservoir Elevation = 1969.1
Tailwater Elevation = 1878.4, for all records

1:30 SCALE HYDRAULIC MODEL STUDIES

TWIN BUTTES DAM OUTLET WORKS

OSCILLOGRAPH RECORDS FOR PRESSURE TESTS ON STILLING BASIN TRAINING WALLS

616



1:30 SCALE HYDRAULIC MODEL STUDIES

TWIN BUTTES DAM OUTLET WORKS

RADIAL GATE DISCHARGE CAPACITY

FIGURE 26
REPORT HYD. 463

GPO 841003