UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS CANADIAN RIVER PROJECT, TEXAS

Hydraulics Branch Laboratory Report No. Hyd. 491

DIVISION OF RESEARCH

OFFICE OF ASSISTANT COMMISSIONER AND CHIEF ENGINEER
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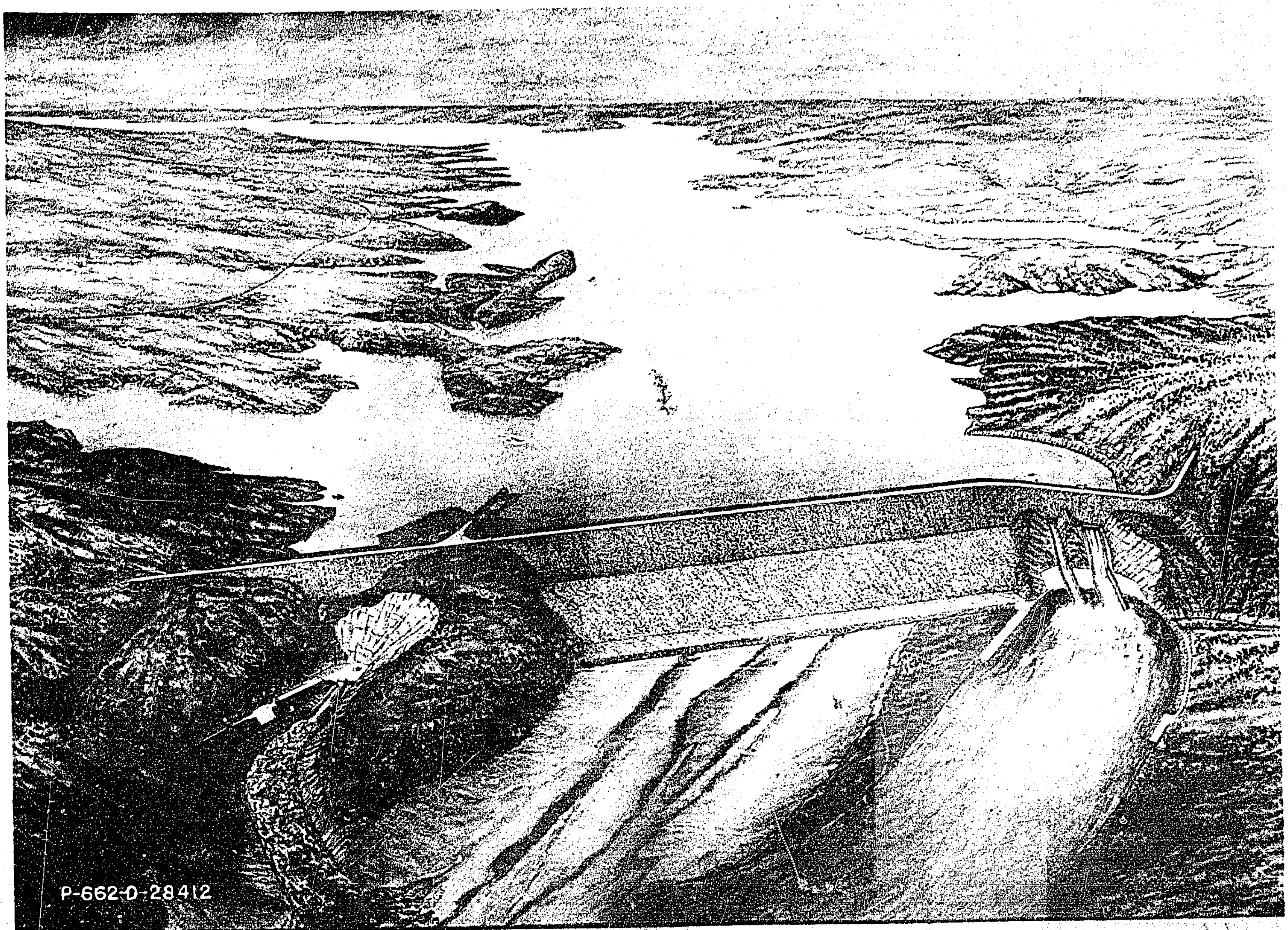

SANFORD DAM

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Office of Assistant Commissioner
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Subject: Hydraulic model studies of the Sanford Dam spillway and flood control outlet works--Canadian River Project, Texas

PURPOSE

The purpose of this study was to investigate the hydraulic features of the morning-glory spillway and the flood control outlet works to determine possible improvements in the operating flow conditions.

CONCLUSIONS

1. The proposed tunnels, chutes, and stilling basins functioned satisfactorily for all operating conditions.

2. The operation of the flood control outlet works was satisfactory for all discharges. A clockwise vortex formed over the entrance and was dissipated as it passed through the trashrack structure.

3. The tapered piers between adjacent tunnels at the outlet portal of the flood control outlet works were increased in length to 40 feet to ensure good chute flow conditions for one gate or two gate operation.

4. Three 30-foot-high piers with "coattail" extensions, installed on the morning-glory crest, improve the tunnel flow conditions for discharges at which the spillway operates unsubmerged and reduce the size of the vortex which forms for discharges at which the spillway operates submerged.

5. Scour tendencies were found to be relatively slight; however, the model indicated that large riprap should be hand placed immediately downstream from the stilling basins.

6. Because of the proximity of the entrances of the spillway and flood control outlet works and the shape of the topography in the reservoir area adjacent to the entrances, the effect of one

entrance on the other during simultaneous operation will be very important. Tests indicated that the vortices which form over the entrances will both rotate in a clockwise direction for all reservoir elevations above 2975, thus tending to weaken each vortex. A small counterclockwise vortex formed over the spillway entrance between reservoir elevations 2973 and 2975.

7. Simultaneous operation of the spillway and flood control outlet works had no apparent adverse effect on the efficiency of the stilling basins.

8. The rate of airflow into the model spillway vertical bend was found to be too small to measure indicating that the prototype air demand rate would also be very small.

INTRODUCTION

Sanford Dam is located in Texas about 45 miles northeast of Amarillo, Figure 1, and is the principal feature of the Canadian River Project. The dam is an earthfill structure about 200 feet high and 6,000 feet long and includes a morning-glory spillway, 58 feet in diameter, with a discharge capacity of 19,300 cubic feet per second; a flood control outlet works consisting of three conduits controlled with top-seal radial gates, with a discharge capacity of 36,400 cubic feet per second; and a river outlet works utilizing twin slide gates with a discharge capacity of 3,400 cubic feet per second. An artist's conception of the dam and its appurtenant features is shown immediately after the table of contents.

The spillway and flood control outlet works entrances are located on the north abutment of the dam, Figures 2 and 3, and are spaced approximately 250 feet apart. The flood control outlet works inlet is approximately 300 feet from the centerline of the dam crest and the spillway entrance is about 200 feet from the crest. The centerlines of the two features converge at an angle of 5° causing the centers of the stilling basins to be about 150 feet apart at their downstream ends.

The morning-glory spillway, Figure 4, is approximately 58 feet in diameter at the crestline and has three piers on the crest, spaced at 120°, with one pier on the tunnel centerline at the upstream side of the entrance. The piers include "coattail" extensions which act as guide vanes. The entrance profile becomes tangent to an approximately 90° vertical bend at a distance of 27 feet below the crest. The bend is joined at its downstream end by a 22-foot-long transition in which the tunnel diameter is changed from 23 to 22 feet. The 22-foot diameter is maintained throughout the tunnel except for a 45-foot 6-inch-long circular-to-horseshoe transition at the exit portal. Immediately downstream from the exit portal is a diverging chute which discharges into a 54-foot-wide stilling basin and then into the river channel.

The flood control outlet works, Figure 5, consists of three rectangular bellmouth entrances which lead into three identical conduits. A transition connects the entrances to three 15-foot 6-inch-diameter circular conduits. Each circular conduit is followed by a top-seal radial gate control section and a transition to a horseshoe shaped tunnel. The horseshoe tunnel, with a bottom width of 17 feet and a top radius of 8 feet 6 inches, continues to the exit portal. The three tunnels discharge onto a diverging chute which discharges into a 100-foot-wide stilling basin and subsequently into the river channel.

The flood control outlet works is controlled to pass the river channel capacity of 25,000 second-feet up to reservoir elevation 2965 at which the spillway begins to operate. Above elevation 2965 the flood control outlet works gates are fully opened.

The tailwater elevation is expected to be lowered 16.5 feet due to degradation in the downstream channel. A curve of discharge versus tailwater elevation before degradation is shown on Figure 3.

This investigation was concerned with determining (1) a correct shape of spillway entrance profile to avoid adverse subatmospheric pressures and cavitation, (2) a means of controlling the vortex which forms over the spillway during submerged conditions, (3) a means of ensuring good tunnel flow conditions for all discharges, (4) the adequacy of the circular-to-horseshoe transition in the spillway tunnel, (5) the adequacy of the chutes and stilling basins, (6) the influence of one entrance on the other during simultaneous operation, and (7) discharge capacities of the spillway and the flood control outlet works. Specific features of the flood control outlet works entrance and conduits were not studied since these features are very similar to those used on a previous project.

THE MODEL

To facilitate a study of the effects of simultaneous operation, the spillway and flood control outlet works were assembled in a common head box and a common tail box, Figure 6. The model, which was constructed on a scale of 1:46.42, consisted of the immediate reservoir topography adjacent to the entrances, the tunnels, chutes, and stilling basins, and the topography in the vicinity of the stilling basins, including a portion of the downstream river channel. The river outlet works was not included in the model study because of its relatively low discharge and because it is located on the opposite abutment of the dam from the spillway and flood control outlet works.

The topography of the reservoir area in the head box consisted of cement mortar placed on metal lath supported by wood framing and the downstream river channel topography was formed using river sand with an average size of 0.8 mm. These features are shown in Figure 7.

The preliminary spillway crest was formed with concrete and included 28 piezometers in 4 rows of 7 each. The preliminary spillway crest before installation of the topography is shown in Figure 8A.

The vertical bend and spillway tunnel were constructed of transparent plastic to permit observation of the tunnel flow conditions. Eight piezometers were installed in the circular-to-horseshoe transition to determine whether or not objectionable pressures would be induced by the change in shape.

The flood control outlet works entrance and conduits, Figure 8B, were constructed of sheet metal, and slide gates were installed to represent the radial gate section. The top-seal radial gates were similar to those tested in a previous model study1/, and the design was considered acceptable for this structure. The slide gates were used only to establish downstream flow conditions for one gate or two gate operation.

The spillway and flood control outlet works chutes were constructed of plywood and tempered masonite, and the stilling basins were constructed of plywood. Piezometers were installed on the center-line of the flood control outlet works chute to determine the pressure profile. Piezometers were not placed on the spillway chute since the two chutes are very similar in design. Chute blocks and dentated end sills were installed in the stilling basins and piezometers were placed in the stilling basin walls.

Water was supplied from a large sump using a centrifugal pump. Discharges were measured using volumetrically calibrated venturi meters and were controlled by gate valves located downstream from the venturi meters.

Reservoir water surface elevations were measured using a staff gage for approximate measurements and a hook gage in a stilling well for more precise measurements. Tailwater elevations were measured with a staff gage in a stilling well and were controlled with a tail

1/Report Hyd-463 "Twin Buttes Dam Outlet Works" by T. J. Rhone.

gate at the downstream end of the tail box. Tailwater settings were determined from the tailwater rating curve, Figure 3.

THE INVESTIGATION

The investigation was concerned with the operating characteristics of the component features for the full range of discharges up to and including the maximum discharge. The stilling basins and downstream channel were investigated for both the normal and the degraded tailwater conditions.

Preliminary Observations

During the initial operation of the model, it was found that several problems would require investigation.

Flow conditions in the spillway were observed for the full range of discharges with no flow-improving devices on the crest and no deflector or air vent in the throat. For lower discharges, when the entrance operated unsubmerged, it was found that the flow spiraled through the spillway entrance and caused a violent swaying action in the tunnel because of the asymmetry of flow over the crest. This asymmetry of flow was due primarily to the proximity of the flood control outlet works. The curved channel approach to the entrance of the flood control outlet works induced a strong clockwise current which caused a concentration of flow to enter the left side of the spillway crest.

As the discharge was increased and the morning-glory became submerged, a strong vortex formed over the entrance. A portion of the spillway tunnel flowed full immediately after submergence and then flowed freely again as the discharge and reservoir elevation increased. It was noted that near the maximum reservoir elevation the vortex tended to become smaller and less stable due to the large degree of submergence. Calibration tests indicated that the spillway would pass approximately 3,200 cfs more than the design discharge.

The flood control outlet works was operated with the gates 100 percent open and with no trashrack structure over the entrance. It was found that a strong clockwise vortex formed over the entrance and moved from one conduit to another. The vortex pulled a large amount of air into the conduit and, although the conduit flow could not be observed, the flow conditions at the outlet portal indicated that the inclusion of air roughened the flow surface considerably.

Tests indicated that the 20-foot-long tapered piers between adjacent conduits at the outlet portal of the flood control outlet works were not sufficiently long to ensure good flow conditions on the chute during operation with one or two gates closed. With an outside gate open or with the center gate and an outside gate open the flow crossed to the opposite side of the chute and collided with the wall, causing a large fin which at times overtopped the wall. With the two outside gates open a large fin formed in the center of the chute. For all combinations of unsymmetrical operation the depth was not uniform across the chute at the toe of the hydraulic jump.

Preliminary calibration indicated that the flood control outlet works would carry about 41,000 second-feet. The maximum design discharge was calculated to be 36,400 second-feet, using n values of 0.012 for steel and 0.013 for concrete. Using a value of 0.008 for both steel and concrete, the maximum design discharge was found to be 38,650 second-feet. Therefore, the model indicated a maximum discharge above both the calculated values. Inspection of the model construction drawings indicated that the conduits at the gate section had been made slightly larger than the correct size. Since calculations indicated that the maximum discharge would be about 38,600 second-feet, the gates were closed slightly to compensate for the error. Thus, the gates were adjusted to pass a design discharge of 38,650 second-feet at maximum reservoir elevation and all subsequent tests were made under this condition.

In addition to the investigation of the problems that became apparent during the initial operation of the model, tests were made concerning pressure distribution on the spillway crest, pressures in the circular-to-horseshoe spillway tunnel transition, pressure distribution on the flood control outlet works chute, air demand rates in the spillway tunnel, stilling basin operation, pressures on the stilling basin training walls, erosion of the downstream river channel, and the effectiveness of riprap in the downstream channel.

Investigation of the flood control outlet works will be discussed first, followed by discussions of the investigation of the morning-glory spillway, the downstream channel, and the effects of simultaneous operation.

The Flood Control Outlet Works

The discharge curve of the recommended designs, Figure 9, indicates that the flood control outlet works will discharge the safe river channel capacity of 25,000 cfs above reservoir elevation 2846. The gates may be adjusted to maintain this discharge for reservoir elevations up to the spillway crest at elevation 2965. When the spillway begins to operate, the flood control outlet works will discharge about 29,800 cfs with the gates 100 percent open. These figures are based on a maximum flood control outlet works discharge of 38,650 second-feet.

Tests were made for one gate operation and two gate operation using three different sets of tapered piers with lengths of approximately 20, 33, and 46 feet. The tests indicated that the 20- and 33-foot piers were both too short to ensure good flow conditions on the chute and that the 46-foot wall was longer than necessary, Figure 10. Therefore, a 40-foot-long wall was installed and tested and was found to be adequate.

To simulate the effect of the trashrack structure, a box-like framework of 1/4-inch mesh hardware cloth conforming to the outside dimensions of the trashrack was built and installed in front of the flood control outlet works entrance, Figure 11. This structure did not eliminate the vortex, but reduced the vortex size and prevented air from passing through the mesh and into the conduits. The prototype trashrack structure is expected to have approximately the same effect, but no accurate comparison can be made.

Five piezometers, Figure 12, were installed on the flood control outlet works chute centerline to discover any possible adverse subatmospheric pressures. The piezometers indicated that the lowest subatmospheric pressure would occur approximately 50 feet downstream from the point of curvature of the vertical curve for the maximum discharge of 38,650 cfs. This pressure was found to be about 2.8 feet of water below atmospheric and was considered to be safe.

The Spillway

The investigation of the spillway was concerned primarily with developing a means of reducing the size of the vortex which forms when the spillway is operating under submerged conditions, and with ensuring good tunnel flow conditions for all discharges. The development of the spillway was accomplished with a series of test setups which will hereafter be referred to as trials.

Trial 1.--The spillway was operated with no piers or guide vanes on the spillway crest and no deflector or air vent in the throat. Some spinning or zigzag action in the tunnel flow was observed for unsubmerged discharges and a portion of the spillway tunnel flowed full for a brief period after submergence. Pressures as low as 6.0 feet of water below atmospheric were observed at the lowest piezometers on the spillway profile, Figure 13 and Table 1. Preliminary calibration indicated that the spillway would pass a maximum discharge of 22,500 second-feet at reservoir elevation 3005 as compared with the design discharge of 19,300 second-feet, Figure 14. It was apparent at this point that a deflector and air vent should be provided as shown in the preliminary design. In addition to the poor tunnel flow conditions, a strong vortex formed over the spillway entrance after submergence. This vortex was observed to become weak and unstable near the maximum reservoir elevation. Flow conditions at the spillway entrance with no piers or guide vanes on the crest are shown in Figure 15.

Trial 2, Preliminary Design. --A deflector and air vent were installed in the spillway throat, 4.25 feet below the start of the vertical bend. The deflector protruded approximately 0.75 foot into the flow and the air vent was 4 by 4 feet square, based on a suggested preliminary design. The deflector and air vent proved to be very effective in improving the tunnel flow conditions. The water surface was deflected away from the tunnel crown and a free surface was maintained throughout the length of the tunnel, Figure 16. The tunnel would not flow full at any discharge with the air vent closed. The deflector was also effective in reducing the swaying action in the tunnel for unsubmerged discharges. The maximum discharge was reduced to about 19,100 cfs, Figure 14, only slightly less than the design discharge, which indicated that the deflector was properly sized. Pressures as low as 6.5 feet of water below atmospheric were observed at the lowest piezometers on the spillway crest, Table 2. These pressures were considered to be within the limits of safe operation and no further pressure measurements were made before installation of the recommended piers. Although tunnel flow conditions were greatly improved by the addition of the deflector and air vent, a means of reducing the size of the vortex was needed.

Trial 3. --Since it was necessary to reduce the size of the vortex for submerged discharges and also to further improve tunnel flow conditions for unsubmerged discharges, it was decided to install rib vanes similar to those used on the Whiskeytown Dam Spillway. 2/ Six rib vanes, approximately 2-1/2 feet wide and 4 feet high were installed. These rib vanes extended from the crest to elevation 2950, about halfway down the spillway profile, Figure 17. It was found that the rib vanes had some effect in stabilizing the vortex but had no apparent effect in reducing the size of the vortex. The tests also indicated that the vanes were too short in length to provide any improvement in the tunnel flow. The maximum spillway discharge was found to be approximately 19,100 cfs, Figure 14, which indicated that the rib vanes had little effect on the spillway capacity.

2/ Report Hyd-498 "Whiskeytown Dam Spillway" by G. L. Beichley.

Trial 4.--The Whiskeytown-type rib vanes were extended to elevation 2938 at the bottom of the spillway profile. Tests indicated that these rib vanes improved the tunnel flow for unsubmerged discharges, Figure 18. This improvement was due to more even distribution of the flow entering the vertical bend. Previous observations had shown that a large concentration of flow entered the left side of the spillway crest because of the influence of the flood control outlet works. The long rib vanes distributed the flow around the crest more evenly so that the flow concentration was less apparent. The long rib vanes reduced the maximum discharge to about 18,500 cfs, Figure 14, due to their effect in reducing the area of the throat. The rib vanes had no apparent effect in reducing the size of the vortex. At this point it was decided that a means of blocking the rotation near the water surface would be required.

Trial 5.--A single pier extending approximately 20 feet into the reservoir in a radial direction was installed on the spillway crest. The pier was 40 feet high above the spillway crest, 6 feet wide, and was not streamlined except for rounding on the upstream end. The optimum location of the single pier, with regard to reduction of the vortex size, was found to be at 90° clockwise from the centerline of the tunnel on the upstream side of the crest. During operation at weir discharges a strong concentration of flow was observed on each side of the pier, Figure 19A. The single pier tended to strengthen the concentration which was naturally induced by the operation of the flood control outlet works. A very pronounced swaying action was observed in the spillway tunnel, and the spinning action in the vertical bend was strong enough to cause the flow to spin across the crown of the tunnel just downstream from the bend. As the spillway entrance became submerged a strong vortex formed, not as strong as that observed with the rib vanes, but large enough to cause a large rope of air to move through the vertical bend. The rope of air became broken immediately below the vertical bend and the air coming to the surface caused a rough water surface in the tunnel, Figure 19B. As the reservoir rose to the maximum elevation the vortex became small and unstable and was observed to change its direction of rotation periodically. The flow in the vertical bend and tunnel was observed to be very good. Although the single crest pier was found to be fairly effective in reducing the size of the vortex, it worsened the tunnel flow conditions for the unsubmerged discharges. The maximum discharge for this trial was about 19,000 cfs, Figure 20.

Trial 6.--Three crest piers of the same size and shape as the single pier used in Trial 5 were spaced at 120° on the spillway crest. One of the three piers was placed at the optimum location found in Trial 5. For unsubmerged discharges the flow was much more evenly distributed around the crest, Figure 21A. The three piers caused the flow to enter the crest primarily from three segments instead of a single segment as observed earlier; however, a small flow concentration was still apparent on the left side of the spillway crest. Some swaying motion existed in the tunnel, Figure 22A, though much less than that observed with a single crest pier. Just before the spillway submerged it was noticed that most of the flow entered the crest between Piers 1 and 3. As the morning-glory began to submerge a rotating boil formed just above the throat. A very unstable rope of air came through the vertical bend and then disintegrated. The tunnel flow was good except that the pockets of entrained air roughened the surface. As the spillway became completely submerged, a vortex, smaller than those previously observed, was formed, Figure 21B. An unstable rope of air came through the vertical bend, Figure 22B, but the tunnel flow was generally good. As the maximum reservoir elevation was approached a very small unstable vortex formed, Figure 21C, similar to that observed with the single crest pier. The tunnel flow was observed to be very good, Figure 22C. Maximum discharge was found to be about 19,500 cfs, which indicated that the size of the vortex had been reduced, thus increasing the discharge, even though the crest length had been reduced by the total width of the three piers.

<u>Trial 7</u>.--In an attempt to further improve the tunnel flow conditions, "coattail" extensions were added to the three crest piers described in Trial 6. The extensions may have had some effect in stabilizing the vortex but had no apparent effect in reducing the size of the vortex. There was no noticeable difference in the tunnel flow conditions; in general, the performance with the piers and extensions was very similar to the performance with the piers alone.

<u>Trial 8</u>.--Previous trials had indicated that the crest piers could be shortened to 30 feet in height with no reduction in efficiency because of the instability and small size of the vortex near the maximum reservoir elevation. The vortex at the maximum reservoir elevation appeared to be somewhat stronger with the shortened piers but it was felt that the shortened piers would be adequate. Operation with the shortened piers with extensions is shown in Figures 23 and 24.

<u>Trial 9</u>.--Three streamlined piers with "coattail" extensions were installed and tested. These piers were 30 feet in height above the spillway crest, 6 feet wide at the crest, 20 feet long in a radial direction, and extended approximately 10 feet into the reservoir. The coattail extensions tapered to zero thickness at the start of the vertical bend so that there was no reduction in

area at the throat. Performance of the spillway with the streamlined piers, Figures 25 and 26, was very similar to the performance with the piers used in Trial 8; however, the Trial 9 piers would be preferred over the Trial 8 piers because of their streamlined shape, smaller size, and more practical placement on the crest. The maximum discharge was found to be approximately 19,200 cfs.

Trial 10.--In order to test the validity of the assumption that crest piers would be the only effective means of controlling the vortex, a wall approximately 25 feet high extending from the spillway crest to the dam embankment was installed and a riprapped fill was placed along the sides of a portion of the wall, Figure 27. Immediately after submergence of the morning-glory, the wall was effective in appreciably reducing the size of the vortex. It was also observed that the side slopes of the riprapped fill had no apparent bearing on the efficiency and that the wall was just as effective with the fill removed. As the reservoir rose above the top of the wall the vortex again became uncontrolled. A wall extending to the maximum reservoir elevation was found to be effective in controlling the vortex at all times. However, because of the impracticability of building such a high wall and some uncertainty in predicting the flow conditions in the reservoir, it was decided to return to the use of crest piers.

Trial 11.--Four piers spaced at 90°, and identical in design to those used in Trial 9, were placed on the crest with two piers on the tunnel centerline, Figure 28. It was found that four piers were no more effective than three piers. Also, there was some loss in discharge capacity with four piers because of the reduced crest length.

Trial 12, Recommended Design.--Since tests had indicated that the piers could be shortened to 30 feet in height with little loss in effectiveness and that four piers were no more effective than three piers, it was decided to use three 30-foot-high piers. It was also decided to add the "coattail" extensions to gain any possible improvement in the tunnel flow conditions. The three piers used in Trial 9 were slightly modified by further streamlining. Figures 29, 30, and 31 show the operation of the spillway with the recommended piers on the crest. For unsubmerged discharges, a concentration of flow, due to the effect of the flood control outlet works, was observed between Piers 1 and 3. The flow in the vertical bend was observed to pile up on the upstream side of the throat, thus entraining air in the tunnel flow. This entrained air rose to the surface in approximately one-half the tunnel length. The flow conditions in the tunnel were generally good, with some swaying action due to the flow concentration on the crest.

Near the point of submergence, a boil, rotating primarily in a counterclockwise direction, was observed to rise and fall just below the spillway crest. The surging action was not violent and it was felt that no serious vibrations would be induced. The flow concentration observed between Piers 1 and 3 at lower discharges was much less apparent. Several very unstable spirals of air came through the vertical bend and rose to the surface about halfway through the tunnel. Some small pockets of air were carried through the tunnel. The tunnel flow was observed to be very good with very little swaying action.

As the spillway became fully submerged, the vortex was observed to change to a clockwise direction of rotation at approximately reservoir elevation 2975. Up to this point the vortex was very unstable and frequently changed directions. A very unstable rope of air came through the vertical bend, usually very close to the crown side of the tunnel. Most of this air came to the surface at the upstream end of the tunnel, causing a rough water surface at that point. Part of the air traveled through the tunnel in the form of large pockets.

With the reservoir about 4 or 5 feet below the tops of the crest piers, a small, unstable, clockwise vortex was observed. Small vortices formed near the upstream end of Pier 1, moved toward the center of the entrance, and either disappeared or joined the larger vortex. A small rope of air came through the vertical bend and moved up and down in the tunnel flow, either coming to the surface at the upstream end or moving to the outlet portal.

At the maximum reservoir elevation very small, unstable vortices formed over the entrance. These vortices sometimes changed direction or completely disappeared. A very small rope of air appeared at times in the vertical bend, usually near the crown side and the tunnel flow conditions were very good. The head discharge curve for the recommended spillway is shown in Figure 32 and details of the recommended spillway entrance are shown in Figure 33.

Flow on the spillway chute appeared to be satisfactory for all discharges. No piezometers were installed on the spillway chute, because of its similarity with the flood control outlet works chute.

The difference in water surface elevations on each side of the three piers was measured to determine the differential head acting on each pier. The maximum differential head was found to be about 16 inches acting on Pier 2 with . discharge of about 8,000 second-feet.

Average pressures on the crest pier "coattail" extensions were found to be above atmospheric for all flows above 14,000 second-feet and only slightly subatmospheric for lower flows, Figure 34.

Pressures on the spillway profile were observed to be as low as 7.4 feet of water below atmospheric with the spillway operating alone, and as low as 5.7 feet of water below atmospheric during simultaneous operation of the spillway and outlet works, Table 5. Oscillograph recordings of the dynamic pressure fluctuations, Figure 35, indicated a minimum pressure of 7.0 feet of water below atmospheric, Table 6. These data showed that the pressures on the spillway profile were within safe limits.

The minimum subatmospheric pressures occurred 6 feet above the spillway throat; therefore, it was felt that the pressures in the vertical bend immediately below the throat should be investigated. The lowest pressure in this region was found to be 2.3 feet of water below atmospheric, Table 7.

Pressures in the circular-to-horseshoe transition at the spillway tunnel portal, were found to be above atmospheric for all discharges, Figure 36.

Air Demand

An attempt was made, using a 3/4-inch-diameter sharp-edged orifice, to determine the rate of air flow through the vent into the vertical bend of the spillway, Figure 33. The rate of air flow in the model was found to be so small as to be impossible to measure. This small rate of air inflow may be due to the relatively short length of the spillway tunnel, the small depth of flow in the tunnel, the relatively low velocity of flow in the tunnel, or a combination of these factors, and indicates adequate aeration from the downstream tunnel portal.

Stilling Basin Studies

The operation of the stilling basins was observed to determine the energy dissipating efficiency of the hydraulic jump in each basin, the pressure distribution and water surface profiles on the walls of the basins, the possible erosive effects of the flow in the downstream channel, the effectiveness of riprap placed in the downstream channel, size and frequency of waves encroaching on the side slopes of the downstream channel, and the margin of safety between the expected minimum tailwater and the tailwater elevation at which the hydraulic jump begins to move out of the basin.

Efficiency. --The energy dissipating efficiency was determined by general observation of the performance of the hydraulic jumps in the stilling basins. The basins were designed for the minimum tailwater (after degradation) of 2798.1 and were found to perform satisfactorily under this condition.

The basins were also tested with a maximum tailwater elevation (before degradation) of 2814.6. The operation of the basins under this condition was satisfactory except that surging in the basins caused intermittent overtopping of the walls. It was decided that this condition could be disregarded since at the time of maximum discharge the river channel would have degraded enough that the overtopping condition would not exist. The flood control outlet works stilling basin was also tested for the normal operating discharge of 25,000 second-feet and was found to operate satisfactorily for this discharge. Photographs of the operation of the stilling basins are included in this report in the section on erosion tests.

Sweepout tests. --It was found that at the maximum discharge the stilling basins have a safety margin of approximately 6 feet between the expected minimum tailwater elevation after degradation and the tailwater elevation at which the jump begins to move out of the basin. Observations indicated that when the stilling basins are operating simultaneously, the hydraulic jump in the flood control outlet works basin sweeps out before the jump in the spillway basin. The curves shown in Figure 37 indicate that the safety margin of 6 feet exists also for each feature operating alone at maximum discharge. The safety margin for both structures increases quite rapidly with a decrease in discharge.

Pressures and water surface profiles. --Piezometers were placed at strategic locations on the stilling basin walls as shown in Figure 38 to determine the pressure distribution on the training walls as an aid in the structural design of the walls. Water surface profiles, Figure 39, were taken in conjunction with the pressure measurements to determine the variation between the pressure measurements and the hydrostatic head at any point. Electronically recorded dynamic pressure measurements were made to determine instantaneous pressure variations on the walls of the basins. The dynamic pressures and pressure variations are shown in Table 8. Oscillograph records are shown in Figures 40 and 41.

Erosion tests. --A portion of the river channel downstream from the stilling basins was modeled using sand with an average size of 0.8 mm with 90 percent between the No. 8 and the No. 20 Tyler Standard screens. The sand bed, Figure 42, was subjected to 1 hour of erosion with (1) Q = 57,900, maximum tailwater elevation 2814.6, (2) Q = 57,900, minimum tailwater elevation 2798.1, (3) Q = 25,000, maximum tailwater elevation 2813.5, and (4) Q = 25,000, minimum tailwater elevation 2797.0. The sand bed was reformed after each test run. Results of the erosion tests are shown in Figures 43 through 46. The deepest erosion was found to be about 5 feet, occurring at the right corner of the spillway basin for Q = 57,900, both maximum and minimum tailwater, and at the right corner of the flood control outlet works basin for Q = 25,000, maximum tailwater.

Riprap studies.--Riprap protection was placed in the downstream channel, Figure 47A, to approximately Station 26+80 in three layers, hereafter referred to as Zones A, B, and C, Figure 47B. Zone A represented the bottom layer of 18 inches of sand-gravel filter in the prototype and was represented with fine-grained equal sized sand with a mean diameter of 0.2 mm. The middle layer, Zone B, 24 inches of bedding material ranging in size from 3/16 inch to 3-1/2 inches, was simulated with the previously described river sand. The top layer, Zone C, the 48-inch layer of riprap material consisting primarily of rocks ranging in size from 1/2 cubic foot to 1/2 cubic yard, was formed with an aggregate mixture having maximum sized pieces of about 3/4 inch. Zone C was the only layer that could be scaled with any degree of accuracy. One purpose of the tests was to determine whether the sand-gravel filter would leach through the bedding material and riprap. The model showed no indication of leaching except for some removal of Zone B material by the action of waves at the water surface; however, because of the inaccuracies involved in scaling down the particle sizes, a large scale test in a wave flume would be more representative of the prototype action. The riprap bed showed very little erosion after 3 hours operation with a discharge of 57,900 second-feet and a tailwater elevation of 2798.1, Figure 47C. Approximately 2 feet of material were removed from a small area at the right corner of the spillway basin. Hand placement of large riprap is suggested for this region. Figure 47D shows the riprap after the 3-hour erosion test.

Waves.--The sizes and average frequencies of the waves observed in the downstream river channel are shown in Table 9. The maximum wave height measured vertically from trough to crest on the right bank of the channel at approximately Station 26+80 was found to be about 4 feet. These large waves can be seen in Figures 44A and 47C. The beaching of the sand bed by the wave action can be clearly seen in Figure 44B, but the waves apparently had very little effect on the riprap as shown in Figure 47D. The riprap tended to absorb the waves, thus reducing their tendency to pull material down the slopes during the ebb of the wave cycle. The average frequency of waves of all sizes was found to be about 65 to 75 waves per minute. The larger waves had no periodic frequency and encroached on the slopes at random intervals.

Effects of Simultaneous Operation

The effects of simultaneous operation were observed during all phases of the model testing. The most apparent effect was that of the flood control outlet works entrance on the morning-glory spillway entrance.

When the spillway was operating alone, it was found that the flow was well distributed around the spillway crest and that very little swaying action was present in the tunnel. When flow was passing through the flood control outlet works, flow in the curved channel leading to the flood control outlet works induced a clockwise current in the reservoir above the outlet works intake. When the spillway was operating in an unsubmerged condition this clockwise current caused a concentration of flow to enter the left side of the spillway crest. This flow concentration induced a swaying or zigzag action in the spillway tunnel.

After the spillway became submerged, a clockwise vortex formed above the spillway entrance. The clockwise vortex which had formed above the flood control outlet works entrance tended to weaken the spillway vortex because both vortices were rotating in the same direction. By blocking off a portion of the flow in the curved channel, the direction of rotation of the outlet works vortex was forced to change to counterclockwise. When this condition existed the vortices tended to reinforce each other and became much stronger. Allowing the flow conditions in the curved channel to return to the natural state, the outlet works vortex resumed a clockwise direction of rotation. Even though the outlet works vortex returned to its original clockwise direction the spillway vortex had become so strong that it was not appreciably weakened by the opposing forces. However, since this condition was artificially induced it should not occur in the prototype.

Because of the flow conditions in the reservoir, Figure 48, the spillway vortex was in nearly a balanced condition for all reservoir elevations. The vortex was small and unstable and attempted to change its direction of rotation at times. The flood control outlet works entrance apparently had an adverse effect on the spillway before the spillway became submerged and a favorable effect after the spillway became submerged as explained in the two preceding paragraphs. It was apparent that the entrances would have less effect on each other if they were placed farther apart in the reservoir and that a change in the configuration of the curved channel would affect the reservoir flow conditions.

Simultaneous operation also appeared to increase the pressures on the spillway crest above those observed for the spillway operating alone, as shown in Table 5.

Simultaneous operation of the two structures had no apparent effect on the performance of the stilling basins or on flow conditions in the downstream river channel.

APPENDIX A

The purpose of this appendix is to include the results of additional model tests which were requested after completion of the final draft of this report.

Pressures in the spillway tunnel vertical bend. --Piezometers were installed throughout the vertical bend to accurately determine pressures for use in the structural design of the bend invert. The maximum water manometer pressure of 75.2 feet of water occurred on the invert centerline, approximately two-thirds the distance through the bend. Piezometer locations and pressures, for the maximum discharge of 19,300 second-feet, are shown in Figure 49.

Differential pressures on the spillway crest piers. --Water manometer pressures were measured on both side of Piers 1 and 3, Figure 50, to determine the differential head acting on the piers. The maximum differential of 2.3 feet of water occurred on Pier 1, for a discharge of 16,500 second-feet, reservoir elevation 2985.0. At this elevation, vortices form on the left side of the pier (looking toward the center of the morning glory) inducing a drawdown of the water surface in that region.

No measurements were taken on Pier 2 because the flow conditions around this pier appeared identical to those for Pier 3. Figure 50 also includes the differentials for the maximum discharge of 19,300 second-feet.

Additional pressures on the training walls of the stilling basins. --Additional piezometers were installed in the stilling basin training walls downstream from spillway Station 11+56.46 and flood control outlet works Station 22+48.50. These portions of the training walls are not supported by fill outside the basins. Dynamic instantaneous pressures on the inside walls of the basins and simultaneous wave measurements outside the basins were determined for use in an analysis of possible vibrations in the walls. Piezometer locations and pressures are shown in Figure 51, along with a record of dynamic pressure fluctuations and simultaneous wave measurements for one piezometer on one wall of each basin. Pressures were similar to those found in an earlier part of the model study (see Piezometers 36, 37, 38, 44, 45, and 46 in Table 8).

Table 1

PROTOTYPE PRESSURES ON THE SPILLWAY CREST WITH NO APPURTENANCES--FT OF WATER--SPILLWAY OPERATING ALONE

Piezometer No.	Q = 8, 700	Q = 18, 500
1	1. 4	30. 2
2	1. 2	29. 9
3	0. 9	30. 2
4	0. 2	33. 0
5	0. 5	34. 6
6	-0. 9	32. 0
7	-5. 1	20. 9
8	2. 3	31. 1
9	0. 9	29. 5
10	0. 9	29. 7
11	0. 5	33. 0
12	0. 7	35. 1
13	-0. 9	32. 0
14	-4. 6	21. 8
15	2. 8	31. 1
16	1. 2	30. 4
17	0. 9	30. 6
18	0. 2	33. 0
19	0. 7	34. 6
20	-0. 9	32. 0
21	-6. 0	15. 8
22	2. 3	29. 7
23	1. 2	29. 5
24	0. 9	29. 7
25	0. 2	32. 0
26	0. 5	34. 1
27	-0. 9	31. 1
28	-5. 1	18. 6

Piezometer locations shown on Figure 13

Table 2

PROTOTYPE PRESSURES ON THE SPILLWAY CREST WITH NO PIERS--FT OF WATER--SPILLWAY OPERATING ALONE

Piezometer No.	Q = 5,200	Q = 12,700	Q = 18,400
1	2.6	2.3	27.4
2	1.6	0.7	27.2
3	0.9	0.5	26.9
4	0.0	0.0	29.7
5	0.7	0.5	33.2
6	-0.5	-1.4	32.5
7	-2.8	-5.6	26.9
8	2.3	1.9	27.9
9	1.2	1.4	27.2
10	0.9	1.5	27.9
11	0.5	0.0	30.6
12	0.7	0.7	33.7
13	-0.5	-1.6	33.4
14	-2.3	-4.6	26.9
15	2.6	2.3	28.3
16	1.2	0.7	28.1
17	0.9	1.5	28.3
18	0.5	0.0	31.1
19	0.7	0.7	33.7
20	-0.5	-1.4	32.0
21	-3.0	-6.0	22.3
22	2.3	1.9	27.9
23	1.2	1.4	27.6
24	0.9	1.4	27.9
25	0.2	0.0	31.1
26	0.2	0.4	33.7
27	-0.5	-1.9	33.0
28	-3.0	-6.5	25.1

Piezometer locations shown on Figure 13

Table 3

PROTOTYPE PRESSURES ON THE SPILLWAY CREST WITH NO PIERS--FT OF WATER--COMBINED OPERATION

Piezometer No.	Spillway discharge			
	Q = 3,500	Q = 7,500	Q = 12,800	Q = 15,200
1	2.3	2.3	1.9	5.6
2	1.6	1.2	1.2	5.8
3	0.9	0.9	0.9	6.5
4	0.5	0.5	1.4	11.1
5	0.7	1.2	4.4	16.0
6	0.0	-0.5	8.4	16.7
7	-0.9	-3.3	9.3	14.9
8	2.3	2.3	1.9	4.6
9	1.2	1.2	-0.2	4.0
10	0.9	0.9	0.0	5.1
11	0.5	0.5	2.3	9.8
12	0.7	0.7	4.9	14.6
13	-0.5	-0.9	7.4	16.3
14	-1.4	-2.8	6.5	14.4
15	2.3	2.3	1.4	6.5
16	1.6	1.6	0.7	6.3
17	0.9	1.4	0.9	7.0
18	0.5	0.5	1.4	11.6
19	0.7	1.2	3.5	15.6
20	0.0	0.0	6.5	16.3
21	-0.9	-2.3	10.2	11.6
22	2.3	2.3	1.9	7.0
23	1.6	1.6	1.2	7.7
24	0.9	1.4	0.9	7.9
25	0.5	0.5	1.4	12.1
26	0.7	1.2	5.3	15.6
27	0.0	0.0	9.8	17.2
28	-1.9	-3.3	10.2	14.4

Piezometer locations shown on Figure 13

Table 4

PROTOTYPE PRESSURES ON THE SPILLWAY CREST--FT OF WATER--
SPILLWAY OPERATING ALONE--RECOMMENDED PIERS

Piezometer No.	Q = 5,000	Q = 10,000	Q = 15,000
8	2.3	2.3	7.0
9	0.9	0.5	6.5
10	0.9	0.5	7.4
11	0.5	0.0	12.1
12	0.5	0.5	15.8
13	-0.5	-1.4	17.2
14	-2.3	-5.6	15.3
15	2.3	2.7	8.4
16	1.4	0.9	7.9
17	0.9	0.9	8.4
18	0.5	0.0	12.5
19	0.5	0.5	16.3
20	-0.5	-0.9	16.7
21	-3.3	-6.0	11.6
22	2.3	2.3	7.0
23	0.9	0.5	6.0
24	0.9	0.5	7.4
25	0.0	0.0	12.1
26	0.0	0.0	15.8
27	-0.5	-1.4	17.2
28	-3.3	-6.5	14.9

Note: Piezometers 1-7 covered by pier extensions

Piezometer location shown on Figure 13

Table 5

PROTOTYPE PRESSURES ON THE SPILLWAY CREST FOR UNSUBMERGED DISCHARGES--FT OF WATER--RECOMMENDED PIERS

	Spillway operating alone			Combined operation		
Spillway discharge	Piezometer No. 14	Piezometer No. 21	Piezometer No. 28	Piezometer No. 14	Piezometer No. 21	Piezometer No. 28
1,000	-0.5	-0.5	-0.9	0.0	0.0	-0.9
2,000	-0.5	-0.9	-0.9	-0.5	-0.5	-0.9
3,000	-1.4	-1.9	-1.9	-0.9	-0.9	-0.9
4,000	-1.9	-2.3	-2.3	-1.4	-0.9	-2.8
5,000	-2.3	-3.3	-3.3	-1.9	-1.4	-2.8
6,000	-3.3	-3.7	-3.7	-2.8	-1.9	-3.3
7,000	-3.7	-4.6	-4.6	-3.3	-2.3	-3.7
8,000	-4.2	-5.1	-5.1	-3.7	-2.8	-4.2
9,000	-4.6	-5.6	-5.6	-4.2	-3.3	-4.6
10,000	-5.6	-6.0	-6.5	-4.6	-3.3	-4.6
11,000	-6.0	-6.5	-7.0	-5.1	-4.6	-5.1
12,000	-6.5	-6.5	-7.4	-5.6	-5.1	-5.6
13,000	-6.0	-7.0	-7.0	-4.2	-5.6	-2.3
14,000	-4.2	-6.5	-4.2	-4.2	-4.6	-2.8

Piezometer locations shown on Figure 13

Table 6

MINIMUM INSTANTANEOUS PROTOTYPE PRESSURES ON THE SPILLWAY CREST--FT OF WATER--UNSUBMERGED DISCHARGES--RECOMMENDED PIERS

Spillway discharge	Spillway operating alone				Combined operation			
	Piezometer No.				Piezometer			
	13	14	27	28	13	14	27	28
5,000	-0.2	-1.9	-0.4	-2.3	0.0	-2.8	-1.4	-4.2
10,000	-0.9	-4.3	-1.2	-5.8	-1.4	-4.2	-2.3	-7.0
11,000	-1.5	-5.0	-1.6	-6.6	-1.4	-4.6	-1.9	-5.2
12,000	-1.5	-5.0	-1.8	-6.6	-1.4	-5.2	-1.9	-6.5
13,000	-1.9	-5.6	-1.9	-6.5	-1.4	-4.6	-1.9	-6.5

Piezometer locations shown on Figure 13

Table 7

PROTOTYPE PRESSURES IN THE VERTICAL BEND IMMEDIATELY BELOW THE SPILLWAY PROFILE--FT OF WATER--RECOMMENDED PIERS

Spillway discharge	Spillway operating alone		Combined operation	
	Piezometer No. 29	Piezometer No. 30	Piezometer No. 29	Piezometer No. 30
1,000	1.9	0.9	1.9	0.9
2,000	1.9	0.9	2.3	0.9
3,000	2.3	0.5	1.4	0.9
4,000	1.9	0.0	1.4	0.9
5,000	0.5	-0.5	1.4	0.5
6,000	0.0	-0.9	1.9	0.5
7,000	-0.5	-1.4	1.9	0.5
8,000	-0.9	-1.9	1.9	0.5
9,000	-1.4	-2.3	2.3	0.0
10,000	-1.4	-2.3	2.3	0.0
11,000	0.0	-1.4	2.8	0.9
12,000	5.1	0.0	5.7	0.5
13,000	7.0	0.5	21.8	14.4
14,000	9.3	1.4	--	--
15,000	21.4	13.9	--	--

Piezometer locations shown on Figure 13

Table 8

INSTANTANEOUS DYNAMIC PRESSURES ON
STILLING BASINS TRAINING WALLS--FT OF WATER

Combined discharge	Tailwater elevation	Piezometer No.	Maximum pressure	Minimum pressure	Average pressure
57,900	2798.1	32	0.0	0.0	0.0
		33	0.0	0.0	0.0
		34	13.9	-2.3	2.3
		35	60.3	-13.9	23.2
		36	0.0	0.0	0.0
		37	23.2	11.6	18.5
		38	48.7	37.1	44.1
		40	0.0	0.0	0.0
		41	0.0	0.0	0.0
		42	13.9	0.0	7.0
		43	51.0	-4.6	23.2
		44	0.0	0.0	0.0
		45	23.2	9.3	16.2
		46	46.4	32.5	39.5
		47	32.5	0.0	4.6
		48	67.3	-2.3	2.3
		49	60.3	0.0	4.6
		50	9.3	-9.3	-2.3
	2814.6	32	92.7	-9.3	23.2
		33	11.6	0.0	4.6
		34	37.1	16.2	25.5
		35	64.9	23.2	46.4
		36	13.9	7.0	11.6
		37	37.1	27.9	32.5
		38	62.7	53.4	58.0
		40	84.9	-27.9	18.6
		41	11.6	2.3	7.0
		42	37.1	13.9	25.5
		43	55.6	27.8	44.1
		44	13.9	4.6	9.3
		45	39.5	30.2	26.5
		46	62.7	51.1	58.0
		47	74.3	-4.6	30.2
		48	67.3	-6.5	25.5
		49	60.3	-4.6	25.5
		50	65.0	-9.3	23.2
25,000	2797.0	40	20.0	-9.3	4.6
		41	0.0	0.0	0.0
		42	18.5	11.6	16.2
		43	41.7	32.5	37.1
		44	0.0	0.0	0.0
		45	18.6	13.9	16.2
		46	44.1	39.5	41.8
		49	55.7	-32.5	9.3
		50	51.1	-23.2	4.6
	2813.5	40	47.8	9.3	23.2
		41	11.6	4.6	9.3
		42	37.1	27.9	30.1
		43	58.0	48.7	53.4
		44	11.6	7.0	9.3
		45	37.1	32.5	34.8
		46	58.0	58.0	58.0
		49	46.4	18.6	32.5
		50	46.4	13.9	30.2

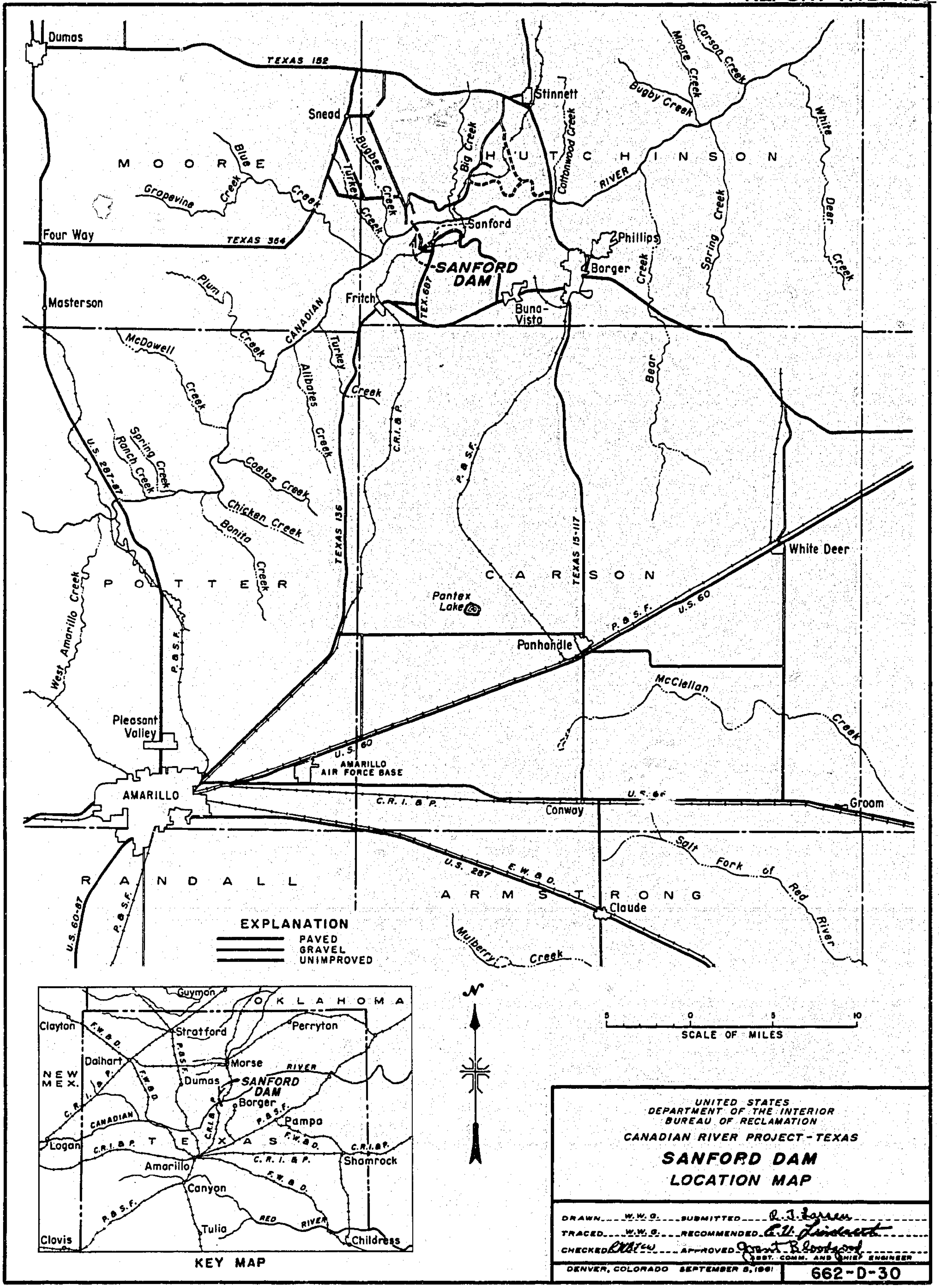
Piezometer locations shown on Figure 38

Table 9

HEIGHT AND FREQUENCY OF WAVES
IN THE DOWNSTREAM CHANNEL

Discharge	Tailwater elevation	Wave height in feet: North bank	Wave height in feet: South bank	Frequency Waves per minute
57,900	2814.6	3.6	2.7	65
57,900	2798.1	4.1	3.6	75
25,000	2813.5	0.9	0.7	65
25,000	2797.0	2.0	1.6	75

FIGURE 1
REPORT HYD. 491



UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CANADIAN RIVER PROJECT - TEXAS

SANFORD DAM
LOCATION MAP

DRAWN W.W.G. SUBMITTED R. J. Larsen
TRACED W.W.G. RECOMMENDED E. V. Lindsey
CHECKED R.M.S. APPROVED Grant Bloodgood, ASST. COMM. AND CHIEF ENGINEER

DENVER, COLORADO SEPTEMBER 8, 1961 — 662-D-30

FIGURE 2
REPORT HYD. 4

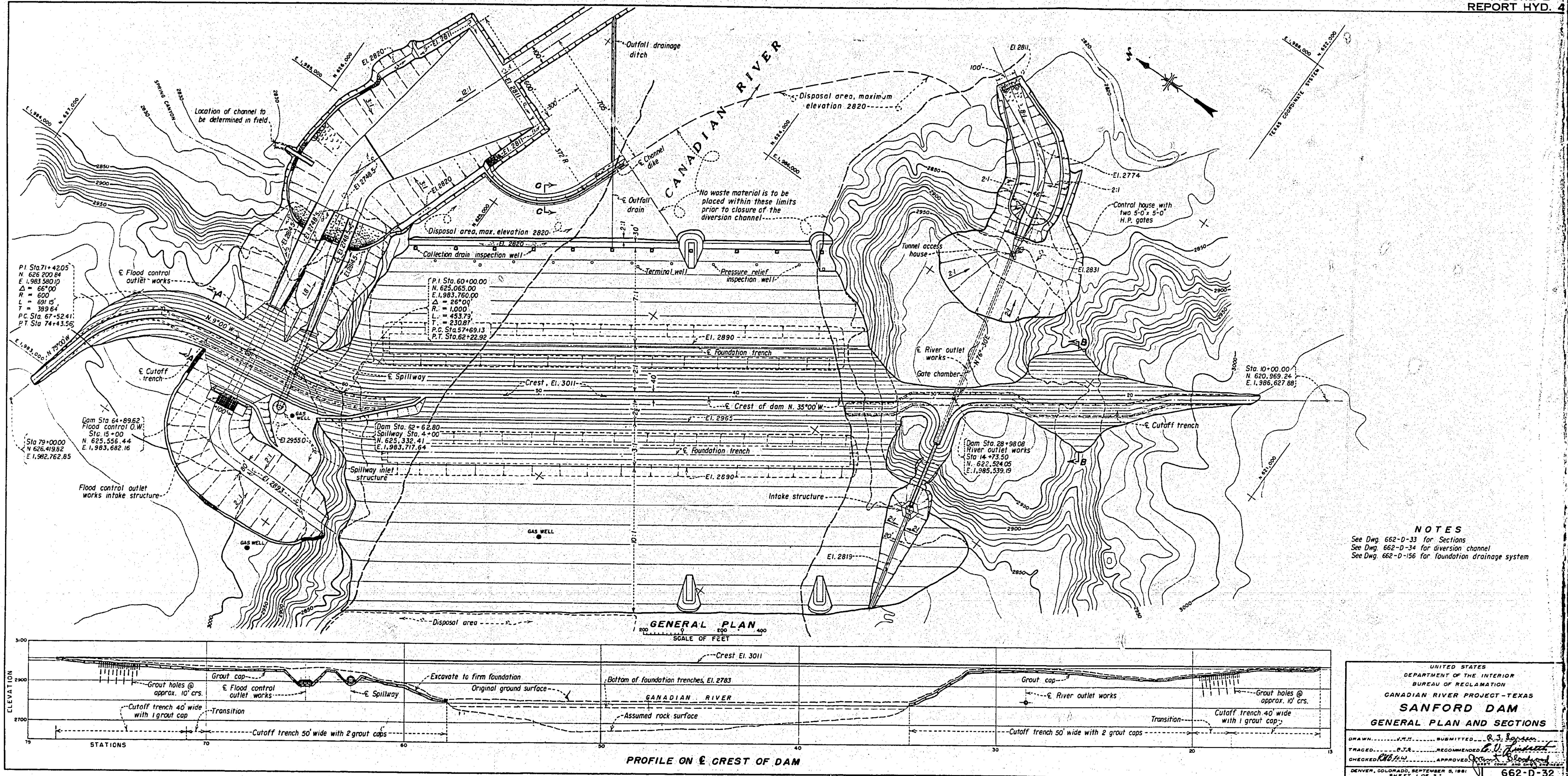


FIGURE 3
REPORT HYD. 491

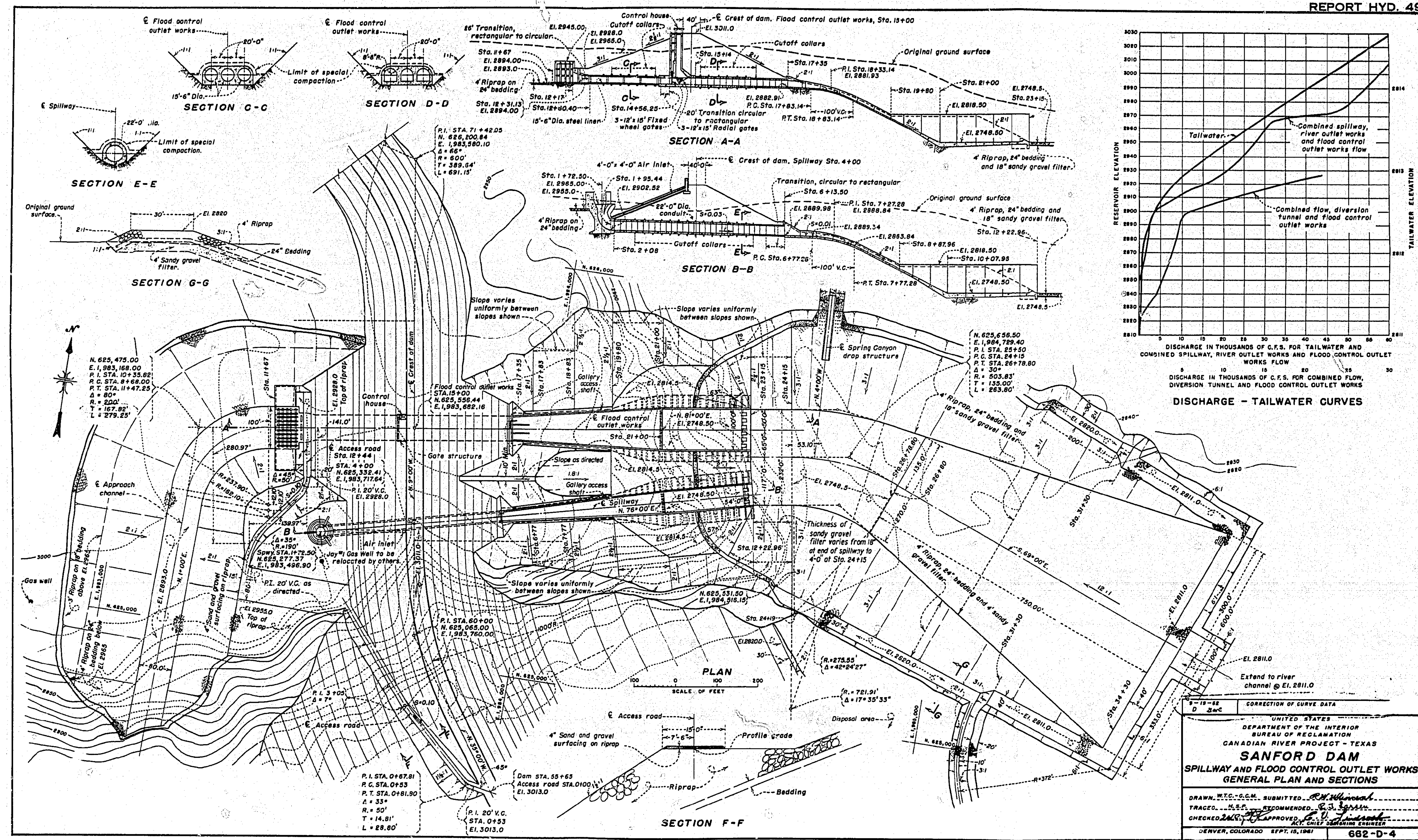
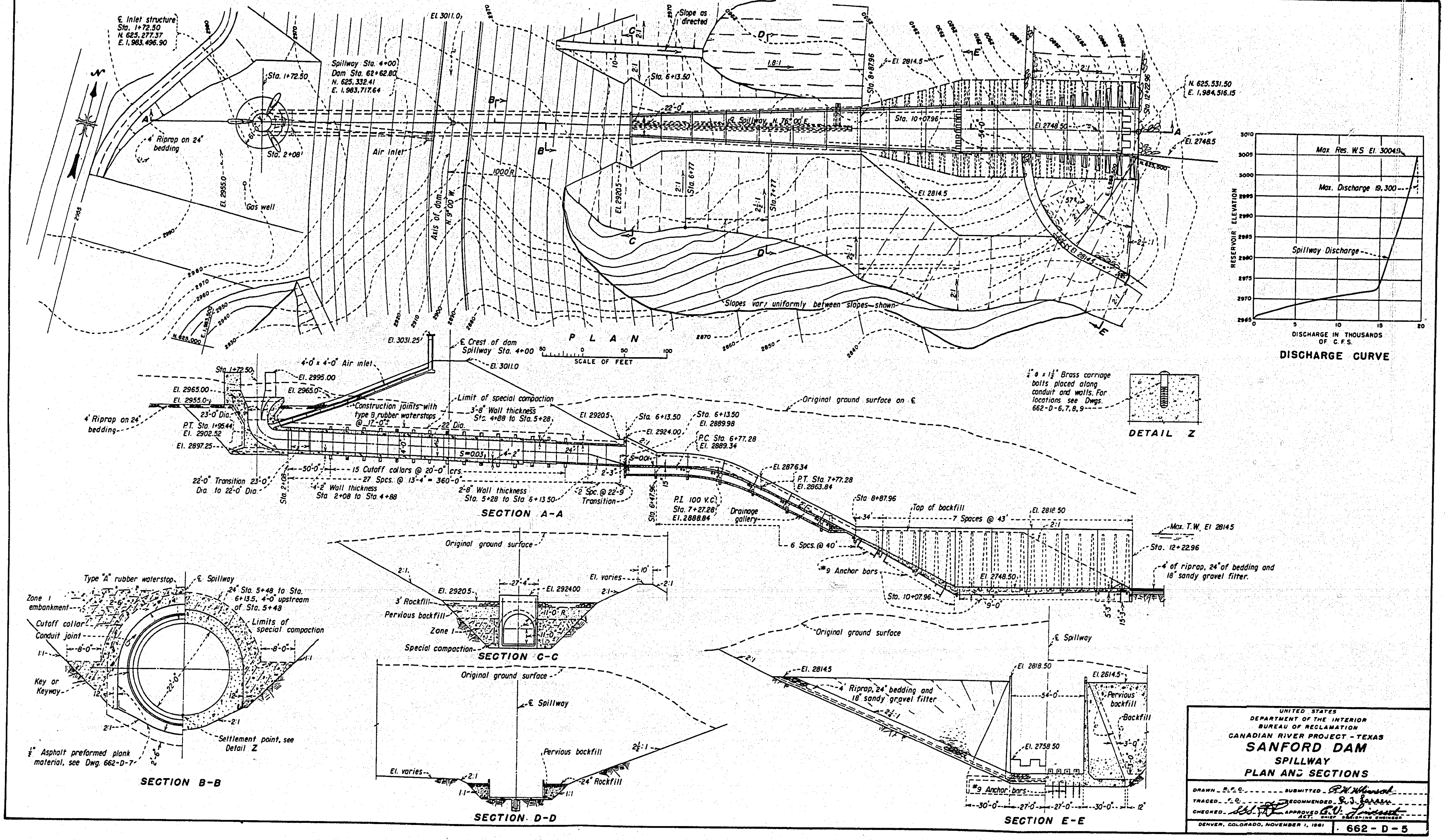
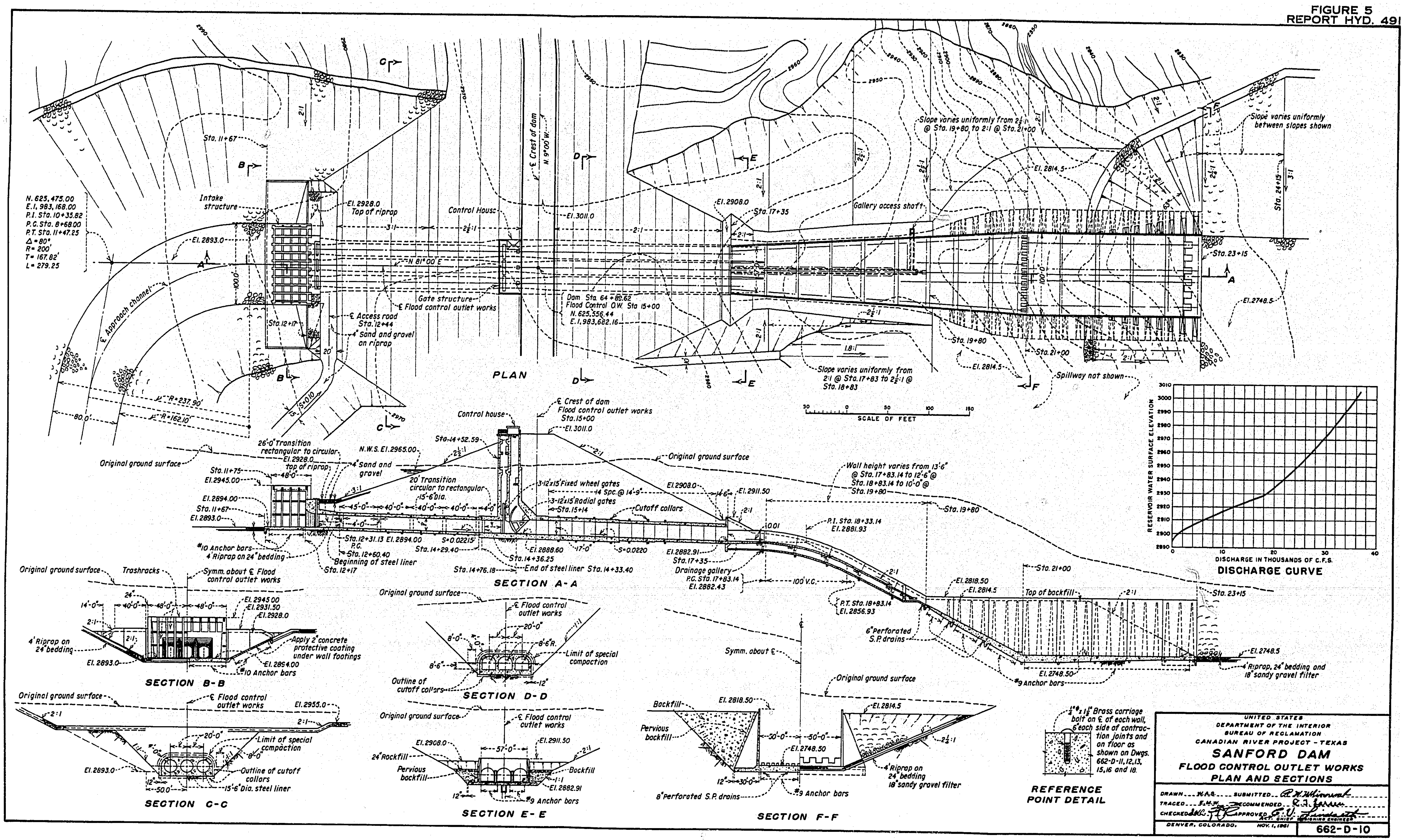
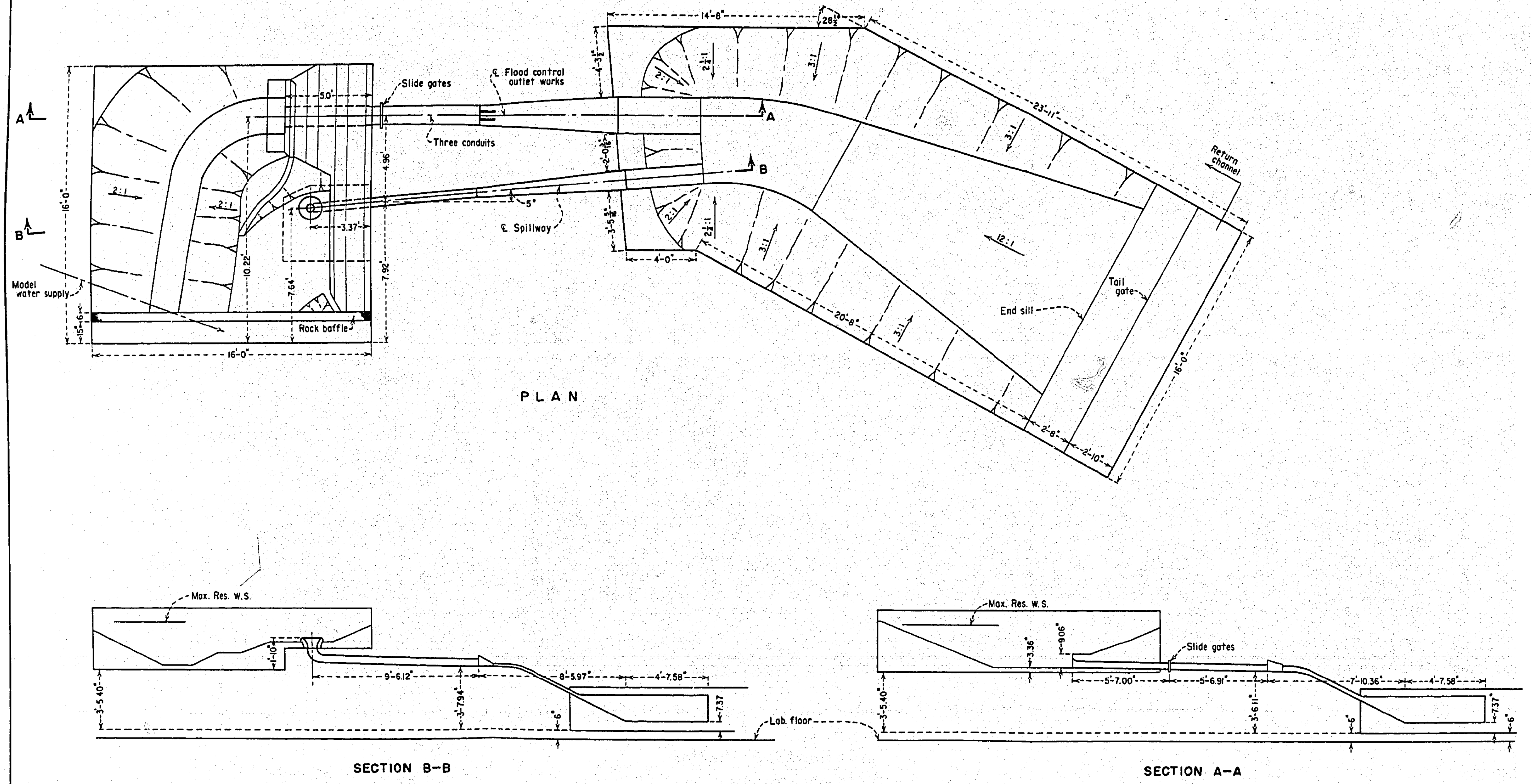


FIGURE 4
REPORT HYD. 491

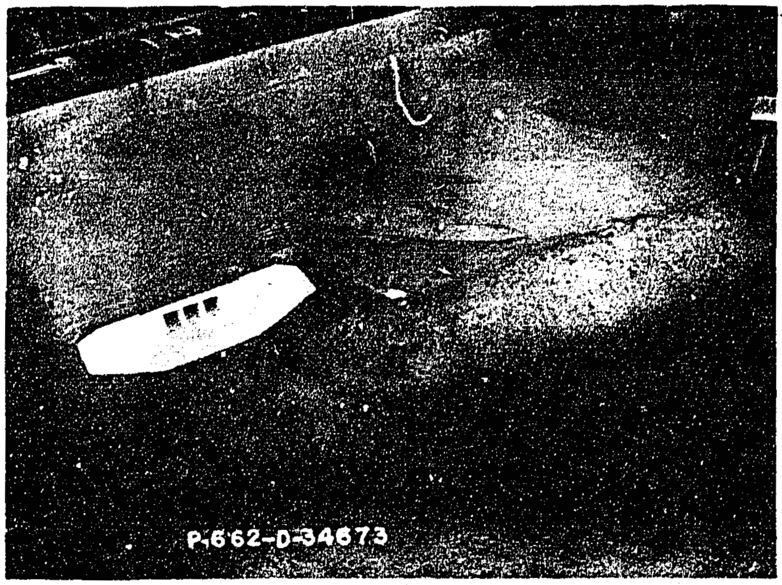



PLAN
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F
REFERENCE POINT DETAIL
DISCHARGE CURVE
RESERVOIR WATER SURFACE ELEVATION
DISCHARGE IN THOUSANDS OF C.F.S.
SCALE OF FEET
Intake structure
Control House
Gate structure
Gallery access shaft
Approach channel
Original ground surface
Spillway not shown
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CANADIAN RIVER PROJECT - TEXAS
SANFORD DAM
FLOOD CONTROL OUTLET WORKS
PLAN AND SECTIONS
DENVER, COLORADO. NOV. 1, 1961
662-D-10

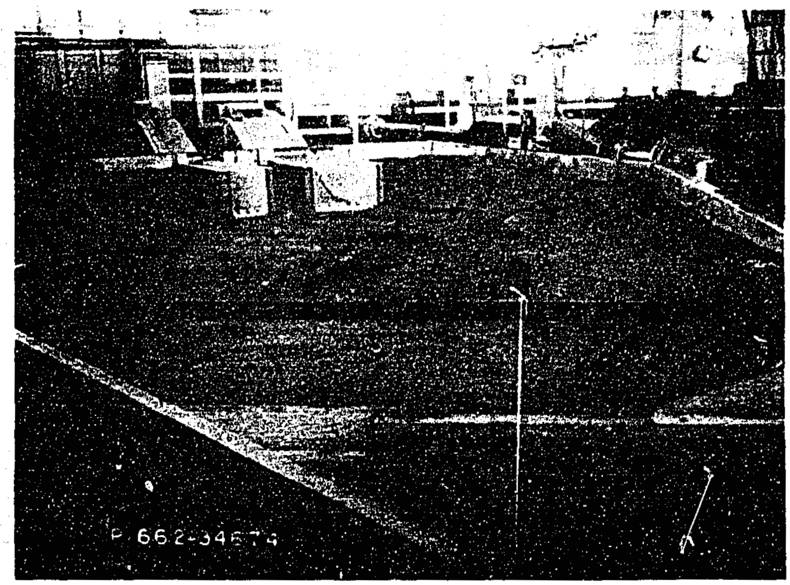


SANFORD DAM SPILLWAY AND
FLOOD CONTROL OUTLET WORKS
1:46.42 SCALE MODEL

MODEL ARRANGEMENT

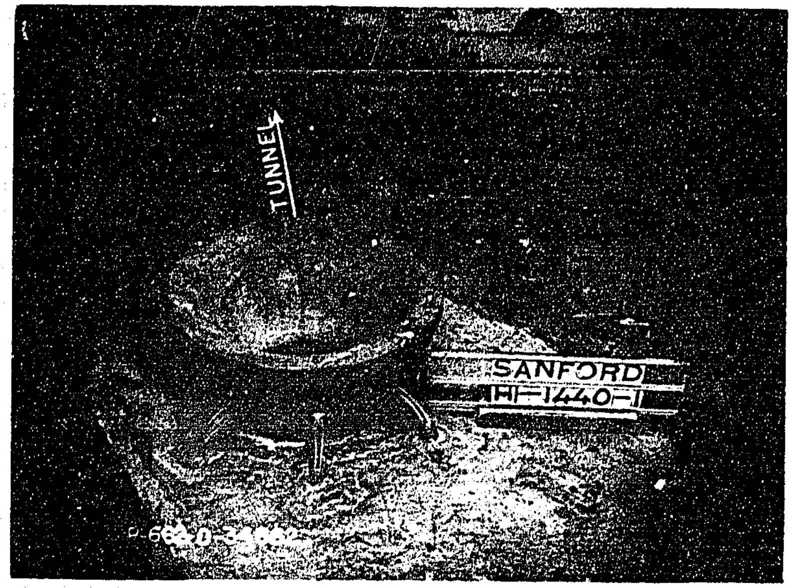


A. Headbox topography, spillway entrance, and outlet works entrance



B. Tailbox topography and spillway and outlet works stilling basins

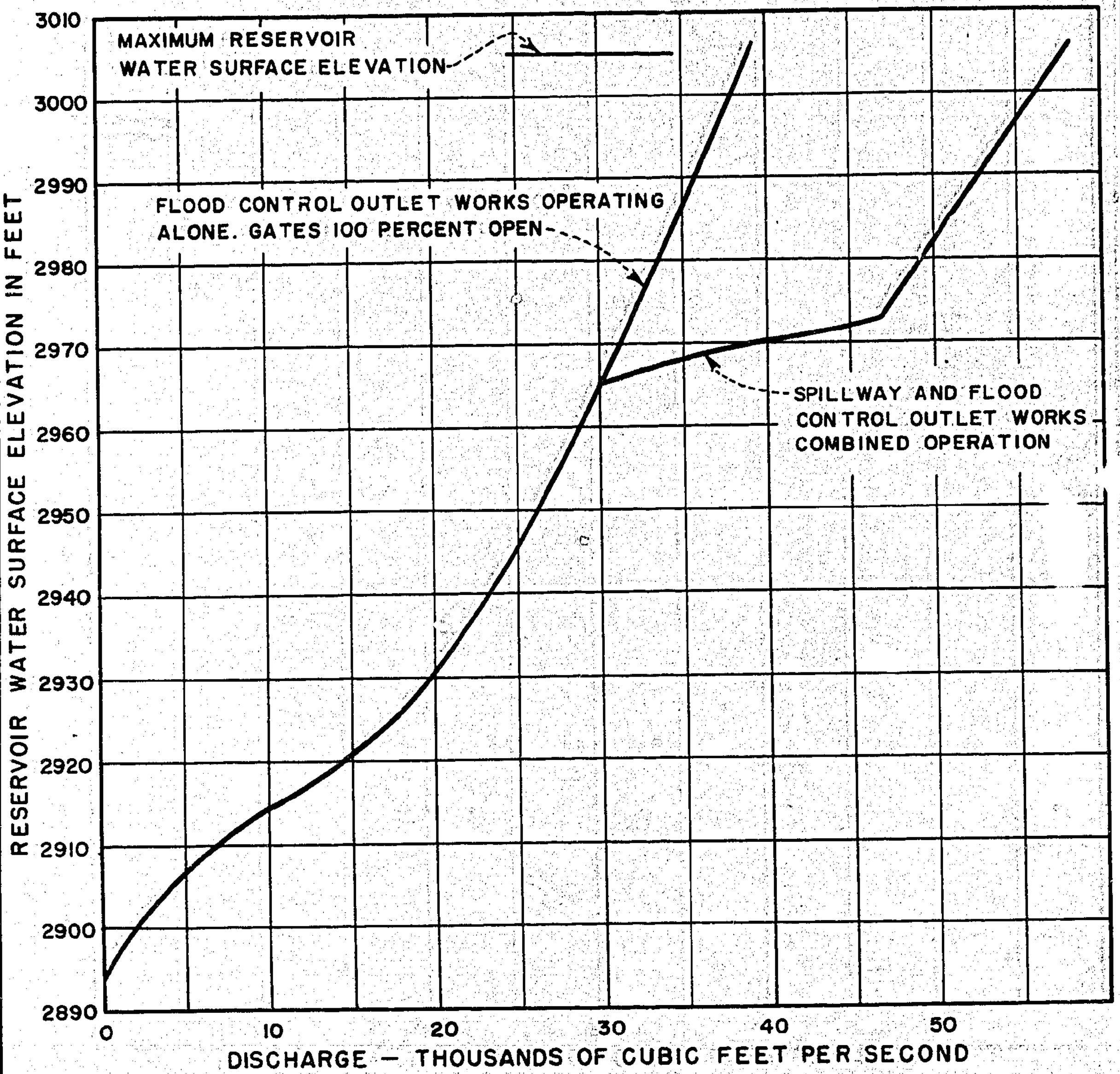
SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS
RESERVOIR AND DOWNSTREAM CHANNEL TOPOGRAPHY
Spillway and Outlet Works Structures
1:46.42 MODEL



A. Morning-glory spillway entrance

B. Flood control outlet works entrance

SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS
ENTRANCES BEFORE INSTALLATION OF TOPOGRAPHY
1:46.42 MODEL



SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS

1:46.42 SCALE MODEL

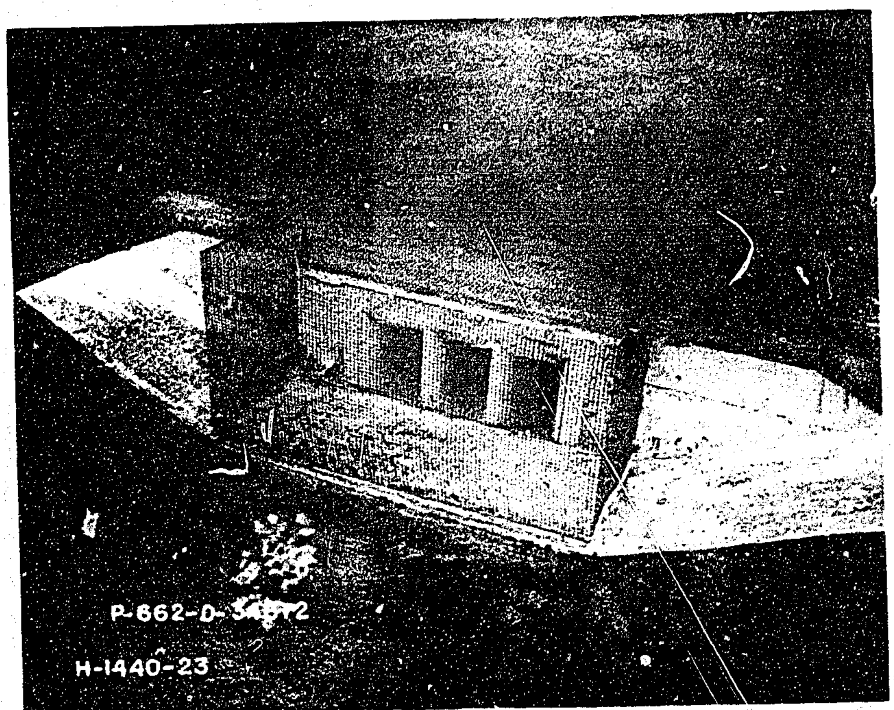
HEAD — DISCHARGE CURVES FOR RECOMMENDED DESIGN

A. Unsymmetrical gate operation with recommended 40-foot-long walls

B. Unsymmetrical gate operation with preliminary 20-foot-long walls

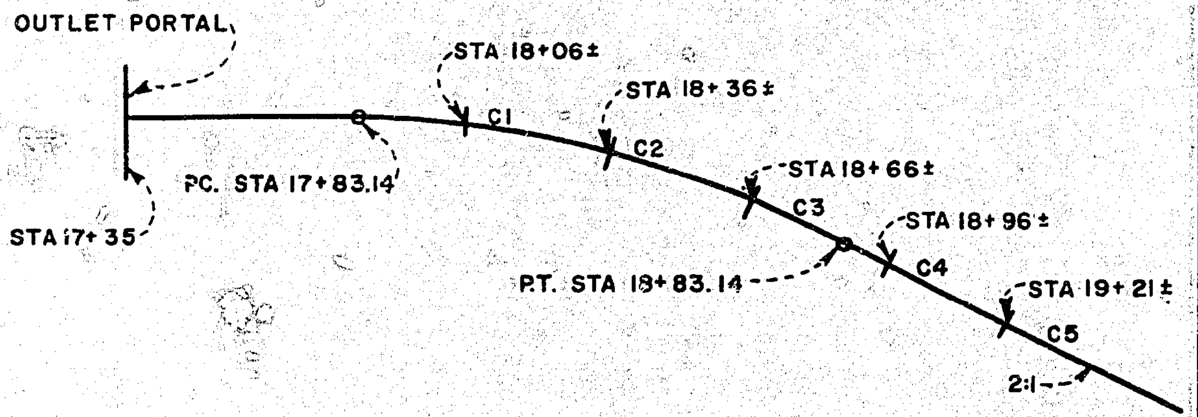
Right gate closed	Right and center gates closed	Center gate closed	Right and left gates closed	All gates open

SANFORD DAM FLOOD CONTROL OUTLET WORKS
FLOW DISTRIBUTION FOR UNSYMMETRICAL GATE OPERATION
1:46.42 MODEL



SANFORD DAM FLOOD CONTROL OUTLET WORKS
SIMULATED TRASH RACK STRUCTURE
1:46.42 MODEL

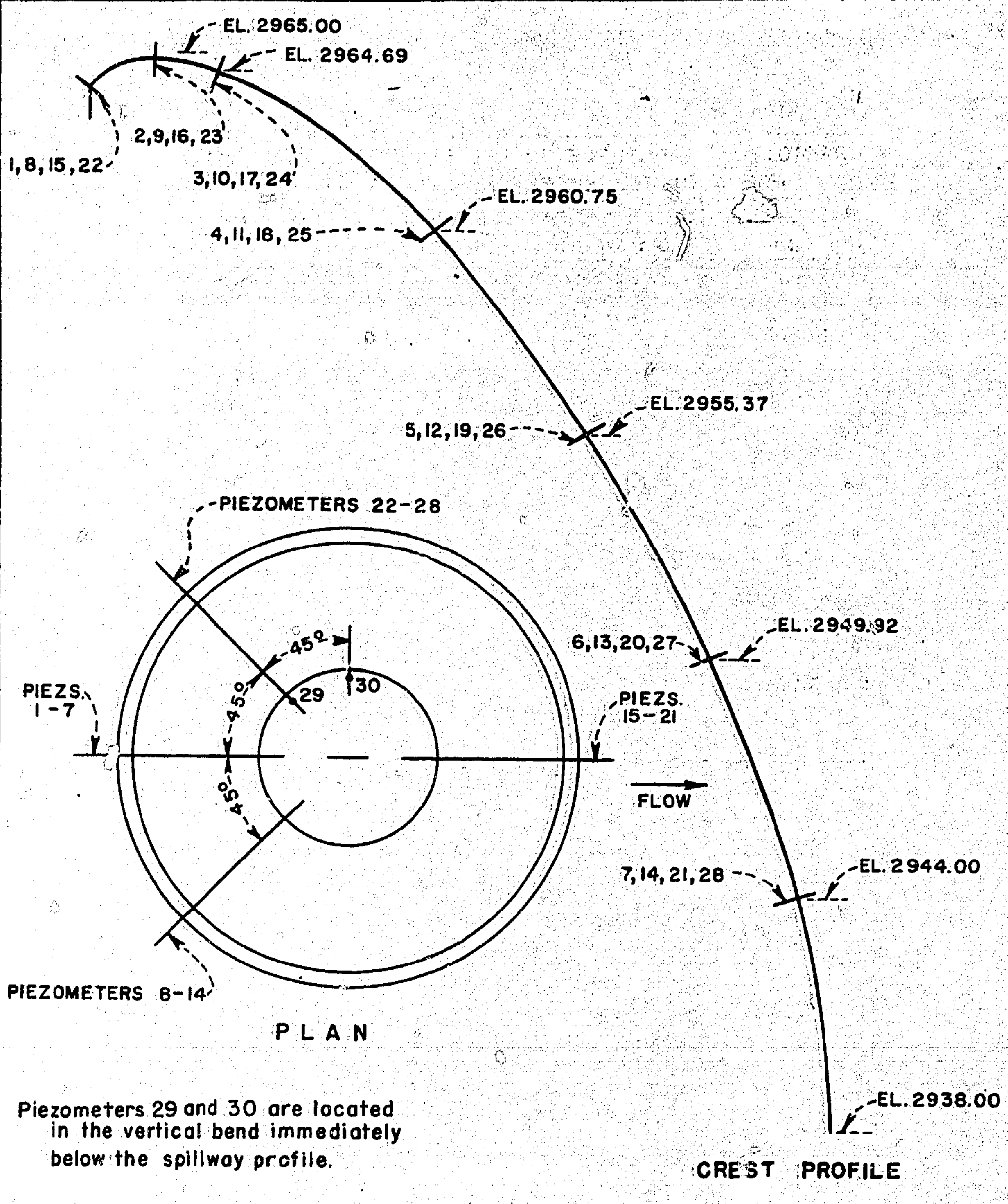
PROTOTYPE PRESSURES ON THE FLOOD CONTROL OUTLET WORKS CHUTE CENTERLINE -- FEET OF WATER			
PIEZOMETER NO.	FLOOD CONTROL OUTLET WORKS DISCHARGE		
	Q = 38,650	Q = 31,000	Q = 25,000
C1	0.9	3.0	3.7
C2	-2.8	0.5	1.9
C3	3.3	3.7	3.7
C4	3.7	3.7	3.7
C5	5.1	4.2	3.7



ELEVATION SHOWING PIEZOMETER LOCATIONS ON CENTERLINE OF CHUTE

SANFORD DAM FLOOD CONTROL OUTLET WORKS
1:46.42 SCALE MODEL

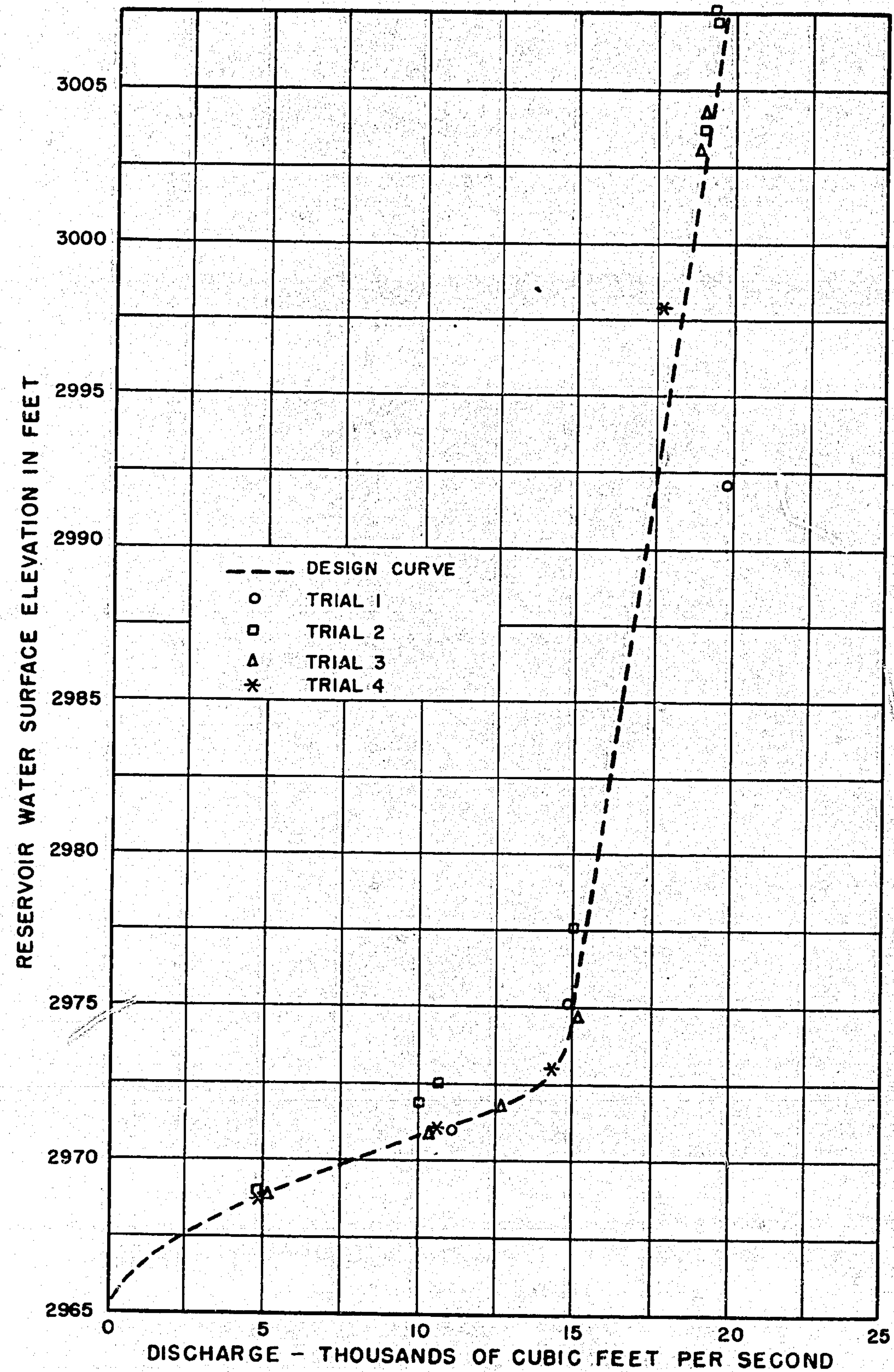
PRESSURES ON THE FLOOR OF THE FLOOD CONTROL OUTLET WORKS CHUTE



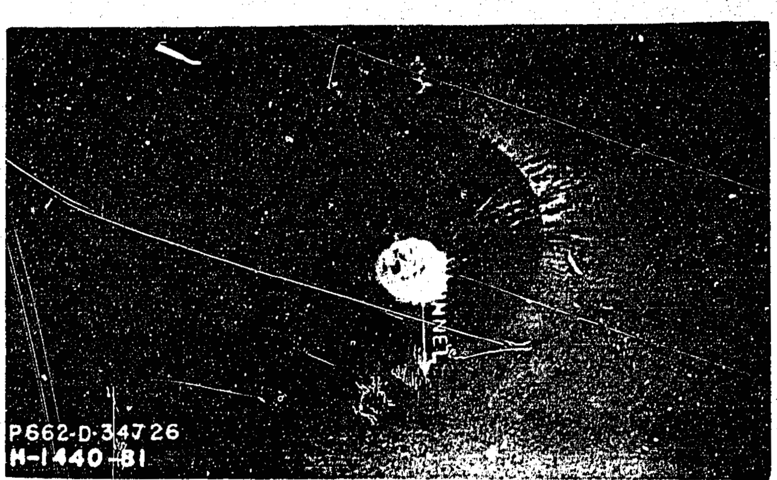
SANFORD DAM SPILLWAY

1:46.42 SCALE MODEL

SPILLWAY PROFILE PIEZOMETER LOCATIONS



SANFORD DAM SPILLWAY
1:46.42 SCALE MODEL
HEAD - DISCHARGE CURVES — TRIALS 1-4



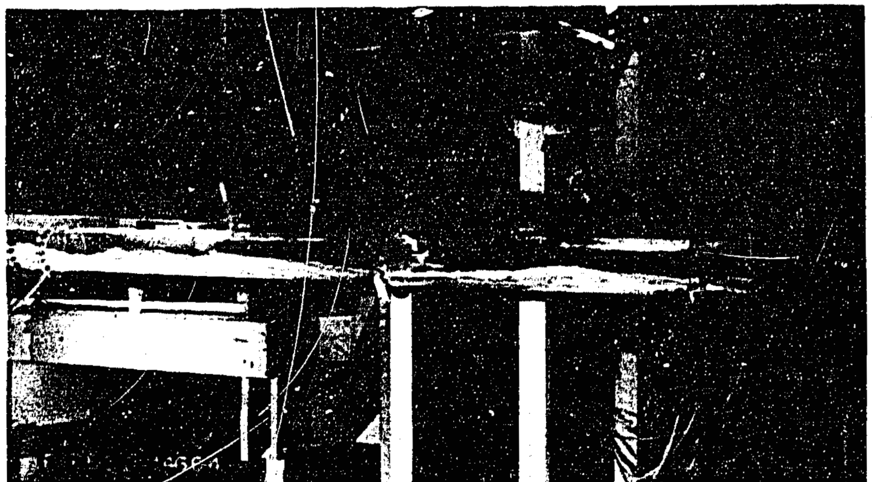
A. Q = 6,000 CFS, Reservoir El. = 2969.0

B. Q = 16,500 CFS, Reservoir El. = 2985.0

C. Q = 19,250 CFS, Reservoir El. = 3005.0

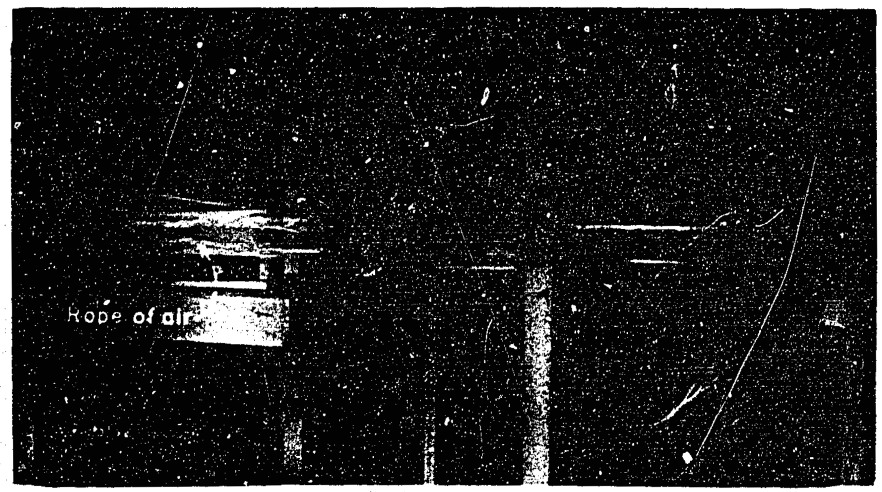
SANFORD DAM SPILLWAY
FLOW CONDITIONS AT THE ENTRANCE WITH
DEFLECTOR AND AIR VENT IN THROAT
1:46.42 MODEL

Figure 15
Report Hyd-491



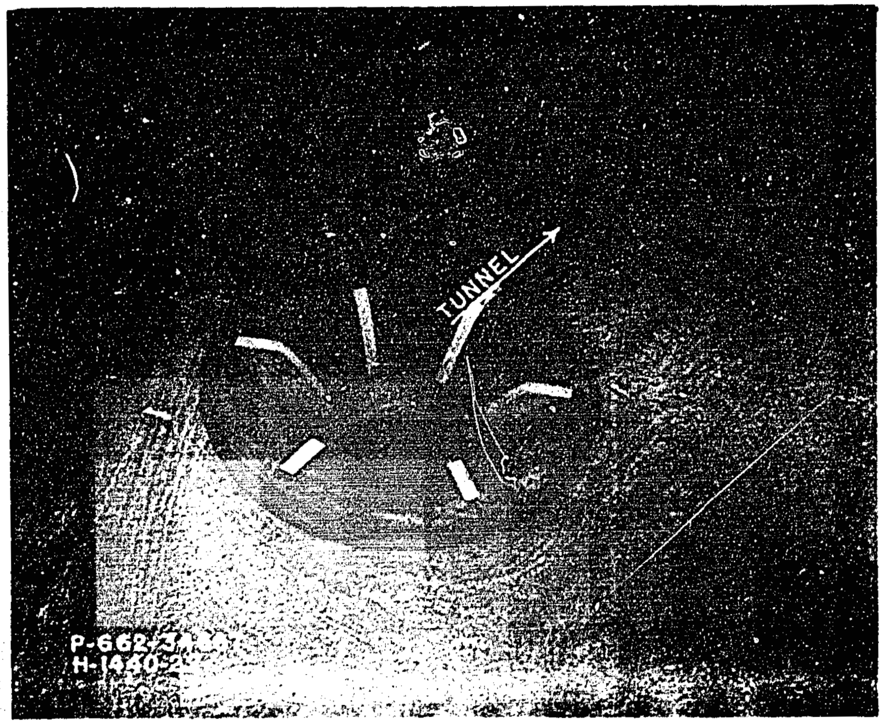
A. Q = 6,000 CFS

B. Q = 16,500 CFS

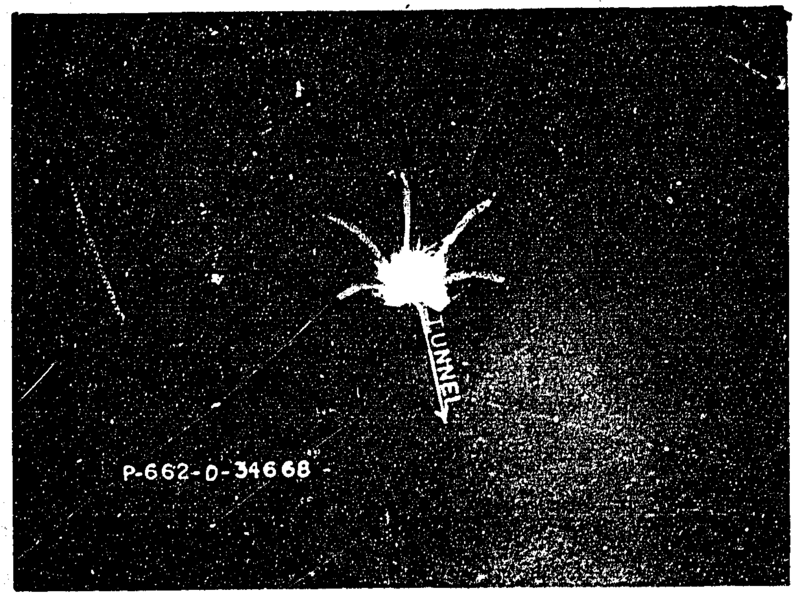


C. Q = 19,250 CFS

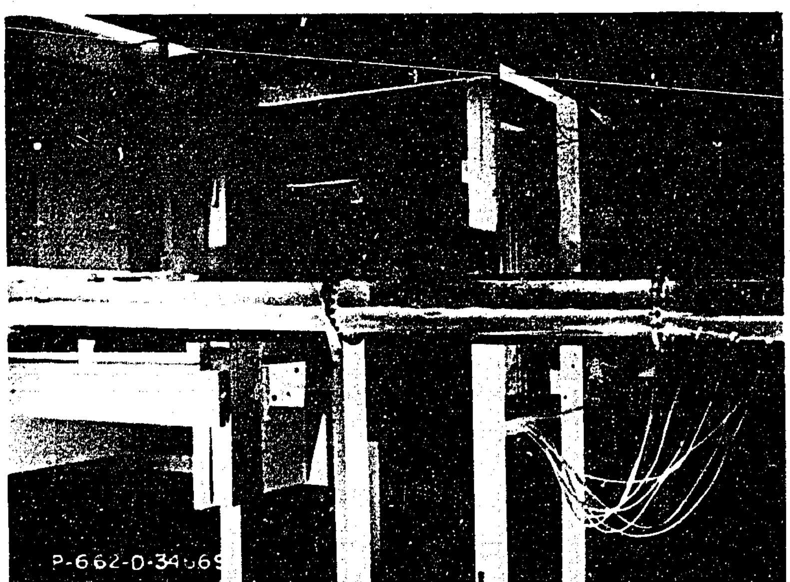
SANFORD DAM SPILLWAY
TUNNEL FLOW CONDITIONS
DEFLECTOR AND AIR VENT IN THROAT
1:46.42 MODEL



SANFORD DAM SPILLWAY
RIB VANES TO EL. 2950
(FLOW CONDITIONS WERE SIMILAR TO THOSE OBSERVED
WITH NO VANES ON THE CREST)
1:46.42 MODEL

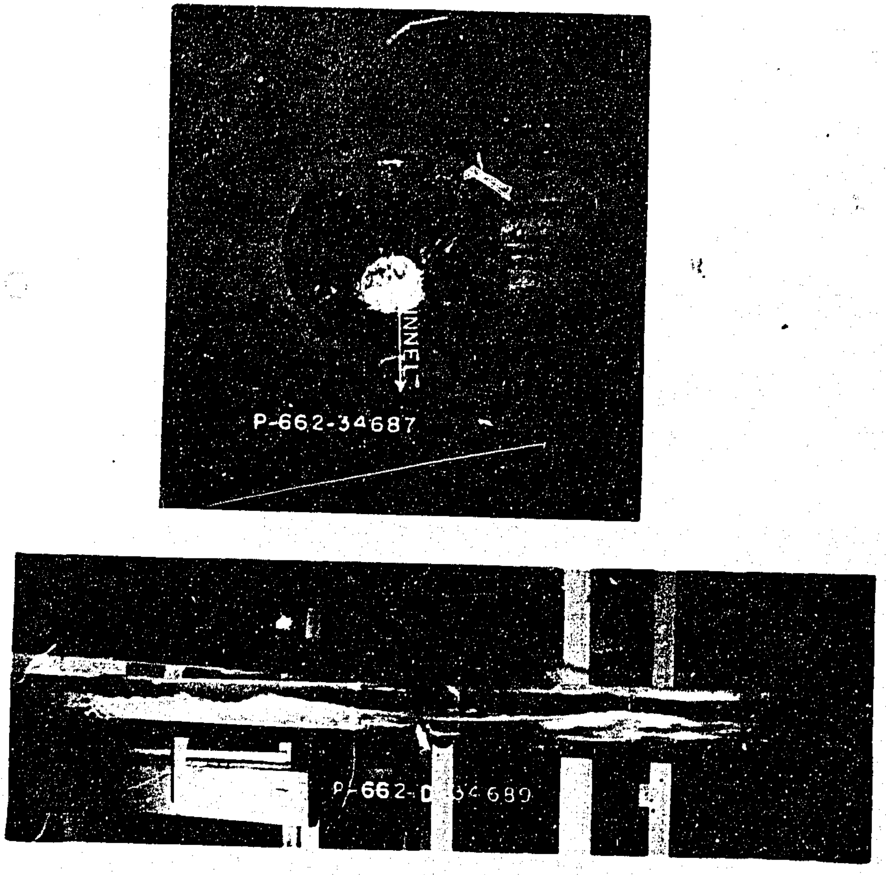


A. Crest flow conditions
Reservoir El. 2969

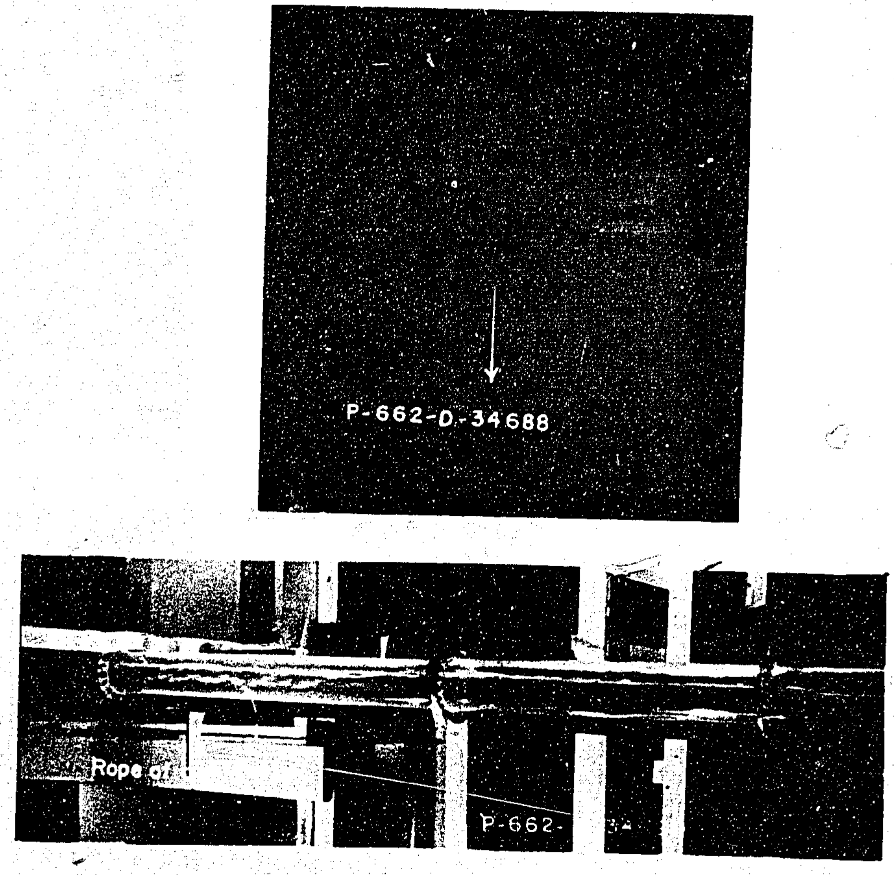


B. Tunnel flow conditions

SANFORD DAM SPILLWAY
RIB VANES TO EL. 2938 - Q = 6,000 CFS
1:46.42 MODEL



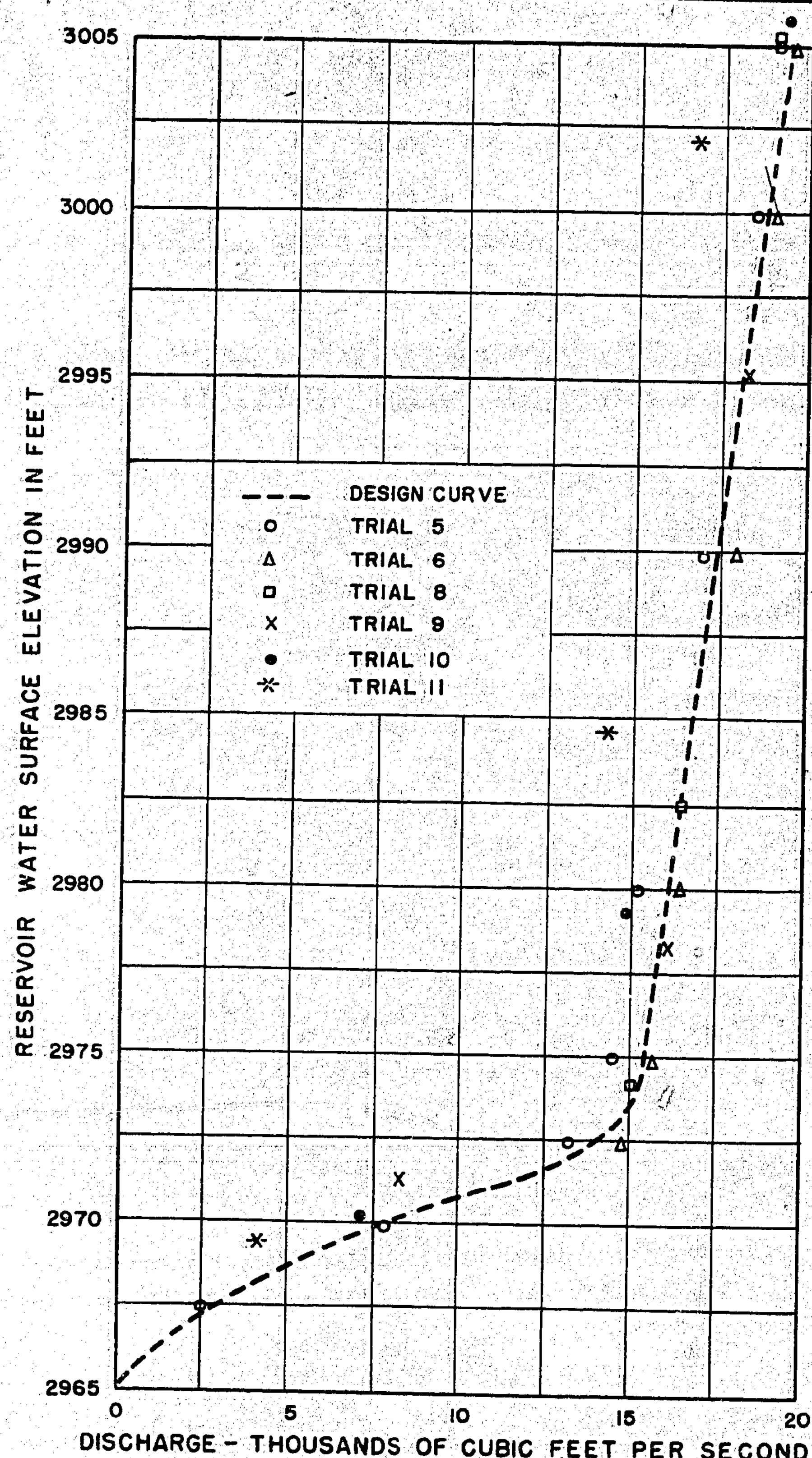
A. Q = 5, 600 CFS, Reservoir El. 2969. 5
(Note swaying action in tunnel)



B. Q = 15, 000 CFS, Reservoir El. 2980. 5

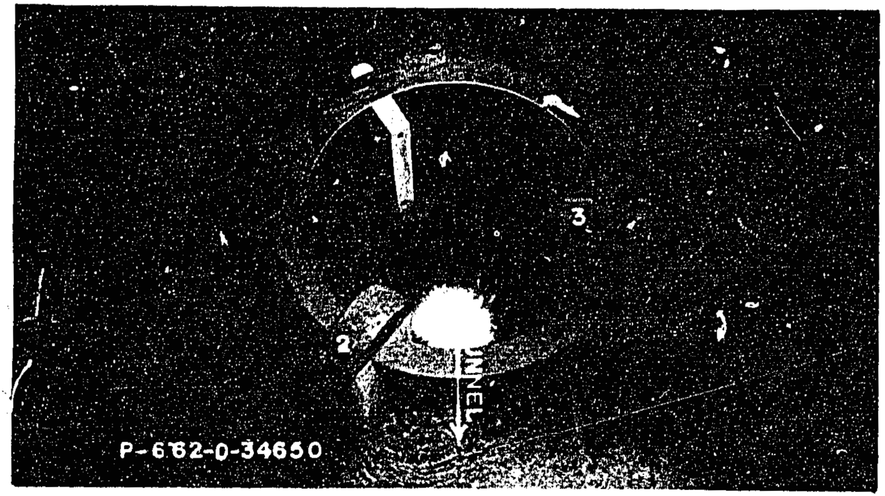
SANFORD DAM SPILLWAY
FLOW CONDITIONS IN ENTRANCE AND TUNNEL
WITH SINGLE PIER ON CREST
1:46. 42 MODEL

Figure 19
Report Hyd-491



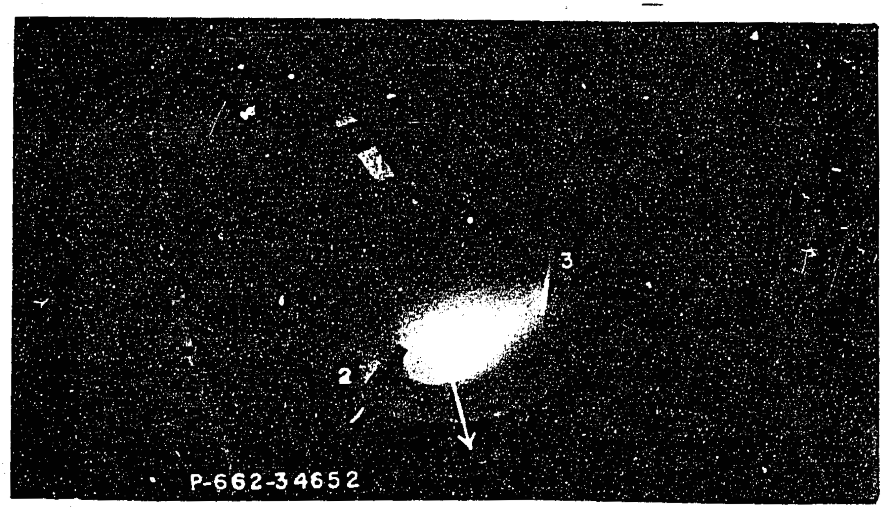
SANFORD DAM SPILLWAY
1:46.42 SCALE MODEL

HEAD - DISCHARGE CURVES — TRIALS 5-11



A. Q = 6,000 CFS, Reservoir El. = 2970

B. Q = 15,000 CFS, Reservoir El. = 2974



C. Q = 19,250 CFS, Reservoir El. = 3006

SANFORD DAM SPILLWAY
FLOW CONDITIONS AT ENTRANCE WITH THREE PIERS ON CREST
1:46.42 MODEL

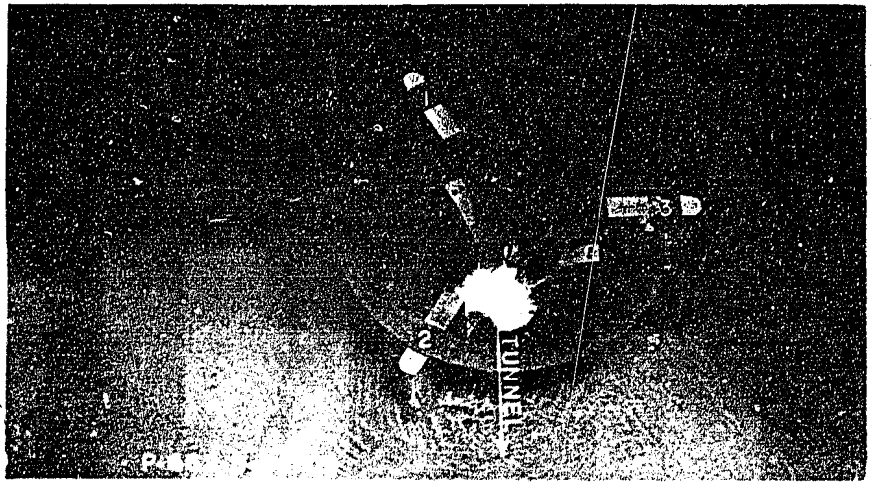
Figure 21
Report Hyd-491

A. Q = 6,000 CFS

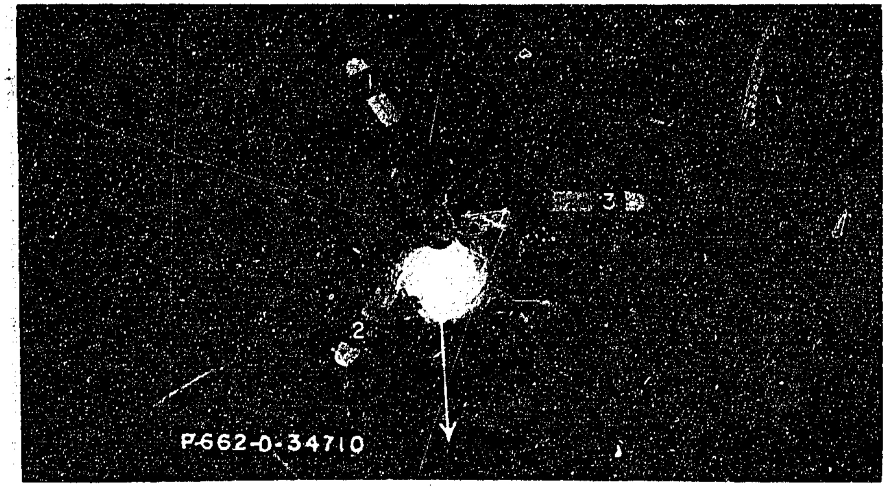
B. Q = 15,000 CFS

C. Q = 19,250 CFS

SANFORD DAM SPILLWAY
TUNNEL FLOW CONDITIONS WITH THREE PIERS ON CREST
1:46.42 MODEL



A. Q = 7,500 CFS, Reservior El. = 2970

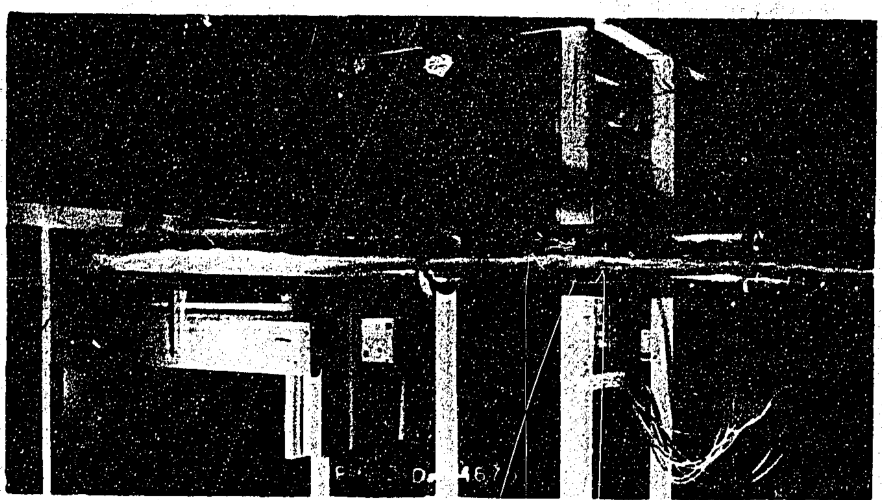


B. Q = 15,500 CFS, Reservoir El. = 2978

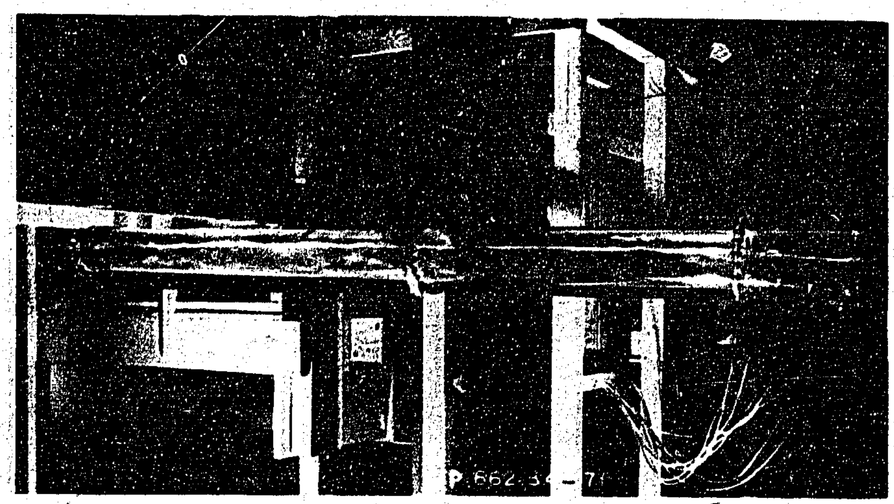
C. Q = 19,250 CFS, Reservoir El. = 3006

SANFORD DAM SPILLWAY
FLOW CONDITIONS AT ENTRANCE WITH EXTENSIONS ATTACHED TO THREE PIERS
1:46.42 MODEL

Figure 23
Report Hyd-491



A. Q = 7,500 CFS

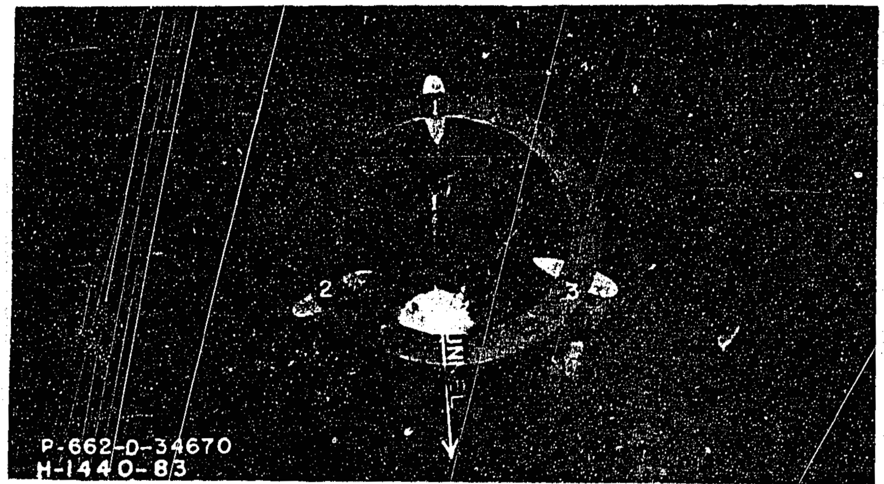


B. Q = 15,500 CFS

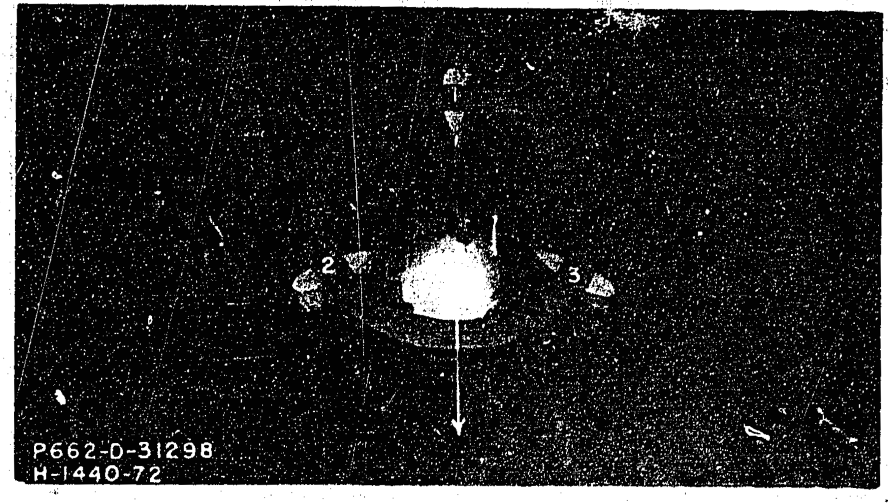
C. Q = 19,250 CFS

SANFORD DAM SPILLWAY
TUNNEL FLOW CONDITIONS WITH EXTENSIONS ATTACHED TO THREE PIERS
1:43.42 MODEL

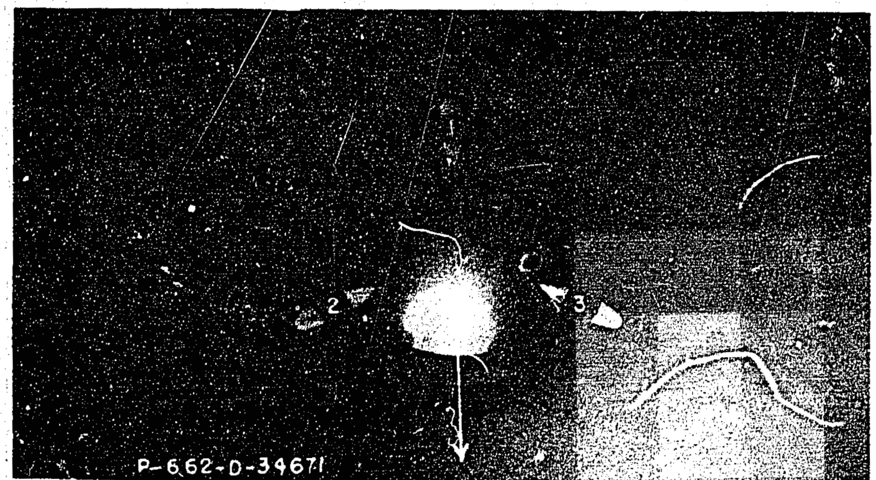
Figure 24
Report Hyd-491



A. Q = 6,000 CFS, Reservoir El. = 2969

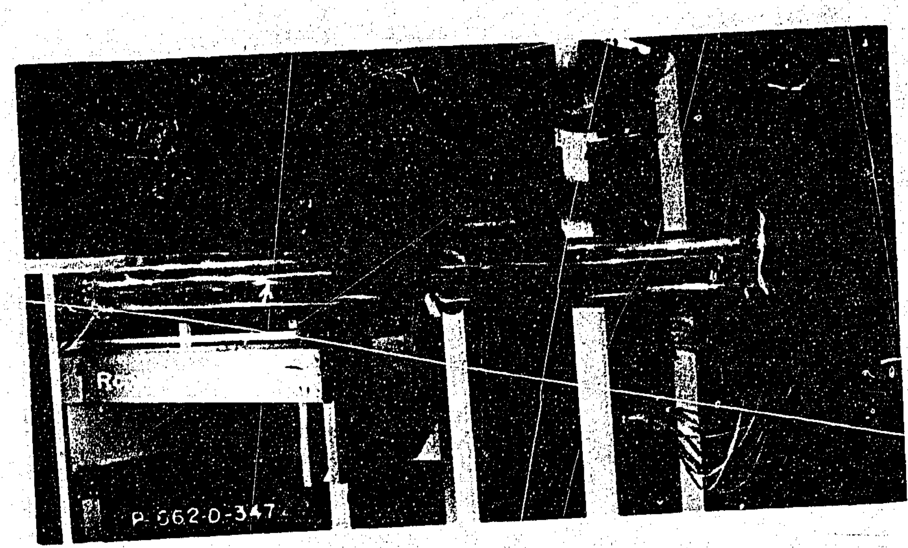


B. Q = 16,500 CFS, Reservoir El. = 2981



C. Q = 19,250 CFS, Reservoir El. = 3002

SANFORD DAM SPILLWAY
FLOW CONDITIONS AT ENTRANCE WITH THREE STREAMLINED PIERS AND EXTENSIONS
1:46.42 MODEL



A. Q = 16,500 CFS

B. Q = 19,250 CFS

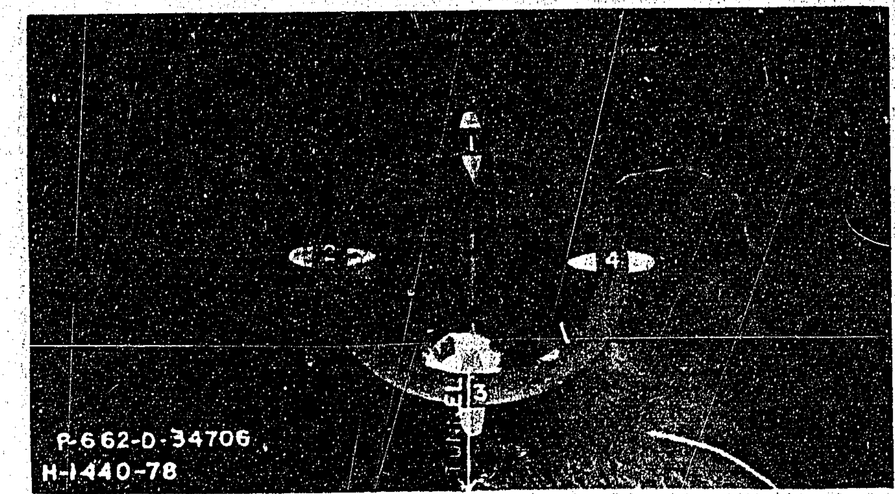
Note: Flow conditions for Q = 6,000 CFS are similar to those for 7,500 CFS with three unstreamlined piers and extensions

SANFORD DAM SPILLWAY
TUNNEL FLOW CONDITIONS WITH THREE
STREAMLINED PIERS AND EXTENSIONS
1:46.42 MODEL

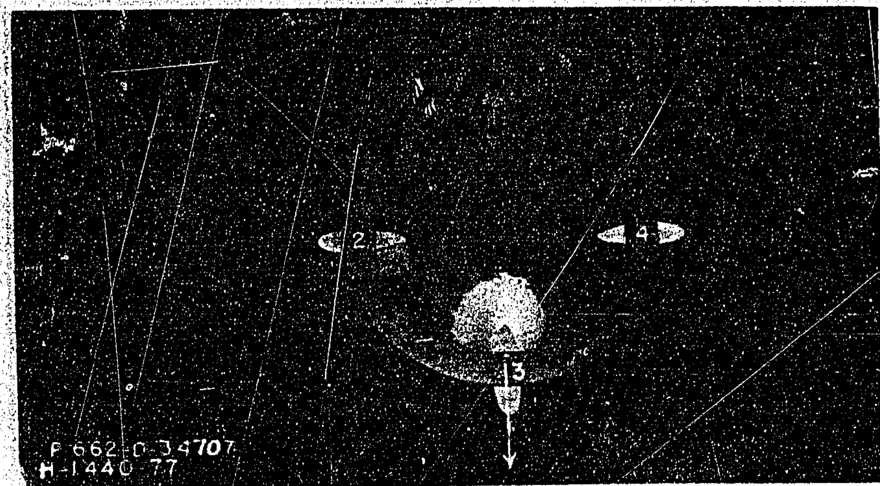
SANFORD DAM SPILLWAY
FLOW CONDITIONS AT ENTRANCE WITH
RIP RAPPED EMBANKMENT

Q = 15,500 CFS
Reservoir El. = 2978

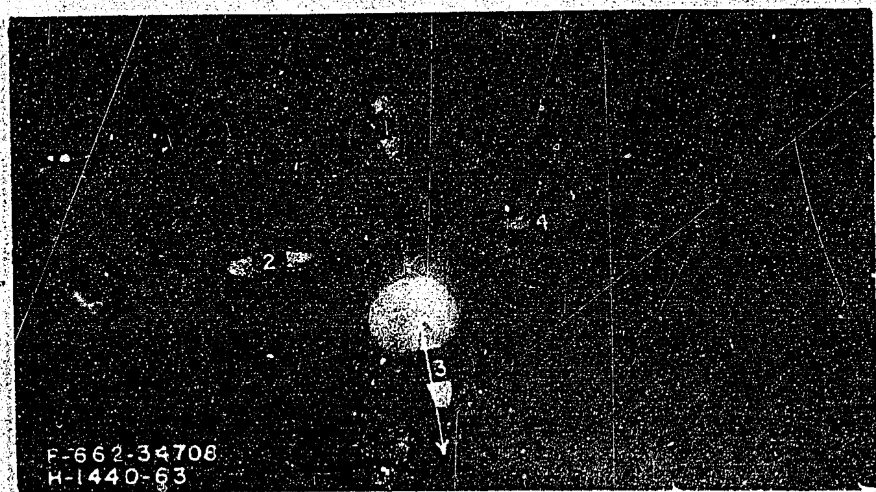
1:46.42 MODEL



A. Q = 6,000 CFS, Reservoir El. = 2969



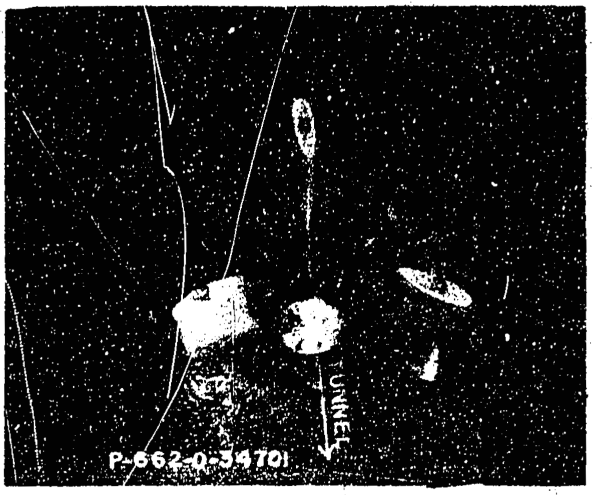
B. Q = 16,500 CFS, Reservoir El. = 2981



C. Q = 19,250 CFS, Reservoir El. = 3002

SANFORD DAM SPILLWAY
FLOW CONDTIONS AT ENTRANCE WITH FOUR STREAMLINED PIERS AND EXTENSIONS
1:46.42 MODEL

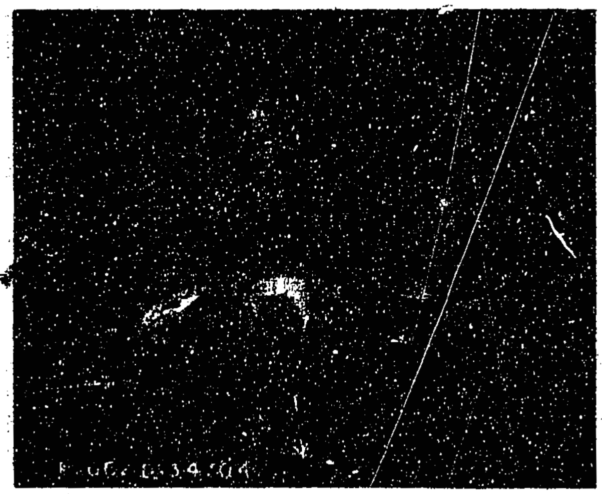
Figure 28
Report Hyd-491



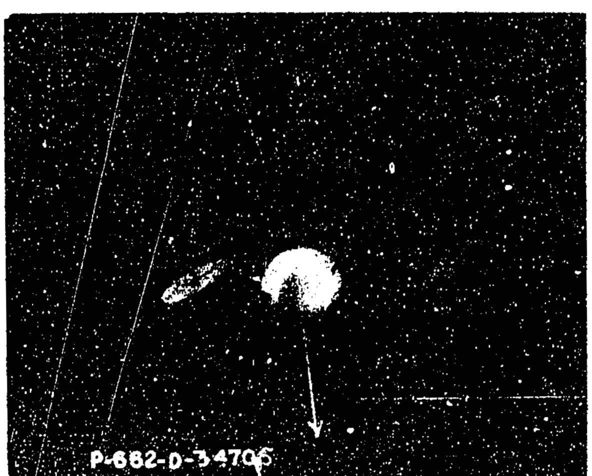
A. Q = 7,100 CFS,
Reservoir El. 2970.1

B. Q = 14,500 CFS,
Reservoir El. 2972.9

C. Q = 15,100 CFS,
Reservoir El. 2975.1

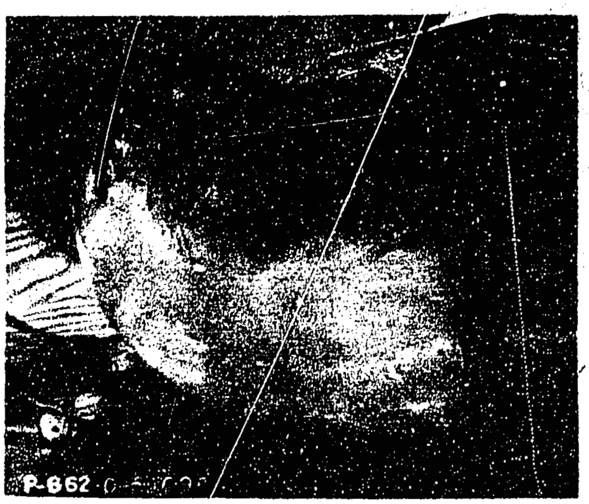


D. Q = 17,200 CFS,
Reservoir El. 2990.9



E. Q = 19,250 CFS,
Reservoir El. 3005.5

SANFORD DAM SPILLWAY
RECOMMENDED DESIGN
MORNING-GLORY PERFORMANCE
1:46.42 MODEL

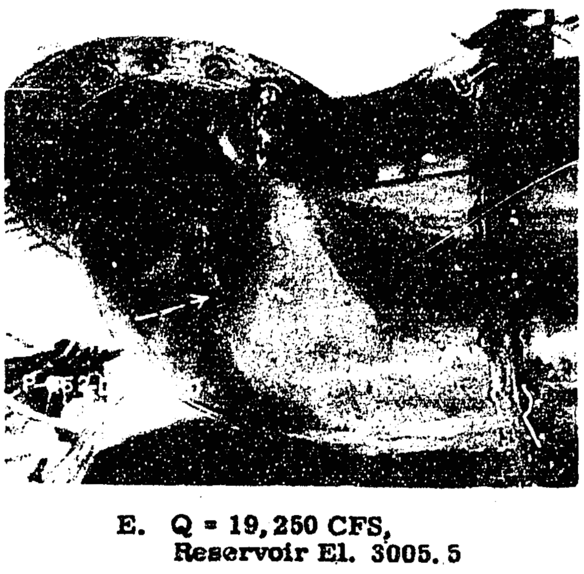


A. Q = 7,100 CFS,
Reservoir El. 2970.1

B. Q = 14,500 CFS,
Reservoir El. 2972.9

C. Q = 15,100 CFS,
Reservoir El. 2975.1

D. Q = 17,200 CFS,
Reservoir El. 2990.9



E. Q = 19,250 CFS,
Reservoir El. 3005.5

SANFORD DAM SPILLWAY
RECOMMENDED DESIGN
FLOW IN THE VERTICAL BEND
1:46.42 MODEL

A. Q = 7,100 CFS, Reservoir El. 2970.1

B. Q = 14,500 CFS, Reservoir El. 2972.9

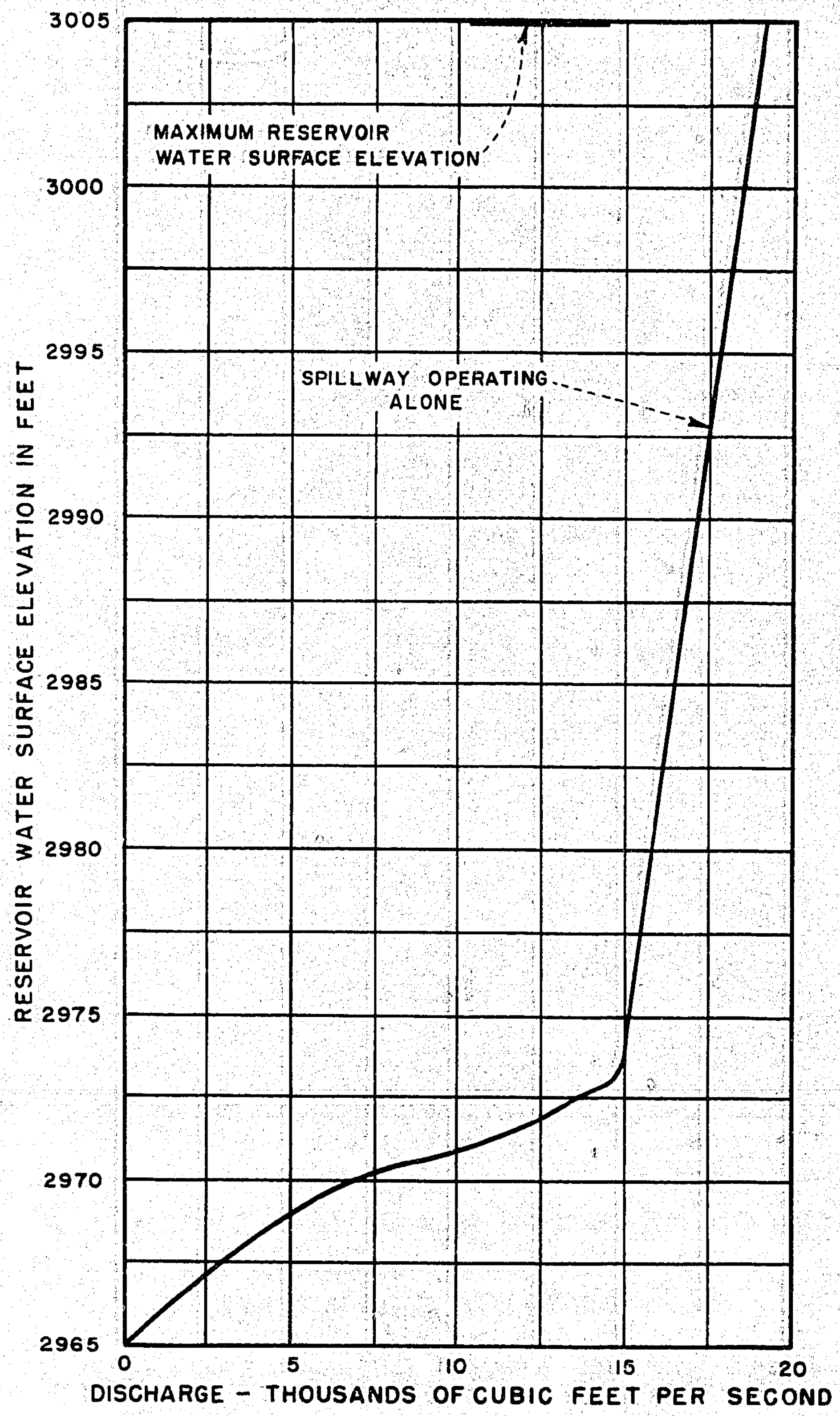
C. Q = 15,100 CFS, Reservoir El. 2975.1

D. Q = 17,200 CFS, Reservoir El. 2980.9

E. Q = 19,250 CFS, Reservoir El. 3005.5

SANFORD DAM SPILLWAY
RECOMMENDED DESIGN
TUNNEL FLOW
1:46.42 MODEL

Figure 31
Report Hyd-481



SANFORD DAM SPILLWAY
1:46.42 SCALE MODEL
HEAD - DISCHARGE CURVE FOR RECOMMENDED DESIGN

FIGURE 33
REPORT HYD. 491

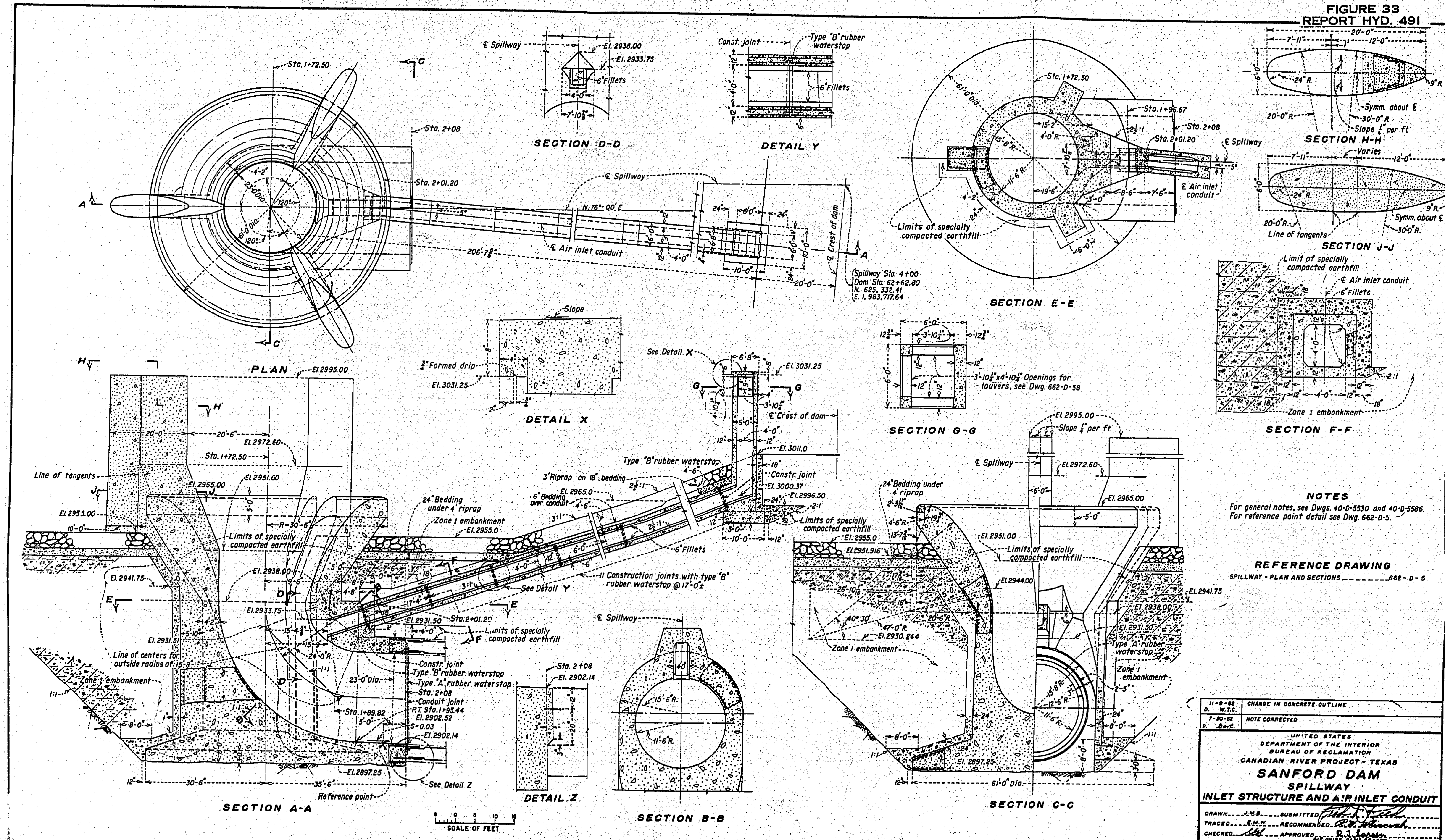
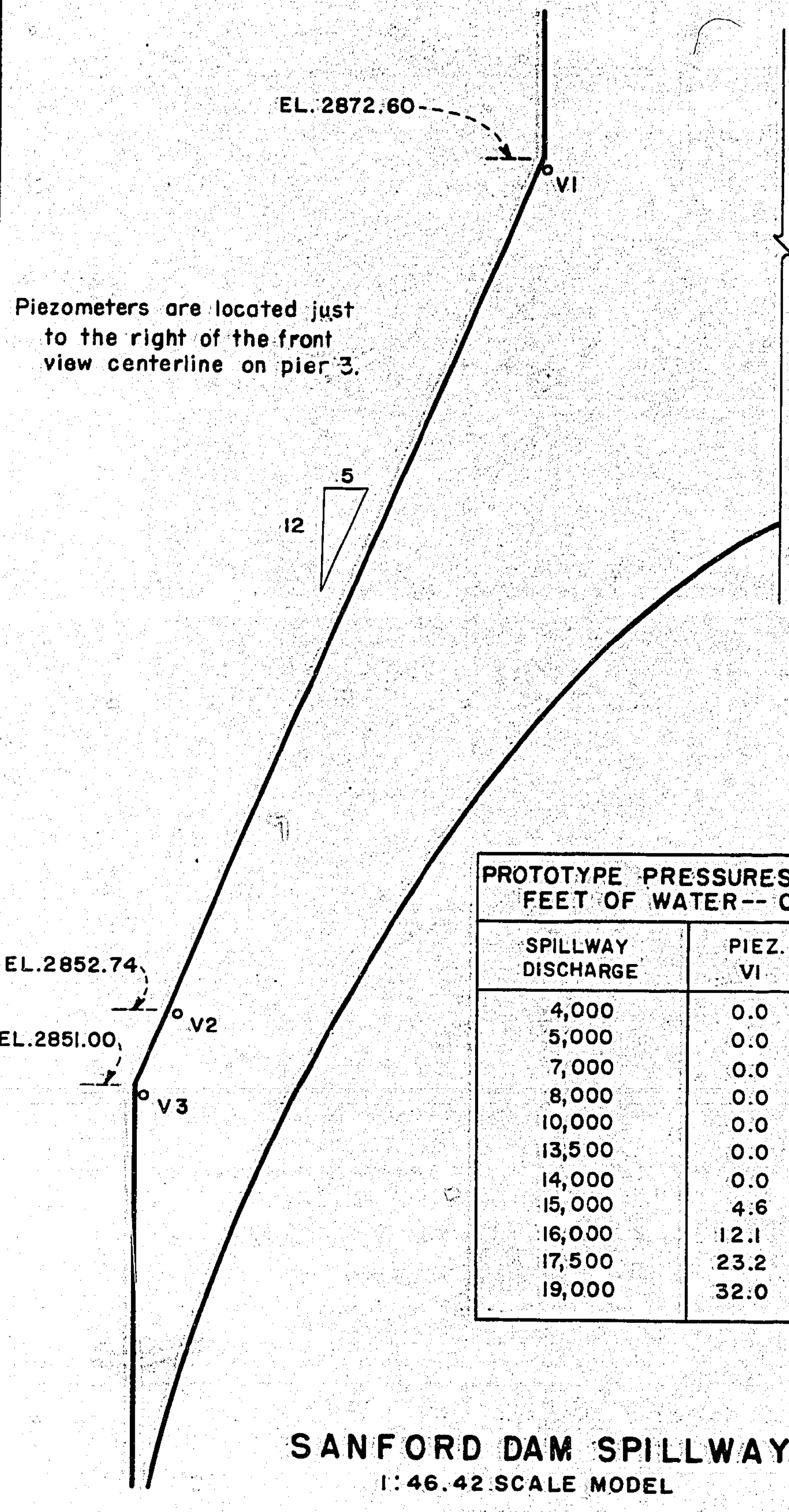


FIGURE 34
REPORT HYD. 491



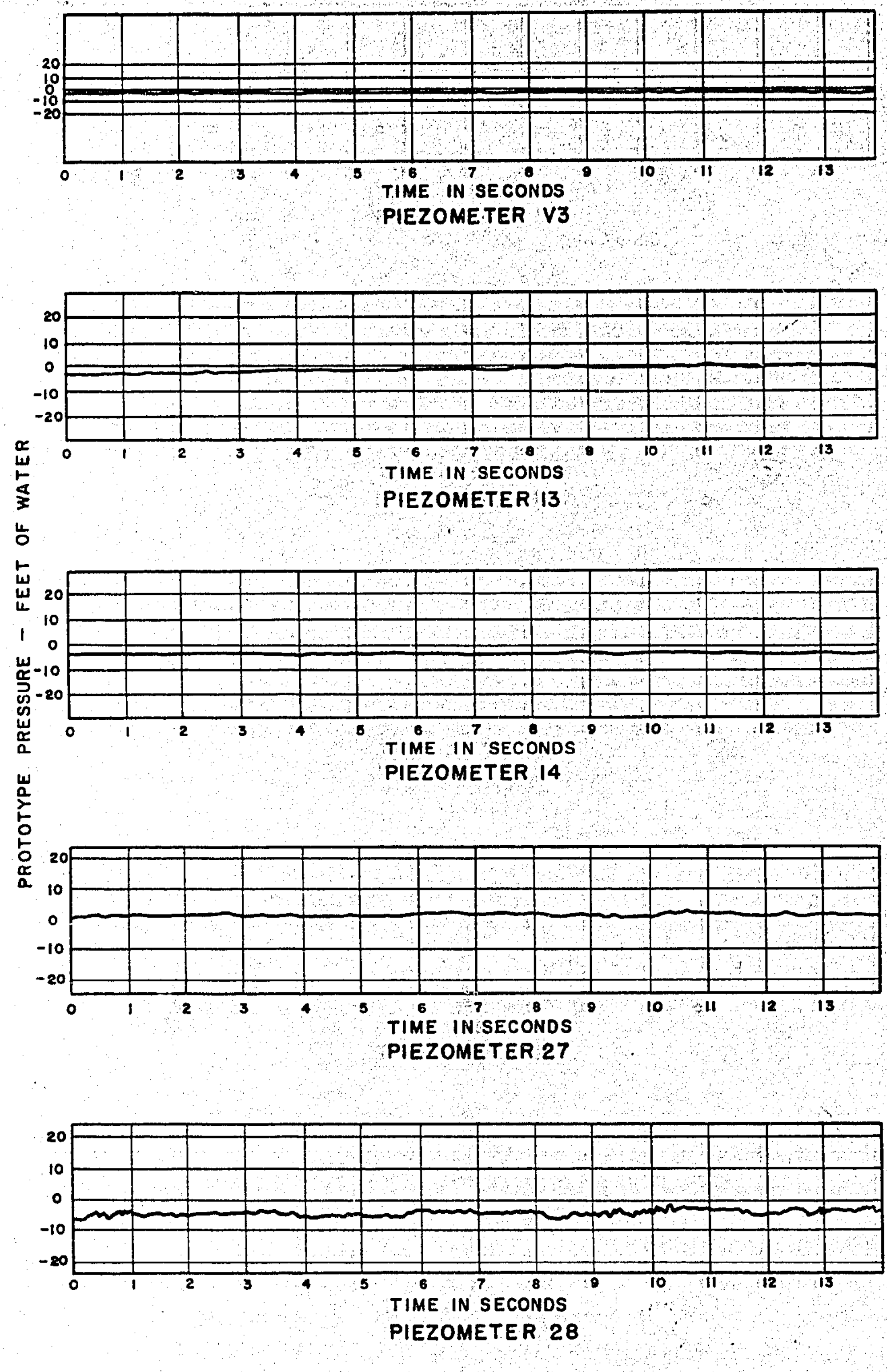
PROTOTYPE PRESSURES ON PIER EXTENSIONS FEET OF WATER-- COMBINED OPERATION			
SPILLWAY DISCHARGE	PIEZ. V1	PIEZ. V2	PIEZ. V3
4,000	0.0	0.0	0.0
5,000	0.0	0.0	-0.9
7,000	0.0	0.0	-0.9
8,000	0.0	0.0	-0.9
10,000	0.0	0.0	-0.9
13,500	0.0	0.0	0.9
14,000	0.0	10.7	10.7
15,000	4.6	18.1	17.2
16,000	12.1	24.6	23.2
17,500	23.2	34.4	32.5
19,000	32.0	41.3	38.5

SANFORD DAM SPILLWAY

1:46.42 SCALE MODEL

PRESSURES ON THE PIER EXTENSIONS

FIGURE 35
REPORT HYD. 491

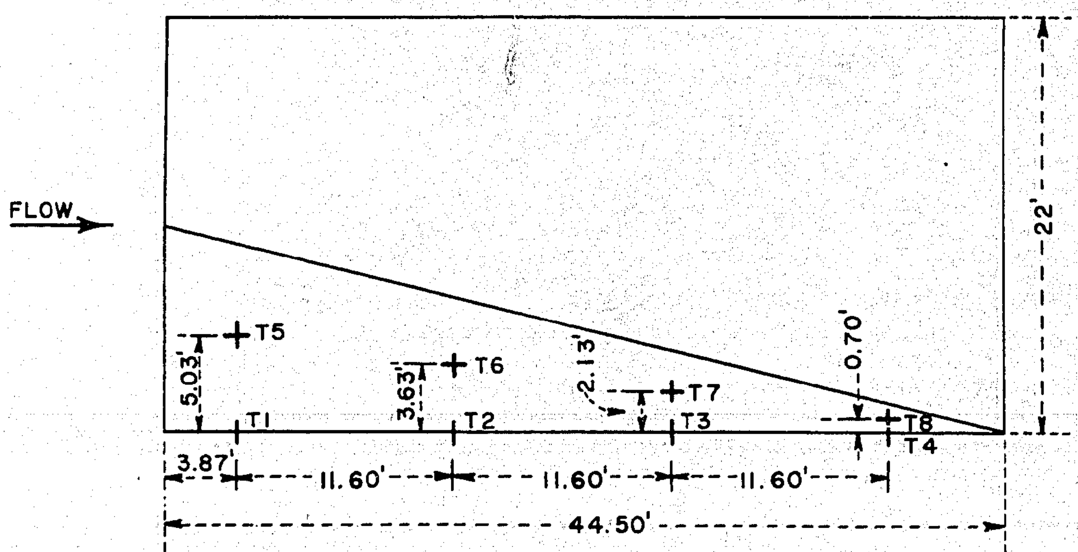


SANFORD DAM SPILLWAY

1:46.42 SCALE MODEL

RECORDS OF INSTANTANEOUS DYNAMIC PRESSURES ON THE SPILLWAY CREST PROFILE

PROTOTYPE PRESSURES IN THE SPILLWAY TUNNEL ROUND-TO-HORSESHOE TRANSITION--FEET OF WATER				
PIEZOMETER NO.	Q = 5,000	Q = 10,000	Q = 15,000	Q = 19,300
T1	5.1	7.9	8.8	8.4
T2	5.1	9.3	12.1	12.1
T3	6.0	10.7	14.4	15.8
T4	9.8	14.4	15.8	17.6
T5	0.9	3.7	4.6	3.7
T6	1.9	6.5	9.3	9.8
T7	4.2	8.8	12.1	13.5
T8	5.1	9.8	13.5	14.9

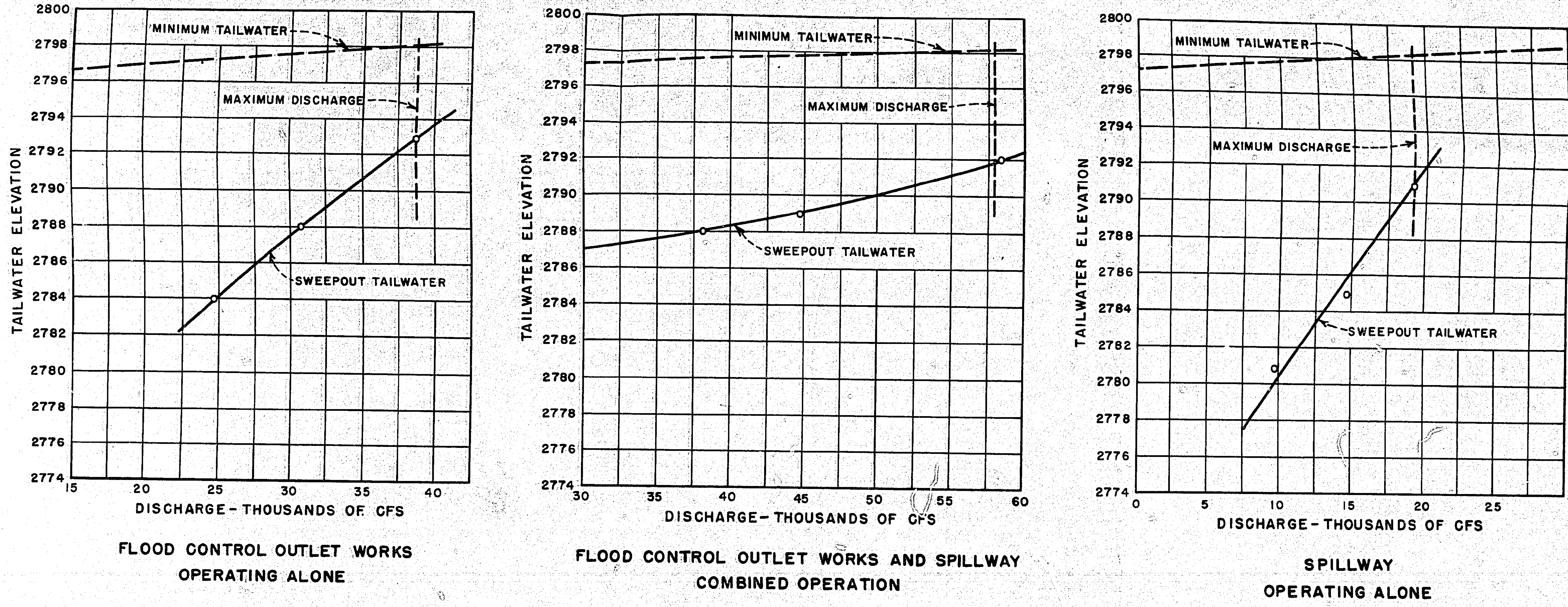


ELEVATION SHOWING PIEZOMETER LOCATIONS

SANFORD DAM SPILLWAY

1:46.42 SCALE MODEL

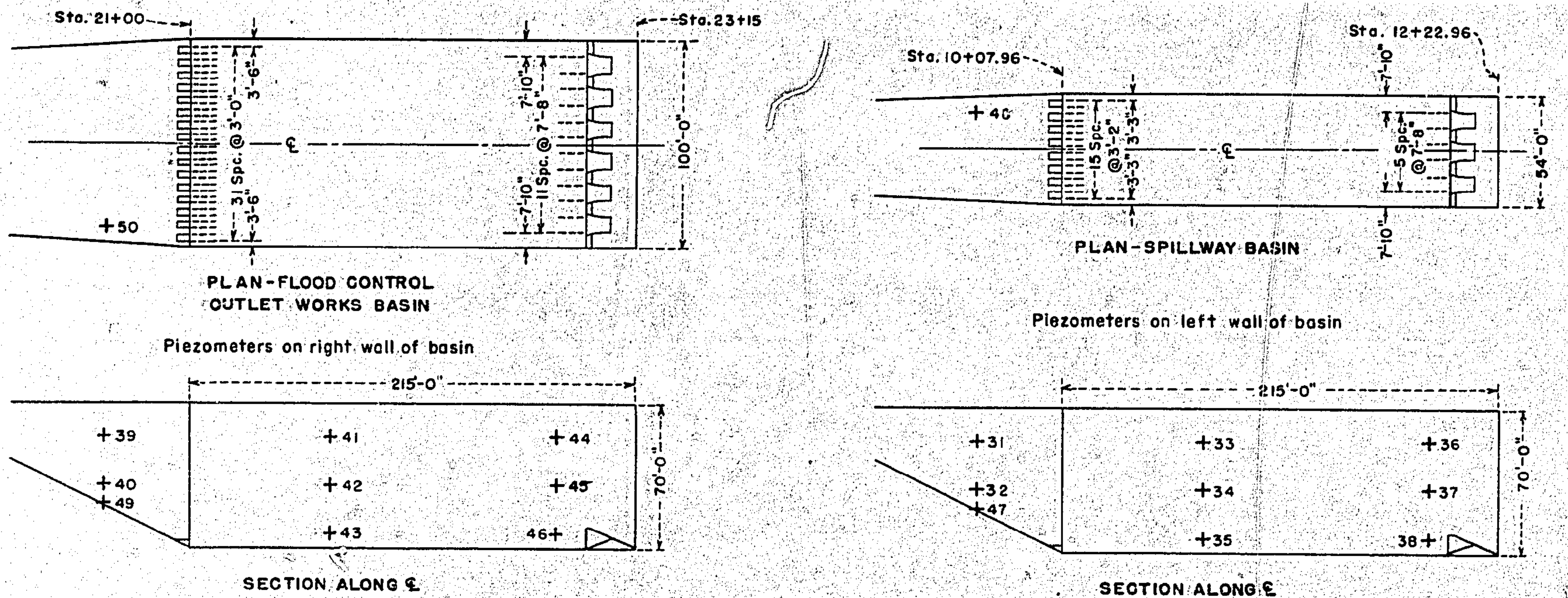
PRESSURES IN THE SPILLWAY TUNNEL
ROUND-TO-HORSESHOE TRANSITION



SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS

1:46.42 SCALE MODEL

STILLING BASIN TAILWATER SWEEPOUT TESTS



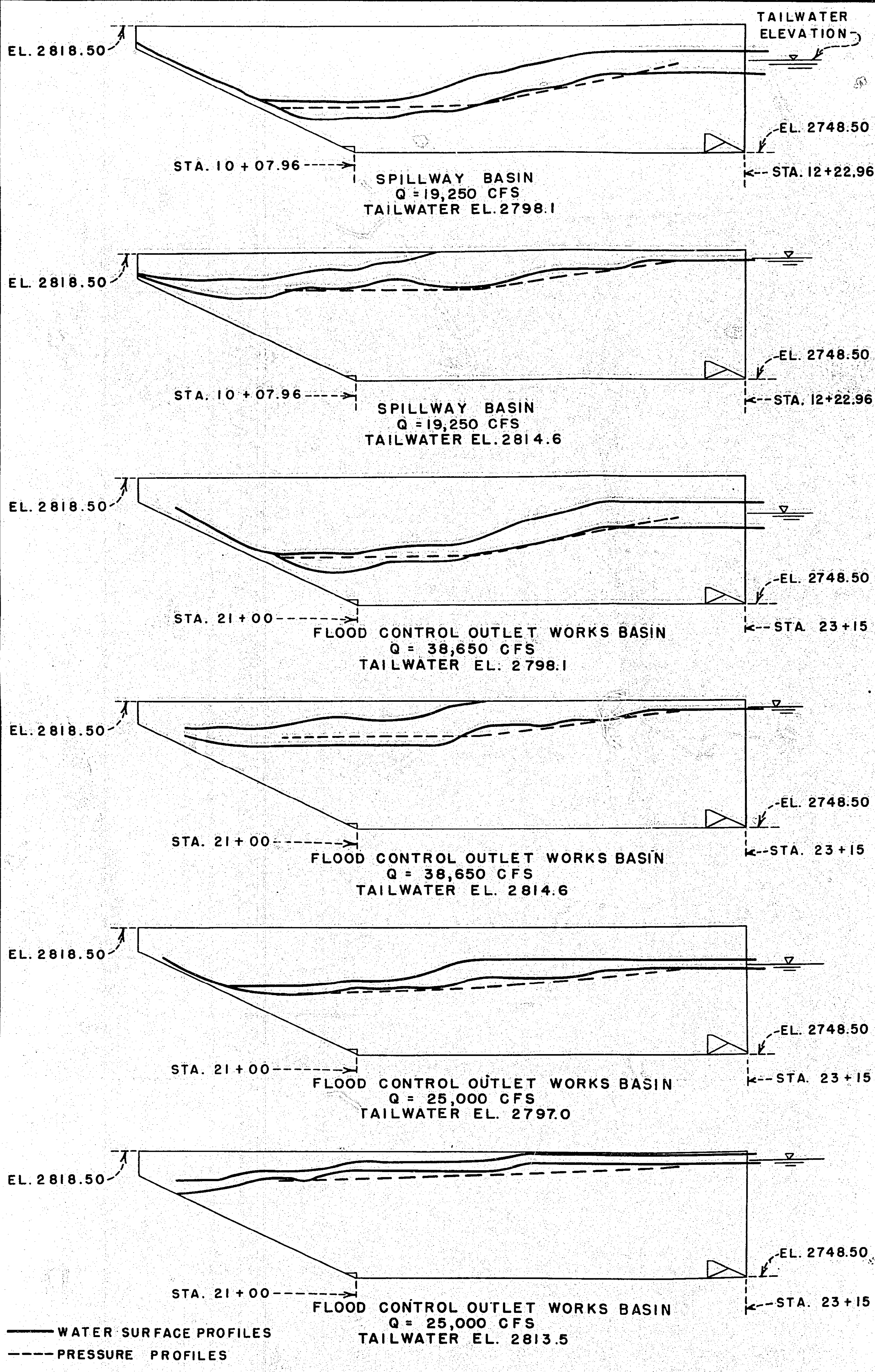
STATIC PROTOTYPE STILLING BASIN PRESSURES - FEET OF WATER				
PIEZOMETER NO.	Q=25,000 C.F.S.		Q=57,900 C.F.S.	
	TAILWATER ELEV. 2797.0	TAILWATER ELEV. 2813.5	TAILWATER ELEV. 2798.1	TAILWATER ELEV. 2814.6
31	—	—	0.0	0.4
32	—	—	0.0	19.0
33	—	—	0.0	3.5
34	—	—	3.9	25.1
35	—	—	17.4	41.8
36	—	—	0.8	9.7
37	—	—	17.0	32.1
38	—	—	39.9	66.1
39	0.4	3.1	2.7	3.1
40	1.9	23.2	0.0	16.2
41	0.0	5.8	0.8	5.4
42	10.1	26.3	3.9	24.4
43	29.4	49.2	18.2	43.3
44	0.0	10.1	1.5	12.0
45	14.7	30.2	16.2	31.7
46	38.7	55.0	39.5	56.5
47	—	—	7.0	31.3
48	—	—	5.8	28.2
49	16.2	35.6	8.0	30.9
50	11.2	32.5	5.9	22.8

SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS

1:46.42 SCALE MODEL

PIEZOMETER LOCATIONS AND STATIC PROTOTYPE STILLING BASIN PRESSURES

FIGURE 38
REPORT HYD. 491

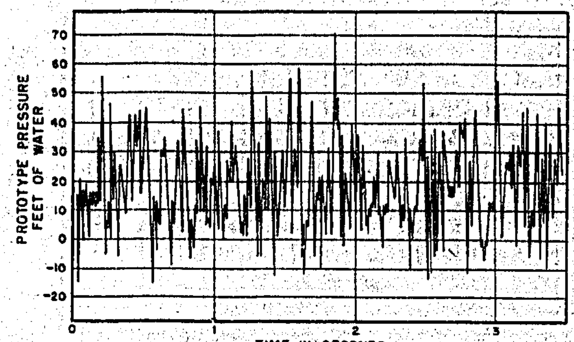


SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS

1:46.42 SCALE MODEL

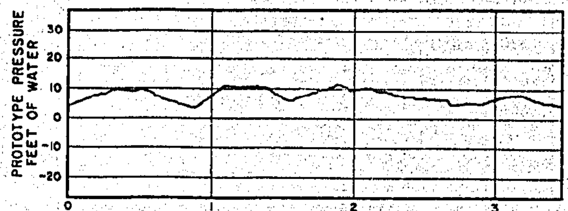
STILLING BASIN WATER SURFACE AND PRESSURE PROFILES

FIGURE 39
REPORT HYD. 491



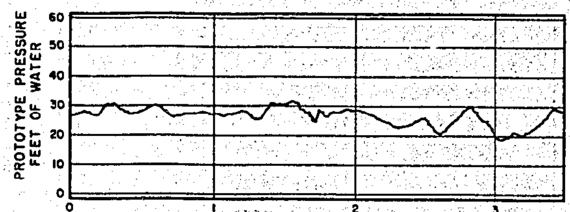
PIEZOMETER 40

Q_T = 57,900 CFS; TAILWATER EL. 2814.6



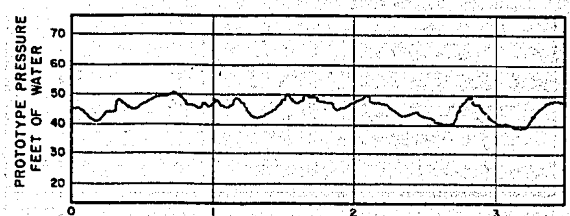
PIEZOMETER 41

Q_T = 57,900 CFS; TAILWATER EL. 2814.6



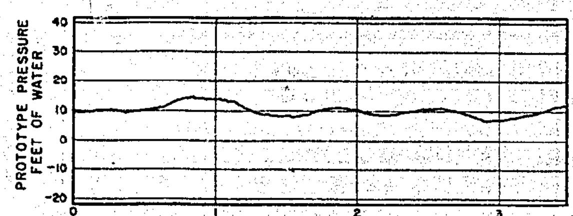
PIEZOMETER 42

Q_T = 57,900 CFS; TAILWATER EL. 2814.6



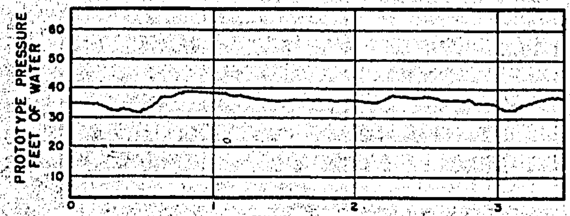
PIEZOMETER 43

Q_T = 57,900 CFS; TAILWATER EL. 2814.6



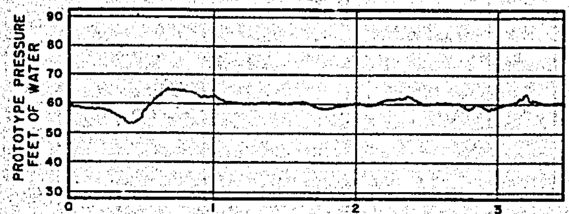
PIEZOMETER 44

Q_T = 57,800 CFS; TAILWATER EL. 2814.6



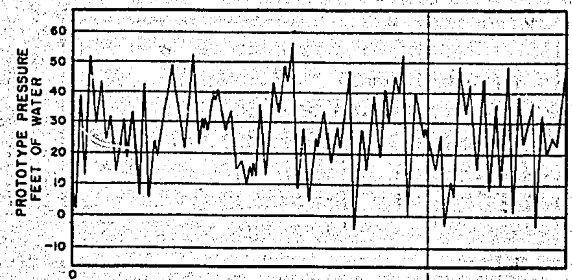
PIEZOMETER 45

Q_T = 57,900 CFS; TAILWATER EL. 2814.6



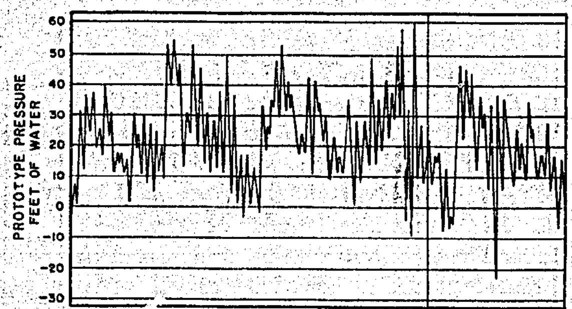
PIEZOMETER 46

Q_T = 57,900 CFS; TAILWATER EL. 2814.6



PIEZOMETER 49

Q_T = 57,900 CFS; TAILWATER EL. 2814.6



PIEZOMETER 50

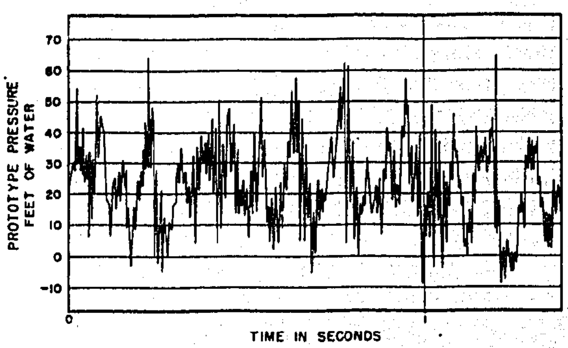
Q_T = 57,900 CFS; TAILWATER EL. 2814.6

SANFORD DAM FLOOD CONTROL OUTLET WORKS

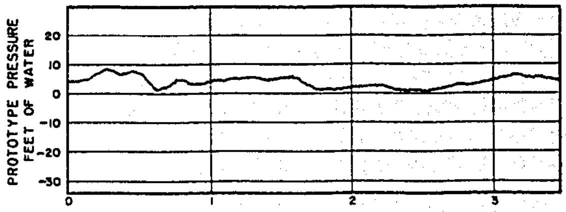
1:46.42 SCALE MODEL

RECORDS OF DYNAMIC PRESSURES ON THE STILLING BASIN RIGHT TRAINING WALL

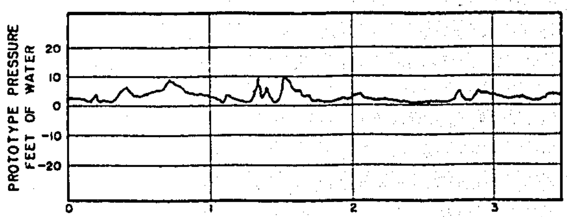
See Figure 38 for Piezometer Locations



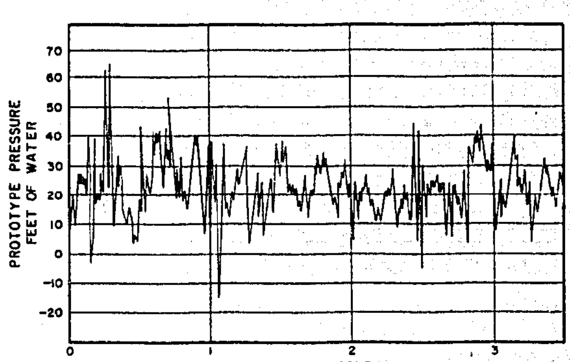
TIME IN SECONDS
PIEZOMETER 32
Q_T = 57,900 CFS; TAILWATER EL. 2814.6



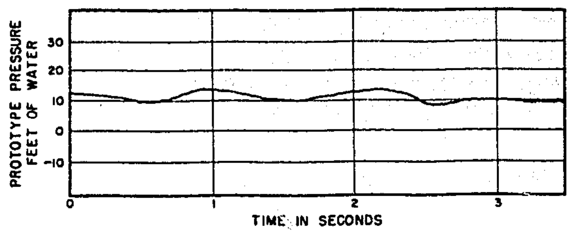
TIME IN SECONDS
PIEZOMETER 33
Q_T = 57,900 CFS; TAILWATER EL. 2814.6



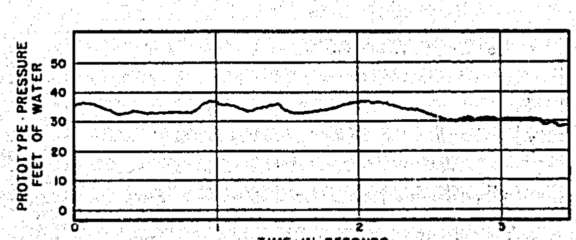
TIME IN SECONDS
PIEZOMETER 34
Q_T = 57,900 CFS; TAILWATER EL. 2798.1



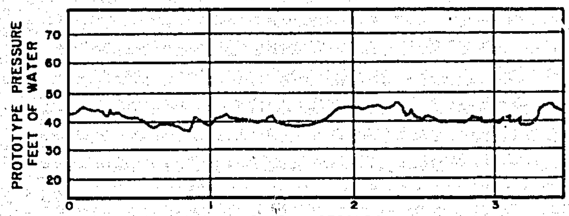
TIME IN SECONDS
PIEZOMETER 35
Q_T = 57,900 CFS; TAILWATER EL. 2798.1



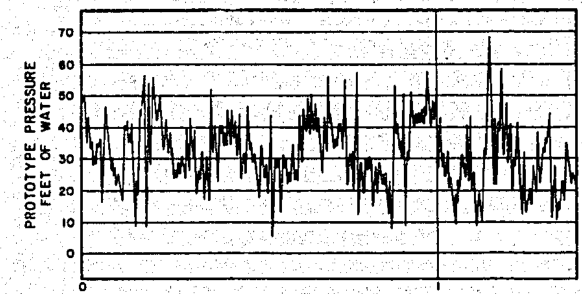
TIME IN SECONDS
PIEZOMETER 36
Q_T = 57,900 CFS; TAILWATER EL. 2814.6



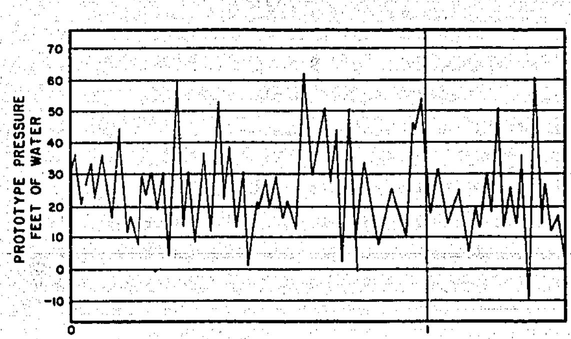
TIME IN SECONDS
PIEZOMETER 37
Q_T = 57,900 CFS; TAILWATER EL. 2814.6



TIME IN SECONDS
PIEZOMETER 38
Q_T = 57,900 CFS; TAILWATER EL. 2798.1



TIME IN SECONDS
PIEZOMETER 47
Q_T = 57,900 CFS; TAILWATER EL. 2814.6



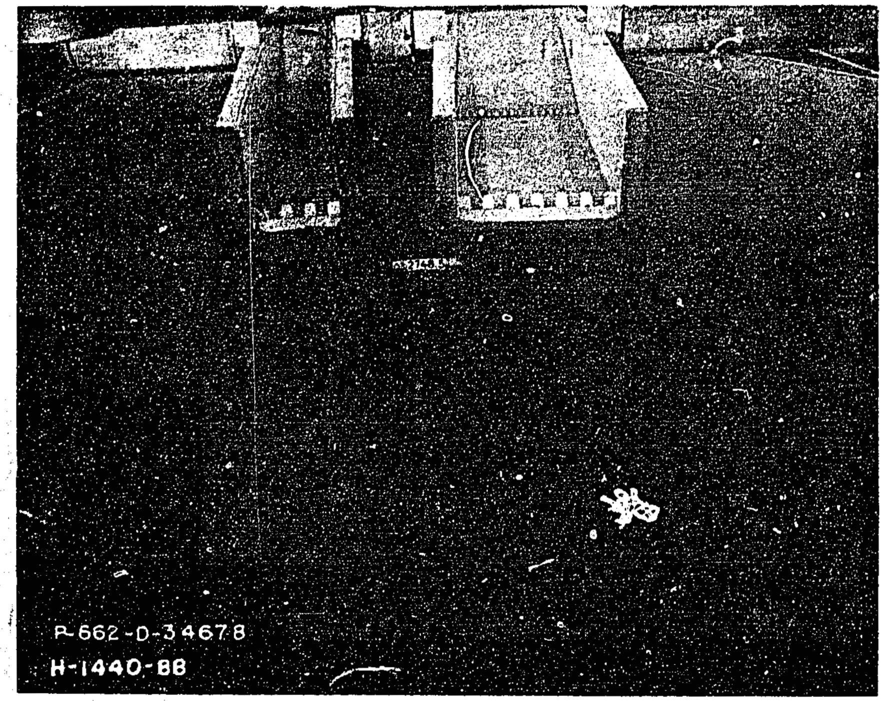
TIME IN SECONDS
PIEZOMETER 48
Q_T = 57,900 CFS; TAILWATER EL. 2814.6

SANFORD DAM SPILLWAY

1:46.42 SCALE MODEL

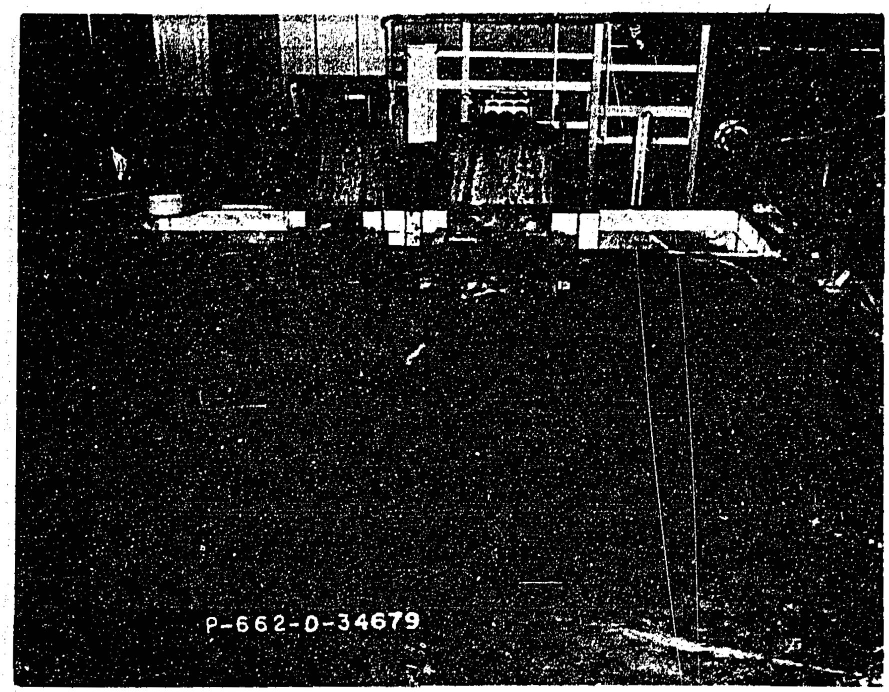
RECORDS OF DYNAMIC PRESSURES ON THE STILLING BASIN LEFT TRAINING WALL

See Figure 38 for Piezometer Locations



Downstream channel with river sand
before erosion tests

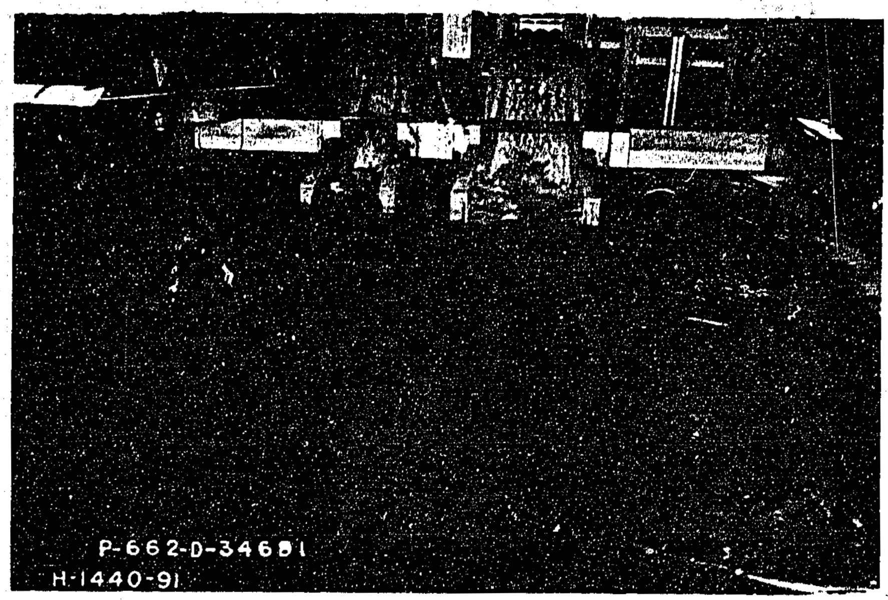
SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS
STILLING BASIN STUDIES
EROSION TESTS
1:46.42 MODEL



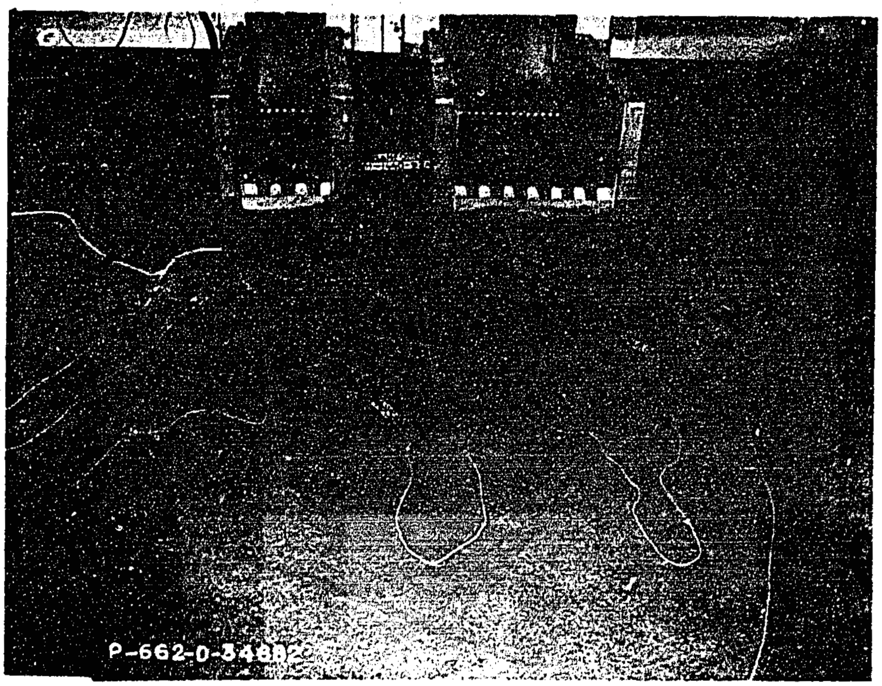
A. Downstream channel. Q = 57,900
Tailwater Elevation 2814.6

B. Downstream channel after one hour operation with flow shown in A

SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS
STILLING BASIN STUDIES
EROSION TESTS
1:46.42 MODEL

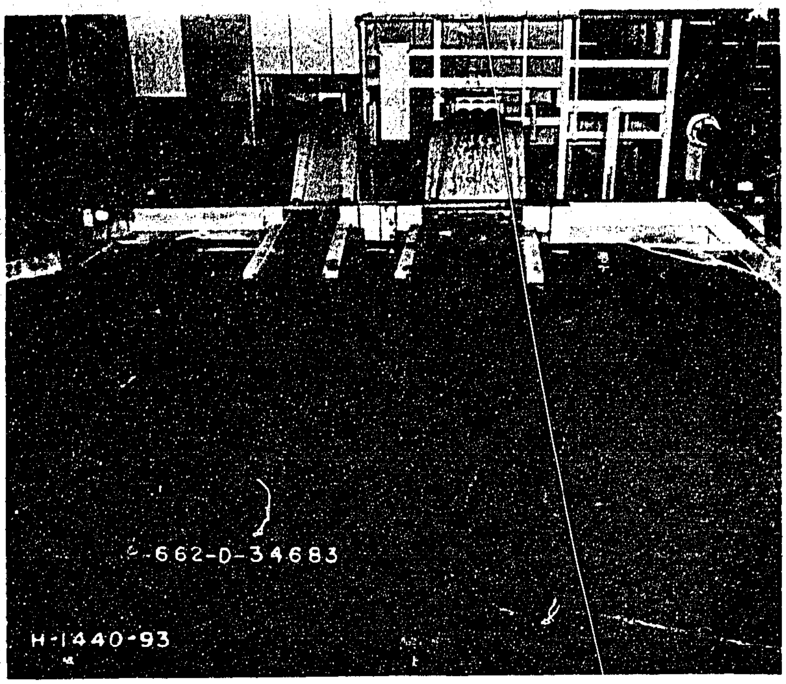


A. Downstream channel. Q = 57, 900
Tailwater Elevation 2798. 1



B. Downstream channel after one hour
operation with flow shown in A

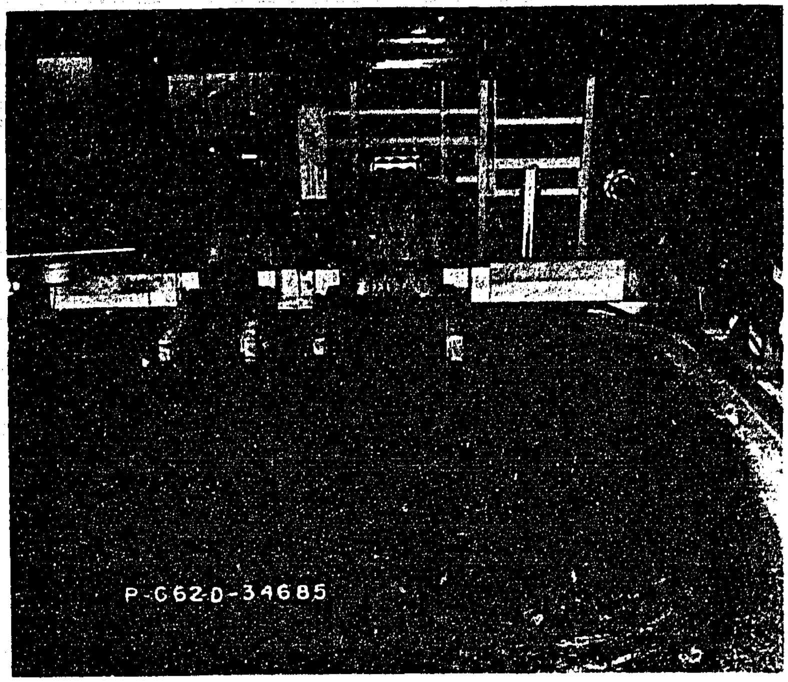
SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS
STILLING BASIN STUDIES
EROSION TESTS
1:46. 42 MODEL



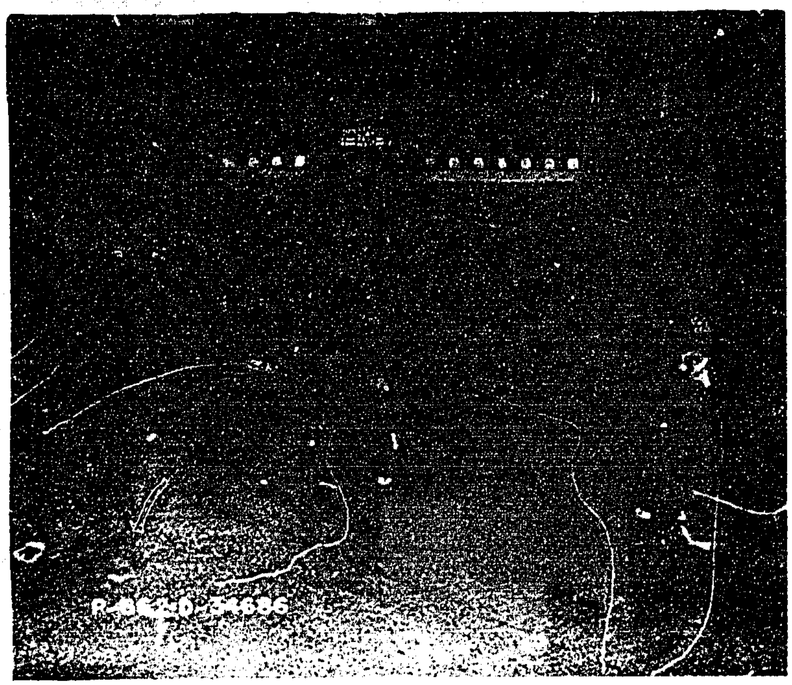
A. Downstream channel. Q = 25,000
Tailwater Elevation 2813.5

B. Downstream channel after one hour
operation with flow shown in A

SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS
STILLING BASIN STUDIES
EROSION TESTS
1:46.42 MODEL

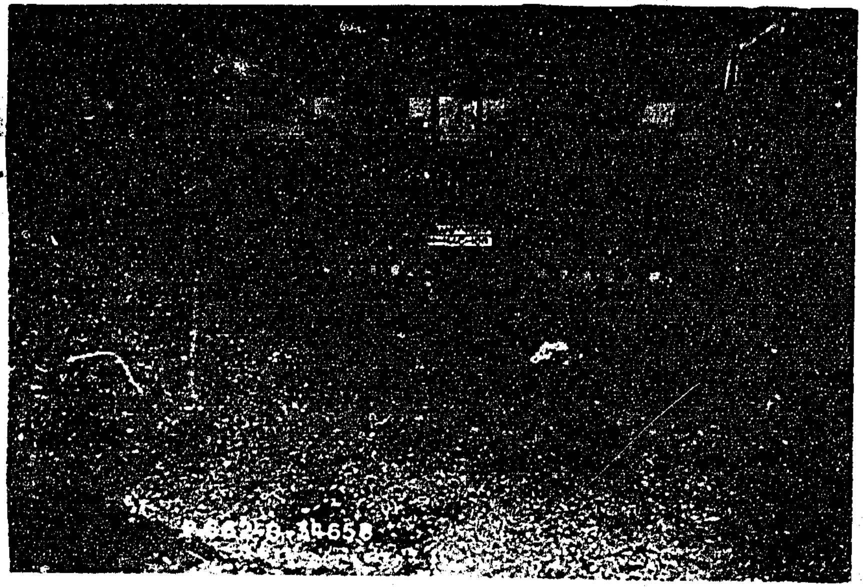


A. Downstream channel. Q = 25,000
Tailwater Elevation 2797.0

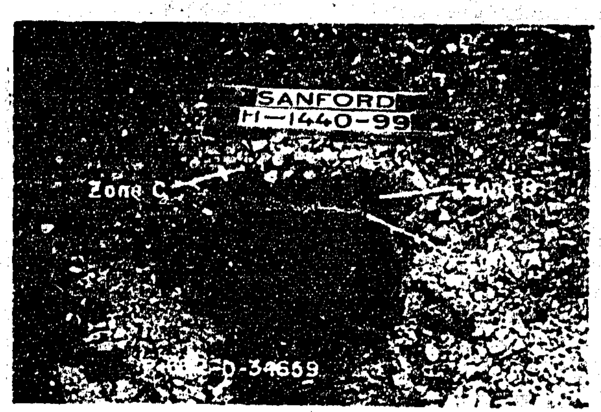


B. Downstream channel after one hour operation with flow shown in A

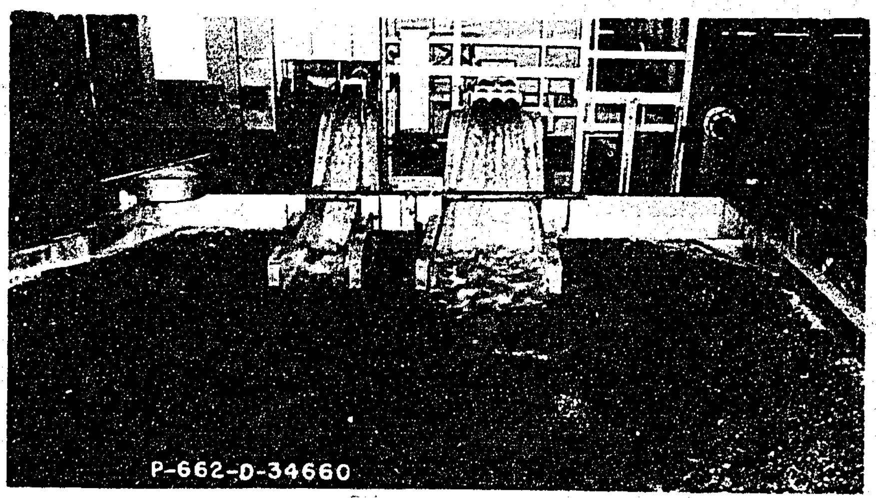
SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS
STILLING BASIN STUDIES
EROSION TESTS
1:46.42 MODEL



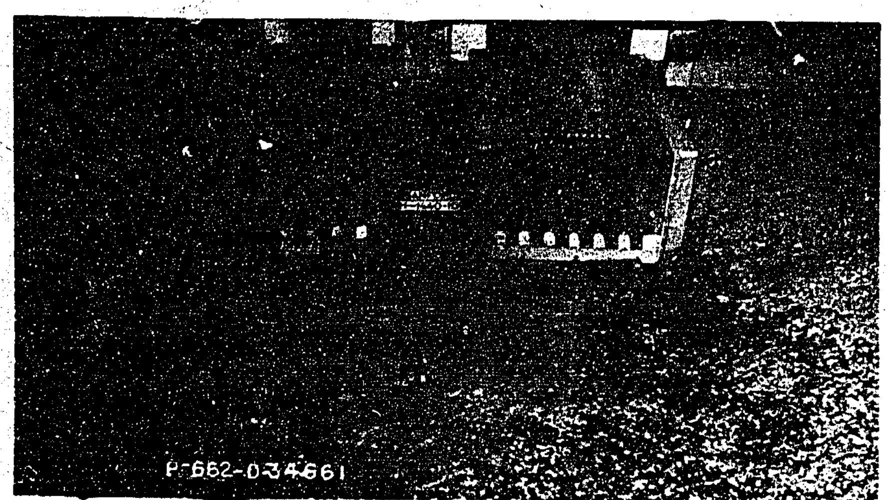
A. Riprap in downstream channel before erosion



B. Section showing filter, bedding material, and riprap



C. Downstream channel. Q = 57, 900 CFS Tailwater Elevation 2798. 1

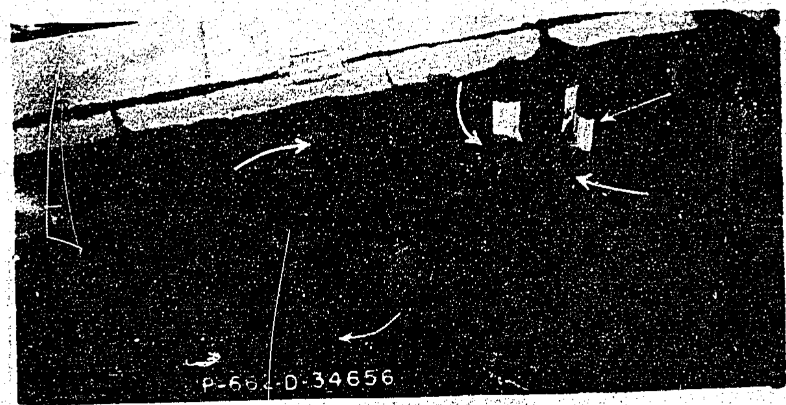


D. Downstream channel after three hours operation with flow shown in C

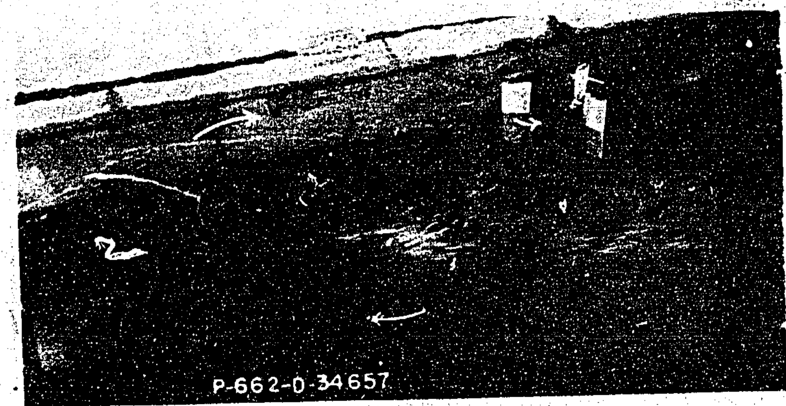
SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS
STILLING BASIN STUDIES
RECOMMENDED DESIGN
RIPRAP TESTS
1:46. 42 MODEL

Figure 47
Report Hyd-491

A. Q_s = 7,000 CFS, Reservoir El. 2970.1
Q_{ow} = 31,000 CFS



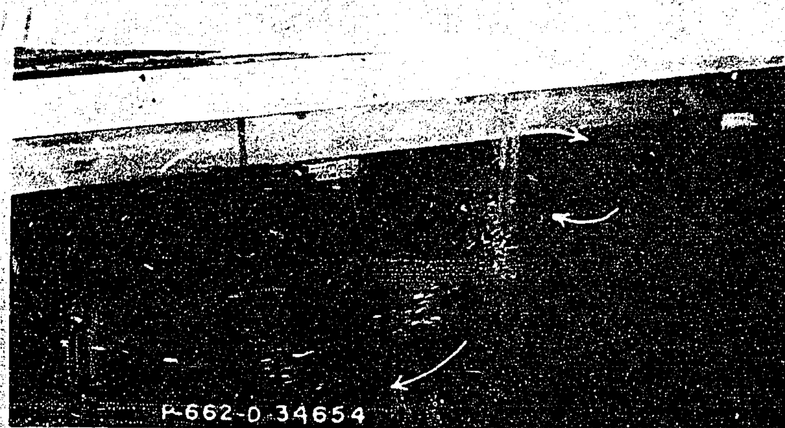
B. Q_s = 15,000 CFS, Reservoir El. 2972.9
Q_{ow} = 31,500 CFS



C. Q_s = 15,000 CFS, Reservoir El. 2975.1
Q_{ow} = 32,000 CFS

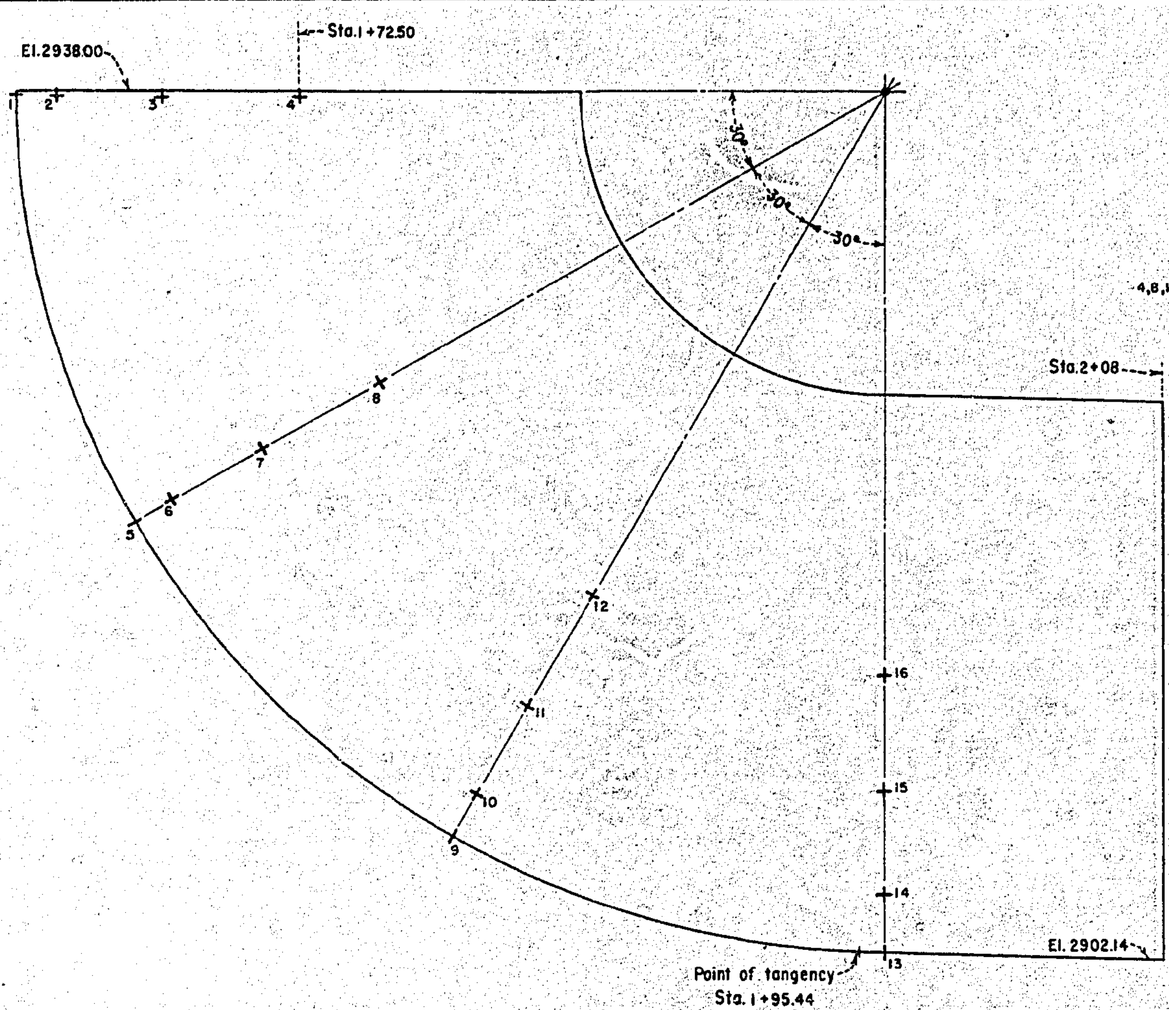
SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS
RESERVOIR FLOW PATTERNS
1:46.42 MODEL

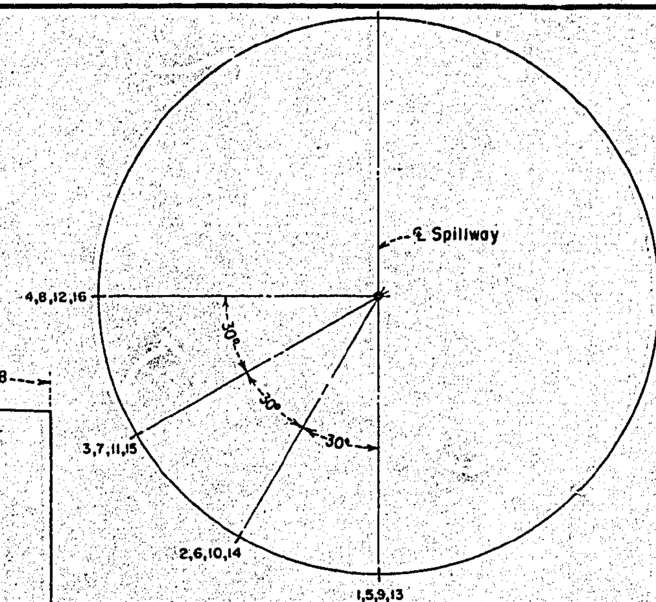
D. Q_s = 17,000 CFS, Reservoir El. 2990.9
Q_{ow} = 36,000 CFS



E. Q_s = 19,250 CFS, Reservoir El. 3005.5
Q_{ow} = 38,650 CFS

SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS
RESERVOIR FLOW PATTERNS
1:46.42 MODEL





NORMAL SECTION THROUGH
ROW OF PIEZOMETERS

PIEZOMETER NUMBER	PRESSURE FT. OF WATER
1	48.3
2	47.3
3	43.2
4	34.8
5	66.4
6	64.5
7	58.0
8	46.4
9	75.2
10	72.4
11	64.5
12	51.1
13	56.2
14	54.8
15	43.6
16	25.1

SANFORD DAM SPILLWAY
1:46.42 SCALE MODEL

PRESSURES IN THE VERTICAL BEND
Q = 19,300 SECOND FEET

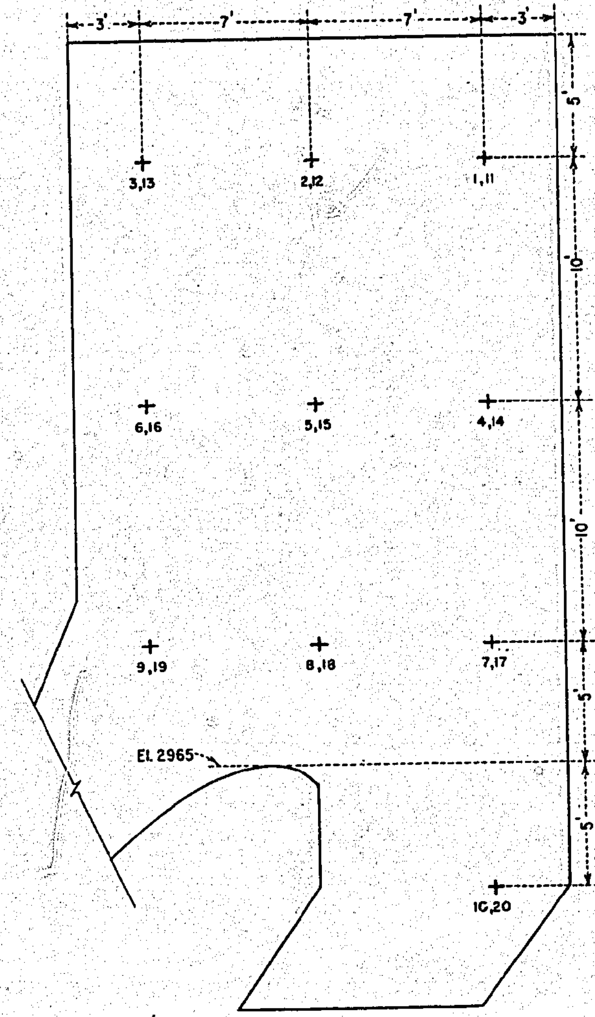
FIGURE 49
REPORT HYD. 481

	DIFFERENTIAL PRESSURE-FEET OF WATER			
	PIER 1		PIER 3	
	PIEZOMETER SET	DIFFERENTIAL	PIEZOMETER SET	DIFFERENTIAL
Q=16,500 CFS	1,11	—	1,11	—
	2,12	—	2,12	—
	3,13	—	3,13	—
	4,14	1.8	4,14	0.9
	5,15	2.3	5,15	0.4
	6,16	2.3	6,16	0.9
	7,17	1.4	7,17	0.4
	8,18	1.8	8,18	0.9
	9,19	1.4	9,19	1.0
	10,20	2.3	10,20	0.0
Q=19,300 CFS	1,11	0.5	1,11	1.0*
	2,12	0.0	2,12	1.0*
	3,13	0.5*	3,13	1.0*
	4,14	0.5	4,14	0.5
	5,15	0.5	5,15	0.9
	6,16	0.5	6,16	0.5
	7,17	0.5	7,17	0.5
	8,18	0.0	8,18	0.9
	9,19	0.5*	9,19	0.9
	10,20	0.4	10,20	0.4

* Differential acting in opposite direction of that for other piezometer sets.

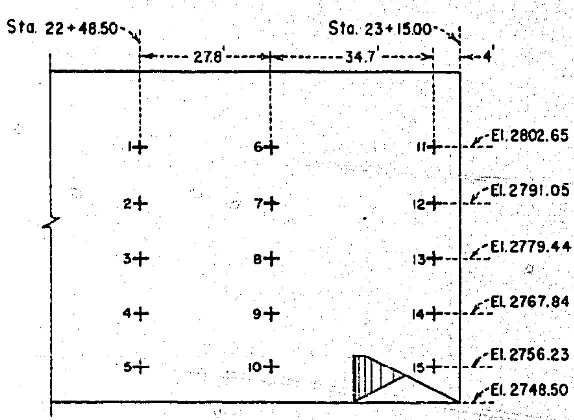
No data taken for pier 2. Flow conditions appeared identical to those for pier 3.

Piezometers 1-10 are located on the left side of the pier, looking toward the center of the morning-glory. Piezometers 11-20 are located on the right side of the pier.

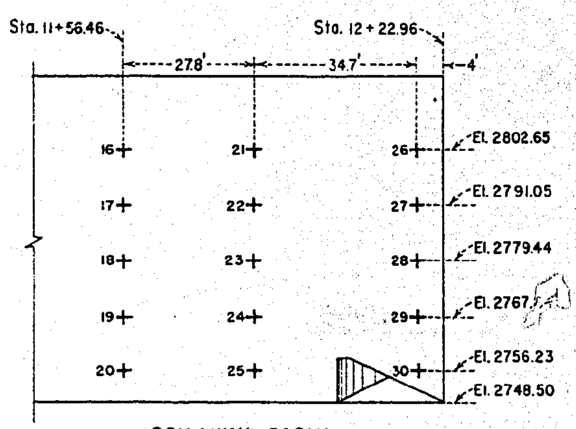


SANFORD DAM SPILLWAY
1:46.42 SCALE MODEL

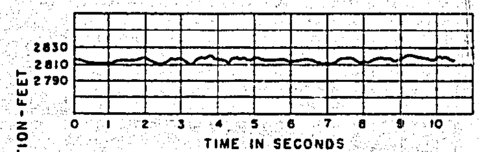
DIFFERENTIAL PRESSURES ON THE CREST PIERS



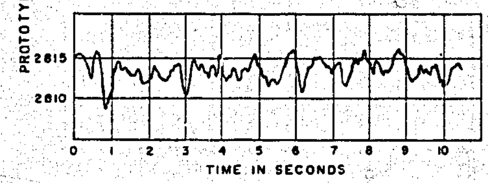
FLOOD CONTROL OUTLET WORKS BASIN



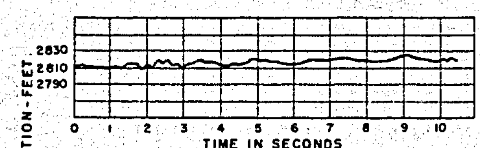
SPILLWAY BASIN



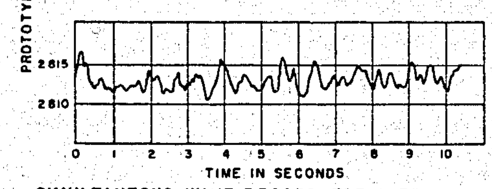
PRESSURE RECORD-PIEZOMETER 1



SIMULTANEOUS WAVE RECORD FOR PIEZOMETER 1



PRESSURE RECORD-PIEZOMETER 16



SIMULTANEOUS WAVE RECORD FOR PIEZOMETER 16

INSTANTANEOUS DYNAMIC PRESSURES

PIEZOMETER NUMBER	PRESSURE-PROTOTYPE FT. OF WATER		
	MAXIMUM	MINIMUM	AVERAGE
1	17.1	3.2	7.8
2	33.4	10.2	19.4
3	40.3	12.5	28.7
4	51.9	24.1	40.3
5	63.5	35.7	51.9
6	15.2	5.9	10.6
7	31.5	12.9	19.9
8	43.1	19.9	31.5
9	54.7	31.5	45.4
10	66.3	47.7	57.0
11	12.5	3.2	7.8
12	28.7	10.2	19.4
13	35.7	26.4	31.0
14	47.3	38.0	42.6
15	58.6	49.6	54.2
16	14.8	0.9	7.8
17	26.4	10.2	19.4
18	40.3	12.5	31.0
19	51.9	28.7	40.3
20	61.2	40.3	49.6
21	14.8	3.2	7.8
22	24.1	14.8	19.4
23	31.0	28.7	29.9
24	47.3	24.1	40.3
25	58.9	47.3	54.2
26	12.5	3.2	7.8
27	24.1	14.8	19.4
28	35.7	26.4	31.0
29	54.2	33.4	42.6
30	56.6	49.6	54.2

SANFORD DAM SPILLWAY AND FLOOD CONTROL OUTLET WORKS

1:46.42 SCALE MODEL

PRESSURES AND WAVE MEASUREMENTS - STILLING BASIN TRAINING WALLS

MAXIMUM DISCHARGE - TAILWATER EL. 2814.6

FIGURE 51
REPORT HYD. 491

GPO 859882