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Division of Research
Hydraulic Laboratory Branch

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF FLAMING GORGE DAM SPILLWAY AND OUTLET WORKS COLORADO RIVER STORAGE PROJECT, UTAH

Report No. Hyd-531

Hydraulics Branch
DIVISION OF RESEARCH

OFFICE OF CHIEF ENGINEER
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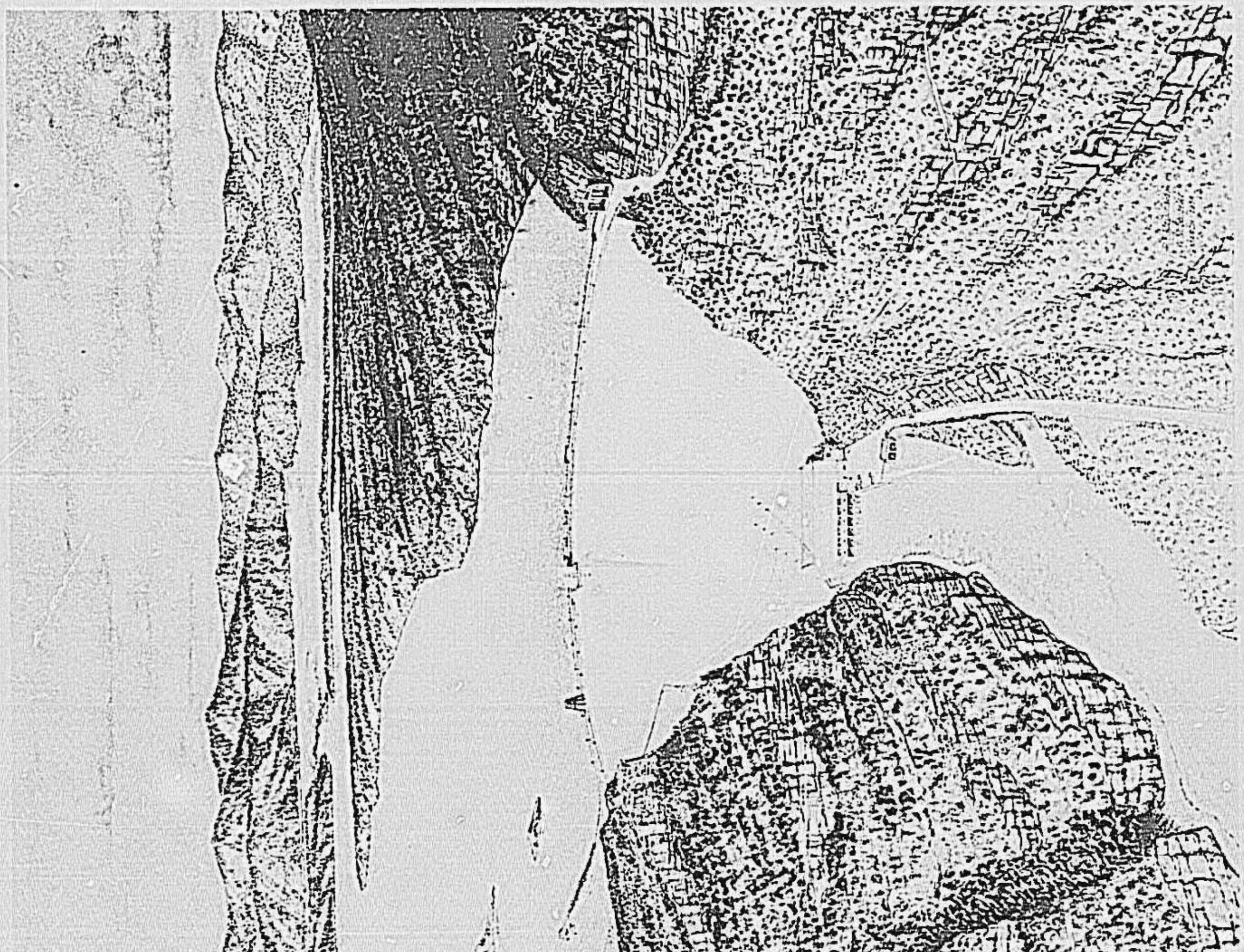
ABSTRACT

Hydraulic model studies were conducted to insure satisfactory performance of the spillway approach channel, spillway crest, tunnel, and flip bucket and to determine the flow characteristics in the downstream river channel when the spillway and/or the outlet works were operating. The spillway approach channel was considerably modified to improve flow conditions. A combination circular pier and angled approach wall on the right side of the spillway entrance reduced the drawdown and improved flow conditions in the right spillway bay. An outward sloping nose on the spillway center pier reduced the water surface contraction in the left spillway bay. The center pier was extended downstream and tapered to eliminate a large fin of water in the tunnel. Pressure measurements and water surface profiles indicated that the spillway ogee and the transition from the rectangular gate section to the circular tunnel were satisfactory. The discharge capacity of the spillway was greater than anticipated, permitting the crest elevation to be raised 1 foot. Flow in the sloping tunnel, vertical bend, and horizontal tunnel were excellent with symmetrical gate operation. Five angles of lift were investigated for the flip bucket at the tunnel portal; a 15° angle provided optimum flow conditions in the river channel. Pressures up to 65 feet of water were measured on the invert of the flip bucket. The minimum pressure observed was 8 feet below atmospheric. The tailwater elevation at the powerplant might be lowered as much as 12.5 feet when the spillway is operating at maximum discharge. The extent of riprap protection needed downstream from the powerplant to prevent damage by the river outlet discharges was determined.

DESCRIPTORS--*Flip buckets/ outlet works/ air demand/ *tunnels/ discharge coefficients/ *hydraulic models/ jets/ pipe bends/ vortices/ water pressures/ afterbays/ bank protection/ open channel flow/ spillway crests/ piezometers/ *riprap/ negative pressures/ drawdown/ water surface profiles/

IDENTIFIER--Spillway profiles/ spillway approaches/ flow contractions/ entrance transitions/ stream lining/ tunnel spillways/ spillway piers/



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HYDRAULIC MODEL STUDIES OF FLAMING GORGE DAM SPILLWAY AND OUTLET WORKS COLORADO RIVER STORAGE PROJECT, UTAH

PURPOSE

Hydraulic model studies were conducted to insure satisfactory performance of the spillway approach channel, spillway crest, tunnel, and flip bucket, and to determine the flow characteristics in the downstream river channel when the spillway and/or the outlet works were operating.

CONCLUSIONS

1. The preliminary approach channel was modified to improve flow conditions in the spillway entrance, Figure 6.

2. The excessive amount of excavation in the spillway approach area to accommodate the contractor's mix plant greatly improved the flow conditions in the approach channel, Figure 19.

3. A combination circular pier and angled approach wall developed for the right side of the spillway entrance, Figure 15, reduced the drawdown and provided smooth flow conditions in the right spillway bay.

4. An outward sloping nose on the spillway center pier, Figure 13, reduced the water surface contraction in the left spillway bay.

5. The center pier was extended downstream 30 feet and tapered to eliminate a large fin of water in the tunnel.

6. Pressure measurements and water surface profiles indicated that the spillway ogee and the transition from the rectangular gate section to the circular tunnel were satisfactory, Figures 16 and 23.

7. Calibration of the spillway showed that the discharge capacity was greater than anticipated. Accordingly, the crest elevation was raised 1 foot, Figure 25.

8. Flow in the sloping tunnel, vertical bend, and horizontal tunnel was excellent, Figure 26. However, unsymmetrical operation of the gates should be avoided.

9. A 15° angle of lift for the flip bucket at the tunnel portal provided optimum flow conditions in the river channel, Figures 33 and 34.

10. Pressures up to 65 feet of water were measured on the invert of the flip bucket, Figure 36. The minimum pressure observed in the flip bucket was 8 feet below atmospheric.

11. Water surface observations in the downstream river channel showed that the tailwater elevation at the powerplant would be lowered as much as 12.5 feet when the spillway is operating at maximum discharge, Figure 35.

12. The extent of riprap protection needed downstream from the powerplant to prevent damage by the river outlet discharges was determined, Figure 42.

13. A temporary access road downstream from the flip bucket should be removed after the construction period.

ACKNOWLEDGMENT

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INTRODUCTION

Flaming Gorge Unit is one of the four storage units of the Colorado River Storage Project. Principal features of the Unit are the Flaming Gorge Dam and Powerplant located on the Green River in northeastern Utah about 40 air miles north of Vernal and 6 miles south of the Wyoming-Utah state line, Figure 1.

Flaming Gorge Dam is an arch-type concrete structure *502 feet high with a crest length of 1,180 feet. The principal hydraulic features of the dam are the tunnel spillway and the river outlets, Figure 2. The tunnel-type spillway, having a capacity of 28,800 cubic feet per second and varying from 26.5 to 18 feet in diameter, is located in the left abutment. Flow through the spillway intake structure is controlled by two 16.75-foot-wide by 34-foot-high fixed-wheel gates, Figure 3. Flow from the tunnel spillway is directed into the river channel by a flip bucket located at the tunnel portal. The two 72-inch river outlets through the dam have a maximum discharge capacity of 4,000 cubic feet per second and are controlled by 66-inch hollow-jet valves which discharge from the left bank of the river channel downstream from the powerplant tailrace.

The model studies described herein were concerned with the tunnel spillway and the river outlets. The studies were made to investigate the flow conditions in the excavated approach channel to the spillway, the gate section and transition to the circular tunnel, the sloping tapered tunnel, the elbow, the constant diameter horizontal tunnel, and the flip bucket at the tunnel portal. Also investigated in the model was the flow from the river outlets.

THE MODEL

The model, built to a geometrical scale of 1:36, included a section of the reservoir area behind the dam, the excavated spillway approach channel in the left abutment, the gate control structure, the tunnel transition, the spillway tunnel, the flip bucket, the powerplant, the river outlets, and a section of the downstream river channel.

The spillway tunnel and transition were fabricated from transparent plastic. The fixed-wheel gates and flip bucket were made of galvanized sheet metal. The piers in the gate control section and the powerplant structure were constructed of wood treated to resist swelling. The spillway crest in the gate section was formed from concrete screeded to sheet metal templates. The topographic features in the reservoir and the rock outline of the downstream river channel were reproduced in rough concrete. The overburden in the river and on the river banks were represented by a 4- to 6-inch layer of sand.

Discharges in the model were measured using calibrated Venturi meters permanently installed in the laboratory. Water surface elevations in the reservoir were measured by point gages. The

*A table of Metric Equivalents of Important Quantities is given following page 21.

tailwater elevations in the river were controlled by an adjustable tailgate at the downstream end of the model and were measured by a staff gage.

Pressure measurements were made in critical flow areas throughout the model by means of piezometers connected to open-tube glass manometers. Piezometers in the flip bucket were also connected to a pressure cell and recording oscillograph and instantaneous dynamic pressure curves were obtained.

The theoretical velocities in the model and prototype spillway tunnels were computed using the Manning equation, $V = \frac{1.49}{n} r^{2/3} S^{1/2}$. A value of n = 0.014 was used for the prototype computation and n = 0.009 was used for the model computations. The computations indicated that at the start of the elbow, Station 3+56.58, the prototype velocity would be 141.4 feet per second. The corresponding model velocity at this station would be equivalent to 138.6 feet per second. The proper prototype velocities were represented in the model by increasing the vertical drop between the model spillway crest and the elbow by 1 foot. The 55° bottom slope of the tunnel was retained, but the convergence of the tunnel diameter was modified so that it reduced from 26.5 to 18 feet in the longer length.

In addition, the length of the horizontal portion of the model tunnel was reduced by 2.43 feet to assure that the portal velocity in the model would be equivalent to the portal velocity in the prototype.

THE INVESTIGATION

General

Although various features of the spillway were studied at the same time and modifications to one feature affected the operation of other features, each will be discussed separately in this report for reasons of clarity.

Spillway Approach

Preliminary.--The entrance to the preliminary approach channel was about 130 feet wide with its centerline approximately 160 feet upstream from the axis of the dam, Figure 4. A gradually converging curved channel 200 feet long terminated at the 50-foot-wide spillway entrance. The floor of the approach channel was at elevation 5995; the sides were excavated on a 1/4:1 slope. The left wall of the channel extended above the maximum water surface, elevation 6045, for practically its full length. The right wall extended to elevation 6030 and was topped off with a flat berm at this elevation. In addition, for construction purposes a notch was excavated between the face of the dam and the portion of the canyon forming the right side of the approach channel, Figure 5. The bottom of the notch was at approximately elevation 5995.

The investigations of the approach channel were made with uncontrolled or weir flow through the spillway section to provide the severest operating conditions in the approach. Normally, releases through the prototype structure will be controlled by the gates to maintain the reservoir at elevation 6040 or higher.

At a discharge of 5,000 cubic feet per second, the flow in the approach channel was very smooth. The flow through the notch caused negligible disturbances at the spillway entrance, Figure 6. At a discharge of 15,000 cubic feet per second, the flow from the reservoir topped the berm on the right side of the approach channel and the flow through the notch became significant. The flows over the berm and through the notch were nearly normal to the flow through the channel and consequently caused considerable disturbance at the right spillway bay, Figure 6. At the maximum discharge, 28,800 cubic feet per second, this condition became very objectionable and caused severe flow asymmetry at the spillway entrance, Figure 6.

First modification.--The notch at the upstream face of the dam was filled in and a training wall was placed on top of the berm, Figure 7. The training wall followed the curve of the original cut and, in cross section, was a continuation of the cut slope of the original wall. The wall extended upstream for about 120 feet and terminated in a small radius curve.

At the maximum discharge, the flow approaching the spillway entrance was greatly improved. However, there was a 1- to 2-foot drop in the water surface where the flow entered the approach channel at the upstream end of the right wall. This drawdown caused considerable surface turbulence that carried downstream along the right wall to the spillway entrance, Figure 7.

Second modification.--The approach channel was further modified by moving both channel walls inward to reduce the channel width. Although the channel entrance was wider than the original, it pinched down almost immediately and formed a constant width channel approaching the gate structure, Figure 7. A training wall with its top at elevation 6046 was placed on the berm on the right side of the channel.

Eddy currents and extreme surface roughness caused by the drawdown still occurred in the flow around the upstream end of the right wall,

Figure 7. To smooth out this condition, a long-radius streamlined nose on the right wall would be necessary, since previous tests had indicated that it was not feasible to provide a satisfactory approach wall utilizing the right canyon wall. The decision was made, therefore, to entirely remove the island between the dam and the channel.

Third modification.--For the third modification, the island on the right side of the approach channel was removed. Provisions were made so that semicircular piers with different radii could be installed on the right side of the spillway entrance, Figure 8.

Four piers with radii of 9, 18, 27, and 36 feet were investigated. The adequacy of each pier was judged on the appearance of the flow along the pier and the effect of the pier on the spillway discharge coefficient. The tests showed that the 27- and 36-foot-radius piers provided very good flow conditions at the gate structure, but that larger discharge coefficients could be obtained with the 9-, 18-, and 27-foot-radius piers. The discharge coefficients related to the pier radii for maximum discharge are shown on Figure 9. Since these test results were encouraging, it was decided to determine how far the left wall could be moved to the right without causing undesirable flow conditions at the spillway entrance or a reduction in the discharge coefficient.

Fourth modification.--For the fourth modification, the left wall was moved to the right so as to provide a constant channel width from the gate structure upstream to about halfway to the reservoir. Farther upstream the left wall gradually diverged to provide approximately the original width at the entrance to the channel, Figure 10. With the 36-foot-radius semicircular pier installed, the appearance of the flow in the approach channel was very good. Some drawdown in the water surface occurred along the pier and carried down into the gate structure, causing a standing wave to form in the center of the right bay. The water surface elevation in the left bay of the spillway was also lowered 1 foot due to the increase in velocity in the narrower channel.

With the 18- and 27-foot-radius semicircular piers installed, the water surface drawdown at the gate structure was about 1 foot greater than that observed for the third modification, but about halfway around the right pier the drawdown was 5 to 6 feet greater. Except for the greater drawdown, the flow distribution and appearance in the approach channel were better than they had been with the previous channel alinement, Figure 11A.

Fifth modification.--The approach channel was widened by moving the left wall about 10 feet to the left for the fifth modification; the 27-foot-radius pier was installed on the right side, Figure 12.

The appearance of the flow in the approach channel was very good with this modification. The water surface was smooth and the flow approached the gate structure linearly except along the right pier. A drawdown in the water surface of about 5 feet occurred midway around the right pier and carried downstream into the gate section and caused some asymmetry in the flow in the right spillway bay, Figure 11B.

Dye streams were introduced at various depths to trace the flow lines in the approach channel. At the maximum discharge, most of the surface flow from about halfway upstream in the approach channel entered the right spillway bay; the flow between middepth to the bottom flowed predominantly toward the left bay; and, generally, the flow velocity in the right half of the channel was higher than the flow velocity in the left half. This flow asymmetry resulted in very poor flow conditions in the transition downstream from the gate section.

Sixth modification.--For the sixth modification, the upstream face of the enter pier in the spillway section was sloped upstream at an angle of 21° from the vertical to form an overhanging nose, Figure 13. A 30-foot-radius semicircular pier was placed at the right side of the spillway entrance. The left wall alinement was the same as in the fifth modification.

The flow appearance in the approach channel was very good at all discharges. The overhanging pier provided adequate flow area at the larger discharges and still retained directional control at the intake entrance. There was a 2- to 4-foot drawdown in the water surface around the pier faces, but the flow entering the spillway was still symmetrical.

Recommended approach channel.--Although the approach channel described in Modification 6 was satisfactory in all respects, the 30-foot-radius semicircular pier would require a considerable quantity of concrete, and it was thought that, if possible, a smaller pier should be used. Further investigations showed that an 18-foot-radius pier could be used if it was placed about 27 feet to the right of the gate section.

For the recommended approach channel, the alinement of the left side was the same as for the previous channel, and the overhanging center pier was used. The right training wall consisted of a vertical wall extending upstream from the end of the right pier at an angle of 30° with the face of the dam and curving on an 18-foot radius to join the face of the dam, Figure 14. The angled wall became tangent to the 18-foot-radius curved wall 27 feet to the right of the spillway.

The flow in the approach channel was satisfactory in all respects with this arrangement. There was some water surface drawdown in the flow around the curved part of the right pier; a slightly disturbed water surface appeared along the angled wall; and another surface depression was present as the flow entered the gate section, Figure 15. A rise in the water surface occurred on the right side of the center pier, caused by the flow impinging on the pier face. On the left side of the center pier, the water surface depressed due to the centrifugal force of the flow going around the pier nose. There was a slight buildup where the flow struck the left pier nose and a slightly depressed water surface as the flow passed around the pier and entered the gate section. The profiles showing the flow along the piers are shown on Figure 16.

An adverse flow condition occurred with this arrangement when a large vortex formed in the right spillway bay for discharges at gate openings over 50 percent, Figure 17.

Construction changes.--During the construction of the dam, the contractor elected to install the concrete mix plant in the vicinity of the spillway approach channel. To obtain sufficient area for the plant, it was necessary to excavate a large area at the canyon edge upstream from the recommended approach channel. This area was excavated to elevation 6007 and was equivalent to the preliminary approach channel, Figure 18. Since the model was operable when this change became known, the prototype excavation was reproduced in the model to determine its effect on the approach flow.

The model was operated at discharges ranging from 7,200 cubic feet per second up to the maximum discharge of 28,800 cubic feet per second. The contractor's excavation opened up the approach area and greatly improved the flow conditions, Figure 19.

Intake Structure and Transition

The intake structure and transition include that portion of the spillway between the approach channel, Station 0+89, and the spillway tunnel portal, Station 1+74.98, Figure 20. The studies of this section were concerned with the center spillway pier, the discharge capacity of the spillway crest, and the pressure investigations of the spillway crest and transition.

Spillway pier.--The center pier on the spillway was 8 feet wide and divided the spillway into two 16.75-foot-wide rectangular passages. In the preliminary design, the nose of the pier was at Station 0+89 and the pier terminated at Station 1+20. Both the nose and the tail end of the pier were streamlined, Figure 21.

While conducting the investigations to develop the approach channel, an overhanging nose was placed on the upstream end of the pier to improve the flow distribution, Figure 13. These tests were described in the previous section. Concurrently with the tests in the approach channel, the pier modifications described in this section were developed and the flow improvements were probably equally due to the improved approach channel and to the modifications of the pier.

Preliminary tests indicated that the center pier, Figure 21, was too short for its width. At all discharges, a fin of water formed immediately downstream from the pier. At the maximum discharge, the fin was very large and occasionally struck the tunnel roof at the tunnel portal, Figure 22. The fin fluctuated from side to side of the tunnel where it struck the walls with considerable force. The fluctuation seemed to be caused by the flow asymmetry in the approach channel and was greatly reduced by the channel modifications previously described. However, it was apparent that modifications to the downstream end of the center pier were necessary to eliminate or reduce the fin.

Pier modifications.--The streamlined tail of the pier was cut off at Station 1+15, leaving a flat face 4 feet wide. Another tail that was 15 feet long and tapered from 4 feet wide at Station 1+15 to 2 feet wide at Station 1+30 was added to the shortened pier. The end of the pier was normal to the tunnel invert, Figure 21. This pier reduced the center fin which still had a tendency to strike the roof of the tunnel portal, Figure 22.

Shortening the pier by 7.5 feet and increasing the convergence of the tail to a width of 1 foot at the end did not improve the performance.

Recommended pier.--Satisfactory entrance conditions had been obtained by modifying the approach channel and placing an overhanging nose on the upstream end of the center pier. However, a center fin still formed in the transition downstream from the pier. The pier tail that was most effective in eliminating this fin was a tapered section extending from Station 1+15 to Station 1+50.0. At the upstream station, the pier was 4 feet wide and 24.50 feet high. At the downstream station, the pier was 2.25 feet wide and 23.81 feet high. The end was vertical, Figure 13.

This pier was very effective in smoothing the flow. A small fin formed downstream from the pier at discharges up to about 20,000 cubic feet per second; above 20,000 cubic feet per second, the flow closed over the top of the pier, the fin disappeared, and the water surface was smooth and level, Figure 22.

When the flow closed over the top of the pier, a whirling rope of air, similar to the submerged portion of a vortex, formed beneath the surface and parallel with the end of the pier, Figure 22. Over a short period of time, the "vortex" grew gradually smaller and moved closer to the pier; above the "vortex" a depression formed in the water surface. As the size of the "vortex" diminished, the depression gradually grew deeper; and about the same time that the "vortex" disappeared, a slug of air broke through the depressed water surface with an audible "pop" and another "vortex" formed and the water surface became level. This cycle consistently repeated itself in about 2- to 3-minute intervals in the model. Piezometers placed in the end of the pier indicated that the pressure gradually reduced from atmospheric when the "vortex" first formed to about 15 feet below atmospheric, when the "vortex" disappeared. When air was supplied to the end of the pier through a vent pipe, the cyclic action stopped and the pressures remained near atmospheric. An air vent in the downstream face of the pier would be desirable to prevent possible vibrations from occurring. However, due to the infrequency of operation at the maximum discharge, the air vent was not installed.

This pier was also tested with the downstream end normal to the tunnel invert rather than vertical. With this end shape, the water surface was not as smooth but was still adequate. The "vortex" cycle at the end was still present but was not as severe.

Although either end shape would be satisfactory, the vertical end was chosen for prototype construction, since it would be simpler to construct.

Intake transition.--The spillway tunnel transition extended from the end of the gate section at Station 1+10 to the tunnel portal at Station 1+74.98, Figure 20. The transition changed the shape of the flow passage from two 16.75-foot-wide rectangular sections to a single 26-foot-diameter circular conduit. The center pier, which formed a common wall for the rectangular passages, extended into the transition to Station 1+50.

With the recommended approach channel and center pier in place, the flow in the transition was very good, Figure 20. At the maximum discharge, 28,800 cubic feet per second, surface fins of water formed along the sidewalls in the transition. As the walls pinched in, the fins became larger and impinged on the headwall of the tunnel intake. Although the fins did not fill the tunnel, it was desirable to reduce as much as possible the large amount of splashing and spray at the tunnel portal. To reduce the splash, the fins were contained and directed into the tunnel by reducing the width of the headwall at the tunnel intake from about 18.9 feet to 12 feet 3 inches, Figure 20 and Section H-H of Figure 3. This change in effect supported the flow over more of the tunnel periphery. The additional support was very effective in preventing the excessive spray and splashing at the portal and the change was incorporated in the recommended design.

Pressures on the invert and sidewalls of the right half of the transition were obtained for gate-controlled discharges at the maximum reservoir elevation. Pressures were recorded for flows with both gates at equal openings from 2 feet to fully open, Figure 23. The pressures were either near or above atmospheric for all gate openings. The lowest pressure, 2.1 feet of water below atmospheric at Piezometer 40, was well above the cavitation range. A complete tabulation of the pressures is given on Figure 23.

Intake structure.--The intake structure includes the portion of the structure between Stations 0+89 and 1+15, Figure 20, and consists of the overfall spillway crest and the two fixed-wheel gates. The investigations of the intake structure were concerned with the flow characteristics in the gate section, the pressures along the spillway crest, and the discharge capacity of the spillway.

Although water surface drawdown occurred at the center and side piers, the flow through the intake structure was generally satisfactory. The profile of the water surface at the upstream edge of the fully open gates is shown on Figure 24. This profile may be used to insure that the gates will clear the flow at maximum discharge. Piezometers were installed along the invert centerline of the right bay, Figure 23. Pressure measurements were made for gate-controlled flow at the maximum reservoir elevation and for the maximum discharge with the gates fully opened. The pressures were near or above atmospheric for all discharge conditions. A complete tabulation of the pressures is shown on Figure 23.

After the recommended approach channel and center pier had been developed, the spillway was calibrated to determine the discharge capacity and head discharge curves over the full range of reservoir elevations for regulated and free flow. The discharge capacity for regulated flow was measured for gate opening intervals of 2 feet with both gates equally opened.

The results of the calibration are shown by the curves of Figure 25. The calibration showed that for uncontrolled flow the maximum discharge was 28,800 cubic feet per second and occurred at a head of 39 feet above the crest. The coefficient of discharge at maximum reservoir was determined to be 3.527 from the equation

$$C = \frac{Q}{LH^{3/2}}$$

where

C = coefficient of discharge
Q = discharge, 28,800 cubic feet per second
L = spillway width at crest axis, 33.5 feet
H = total head, 39 feet

For design purposes, a value of 3.40 had been used for the coefficient of discharge. The larger coefficient determined from the model permitted the spillway crest to be raised from elevation 6005 to elevation 6006, which reduced the amount of rock excavation in the approach channel and spillway.

Flow in Tunnel

Tapered shaft.--From the tunnel portal, Station 1+74.98 to the PC of the elbow, Station 3+56.58, the tunnel diameter decreased from 26.50 to 18.00 feet, Figure 3. In this length, the tunnel invert followed a 55° slope from elevation 5952.46 to elevation 5693.25.

When the gates were equally opened, the flow in this section was very uniform with no vacillation from side to side throughout the tapered length. Water surface profiles in the tunnel for discharges of 15,000 and 28,800 cubic feet per second are shown on Figure 26.

Vertical bend.--The 18-foot-diameter tunnel changed direction from the 55° slope to near horizontal with a 200-foot-radius vertical bend or elbow, Figure 3.

The flow in the elbow was excellent in all respects for all discharges, Figure 26. The change in direction of the flow was smoothly accomplished with a uniform flow depth and no disturbances.

Horizontal tunnel.--The near horizontal tunnel extended from Station 5+18.31 to Station 7+50 on a 0.01 bottom slope, Figure 26.

The flow distribution was good for the full length of this section, and the flow appearance was satisfactory in every respect, Figure 26. At the maximum discharge, the depth of flow in the horizontal section was about 0.85 of the tunnel diameter, but because of the smooth flow conditions the space above the water surface was considered adequate. The profiles of the water surface in the horizontal section for discharges of 15,000 and 28,800 cubic feet per second are shown on Figure 26.

Unequal gate openings.--Flow conditions with unequal gate openings were very poor. The flow switched from side to side in the tunnel and on occasion would swirl over the crown. Unsymmetrical gate operation should be avoided.

Flip Bucket Studies

A flip bucket was used at the downstream portal of the tunnel spillway to direct the flow into the river channel where the high-velocity flow was dissipated. In place of a transition between the circular tunnel and the rectangular flip bucket, the semicircular invert of the tunnel intersected the upward curve of the flip bucket. The river channel downstream from the flip bucket was in a comparatively narrow canyon having steep sides. Downstream from the tunnel portal, the centerline of the river was almost directly in line with the centerline of the tunnel, Figure 2. The bucket lip was at elevation 5614.62, about 10 feet above the normal river water surface elevation with the powerplant discharging 4,260 cubic feet per second. The powerplant and river outlets were about 600 feet upstream of the flip bucket.

The canyon walls have a comparatively small amount of overburden or talus over fairly sound rock; however, most of the overburden is found near the riverbanks and will be easily moved by flowing water. A large fault is located in the canyon wall on the right side of the river approximately 500 feet downstream from the tunnel portal. In the model, the approximate sound rock outline was reproduced in concrete. River sand placed over the concrete represented the talus.

The model studies of the flip bucket were directed toward developing a bucket that would deflect the flow away from the tunnel portal without impinging near the fault zone or causing excessive tailwater drawdown or eddies at the powerplant.

Preliminary bucket, 35° flip angle.--The PC of the preliminary bucket was located at elevation 5605.65, 14.10 feet downstream from the tunnel portal. The bucket invert curve was formed by a 48.00-foot-radius circular arc which terminated in a 2-foot-long horizontal lip at elevation 5614.62 and provided a lift of 35° above the horizontal, Figure 27.

The jet was well dispersed at the lower discharges and became more compact as the discharge increased, Figure 28. The water rose on the sides of the bucket at low flows, forming in effect a "U" shaped sheet of water in which the bottom and sides were of equal thickness. The vertical sides of the "U" followed the line of the bucket sidewalls after leaving the bucket, while the bottom sheet tended to diverge. The vertical fins had a shorter trajectory than the lower sheet, and on falling penetrated the lower jet, tending to spread or disperse it, Figure 28. As the discharge increased, the size of the fins relative to the thickness of the lower sheet became insignificant and the jet no longer tended to spread. At the maximum discharge, 28,800 cubic feet per second, the jet impact area was slightly downstream from the fault area; at 15,000 cubic feet per second it was opposite the fault; and at 7,200 cubic feet per second it was slightly upstream from the fault.

Second bucket, 30° flip angle.--Since it was desirable to move the point of impact of the jet away from the fault zone, the angle of flip of the bucket was reduced to 30° above the horizontal. The invert curve was formed with a 67.00-foot-radius circular arc. The elevation of the bucket lip was not changed, but the bucket lip was 1.5 feet long and sloped downward 12.5° below the horizontal, Figure 27.

The appearance of the jet from this bucket was similar to that with the previous bucket. The jet was greatly dispersed at the lower flows and became more compact as the discharge increased, Figure 29.

The "U" shaped sheet of water in the bucket was also similar in appearance and effect. The jet impact area had moved upstream a small amount, but it still impinged very close to the fault zone.

Third bucket, 30° flip angle with wall deflector.--In an attempt to move the impact area of the jet away from the fault zone, the final 10 feet of the right wall of the second bucket was turned inward 8°30' toward the tunnel centerline. The deflector wall directed the right side of the jet toward the center of the river channel; however, the jet was more compact and caused considerable turbulence and swirling action in the river near the fault zone, Figure 29.

Fourth bucket, 25° flip angle.--For the fourth bucket, the angle of the flip was reduced to 25° above the horizontal. The PC of the bucket invert was moved to the tunnel portal, Station 7+50. The invert curve was formed with a 96.00-foot-radius circular arc. A 2.34-foot-long horizontal lip at elevation 5614.65 was at the end of the bucket, Figure 27.

The appearance of the jet from this bucket was also very similar to that from the 35° and 30° buckets. The flow dispersion was not as extensive and the point of impact was farther upstream than with the previous buckets, Figure 29. However, the turbulent area in the river where the jet struck was still considered too close to the fault zone.

Fifth bucket, 15° flip angle.--The angle of flip was reduced to 15° above the horizontal for the fifth bucket, which was formed with a 160-foot-radius circular arc. The bucket lip was 1.5 feet long and at elevation 5611.11, Figure 27.

The jet from this bucket was not as well dispersed as the jets from the previous buckets, and the jet impact area was considerably upstream from the fault at all discharges, Figure 30. Because of the flat angle of the jet trajectory, the jet did not penetrate the water surface and caused considerable surface turbulence downstream from the point of impact. However, the surface turbulence was not accompanied by the severe swirling and eddy action that had occurred with the other buckets and consequently would be less apt to cause damage in the fault area. The comparatively flat angle of the 15° trajectory caused greater water surface drawdown in the powerplant afterbay, but the difference was considered minor when compared to the better overall performance of the bucket in other respects.

No flip bucket.--To determine whether it would be possible to eliminate the flip bucket, the bucket was removed and the circular tunnel was extended 72 feet downstream from the portal to the edge of the river channel.

The performance was adequate for discharges below 10,000 cubic feet per second, Figure 31. Above this discharge, the flow in the river channel became quite rough. The turbulent area downstream from the jet impact area shifted from one side of the river to the other, causing severe eddies along the banks. Although discharges above 10,000 cubic feet per second will seldom occur, it was decided that a flip bucket was needed to lift the flow into the downstream river channel and provide better flow conditions over the full discharge range.

Recommended bucket.--The bucket with the 15° flip angle was selected for the prototype structure since it most nearly provided the desired flow conditions in the downstream river channel. However, due to foundation and structural considerations, it was necessary to make minor modifications to the basic bucket design used in the previous tests. Accordingly, the bucket was rebuilt to reflect these modifications.

The PC of the invert curve was moved upstream in the tunnel to Station 7+37.50, Figure 32. The radius of the bucket was 145 feet. The lip at the end of the bucket was at elevation 5610.34 and sloped downward on a 1:1 slope but the sidewalls extended beyond the end of the bucket a distance of 1.42 feet. Because of the position of the bedrock on the right side, the right wall of the bucket was constructed only as high as the springline of the circular tunnel downstream from Station 7+60.00. The top of the left wall was at elevation 5626.00, approximately 11.00 feet above the springline.

Flow conditions in the river channel with the recommended bucket were identical to the flow conditions with the initial 15° bucket for discharges up to and including 15,000 cubic feet per second, Figure 33. The point of impact of the jet was well upstream from the fault area and, due to the flat trajectory of the jet, considerable turbulence occurred downstream from the point of impact. The jet was very compact with no appreciable divergence. At the maximum discharge of 28,800 cubic feet per second, the water surface in the bucket was higher than the right wall and consequently the top portion of jet spread to the right, Figure 34. The divergence of the right side of the jet caused no adverse flow conditions and the greater part of the jet remained compact. Generally, the flow conditions were excellent.

Water surface drawdown.--A series of tests was made to determine the amount the tailwater elevation at the powerhouse was lowered when the spillway was operating. The test procedure was as follows: With a known discharge from the spillway, the design tailwater elevation was set at the downstream end of the tailbox, or a distance equivalent to about 800 feet downstream from the flip bucket; after the water surface level in the channel had become stable, the water surface elevation at the powerhouse was measured. The difference in the two elevations was the amount of drawdown. The measurements were made with no flow through the river outlets and with either no flow or 3,000 cubic feet per second discharging through the powerplant. The measured water surface elevations at the powerplant and downstream river channel and the amount of drawdown are shown on Figure 35. The greatest drawdown, about 12.5 feet, occurred at the maximum discharge with the powerplant shut down. At the maximum discharge and with the powerplant discharging 3,000 cubic feet per second, the drawdown was 9.6 feet.

Pressure investigations.--During the flip bucket tests, extensive pressure measurements were made on the 35° bucket and on the preliminary and final 15° buckets. The purpose of the tests was to determine the pressure distribution on the invert curves and on the bucket sidewalls.

A total of 17 piezometers was installed on the invert of the 35° bucket, Figure 36. Ten piezometers were spaced along the centerline, 5 piezometers were spaced in a row midway between the centerline and the wall, and 2 piezometers were placed at the base of the wall. In addition, 6 piezometers were placed on the right wall of the bucket. Thirty-five piezometers were placed in the preliminary 15° bucket: 14 on the invert curve and 21 on the sidewall, Figure 36. Seventeen piezometers were installed in the recommended 15° bucket: 12 on the invert curve, 2 on the sidewall, and 3 on the inside top edge of the right wall, Figure 36.

Pressures on the invert of the 35° bucket varied from 12 feet of water below atmospheric to 50 feet of water above atmospheric at a discharge of 5,000 cubic feet per second and from near atmospheric to 156.0 feet of water above atmospheric at the maximum discharge of 28,800 cubic feet per second. The sidewall pressures ranged from near atmospheric to 20 feet of water above atmospheric at the smaller discharge and from 23 feet of water to 125 feet of water above atmospheric at the maximum discharge. The pressure readings are tabulated on Figure 36.

For the preliminary 15° bucket, the invert pressures varied from about 5 to 65 feet of water above atmospheric for the maximum discharge and from near atmospheric to 25.6 feet of water above atmospheric at a discharge of 7,200 cubic feet per second. The sidewall pressures showed similar variation, depending on the location. The location of the piezometers and the pressures for three discharges are shown on Figure 36.

Pressures on the invert of the recommended 15° bucket varied from 8.0 feet of water below atmospheric to 64.7 feet of water above atmospheric for the maximum discharge and from 8.2 feet of water below atmospheric to 23.2 feet of water above atmospheric for a discharge of 7,200 cubic feet per second. The sidewall pressures were about 5 feet of water at the maximum discharge and atmospheric for the 7,200-cubic-feet-per-second discharge. The piezometer locations and the pressure values for three discharges are shown on Figure 36.

The pressures were not excessively high nor dangerously subatmospheric in any of the three buckets. However, some interesting comparisons can be made between the theoretical pressures and the measured pressures on the invert. The maximum theoretical pressure, P_T, given by D. B. Gumensky, 1/ can be expressed as:

$$P_T = (1.94\ w^2 R + 62.5)\ D_1$$

in which

$$w = \frac{V}{R}$$

where V = average flow velocity

R = radius of the invert

and

D_1 = flow depth at the PC

1/ "Design of Sidewalls in Chutes and Spillways," by D. B. Gumensky, Transactions ASCE, vol. 119, 1954.

Using this equation, the highest theoretical pressures for the maximum discharge become 166.9 feet of water for the 35° bucket, 64.7 feet of water for the preliminary 15° bucket and 72.3 feet of water for the recommended 15° flip bucket. The highest measured pressures for the three model buckets were 156.0, 64.5, and 64.7 feet of water, respectively.

Thus, the maximum measured pressures in the model were very close to the theoretical values. These maximum pressures occurred approximately 0.6 of the bucket length from the upstream end. Pressures rapidly decreased near the downstream end of the bucket and reached atmospheric at the bucket lip. The ratio of the measured pressures to the theoretical pressures along the bucket invert is shown on Figure 37. The envelope curve includes pressures on all three buckets operating at four different discharges.

Pressures measured just upstream from the bucket lip were consistently subatmospheric when the lip was horizontal as shown in Figure 37. Experiments on the model buckets showed that the lip pressures were affected by the shape or angle of the downstream portion of the bucket lip. The curve of Figure 38 indicates that for a given angle of inclination of the bucket invert, the downstream portion of the lip should be inclined 35° or more to insure atmospheric or higher pressure at the lip piezometer.

Pressures measured on the sidewalls of the 35° bucket indicated that the maximum pressure was approximately 11 times hydrostatic pressure and occurred near the base of the wall at about three-quarters of the bucket length downstream from the PC. At the end of the bucket, the maximum pressure was 4 times greater than hydrostatic. For the 15° bucket, the maximum pressure was 4 times greater than hydrostatic and occurred from about one-quarter to three-quarters of the bucket length. At the end of the bucket, the maximum sidewall pressure was about twice the hydrostatic pressure.

Pumping plant access road.--To provide access to a water supply pumping plant, a temporary road will be constructed along the left side of the river downstream from the flip bucket. The road will encroach on the river channel and reduce the available flow area. To determine the effect of the road on the riverflow conditions, the outlets were operated with the road installed in the model.

The road fill was reproduced in a weak, easily erodible, sand-cement mixture composed of 1 part aluminous cement to 75 parts of sand and allowed to cure for a minimum of about 24 hours,

Figure 39. The model was operated for several hours with 4,000 cubic feet per second through the outlet works and no flow through the spillway. This operating condition did not erode any material from the road embankment. For the next test the model was operated with the spillway progressively discharging 5,000, 10,000, and 15,000 cubic feet per second. The model was operated for about 3 hours at each discharge and the extent of erosion determined at the end of each period. The two lower discharges caused only moderate erosion of the road fill; however, the 15,000-cubic-feet-per-second discharge eroded a considerable amount of material, Figure 39.

The tests indicated that no harmful effects would occur during the outlet discharges but that large spillway discharges would destroy the roadway, move material into the river channel, and possibly increase the tailwater elevation in the powerplant afterbay. The temporary road, therefore, should be removed after the construction period.

River Outlets and Powerplant Afterbay

The river outlets and powerplant are located at the toe of the dam in the center of the river channel, Figure 2. The powerplant contains three 36,000-kilowatt units with a discharge capacity of 1,420-cubic-feet-per-second per unit. The tailrace downstream from the powerplant is a riprapped channel that slopes upward on a 6:1 slope from elevation 5578.25 to the normal riverbed elevation 5595.0.

Flow through river outlets, located on the left side of the powerplant and about 140 feet downstream, is controlled by two 66-inch hollow-jet valves, Figure 40. The valves are parallel and discharge horizontally into the atmosphere and are alined so that the flow is directed toward the center of the river channel, Figure 2. The maximum design discharge of each valve is 2,000 cubic feet per second.

The model tests were concerned with determining the flow conditions in the river channel and powerplant afterbay when the river outlets are operating and the erosive effects of river outlets flow impacting in the river channel.

Valve alinement.--In the initial tests the valves were operated at total discharges from 400 up to 4,000 cubic feet per second. No riprap protection in the river channel was provided but a 5-foot-thick layer of riprap was placed in the afterbay, Figure 42.

An eddy formed to the right of the jet impact area at all discharges, Figure 41. The eddy moved upstream along the right bank, across the channel in front of the afterbay, then under the jets and downstream into the impact area. At maximum discharge the eddy eroded large quantities of the riverbed material which was carried downstream to form a bar across the full width of the river. The force of the eddy moving in front of the afterbay was sufficient to undercut the riprap and cause extensive damage to the sloped floor.

In an attempt to reduce the force of the eddy, both valves were turned 5° to the right. This change in alinement did not prevent the eddy from forming, Figure 41. Because of design limitations it was not practicable to turn the valves more than 5° so no further tests were made on this type of correction. The original alinement was restored and other methods of protection were investigated.

Riprap protection.--The tests indicated that adequate riprap could be provided to protect the river banks and to prevent erosion in the afterbay, but that riprap protection of the jet impact area would be impracticable for economic reasons. The model operation showed that a considerable erosion hole would be formed and the eroded material would be deposited in the channel and would raise the tailwater for flows through the powerplant. To recover full power head, a channel excavated through the eroded material would be necessary.

Two schemes were proposed for placement of the riprap protection. In Scheme No. 1, 30- to 54-inch-diameter stones were placed on a 1:1 slope along the right bank of the channel between elevations 5600 and 5610 and extended about 200 feet downstream from the afterbay. Stones, 12 to 27 inches in diameter, were placed across the river channel on the 2:1 slope from elevation 5590 to 5600 downstream from the afterbay. It was estimated that this scheme would require over 3,600 cubic yards of riprap.

For Scheme No. 2, 30- to 54-inch-diameter stones were placed on a 2:1 downward slope from elevation 5600 at the downstream end of the afterbay down to elevation 5585 in the river channel and extended to sound rock on both sides of the channel, Figure 42. No riprap was placed along the right bank. It was estimated that this scheme would require about 1,600 cubic yards of riprap.

One test was made with riprap placed as proposed in Scheme No. 1. The discharge was 1,000 cubic feet per second through each valve with no flow through the powerplant. After 2 hours operation (model time) extensive erosion had occurred downstream from the riprap which was relatively intact, Figure 41. The eroded material had moved downstream and formed a sandbar about 5 feet higher than the normal riverbed elevation.

The riprap proposed in Scheme No. 2 was installed in the model and tested at varying discharges. The initial tests were with 1,000 cubic feet per second through each valve and no flow through the powerhouse. After 2-1/2 hours operation (model time) there was extensive erosion downstream from the riprap, but no movement of the riprap was observed, Figure 43. The discharge was increased to 2,000-cubic-feet-per-second per valve, Figure 43, and the test continued for 1-1/2 hours. After this period of operation, the extent of the erosion had not materially increased, Figure 44. The test was continued for an additional 4 hours at the same discharge, during which the erosion in the jet impact area was very extensive but the riprap had not moved, Figure 44.

To further evaluate the riprap protection in Scheme No. 2 and to assure that the powerplant access road along the left bank would not be endangered, the same series of tests was repeated. The results were identical to the previous tests. Another test was made to determine the erosive effects of an extended period of operation with a discharge of 400 cubic feet per second at a low head in the river channel. The model was allowed to operate for a period of 48 hours (model time). The test indicated that the erosion hole would be relatively shallow and that the riprap protection in the tailrace would be unaffected by this discharge.

Recommendations.--Based on this series of tests, it was recommended that the riprap be placed as proposed for Scheme No. 2; the suggested gradation should be as shown in the following table:

Suggested Riprap Gradation

Size	Percent
Larger than 1.5 cubic yards	5 percent
0.6 to 1.5 cubic yards	20 percent
0.2 to 0.6 cubic yards	50 percent
0.1 to 0.2 cubic yards	25 percent

Table 1

METRIC EQUIVALENTS OF IMPORTANT QUANTITIES REFERRED TO IN THIS REPORT

Feature	English units	Metric units
Height of dam	502 feet	153.0 meters
Length of dam at crest	1,182 feet	360.3 meters
Volume of concrete in dam	922,000 cubic yards	705,000 cubic meters
Size of gates	16.75 by 34.00 feet	5.11 by 10.36 meters
Width of piers	4 feet	1.22 meters
Diameter of tunnel spillway	26.5 to 18.0 feet	8.08 to 5.49 meters
Head on crest at maximum discharge	39.0 feet	11.89 meters
Maximum discharge	28,800 cubic feet per second	816 cubic meters per second
Radius of flip bucket	145.0 feet	44.20 meters
Hollow-jet valve diameter	66 inches	1.68 meters
Size of riprap in afterbay	30 to 54 inches	76.2 to 137.2 cms

FIGURE 1
REPORT HYD-531

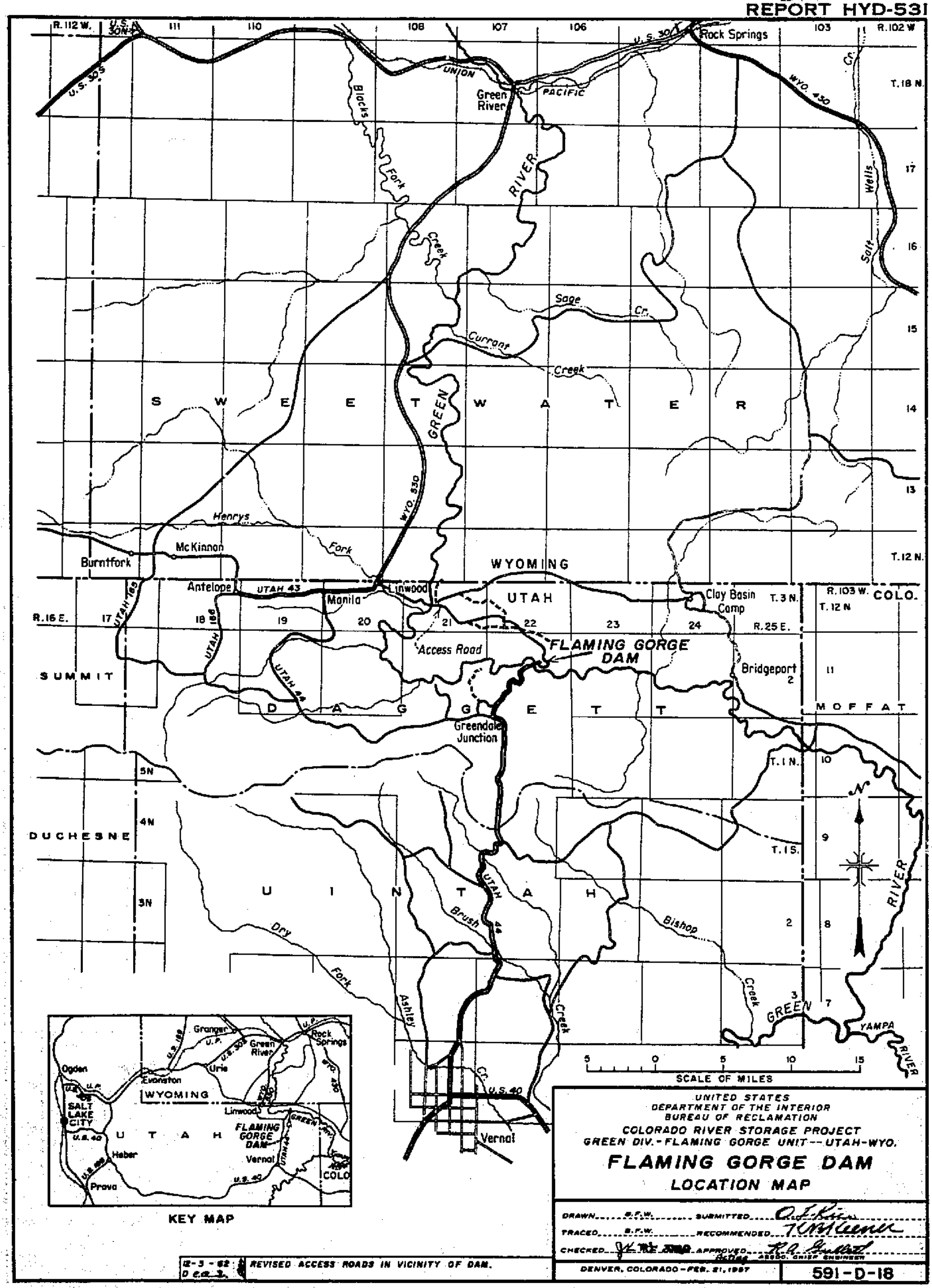


FIGURE 2
REPORT HYD-531

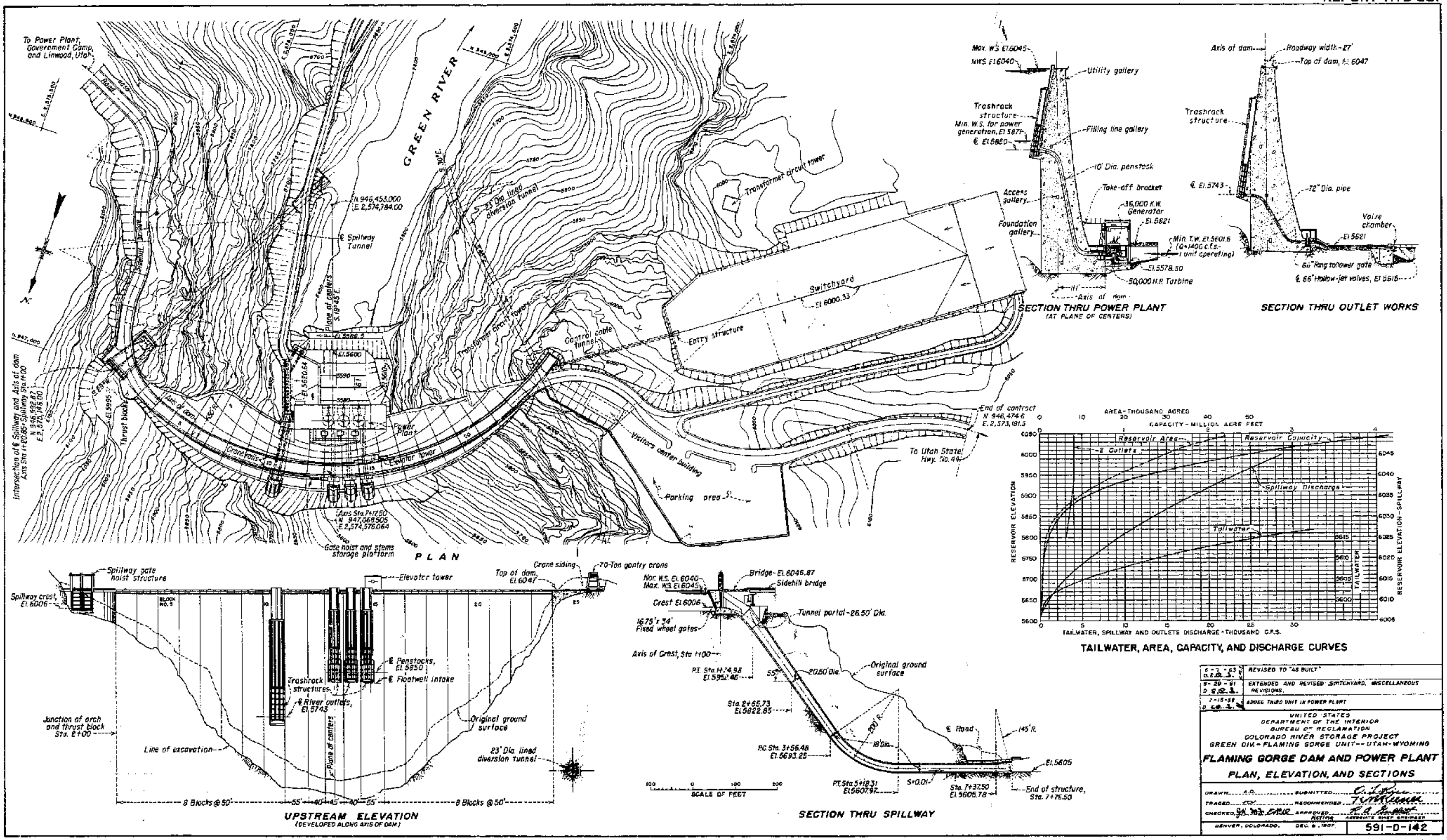
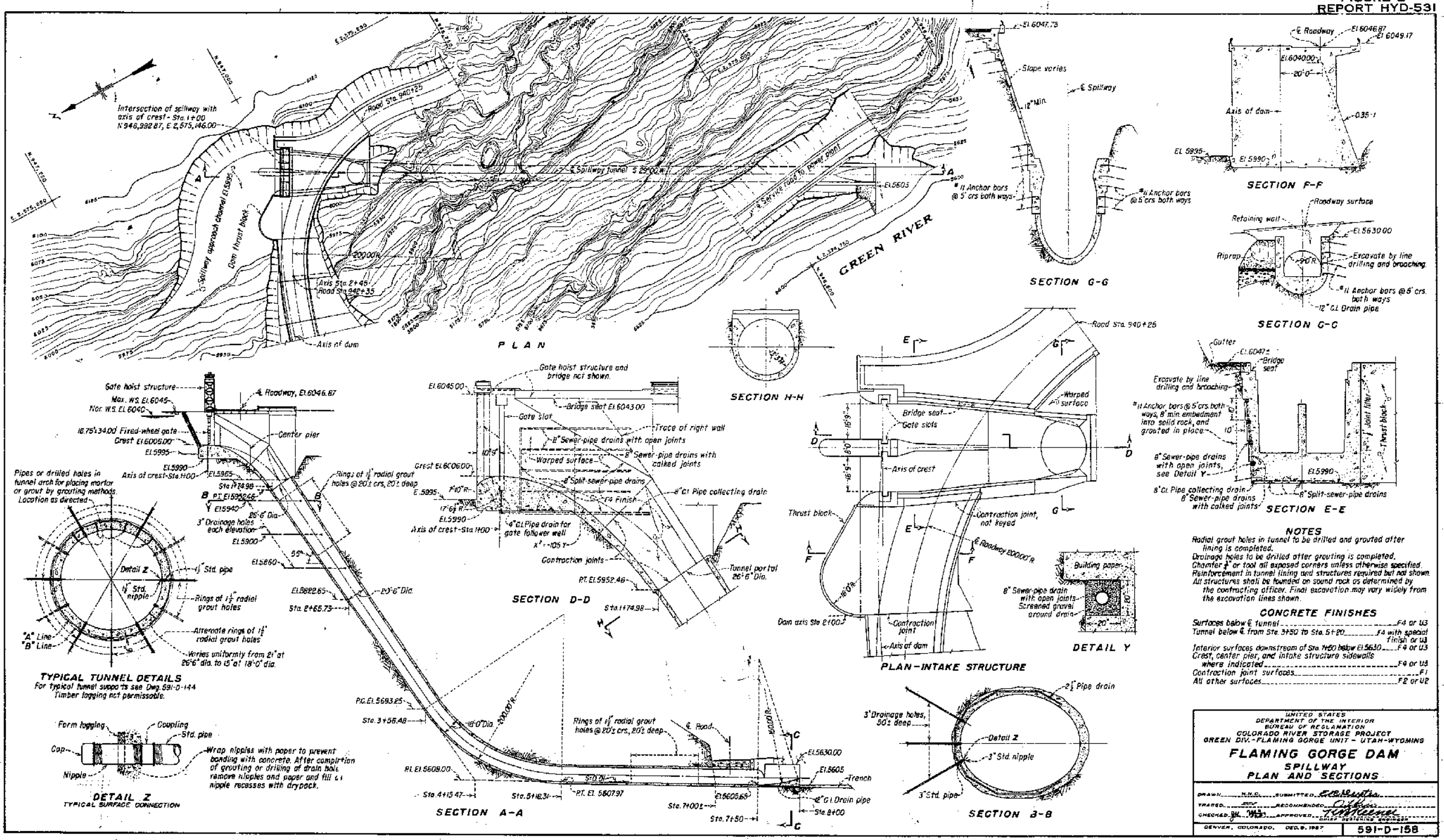
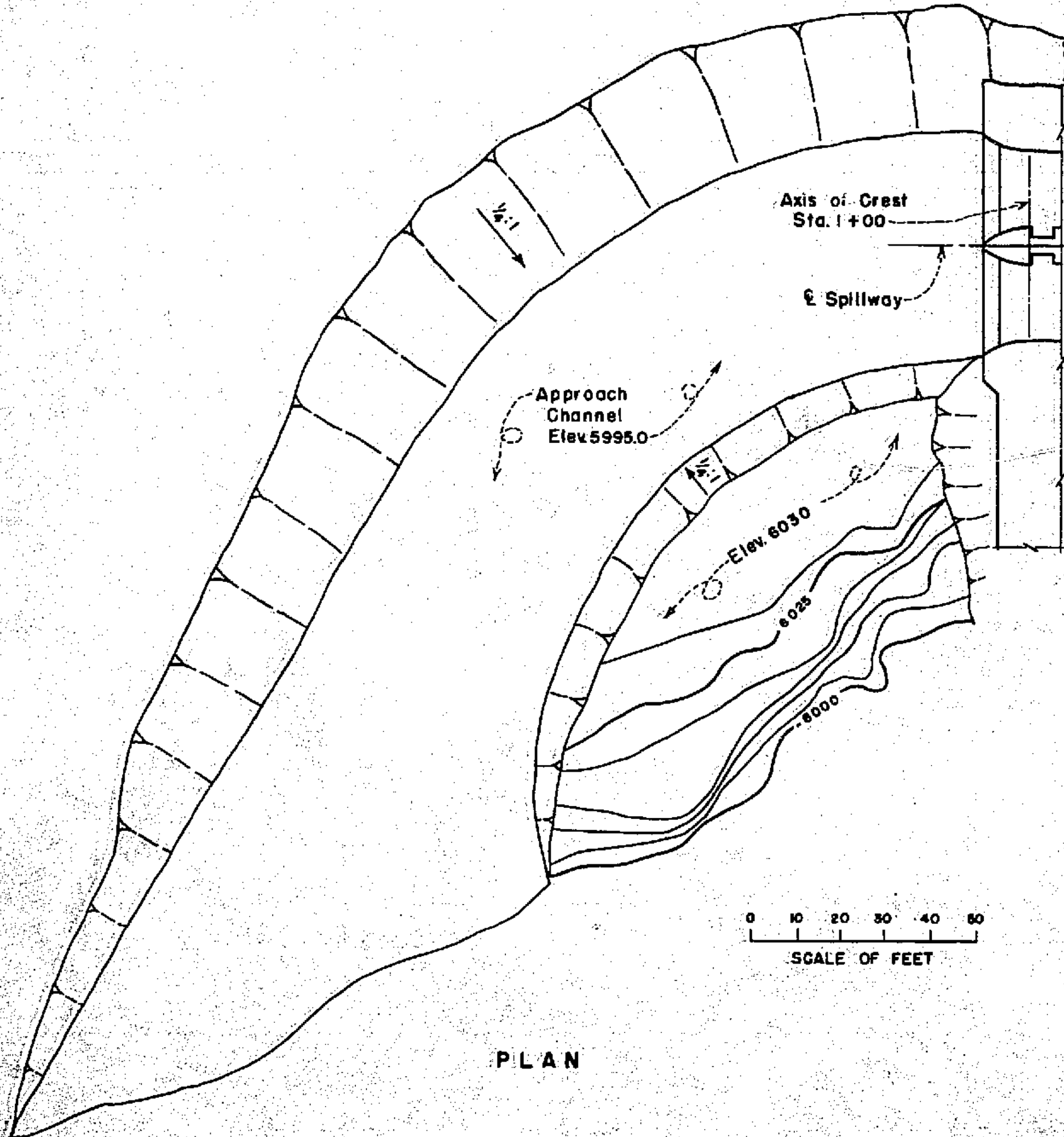


FIGURE 3
REPORT HYD-531

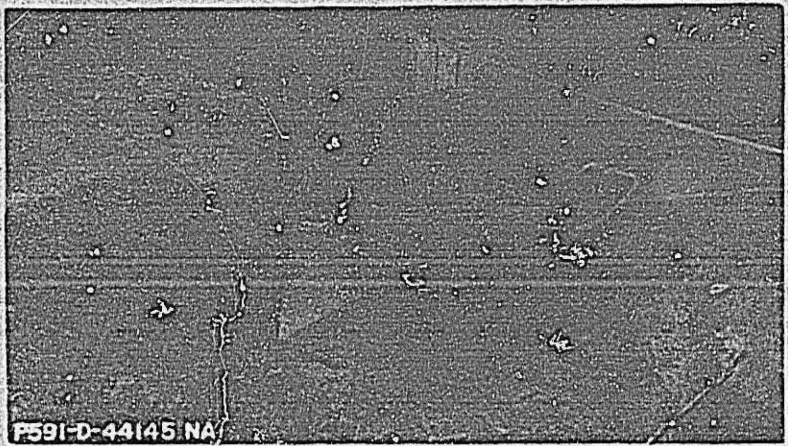


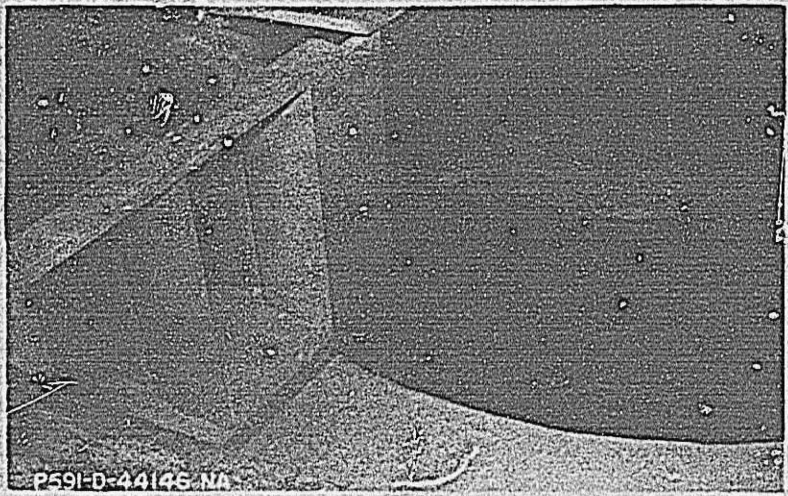


FLAMING GORGE DAM

1 : 36 SCALE MODEL

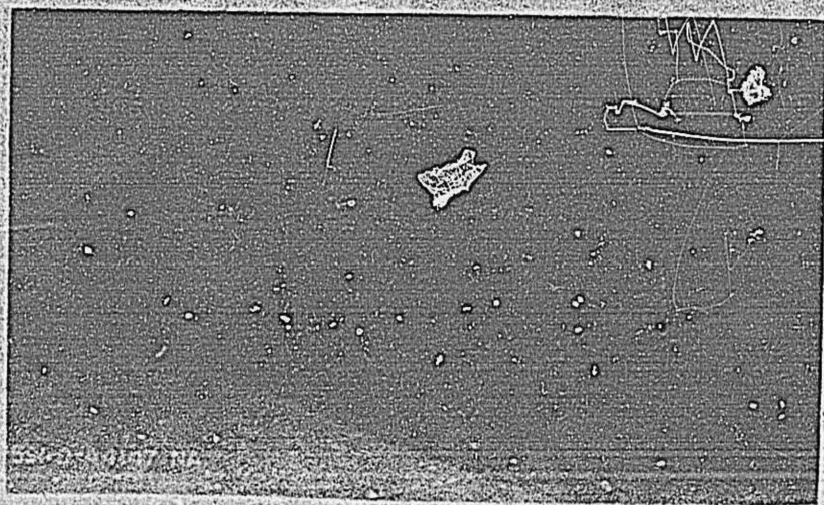
PRELIMINARY SPILLWAY APPROACH CHANNEL





FLAMING GORGE DAM
1:36 Scale Model

Preliminary Spillway
Approach Channel



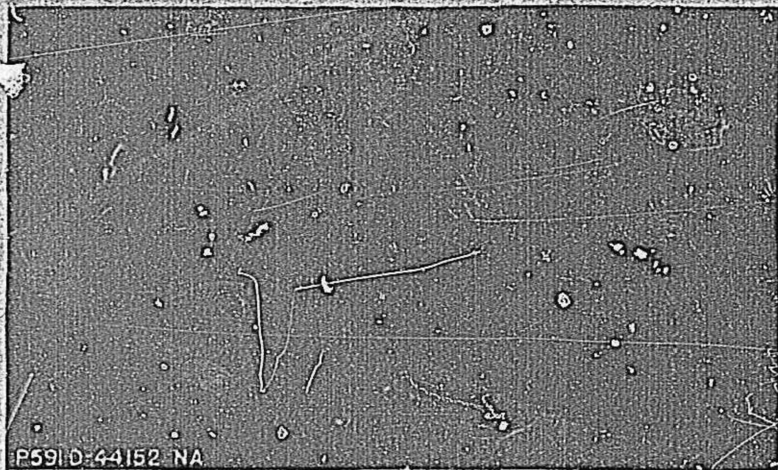
Q = 5,000 cfs

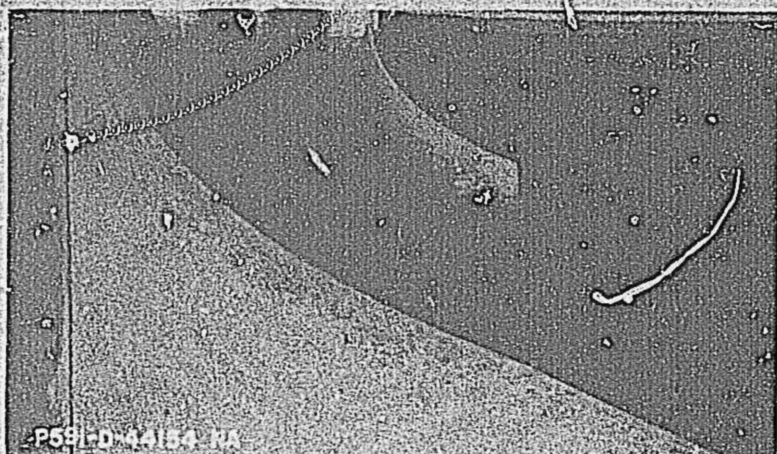
Q = 15,000 cfs

Q = 28,800 cfs

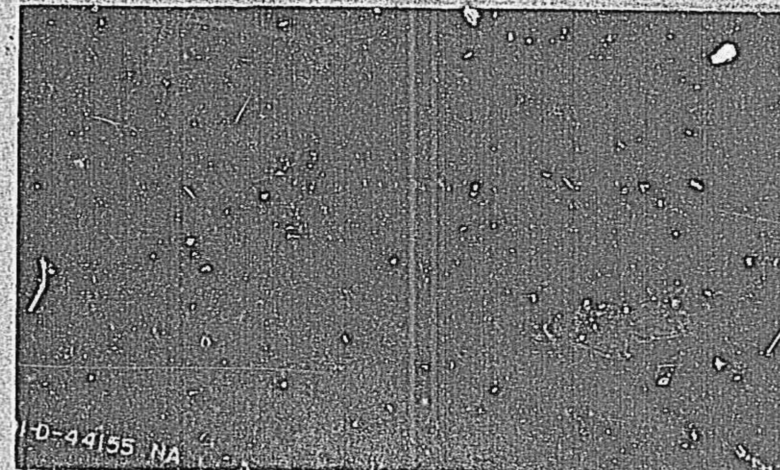
FLAMING GORGE DAM
1:36 Scale Model

Flow in Preliminary Spillway
Approach Channel





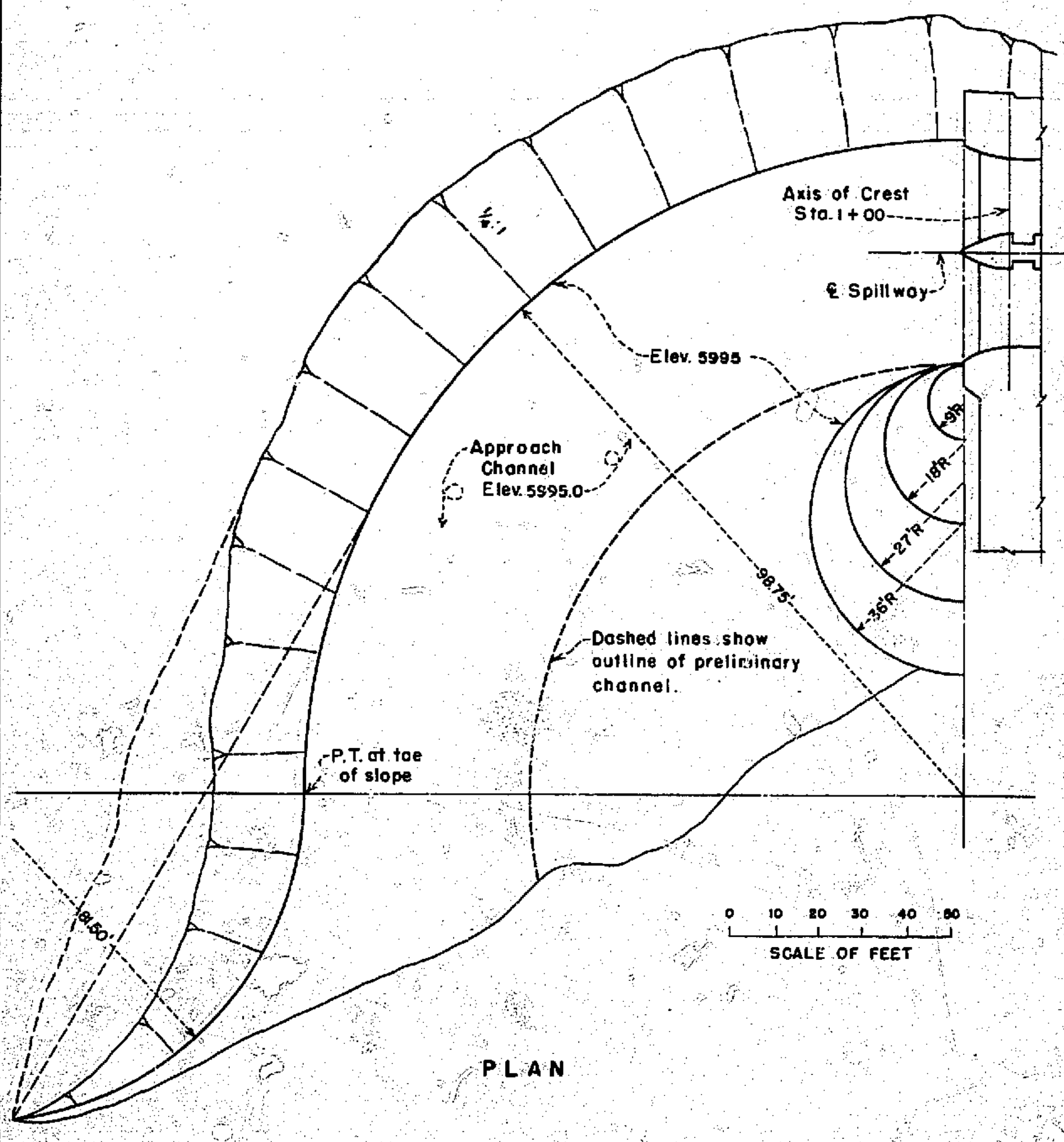
First modification



Second modification

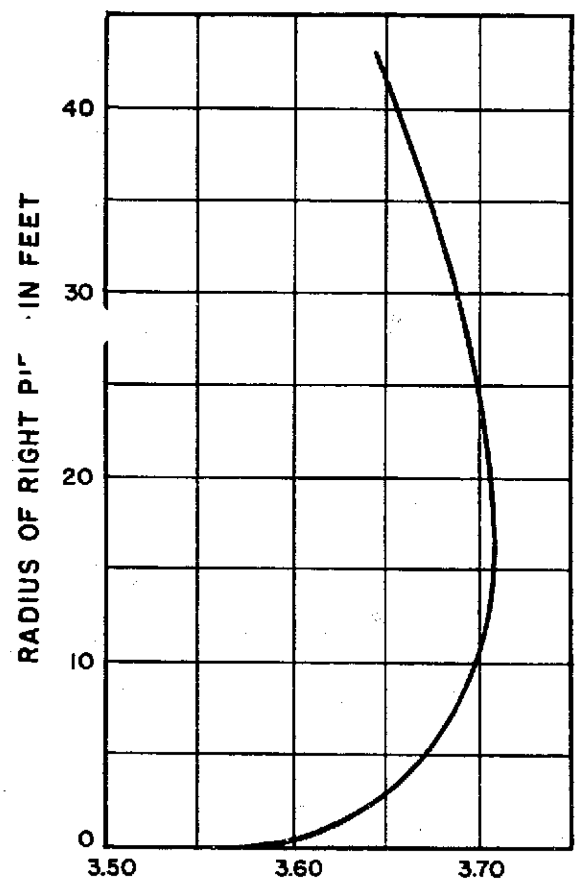
FLAMING GORGE DAM
1:36 Scale Model

Spillway Approach Channel Modifications
Discharge = 28, 800 cfs



PLAN

FLAMING GORGE DAM
1 : 36 SCALE MODEL
SPILLWAY APPROACH CHANNEL
THIRD MODIFICATION



COEFFICIENT OF DISCHARGE
IN $Q = CLH^{3/2}$

WHERE Q = Discharge = 28,800 cfs

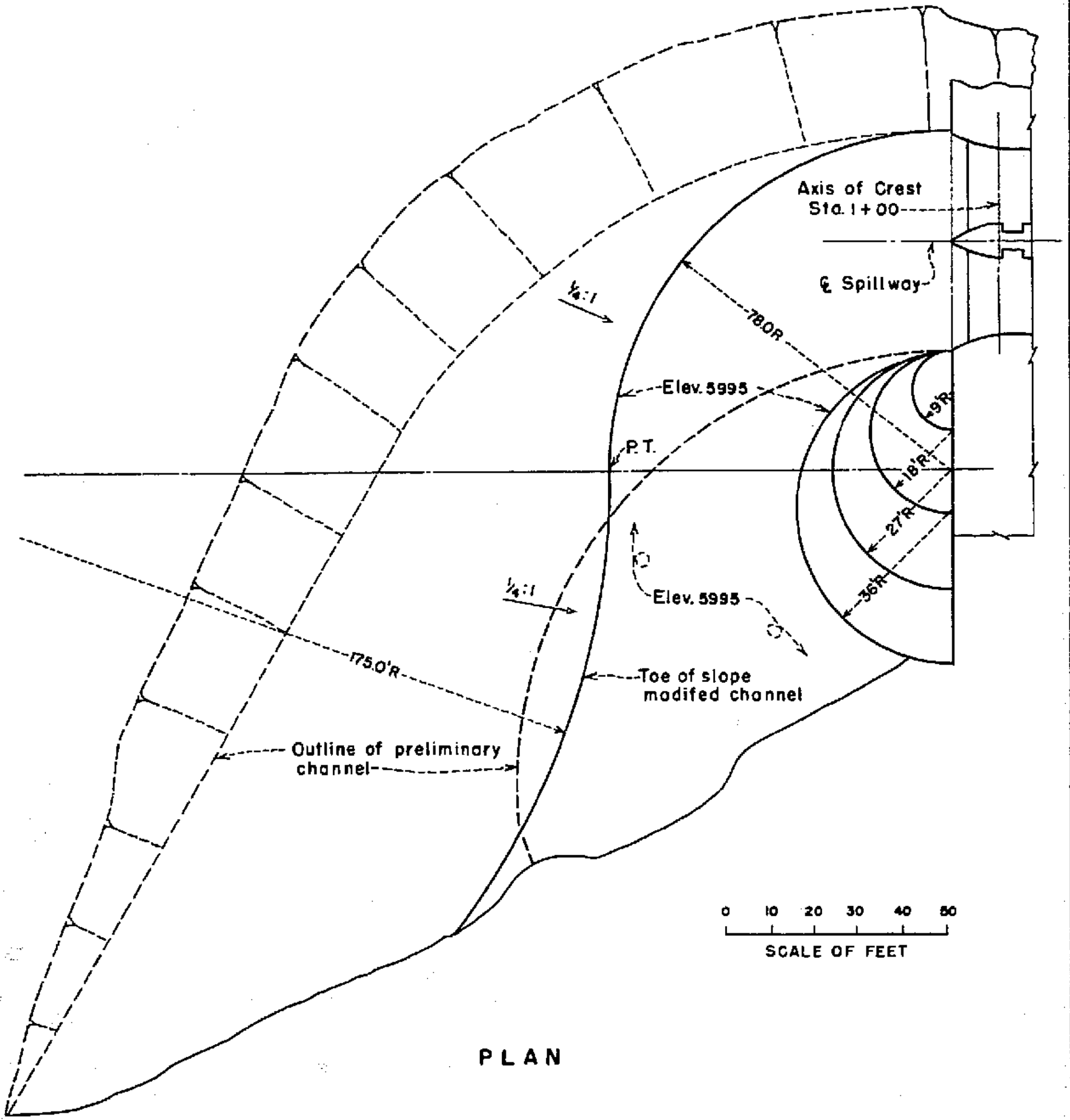
L = Crest length = 33.50'

H = Head above crest (varies)

FLAMING GORGE DAM

1:36 SCALE MODEL

PIER RADIUS VS. COEFFICIENT OF DISCHARGE.



FLAMING GORGE DAM

1 : 36 SCALE MODEL

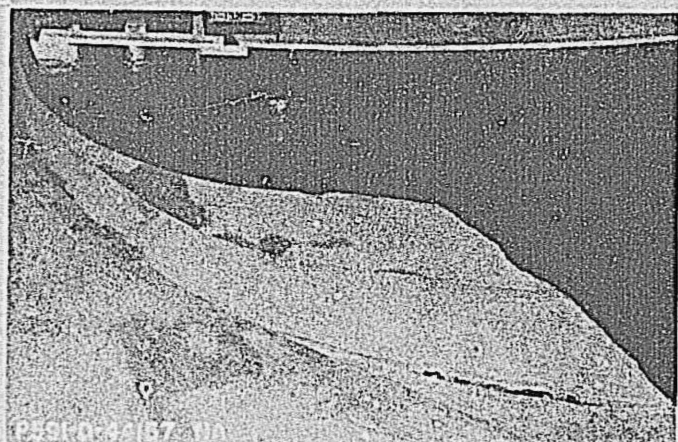
SPILLWAY APPROACH CHANNEL

FOURTH MODIFICATION

580

18-foot-radius pier on right side

18-foot-radius pier on right side



27-foot-radius pier on right side

A. Fourth modification

27-foot-radius pier on right side

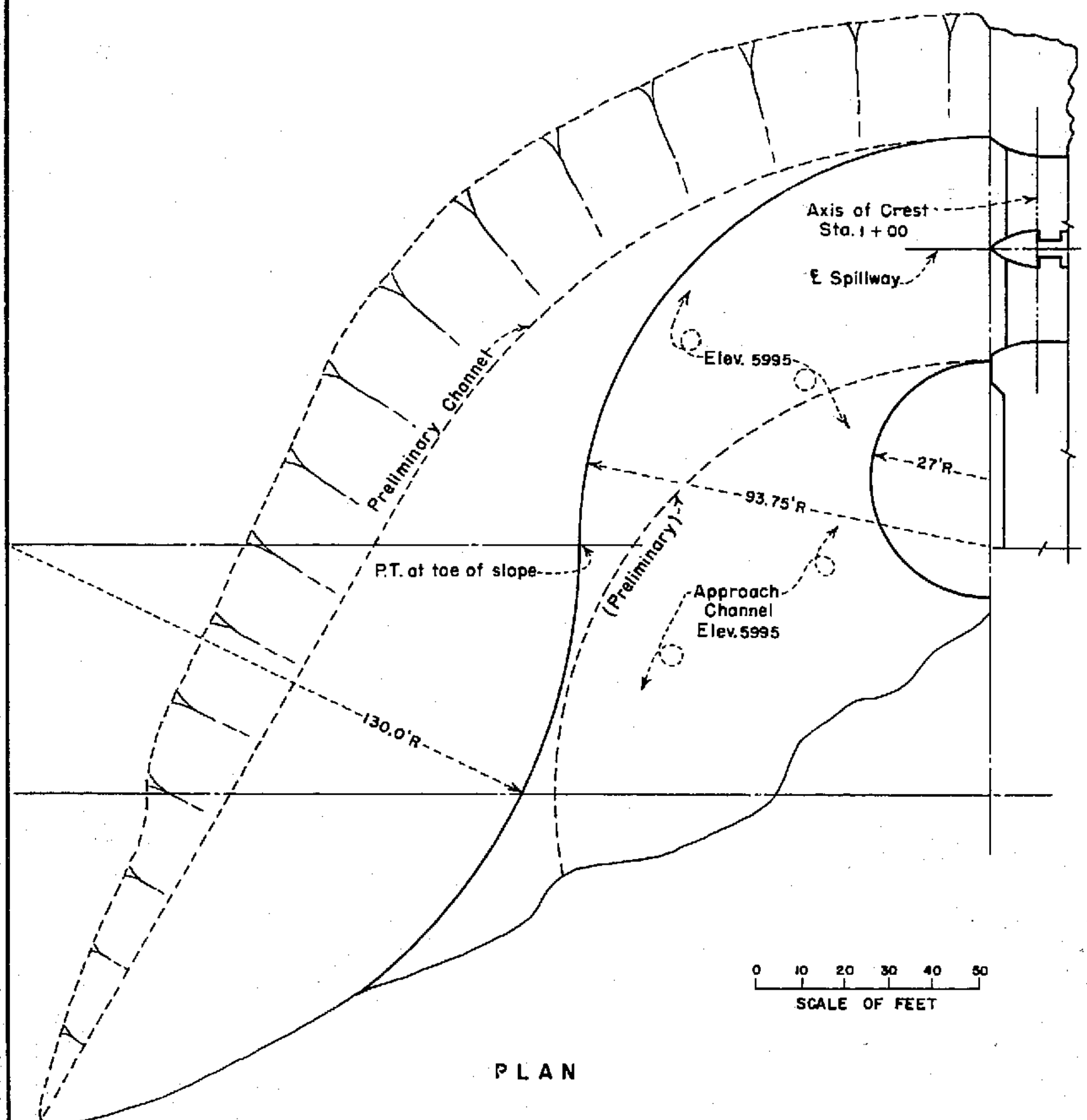
B. Fifth modification

Q = 28,800 cfs

FLAMING GORGE DAM

Spillway Approach Channel Modifications

Figure 11
Report Hyd-531



PLAN

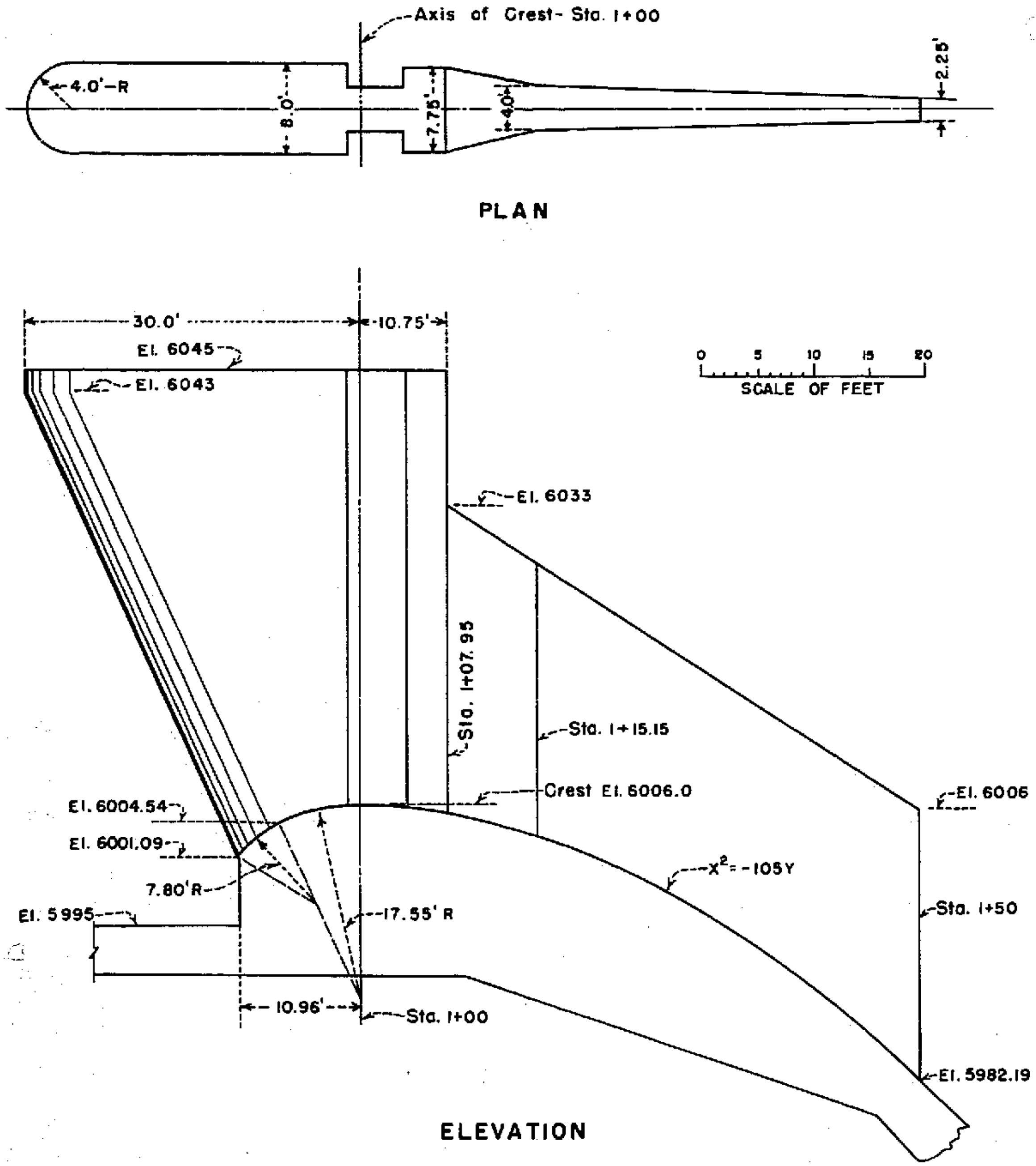
FLAMING GORGE DAM

1 : 36 SCALE MODEL

SPILLWAY APPROACH CHANNEL

FIFTH MODIFICATION

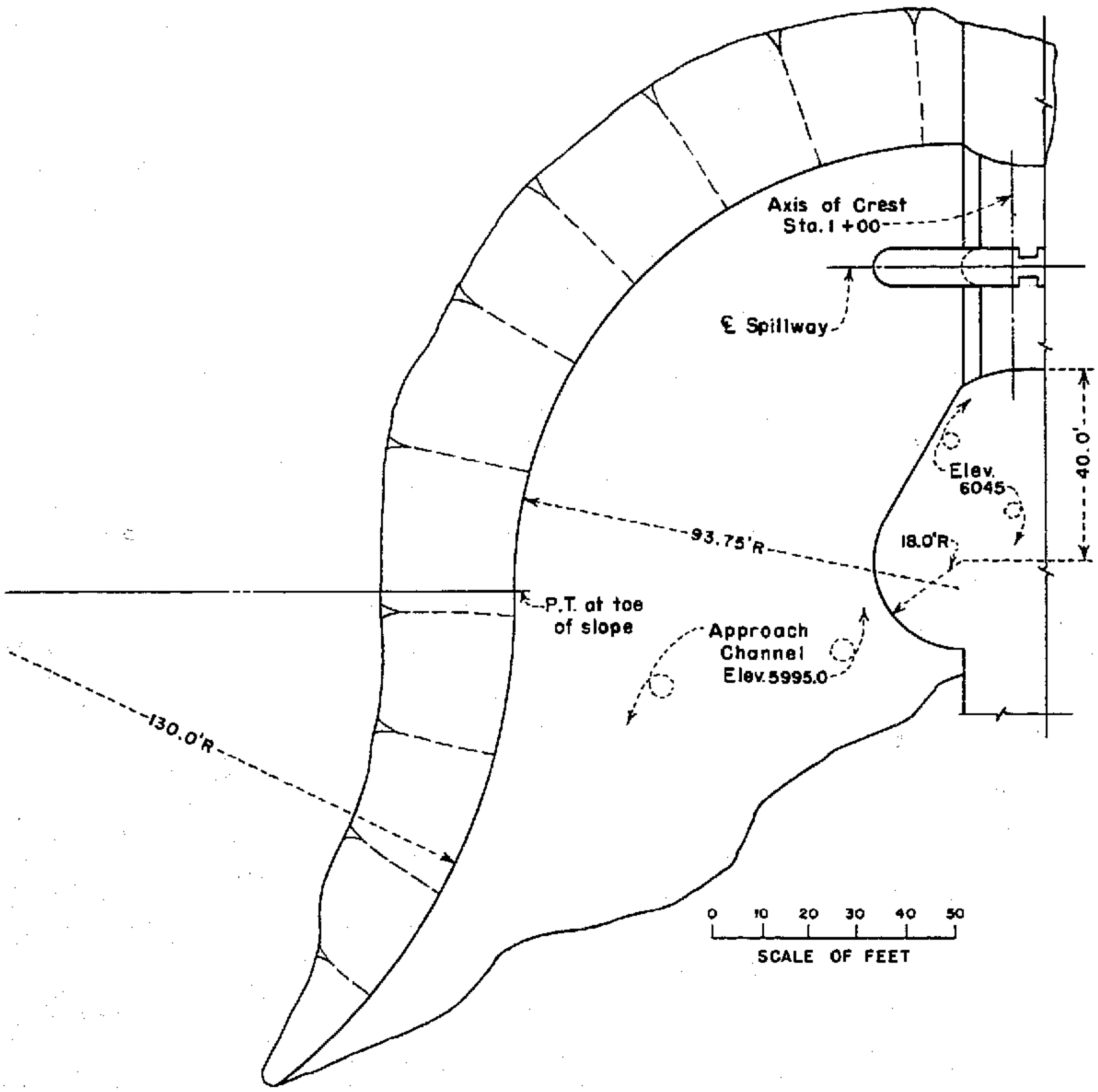
580



FLAMING GORGE DAM

1 : 36 SCALE MODEL

RECOMMENDED CENTER PIER



PLAN

FLAMING GORGE DAM

1 : 36 SCALE MODEL

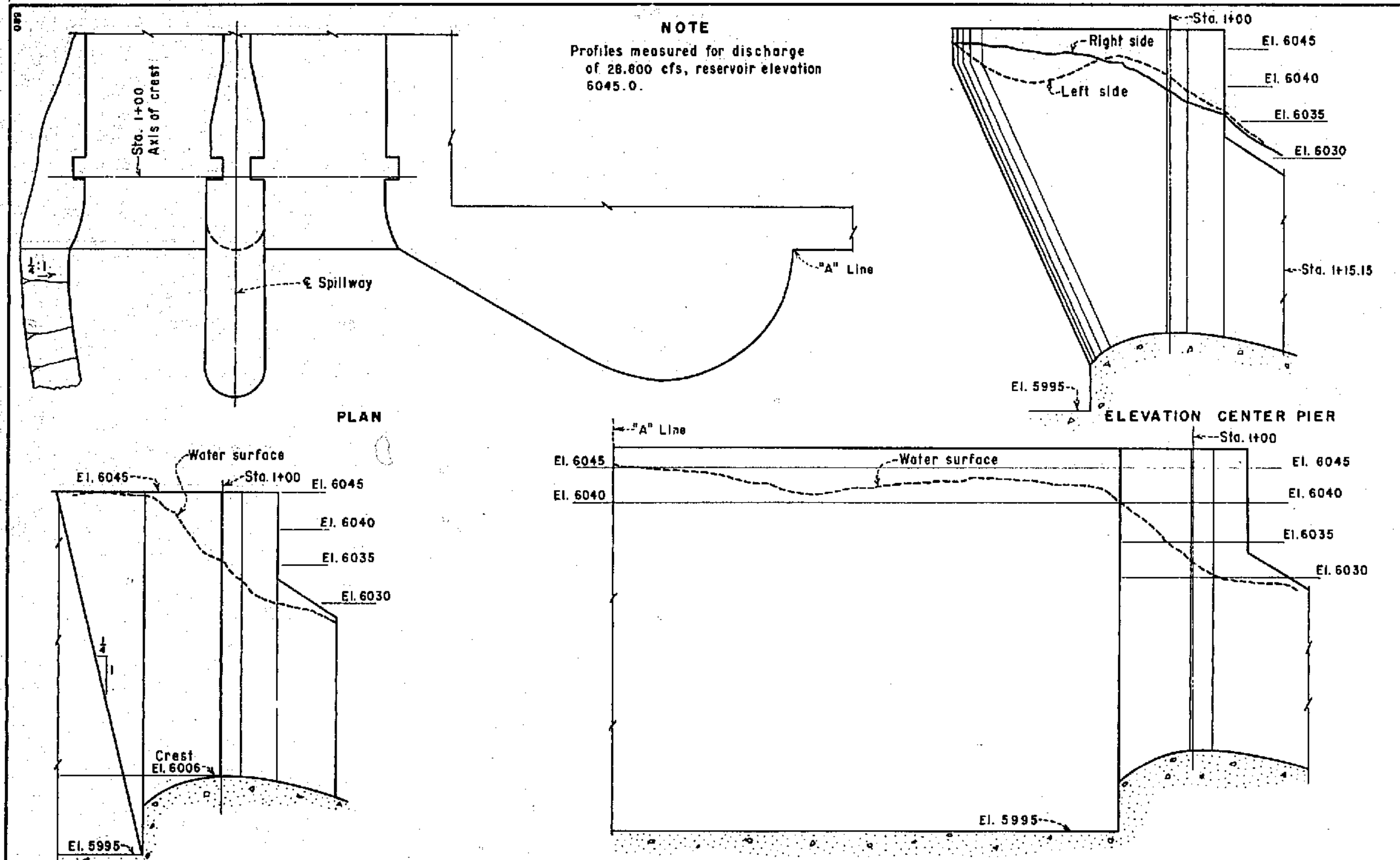
RECOMMENDED SPILLWAY APPROACH CHANNEL

580

Q = 28, 800 cfs

FLAMING GORGE DAM
1:36 Scale Model

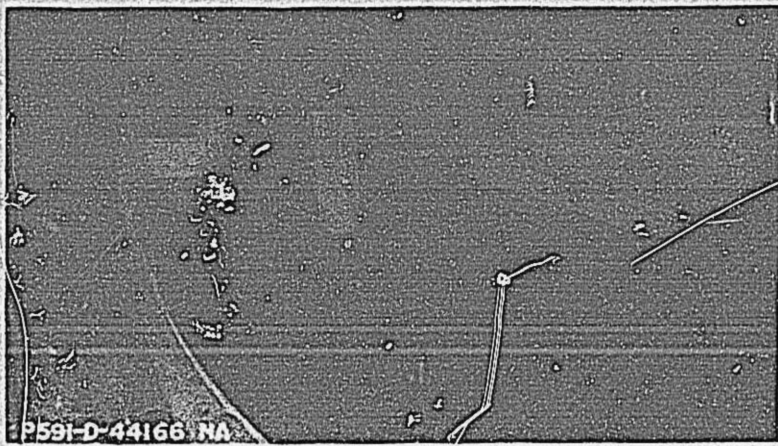
Flow in Recommended Spillway Approach Channel



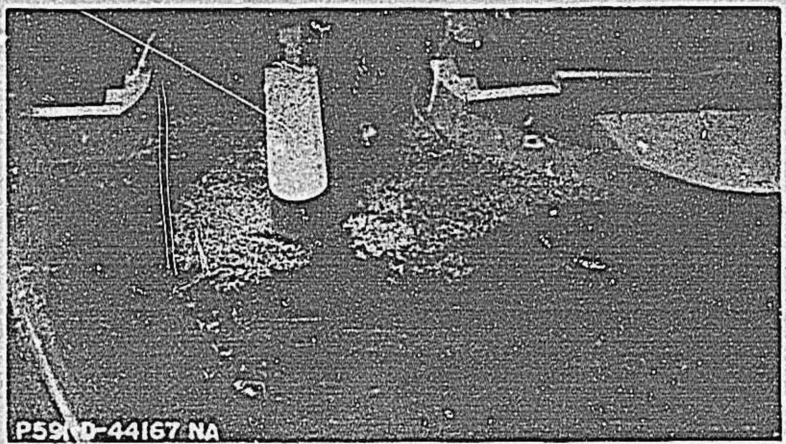
FLAMING GORGE DAM

1 : 36 SCALE MODEL

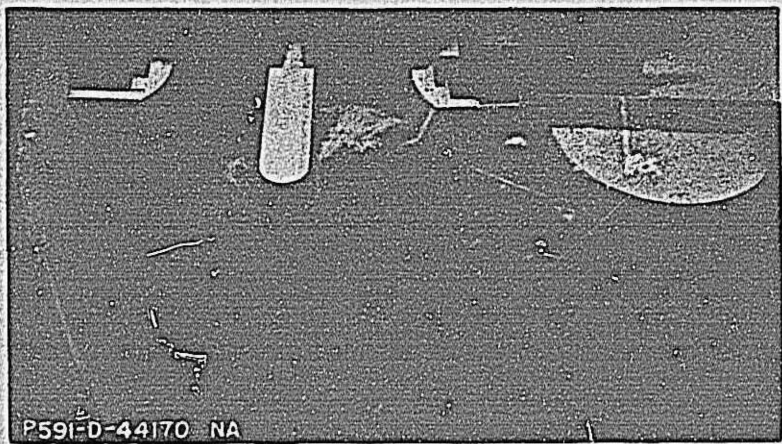
WATER SURFACE PROFILES



10-foot gate opening. Reservoir elevation 6045.0



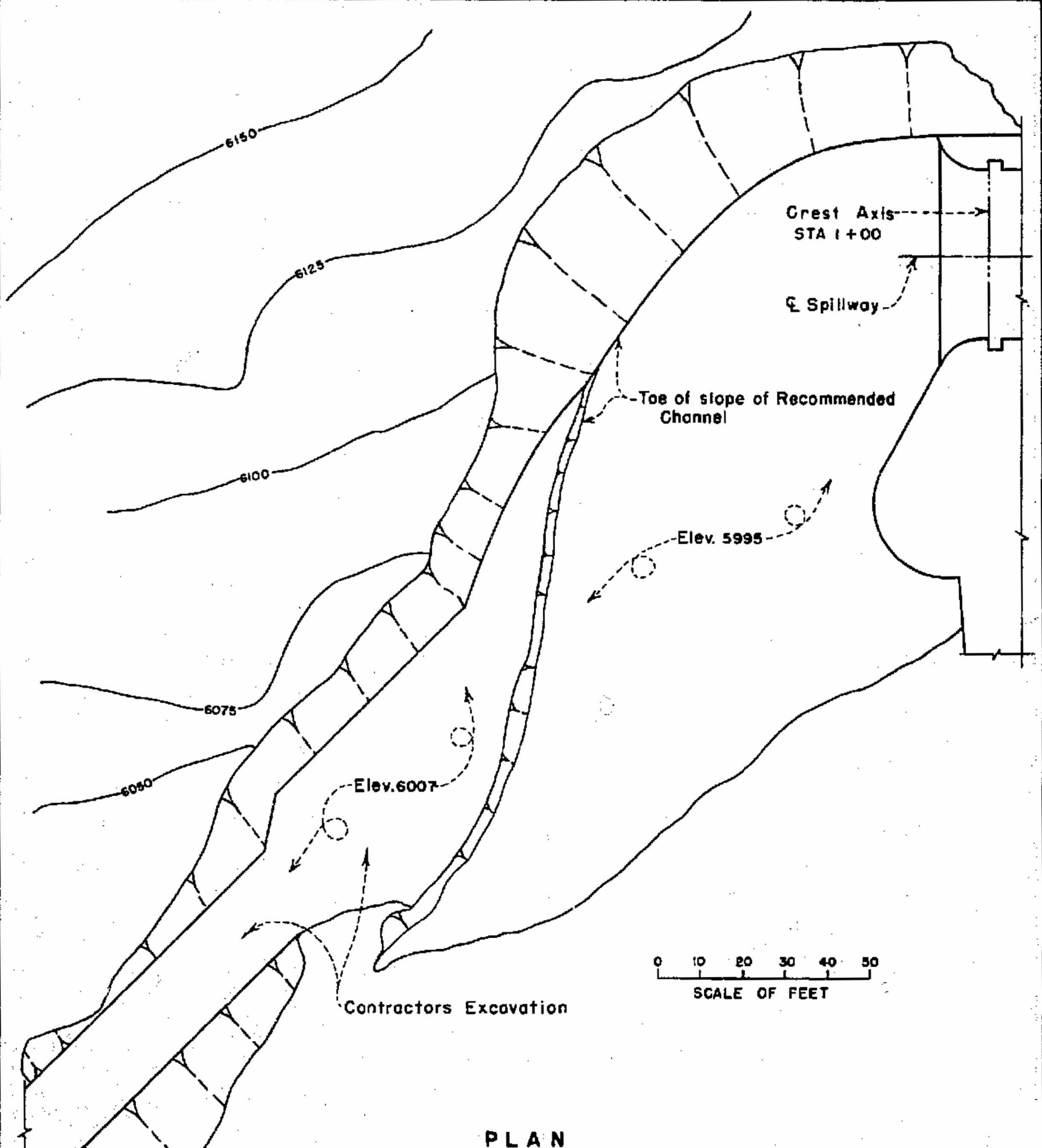
10-foot gate opening. Reservoir elevation 6045.0



20-foot gate opening. Reservoir elevation 6045.0

FLAMING GORGE DAM
1:36 Scale Model

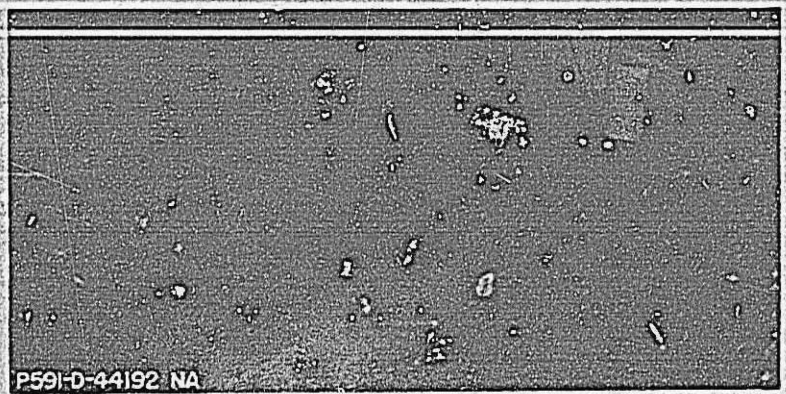
Vortices at Partial Gate Openings



FLAMING GORGE DAM

1 : 36 SCALE MODEL

CONTRACTORS EXCAVATION IN APPROACH AREA



No flow

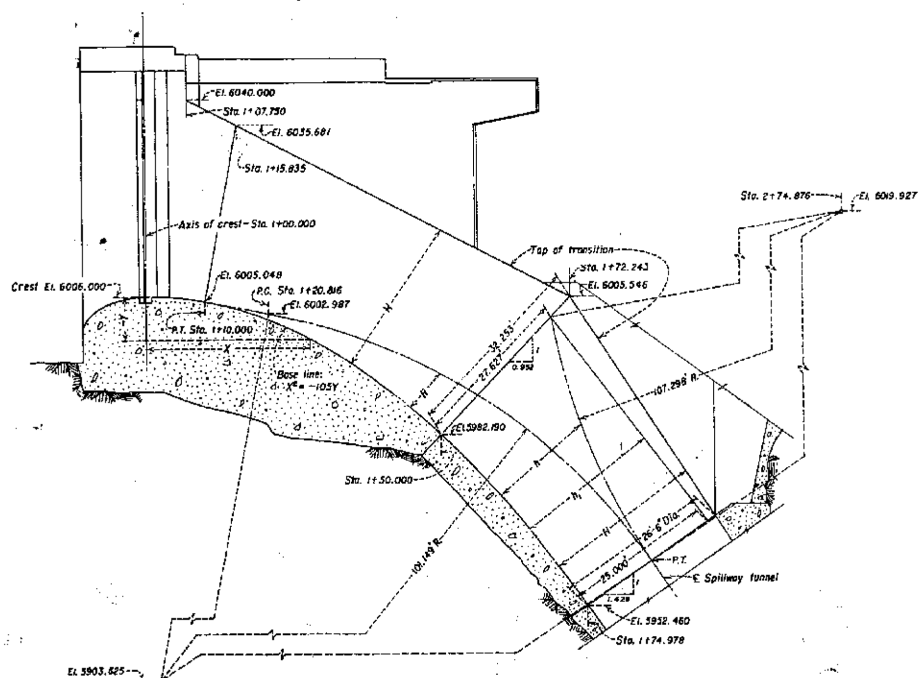
Q = 7, 200 cfs

Q = 28, 800 cfs

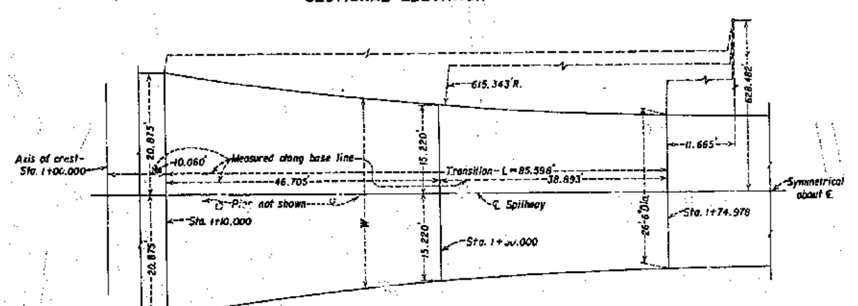
FLAMING GORGE DAM
1:36 Scale Model

Flow in Spillway Approach
with Contractors Excavation

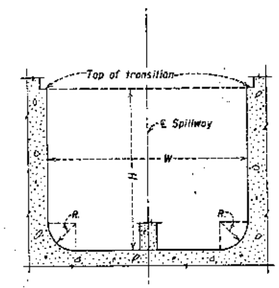
FIGURE 20
REPORT HYD-531



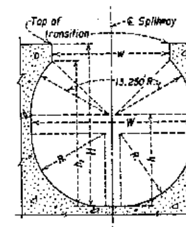
SECTIONAL ELEVATION



DIMENSIONAL DIAGRAM-PLAN
DEVELOPED ALONG BASE LINE



TYPICAL TRANSITION SECTION
STA. 1+10.000 TO STA. 1+50.000
SECTION NORMAL TO BASE LINE



TYPICAL TRANSITION SECTION
STA. 1+50.000 TO STA. 1+74.978
SECTION NORMAL TO BASE LINE

TRANSITION DATA

BASE LINE		TOP OF TRANSITION		TRANSITION ELEMENTS					
STA.	ELEV.	STA.	ELEV.	R	H	h	h_1	W	w
1+10.000	6005.048	1+15.835	6035.681	0'-0"	31'-2 1/4"			41'-9"	
1+11.899	6004.652	1+18.597	6034.205	0'-0 5/8"	30'-3 5/8"			41'-1 3/8"	
1+13.841	6004.175	1+21.368	6032.725	0'-1 5/8"	29'-6 1/4"			40'-6 1/8"	
1+15.766	6003.633	1+24.069	6031.282	0'-3 3/8"	28'-10 3/8"			39'-10 7/8"	
1+17.672	6003.026	1+26.708	6029.872	0'-6 5/8"	28'-3 7/8"			39'-3 3/8"	
1+19.557	6002.357	1+29.292	6028.492	0'-10 3/8"	27'-10 3/8"			38'-8 3/4"	
1+21.420	6001.630	1+31.827	6027.138	1'-2 7/8"	27'-6 3/8"			38'-2"	
1+23.260	6000.847	1+34.318	6025.807	1'-7 5/8"	27'-3 3/8"			37'-7 3/8"	
1+25.077	6000.011	1+36.772	6024.496	2'-0 5/8"	27'-1 5/8"			37'-0 7/8"	
1+26.869	5999.124	1+39.192	6023.203	2'-5 7/8"	27'-0 5/8"			36'-6 3/8"	
1+28.637	5998.190	1+41.584	6021.925	2'-11 3/8"	27'-0 1/2"			36'-0 3/8"	
1+30.381	5997.210	1+43.951	6020.660	3'-4 7/8"	27'-1 1/4"			35'-6 3/8"	
1+32.099	5996.187	1+46.297	6019.407	3'-10 5/8"	27'-2 3/8"			35'-0 1/4"	
1+33.793	5995.124	1+48.624	6018.164	4'-4 3/8"	27'-4 3/4"			34'-6 7/8"	
1+35.463	5994.023	1+50.936	6016.929	4'-10 1/8"	27'-7 3/4"			34'-1 3/8"	
1+37.108	5992.886	1+53.235	6015.701	5'-4"	27'-11 1/4"			33'-8"	
1+38.730	5991.715	1+55.523	6014.478	5'-9 7/8"	28'-3 1/2"			33'-2 3/4"	
1+40.327	5990.512	1+57.802	6013.261	6'-3 1/2"	28'-8 1/4"			32'-9 3/4"	
1+41.902	5989.278	1+60.074	6012.047	6'-9 1/8"	29'-1 3/8"			32'-4 7/8"	
1+43.454	5988.017	1+62.341	6010.836	7'-3 1/8"	29'-7 1/2"			32'-0 1/8"	
1+44.983	5986.729	1+64.604	6009.627	7'-8 3/4"	30'-1 7/8"			31'-7 3/4"	
1+46.491	5985.415	1+66.863	6008.420	8'-2 1/8"	30'-8 1/2"			31'-3 1/8"	
1+47.978	5984.077	1+69.122	6007.213	8'-7 1/2"	31'-4 1/8"			30'-10 7/8"	
1+49.444	5982.717	1+71.379	6006.007	9'-0 5/8"	31'-11 7/8"			30'-6 3/4"	
1+50.000	5982.190	1+72.243	6005.546	9'-2 1/2"	32'-3"	27'-7 1/2"	27'-7 1/2"	30'-5 1/4"	30'-5 1/4"
1+50.890	5981.335	1+73.089	6004.235	9'-5 1/8"	31'-10 3/4"	26'-6 3/8"	27'-4 3/8"	30'-2 7/8"	30'-2 1/4"
1+52.317	5979.933	1+74.443	6002.137	9'-10 5/8"	31'-4 1/8"	24'-11 3/8"	27'-0 1/8"	29'-11 1/8"	29'-7 3/8"
1+53.734	5978.512	1+75.779	6000.065	10'-2 7/8"	30'-10"	23'-6"	26'-8 1/4"	29'-7 1/2"	28'-10 3/8"
1+55.112	5977.073	1+77.100	5998.018	10'-7 1/4"	30'-4 5/8"	22'-2 1/4"	26'-4 5/8"	29'-4"	27'-11 3/8"
1+56.483	5975.616	1+78.406	5995.993	10'-11 3/8"	29'-11 1/8"	21'-0"	26'-1 1/2"	29'-0 3/8"	27'-0"
1+57.836	5974.143	1+79.699	5993.989	11'-3 1/2"	29'-6 3/8"	19'-11"	25'-10 7/8"	28'-9 1/2"	25'-11 1/2"
1+59.172	5972.655	1+80.980	5992.004	11'-6 3/4"	29'-1 7/8"	18'-11 3/8"	25'-8 1/8"	28'-6 1/2"	24'-10 1/2"
1+60.491	5971.151	1+82.250	5990.036	11'-10 1/8"	28'-9 3/4"	18'-0 1/4"	25'-6"	28'-3 3/4"	23'-9"
1+61.794	5969.634	1+83.509	5988.083	12'-1 1/4"	28'-5 7/8"	17'-3 1/4"	25'-4 1/8"	28'-1"	22'-7 3/8"
1+63.081	5968.103	1+84.759	5986.145	12'-4"	28'-2 1/2"	16'-6 3/4"	25'-2 7/8"	27'-10 1/2"	21'-5 1/2"
1+64.353	5966.560	1+86.001	5984.221	12'-6 1/2"	27'-11 1/4"	15'-11 1/4"	25'-1 1/8"	27'-8 1/8"	20'-3 3/4"
1+65.609	5965.004	1+87.235	5982.308	12'-8 3/8"	27'-8 3/8"	15'-4 1/2"	25'-0 1/8"	27'-5 7/8"	19'-2 1/8"
1+66.852	5963.437	1+88.462	5980.407	12'-10 5/8"	27'-5 1/2"	14'-10 3/8"	24'-11 3/8"	27'-3 3/8"	18'-0 7/8"
1+68.080	5961.858	1+89.681	5978.516	13'-0 1/8"	27'-3 3/8"	14'-5 3/8"	24'-10 7/8"	27'-2"	17'-0"
1+69.295	5960.269	1+90.895	5976.635	13'-1 5/8"	27'-1 1/4"	14'-1 3/8"	24'-10 1/2"	27'-0 1/2"	15'-11 3/4"
1+70.496	5958.670	1+92.104	5974.762	13'-2 3/8"	26'-11 1/4"	13'-9 1/2"	24'-10 1/8"	26'-10 3/8"	15'-0 1/4"
1+71.686	5957.061	1+93.307	5972.897	13'-3"	26'-9 3/8"	13'-7 1/8"	24'-10 1/2"	26'-9 1/2"	14'-1 1/2"
1+72.859	5955.443	1+94.505	5971.040	13'-3 1/4"	26'-8 1/8"	13'-5"	24'-10 5/8"	26'-7 3/8"	13'-4 3/8"
1+74.022	5953.816	1+95.698	5969.190	13'-3 1/4"	26'-6 7/8"	13'-3 1/8"	24'-11 3/8"	26'-6 3/4"	12'-8 5/8"
1+74.978	5952.460	1+96.685	5967.560	13'-3"	26'-6"	13'-3"	25'-0"	26'-6"	12'-3"

CONCRETE FINISH

Transition surfaces..F4 or U3

REFERENCE DRAWINGS

SPILLWAY-INTAKE STRUCTURE-
PLAN AND SECTIONS..591-D-1214 AND 1215

6-12-62	REVISED W DIMENSION-STA. 1+63.081 AND R DIMENSION-STA. 1+69.295
2-1-62	ADDED COVER SLAB. MINOR REVISIONS.

THIS DRAWING SUPERSEDES DWG. 591-D-158 IN PART

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
GREEN DIV.-FLAMING GORGE UNIT-- UTAH-WYOMING

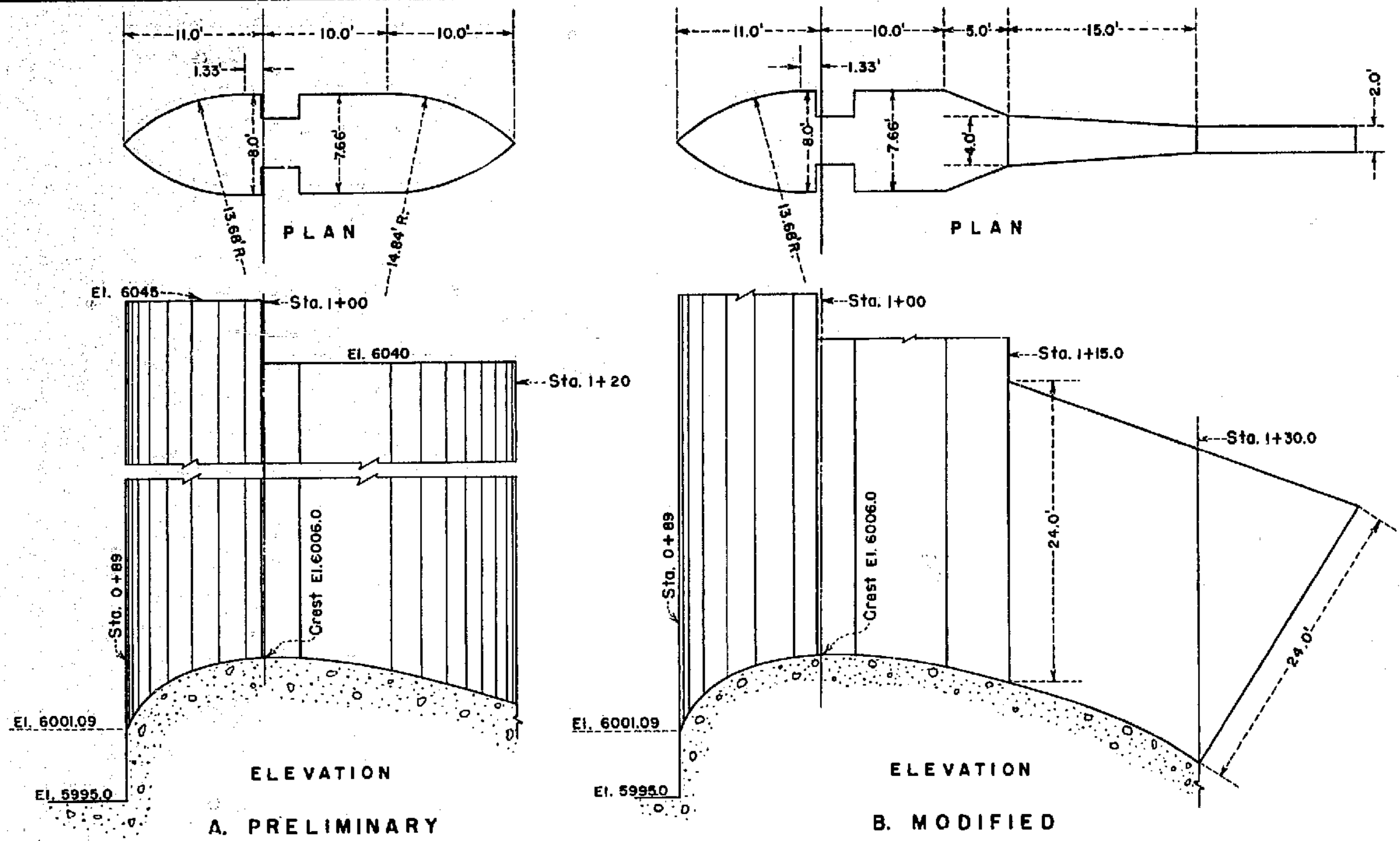
FLAMING GORGE DAM

SPILLWAY

INTAKE STRUCTURE TRANSITION ELEMENTS

DRAWN: E.A.L. SUBMITTED: [signature]
TRACED: RECOMMENDED: [signature]
CHECKED: APPROVED: [signature]
ASST. CHIEF, DAMS BRANCH

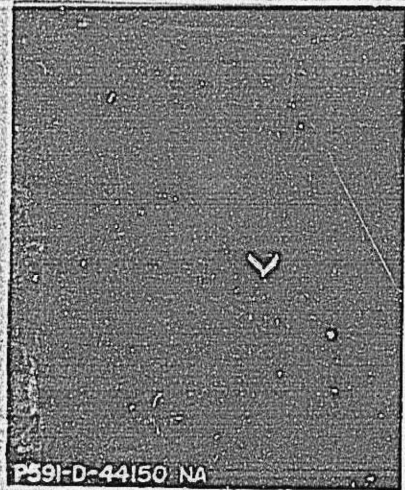
DENVER, COLORADO, DEC. 18, 1961 591-D-1216



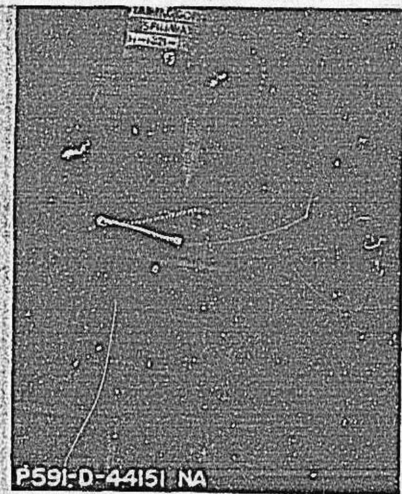
FLAMING GORGE DAM

1 : 36 SCALE MODEL

SPILLWAY PIER STUDIES



Preliminary pier with preliminary approach channel



Preliminary pier and first modification to approach channel

Modified pier with first modification to approach channel

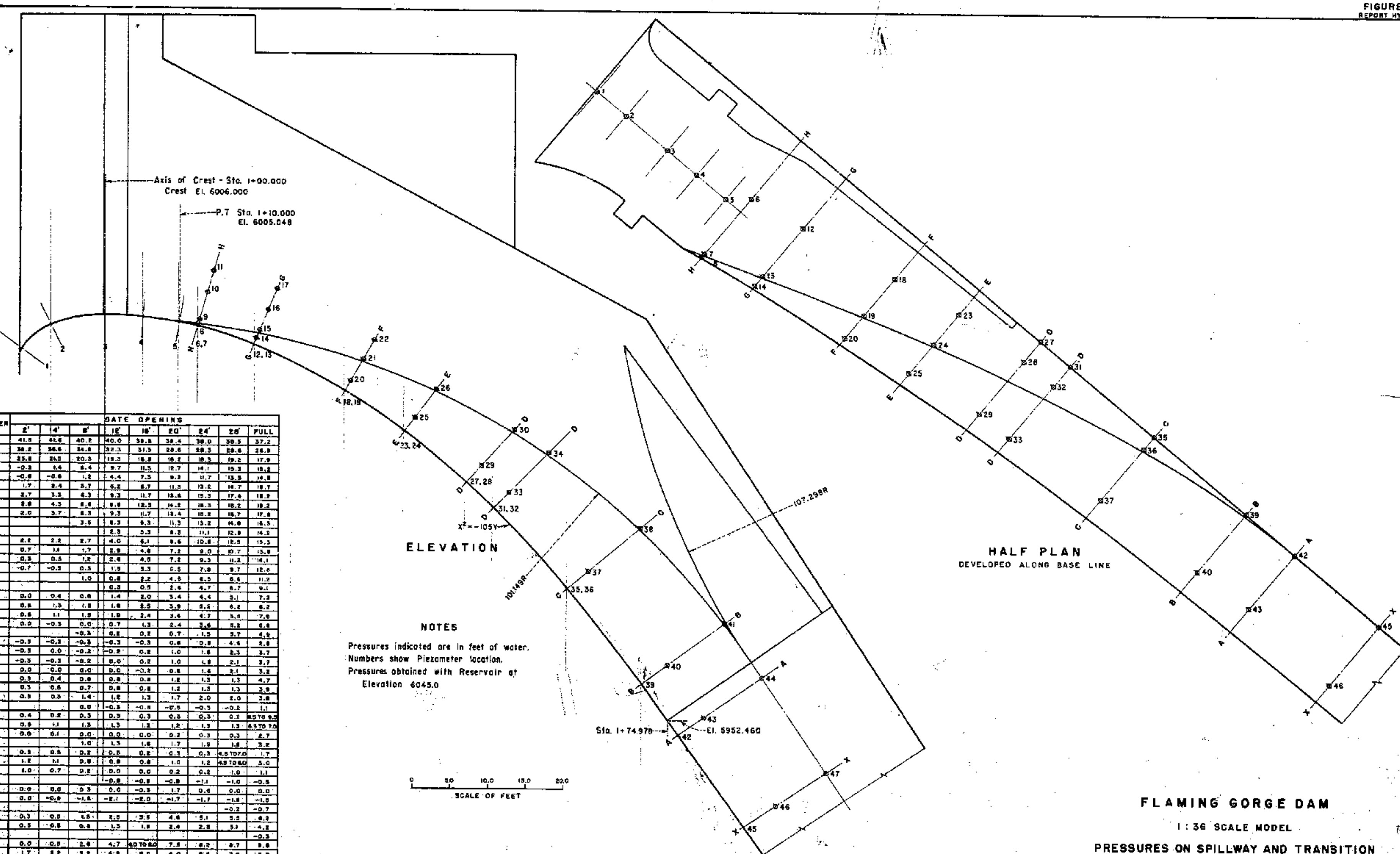
A. Maximum discharge in transition with preliminary and modified pier

Note "rope" of air at end of pier.

B. Maximum discharge in transition with recommended pier and recommended approach channel.

FLAMING GORGE DAM
1:36 Scale Model

Flow with Preliminary Modified and Recommended Center Pier.



PIEZOMETER	GATE OPENING								
	2'	4'	8'	12'	16'	20'	24'	28'	FULL
1	41.8	45.6	40.2	40.0	39.8	39.4	39.0	39.5	37.2
2	38.2	36.6	34.8	32.3	31.5	28.6	28.3	28.6	26.9
3	25.8	24.5	20.3	19.3	18.8	18.2	18.3	19.2	17.9
4	-0.3	4.4	8.4	9.7	11.5	12.7	14.1	15.3	16.2
5	-0.8	-0.6	1.2	4.4	7.3	9.2	11.7	13.5	14.8
6	1.7	2.4	3.7	6.2	8.7	11.3	13.2	16.7	18.7
7	2.7	3.3	4.3	9.3	11.7	13.6	15.3	17.4	18.2
8	2.8	4.3	6.6	8.8	12.3	14.2	16.3	18.2	19.2
9	2.0	3.7	6.3	9.3	11.7	13.4	16.2	16.7	17.8
10			3.6	8.3	9.3	11.3	13.2	14.8	16.5
11				2.3	5.3	8.3	11.1	12.8	14.2
12	2.2	2.2	2.7	4.0	6.1	8.6	10.6	12.5	15.3
13	0.7	1.0	1.7	2.9	4.6	7.2	9.0	10.7	13.8
14	0.3	0.5	1.2	2.4	4.6	7.2	9.3	11.2	14.1
15	-0.7	-0.3	0.3	1.5	3.3	6.5	7.8	9.7	12.6
16			1.0	0.8	2.2	4.8	6.5	6.6	11.2
17				0.3	0.5	2.6	4.7	6.7	9.1
18	0.0	0.4	0.8	1.4	2.0	3.4	4.4	5.1	7.3
19	0.8	1.3	1.8	1.8	2.5	3.9	6.5	6.2	8.2
20	0.6	1.1	1.5	1.9	2.4	3.4	4.7	5.5	7.0
21	0.0	-0.3	0.0	0.7	1.3	2.4	3.6	5.2	6.8
22			-0.3	0.2	0.2	0.7	1.5	3.7	4.9
23	-0.3	-0.3	-0.3	-0.3	-0.3	0.6	0.8	4.4	2.8
24	-0.3	0.0	-0.2	-0.2	0.2	1.0	1.8	2.3	3.7
25	-0.3	-0.3	-0.2	0.0	0.2	1.0	1.8	2.1	3.7
26	0.0	0.0	0.0	0.0	-0.2	0.8	1.6	2.1	3.2
27	0.3	0.4	0.8	0.8	0.8	1.2	1.3	1.3	4.7
28	0.3	0.6	0.7	0.8	0.8	1.2	1.3	1.3	3.9
29	0.5	0.3	1.4	1.2	1.3	1.7	2.0	2.0	3.8
30			0.0	-0.3	-0.8	-0.5	-0.3	-0.2	1.1
31	0.4	0.2	0.3	0.3	0.3	0.3	0.3	0.2	8.5 TO 9.5
32	0.8	1.1	1.3	1.3	1.2	1.2	1.3	1.3	6.5 TO 7.0
33	0.0	0.1	0.0	0.0	0.0	0.2	0.3	0.3	2.7
34			1.0	1.3	1.6	1.7	1.9	1.8	3.2
35	0.3	0.5	0.2	0.5	0.2	0.3	0.3	4.5 TO 7.0	1.7
36	1.2	1.1	0.8	0.8	0.8	1.0	1.2	4.5 TO 6.0	3.0
37	1.0	0.7	0.2	0.0	0.0	0.2	0.2	1.0	1.1
38				-0.8	-0.8	-0.8	-1.1	-1.0	-0.5
39	0.0	0.0	0.3	0.0	-0.3	1.7	0.6	0.0	0.0
40	0.0	-0.9	-1.8	-2.1	-2.0	-1.7	-1.7	-1.8	-1.8
41								-0.2	-0.7
42	0.3	0.5	1.5	2.5	3.5	4.6	5.1	5.5	6.2
43	0.5	0.8	0.8	1.3	1.8	2.4	2.8	3.3	4.2
44									-0.3
45	0.0	0.5	2.8	4.7	4.0 TO 8.0	7.8	8.2	8.7	9.6
46	1.7	2.2	2.2	4.9	5.5	6.0	8.5	7.0	8.0
47					0.7		5.8	1.8	3.2

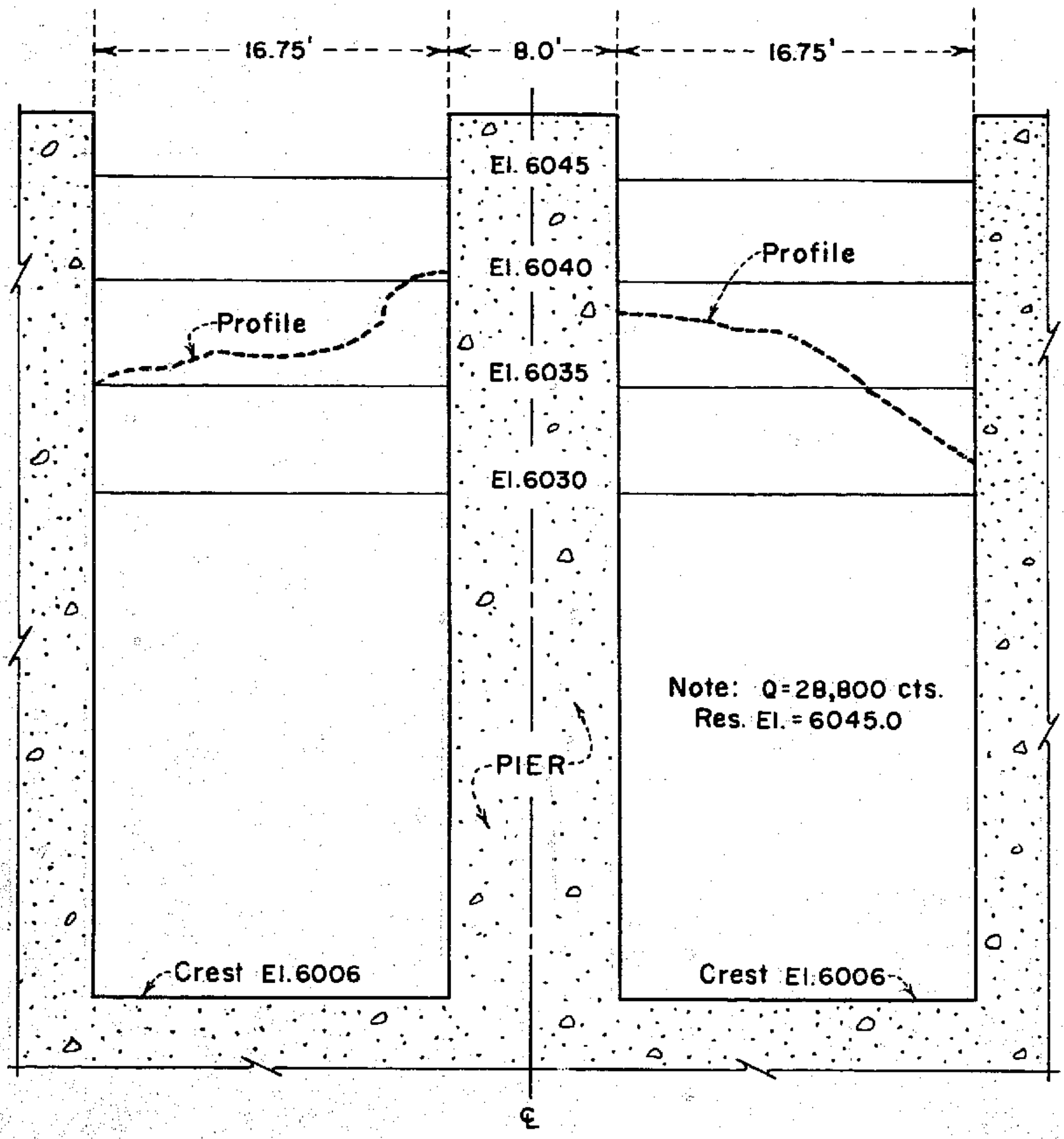
NOTES

Pressures indicated are in feet of water.
Numbers show Piezometer location.
Pressures obtained with Reservoir at Elevation 6045.0

FLAMING GORGE DAM

1:36 SCALE MODEL

PRESSURES ON SPILLWAY AND TRANSITION

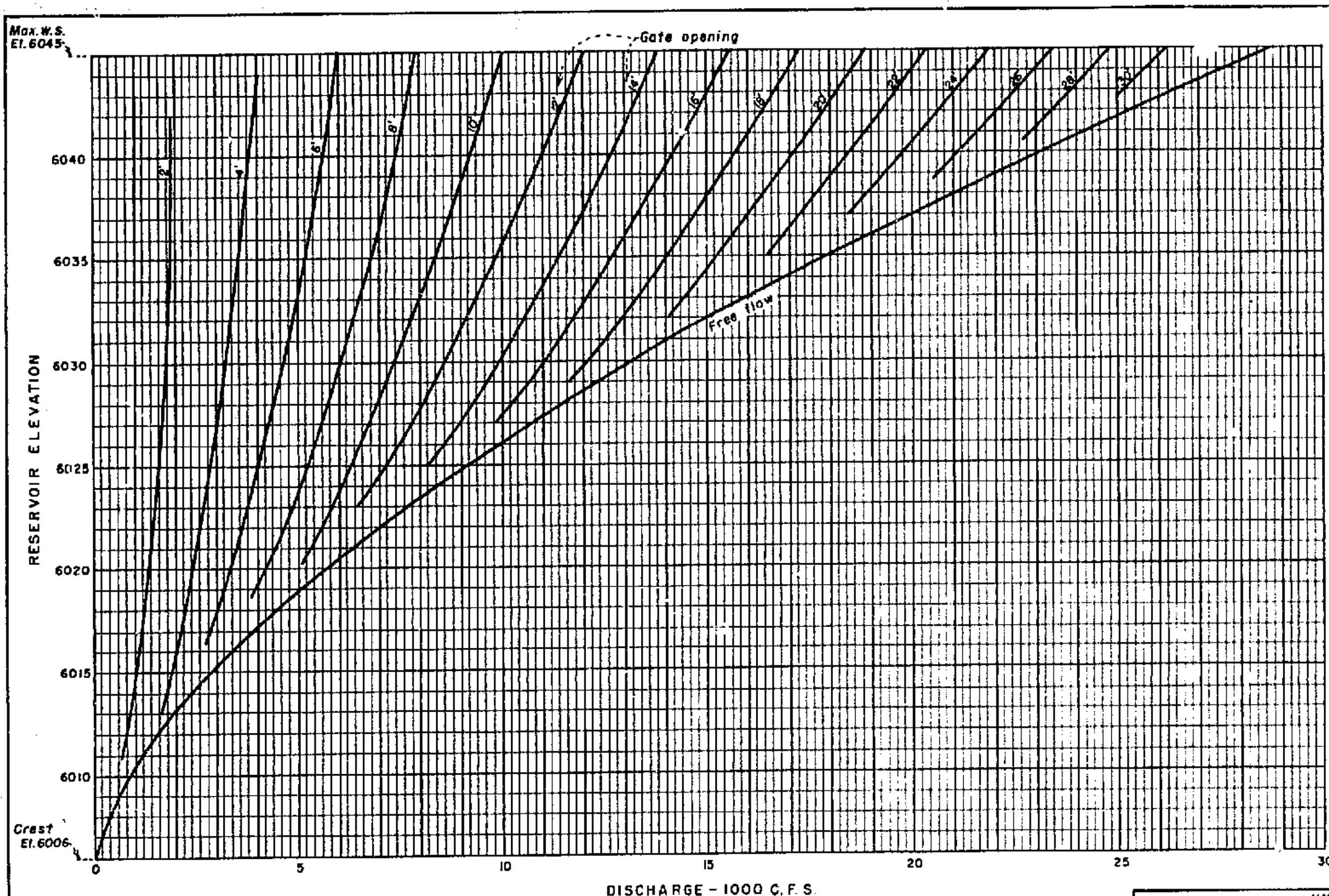


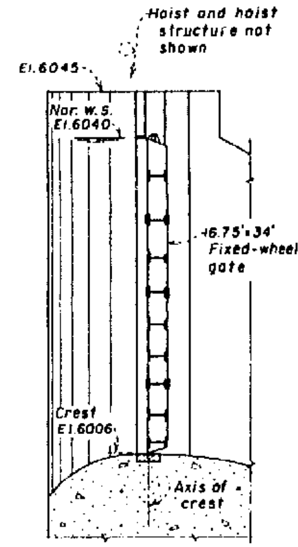
SECTION AT STA 1+00
FACING DOWNSTREAM

FLAMING GORGE DAM

1:36 SCALE MODEL

TRANSVERSE WATER SURFACE PROFILE





SECTION THRU SPILLWAY INTAKE

NOTES

Gates shall be operated simultaneously and discharges shown are for two gates.
The discharge curves shown were obtained from a hydraulic model, scale 1:36.

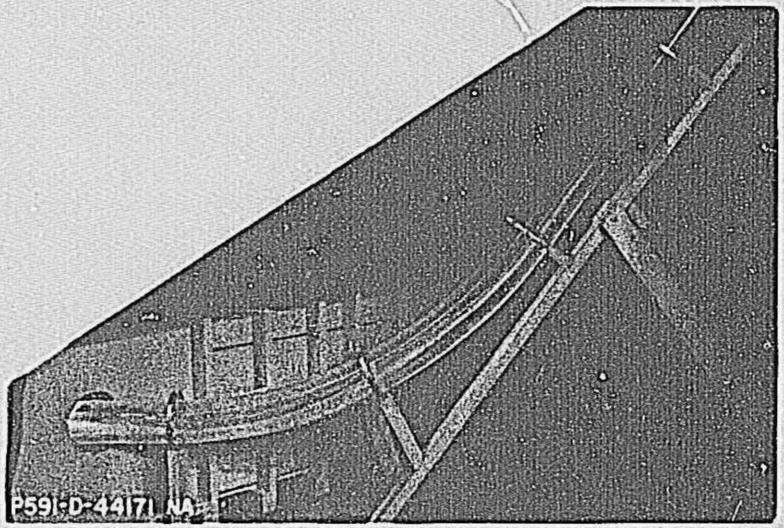
ALWAYS THINK SAFETY

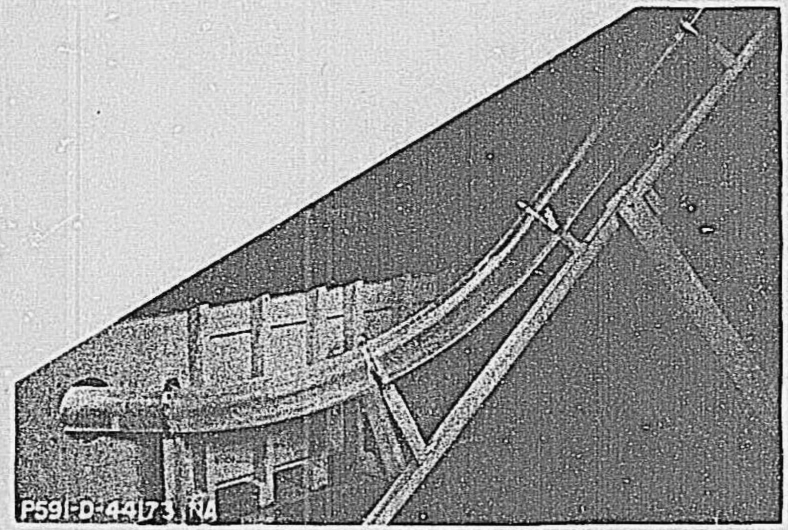
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
GREEN DIV.-FLAMING GORGE UNIT-UTAH-WYOMING

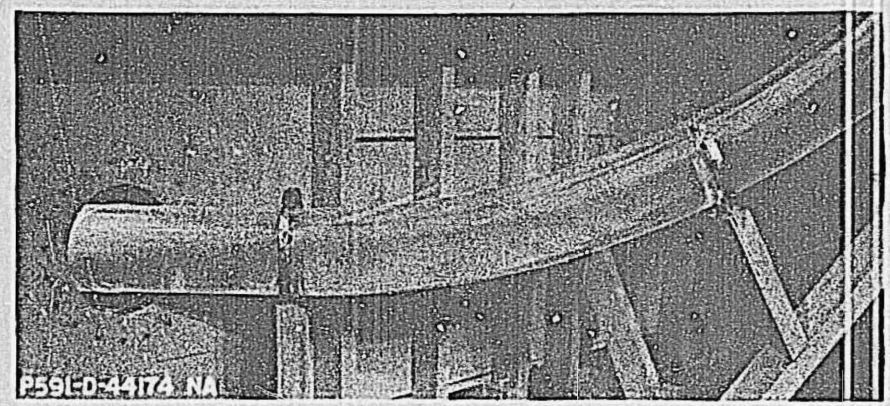
FLAMING GORGE DAM
SPILLWAY

DISCHARGE CURVES FOR 2-16.75'x 34' FIXED-WHEEL GATES

DRAWN A.O. SUBMITTED E.O. Schultz
TRACED C.W.B. RECOMMENDED P.J. Larsen
CHECKED [illegible] APPROVED O.L. Rice CHIEF DESIGNING ENGINEER
DENVER, COLORADO, JUNE 7, 1963 591-D-1221





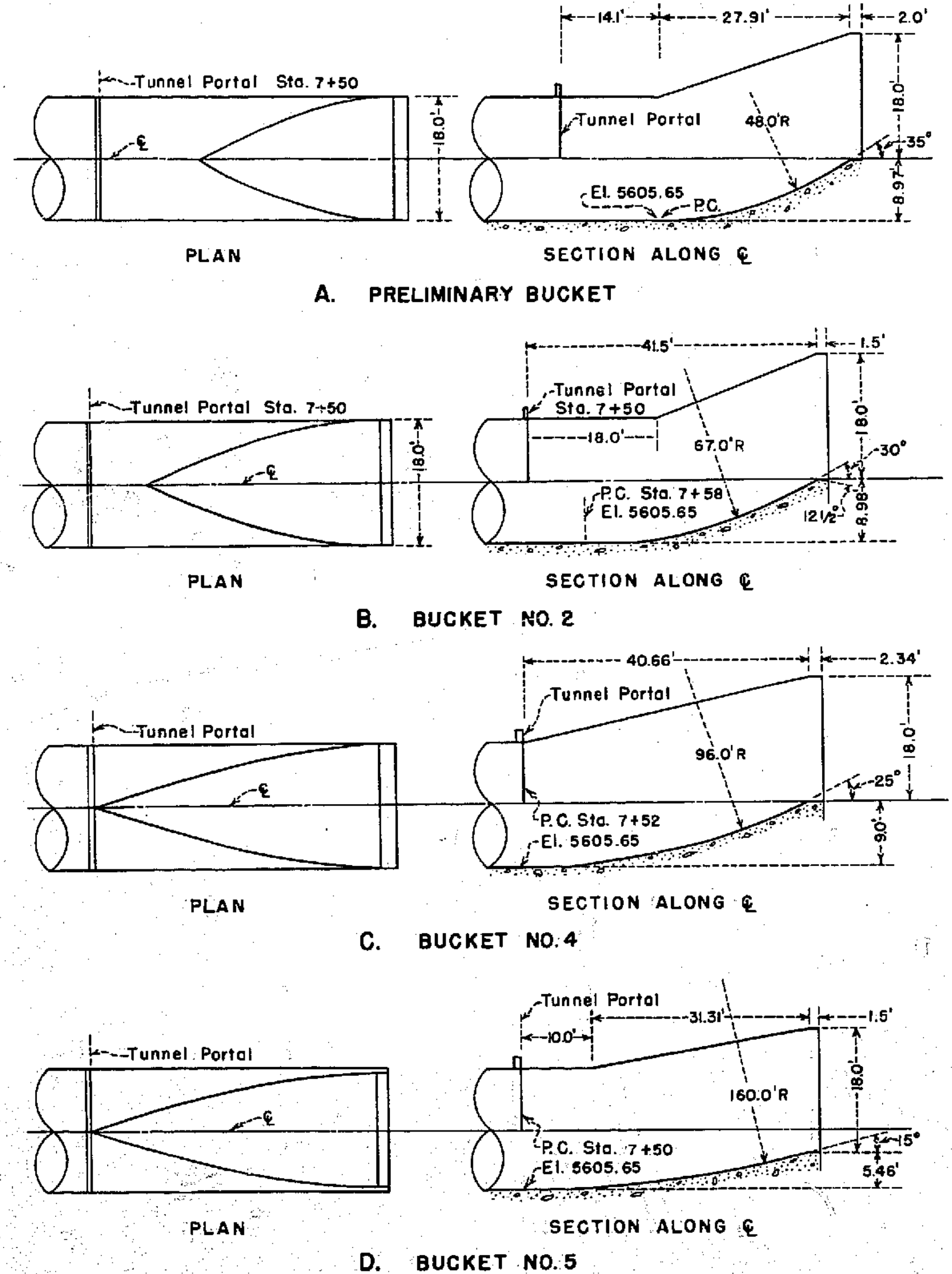


Discharge = 15,000 cfs

Discharge = 28,800 cfs

FLAMING GORGE DAM
1:36 Scale Model

Flow in Sloping Tunnel, Vertical Bend and Horizontal Tunnel

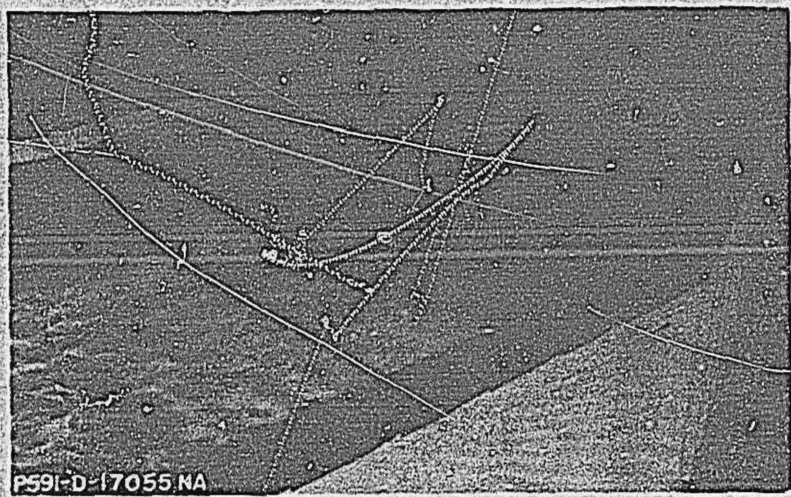


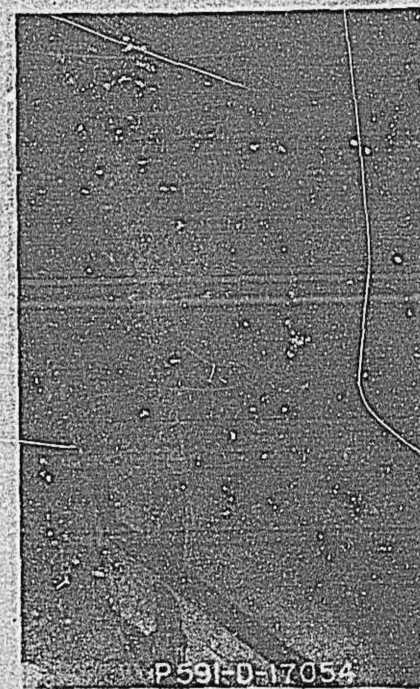
FLAMING GORGE DAM

1 : 36 SCALE MODEL

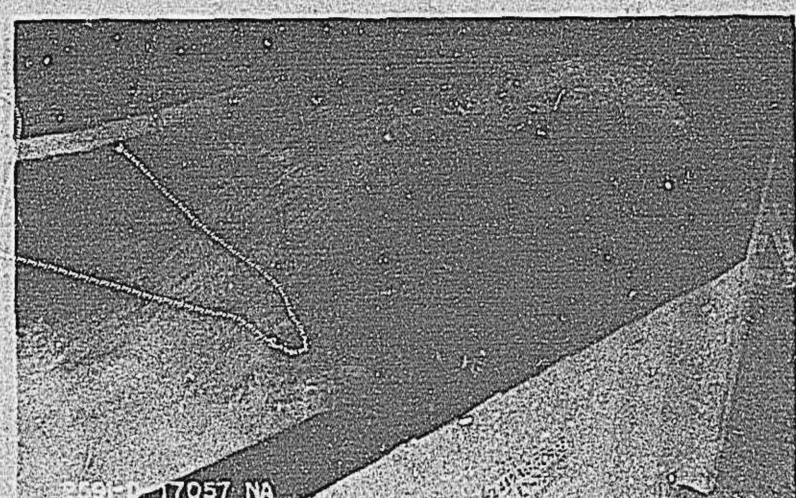
FLIP BUCKET STUDIES

880





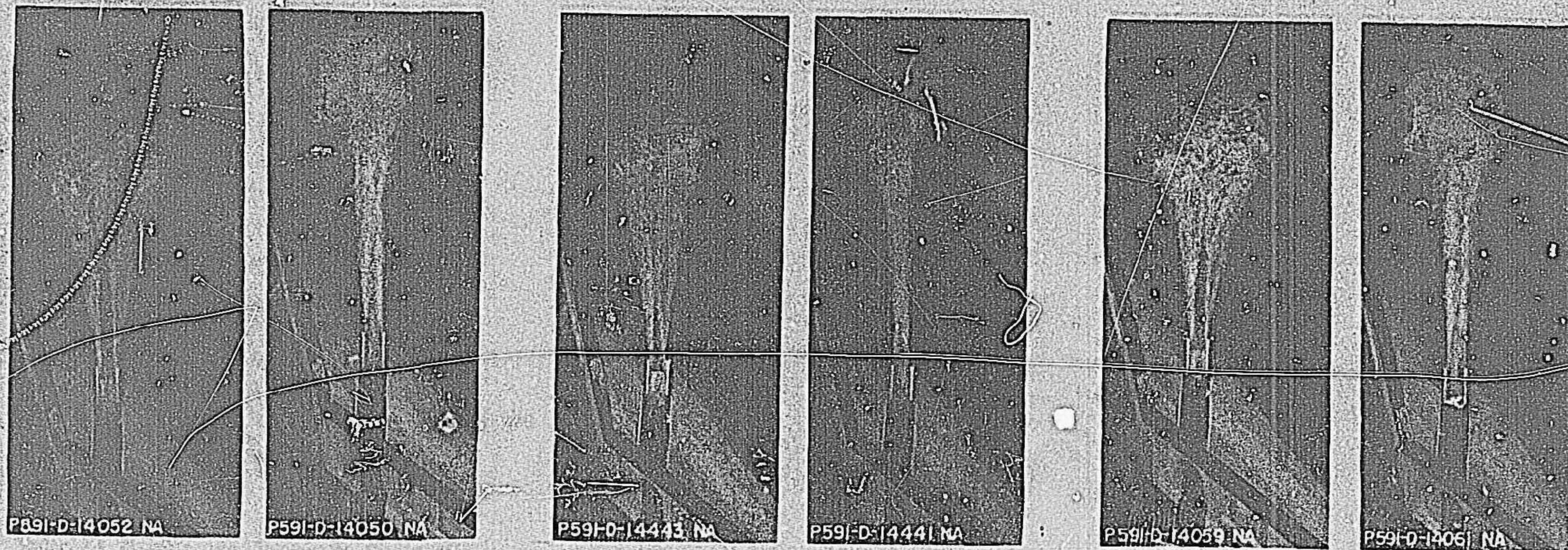
Discharge = 7, 200 cfs



Discharge = 28, 800 cfs

FLAMING GORGE DAM
1:36 Scale Model

Flow in Preliminary Flip Bucket
35° Flip Angle



Q = 7, 200 cfs   Q = 28, 800 cfs   Q = 7, 200 cfs   Q = 28, 800 cfs   Q = 7, 200 cfs   Q = 28, 800 cfs

A. 30° flip angle

B. 30° flip angle with 8°30' deflector wall

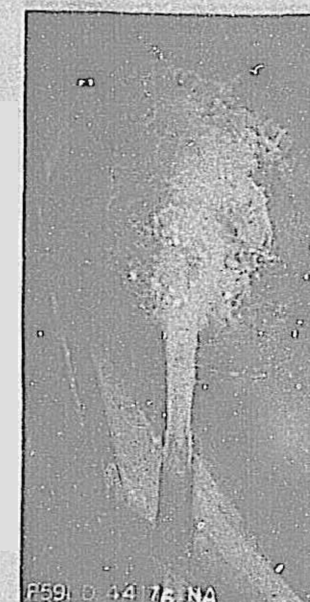
C. 25° flip angle

FLAMING GORGE DAM
1:36 Scale Model

Flow in Flip Bucket Modifications

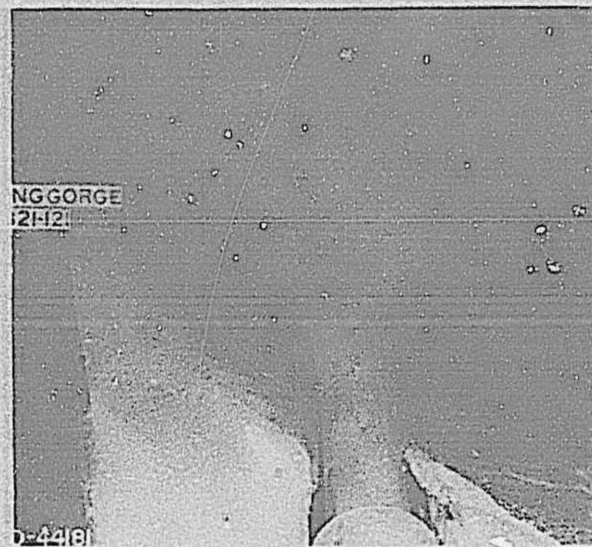
Q = 7, 200 cfs

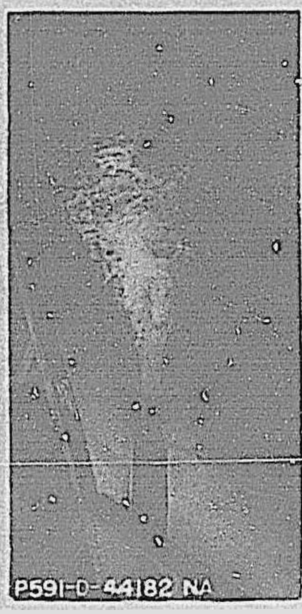
Q = 28, 800 cfs

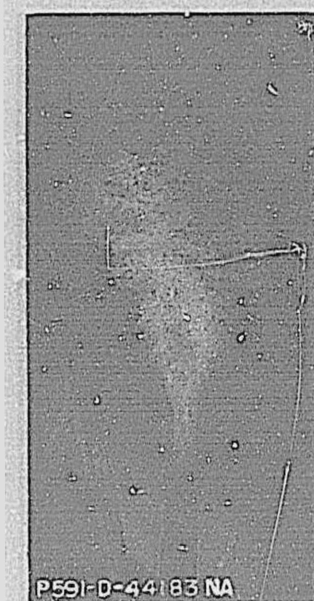


FLAMING GORGE DAM
1:36 Scale Model

Flow in Bucket No. 5
15° Flip Angle





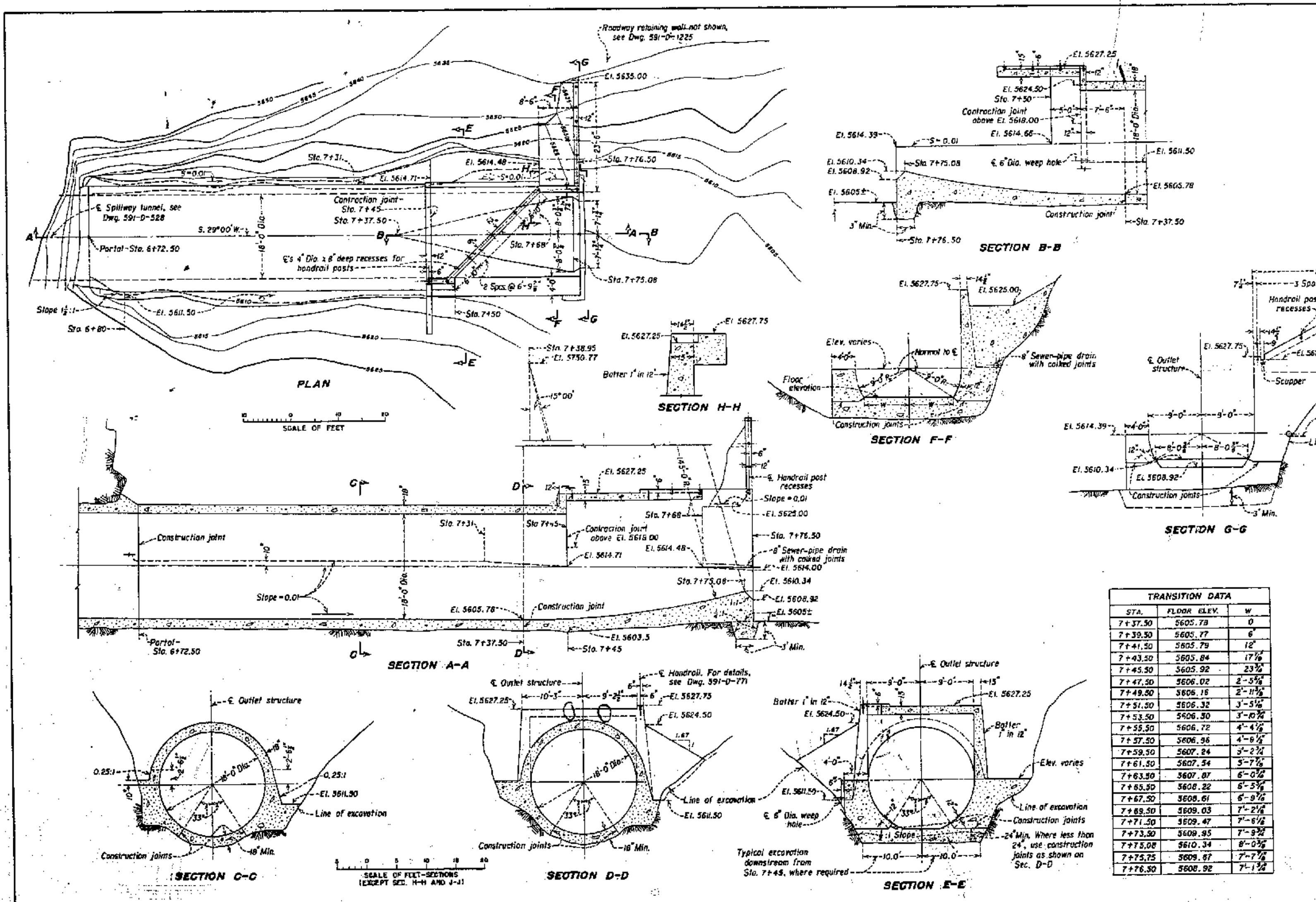


Q = 7,200 cfs

Q = 28,800 cfs

FLAMING GORGE DAM
1:36 Scale Model

Flow from Tunnel
with no Flip Bucket



TRANSITION DATA		
STA.	FLOOR ELEV.	W
7+37.50	5605.78	0
7+39.50	5605.77	6"
7+41.50	5605.79	12"
7+43.50	5605.84	17⅞"
7+45.50	5605.92	23¾"
7+47.50	5606.02	2'-5⅝"
7+49.50	5606.16	2'-11⅝"
7+51.50	5606.32	3'-5⅛"
7+53.50	5606.50	3'-10¾"
7+55.50	5606.72	4'-4⅛"
7+57.50	5606.96	4'-8½"
7+59.50	5607.24	5'-2¾"
7+61.50	5607.54	5'-7⅞"
7+63.50	5607.87	6'-0¾"
7+65.50	5608.22	6'-5⅝"
7+67.50	5608.61	6'-9⅞"
7+69.50	5609.03	7'-2⅛"
7+71.50	5609.47	7'-6½"
7+73.50	5609.95	7'-9¾"
7+75.08	5610.34	8'-0⅝"
7+75.75	5609.67	7'-7⅞"
7+76.50	5608.92	7'-1¾"

CONCRETE REQUIREMENTS

FINISHES
Surfaces below ℄ from portal of tunnel to Sta. 7+45 ---- F4 or U3
Interior surfaces downstream of Sta. 7+45 below El. 5626 ---- F4 or U3
Contraction joint surfaces ---- F1
All other surfaces ---- F2 or U2

STRENGTH
Design of concrete, other than mass, is based on a compressive strength of 3000 p.s.i. at 28 days.

NOTES

Chamfer ¾" or tool all exposed corners unless otherwise specified.
Reinforcement required but not shown.
Concrete, downstream of Sta. 7+37.50, to be placed without transverse construction joints.
Backfill not shown.

2-23-62 D. E.Q.-S.	ADDED SCUPPER ON DOWNSTREAM PARAPET. DELETED HANDRAIL POST RECESSES AT STA. 7+44.50
6-14-61 D. C.G.-J.	TRACED. ADDED CONSTRUCTION JOINTS.

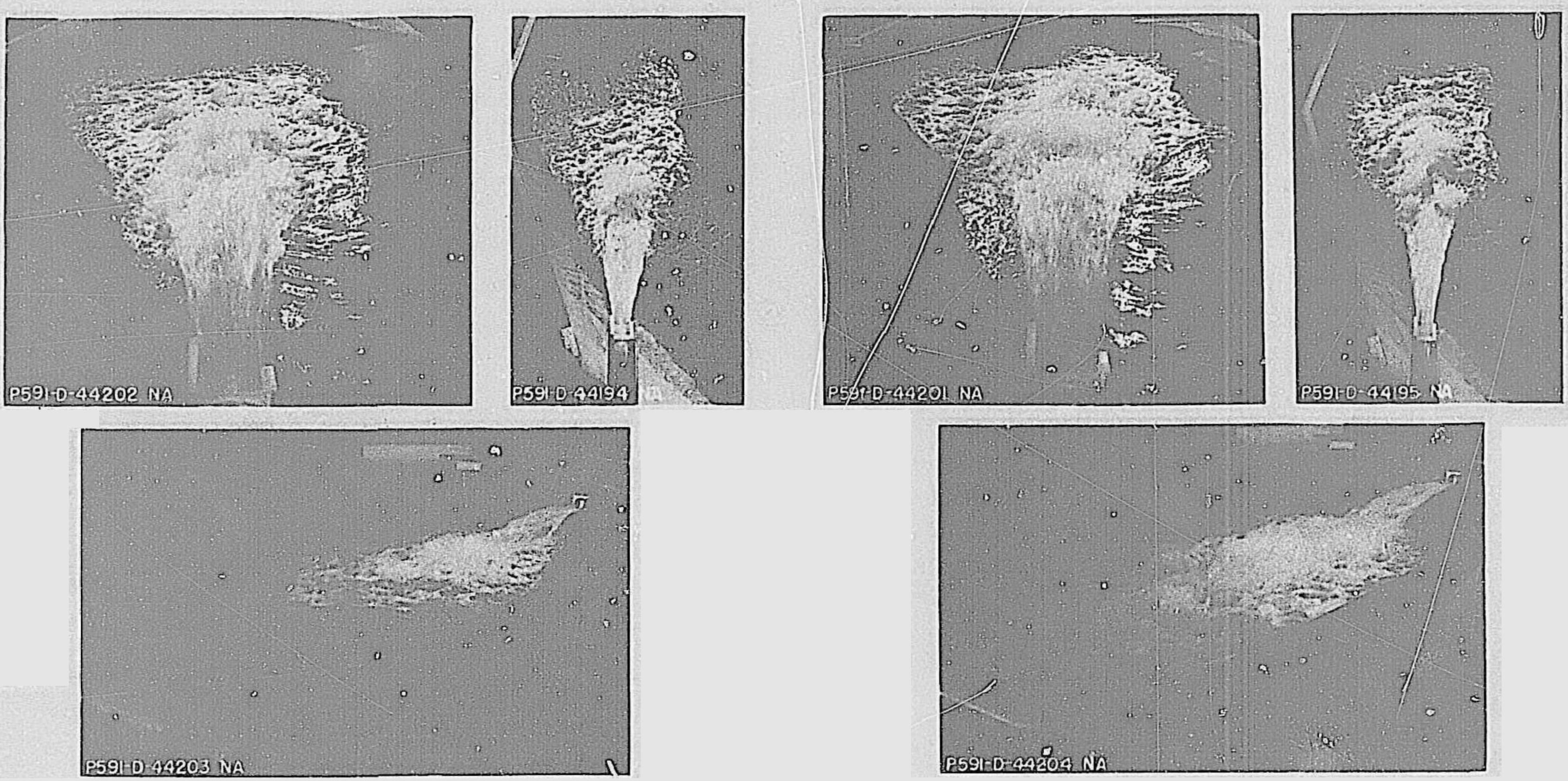
THIS DRAWING SUPERSEDES DWG. 591-D-678

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
GREEN DIV.-FLAMING GORGE UNIT--UTAH-WYOMING

FLAMING GORGE DAM
SPILLWAY
OUTLET STRUCTURE PLAN AND SECTIONS

DRAWN G.E.H.-A.O. SUBMITTED E.R. Schultz
TRACED J.L. RECOMMENDED G.L. Rice
CHECKED J.L.-M.R. APPROVED L.G. Puls
CHIEF DESIGNING ENGINEER
DENVER, COLORADO, MAY 17, 1961

591-D-1051



Discharge = 7,200 cfs
T. W. elevation = 5607.0

Powerplant operating
Discharge = 3,000 cfs

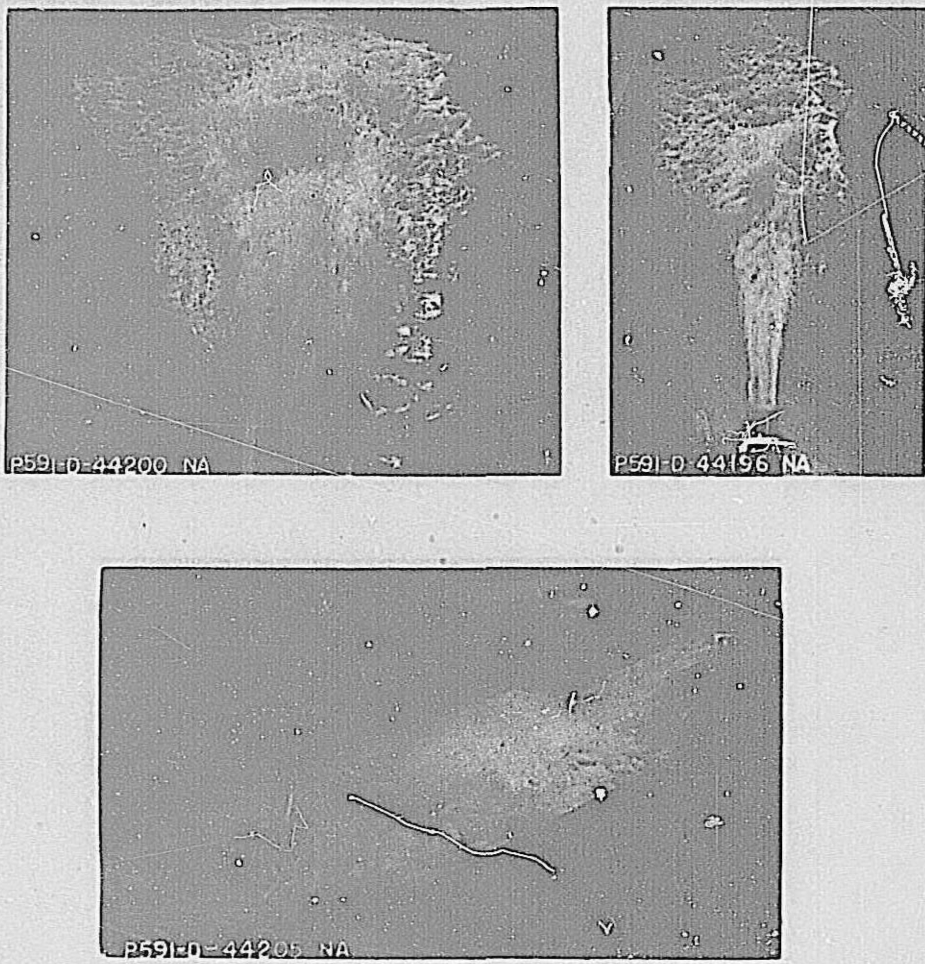
Discharge = 15,000 cfs
T. W. elevation = 5612.0

FLAMING GORGE DAM
1:36 Scale Model

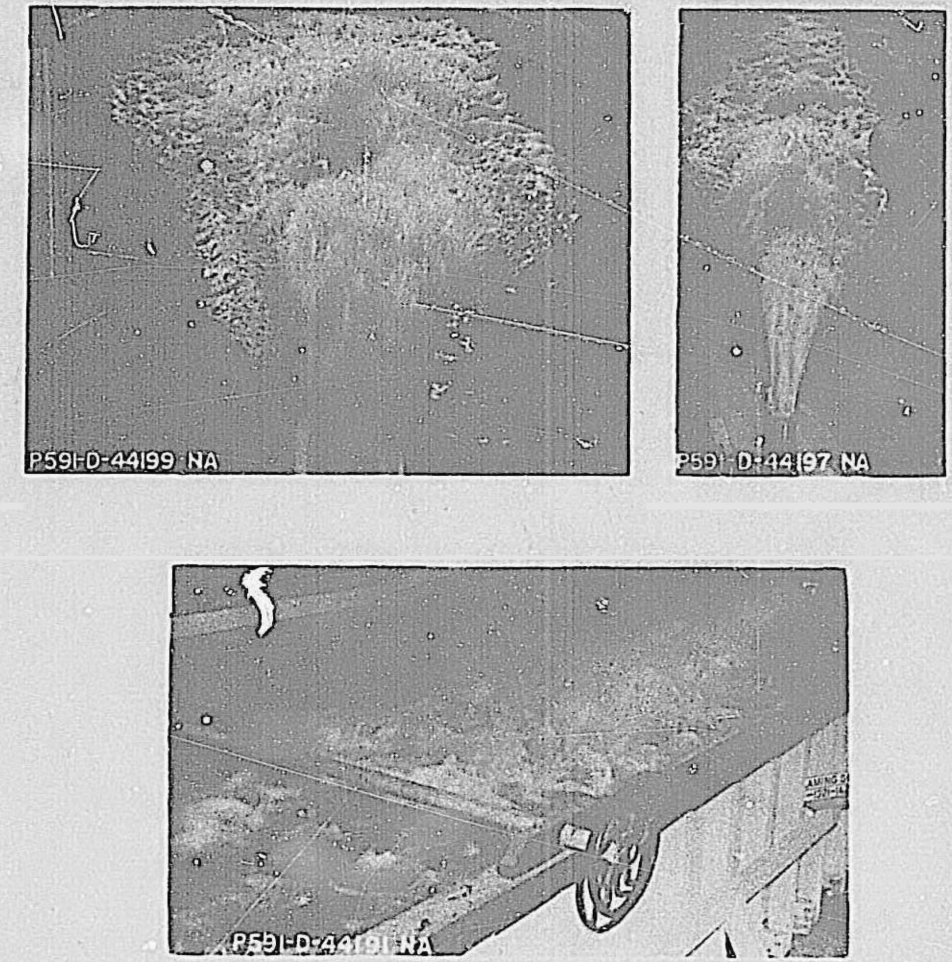
Flow in Recommended Flip Bucket

Figure 33
Report Hyd-531

Figure 34
Report Hyd-531



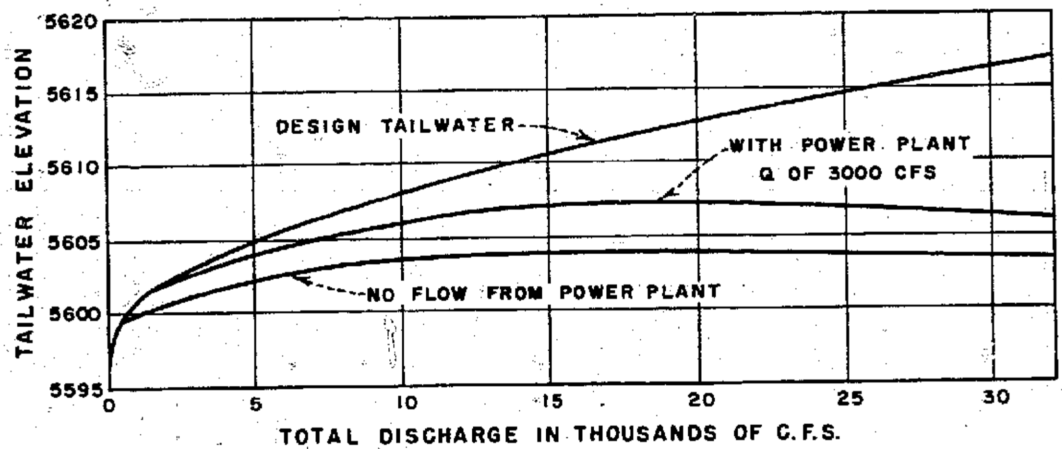
Discharge = 21,600 cfs
T. W. elevation = 5614.5



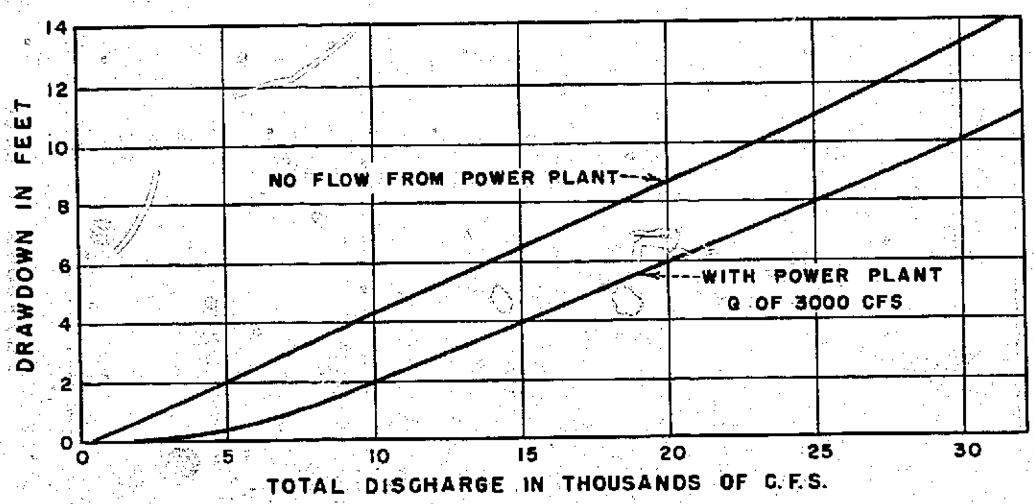
Discharge = 28,800 cfs
T. W. elevation = 5617.0

FLAMING GORGE DAM
1:36 Scale Model

Flow in Recommended Flip Bucket
Powerplant Q = 3,000 cfs



A. TAILWATER ELEVATION



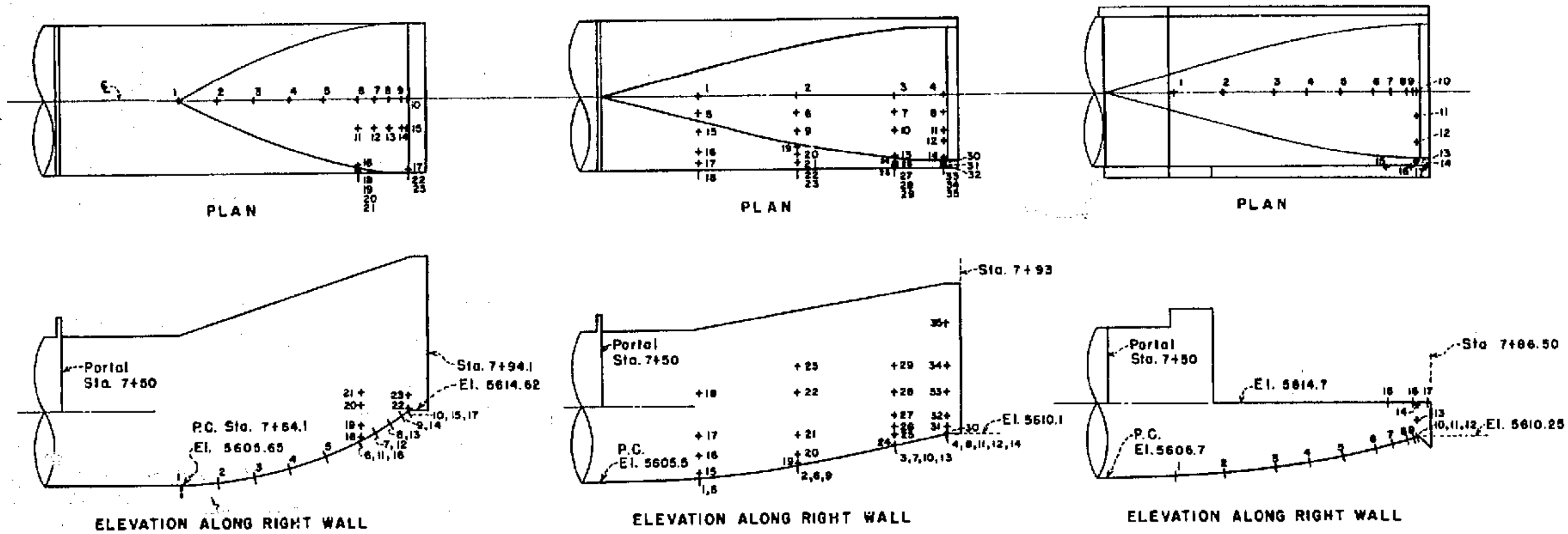
B. TAILWATER DRAWDOWN

Note: Design tailwater measured about 1000 feet downstream from Power Plant. Drawdown is difference in water surface elevation between Power Plant and downstream station. River outlets were not operating.

FLAMING GORGE DAM

1:36 SCALE MODEL

TAILWATER DRAWDOWN TESTS



A. PRELIMINARY BUCKET

35° FLIP ANGLE

PIEZ. NO.	PRESSURE IN FT. OF H_2O		PIEZ. NO.	PRESSURE IN FT. OF H_2O	
	Q = 5000	Q = 28,800		Q = 5000	Q = 28,800
1	18.2	76.1	13	24.2	108.8
2	27.7	111.8	14	5.0	45.0
3	36.3	138.8	15	-12.1	-0.8
4	45.7	156.0	16	30.4	139.5
5	34.3	144.9	17	3.5	11.5
6	50.2	147.1	18	31.6	141.3
7	40.2	130.5	19	20.4	124.9
8	36.1	111.0	20	4.0	96.8
9	22.3	74.3	21	0.5	77.5
10	-11.2	0.3	22	9.3	23.5
11	36.5	145.1	23	17.0	36.0
12	28.8	127.9			

B. BUCKET NO. 5

15° FLIP ANGLE

PIEZ. NO.	PRESSURE IN FT. OF H_2O		PIEZ. NO.	PRESSURE IN FT. OF H_2O		PIEZ. NO.	PRESSURE IN FT. OF H_2O	
	Q = 7200	Q = 28,800		Q = 7200	Q = 28,800		Q = 7200	Q = 28,800
1	18.5	49.9	13	10.8	52.5	25	8.9	51.2
2	22.0	64.5	14	2.8	12.4	26	3.1	40.8
3	25.6	60.1	15	19.1	52.8	27	–	32.9
4	-0.4	4.5	16	4.6	34.2	28	–	16.8
5	16.1	49.3	17	–	21.0	29	–	5.8
6	16.3	60.8	18	–	9.5	30	2.2	9.5
7	18.2	54.7	19	10.9	58.5	31	6.3	16.7
8	1.2	6.7	20	7.0	51.6	32	7.1	17.8
9	13.7	60.9	21	–	34.5	33	–	4.2
10	12.2	54.0	22	–	10.5	34	–	3.0
11	1.6	10.3	23	–	6.2	35	–	1.5
12	3.2	15.0	24	11.2	56.9			

C. RECOMMENDED BUCKET

15° FLIP ANGLE

PIEZ. NO.	PRESSURE IN FT OF H_2O		PIEZ. NO.	PRESSURE IN FT. OF H_2O	
	Q = 7200	Q = 28,800		Q = 7200	Q = 28,800
1	20.0	47.6	10	-8.2	-7.7
2	20.7	55.5	11	-4.7	-3.8
3	16.8	55.6	12	-5.7	-8.0
4	24.0	64.7	13	–	5.3
5	23.2	60.0	14	–	4.5
6	22.7	51.6	15	–	-0.3
7	20.8	43.3	16	–	0
8	10.1	22.2	17	–	0
9	4.2	11.7			

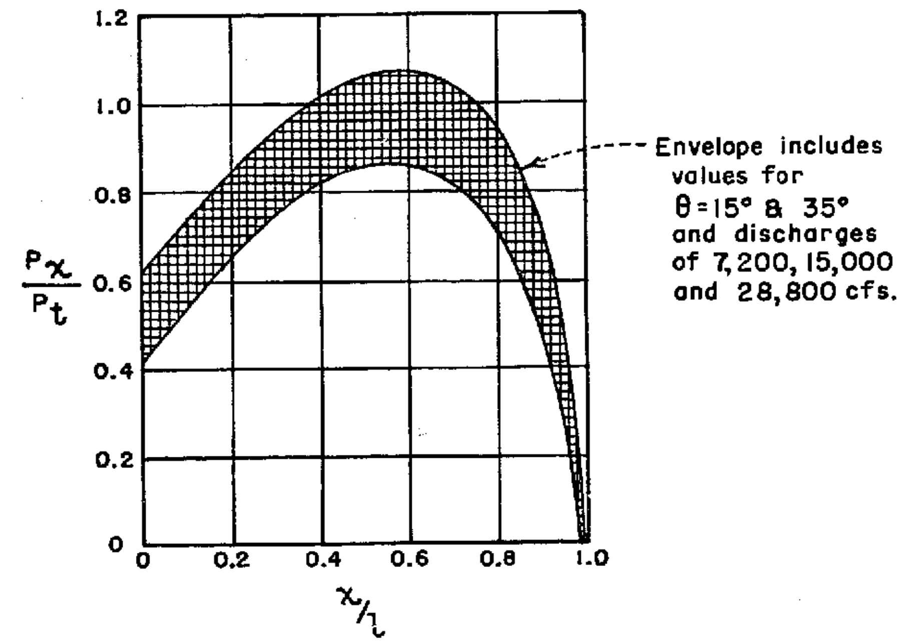
Pressures are above atmospheric unless otherwise noted.

5 0 5 10 15
SCALE IN FEET

FLAMING GORGE DAM

1:36 SCALE MODEL

PRESSURES IN FLIP BUCKETS

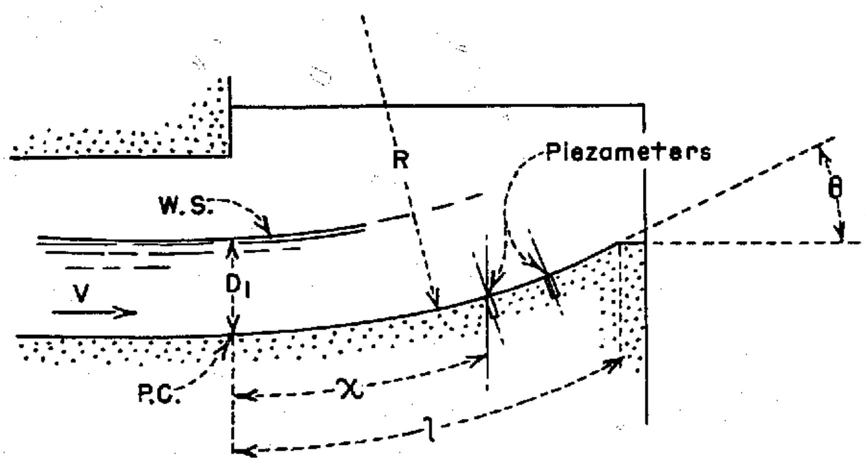


P_x = Measured pressure

P_t = Theoretical pressure; $(1.94\omega^2 R + 62.5)\,D_1$;
where $\omega = V/R$

x = Developed distance from P.C. to piezometer

l = Developed distance from P.C. to end of bucket

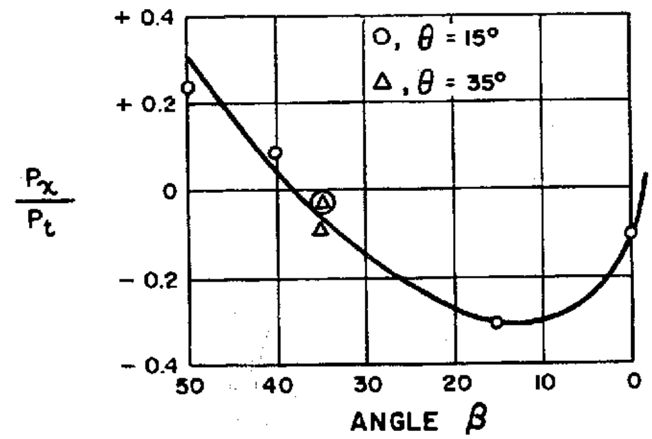


SECTION ALONG ℄

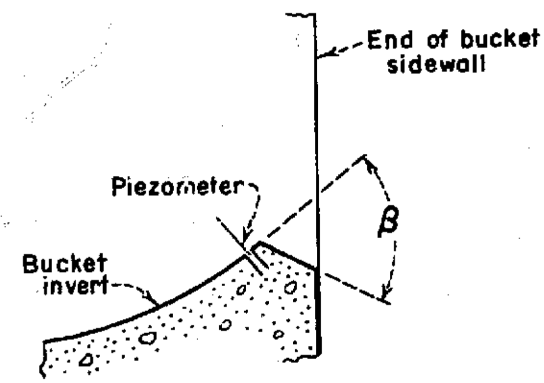
FLAMING GORGE DAM

1:36 SCALE MODEL

PRESSURE DISTRIBUTION ON BUCKET INVERT



P_x = Measured pressure at end of bucket.
P_t = Theoretical pressure.



FLAMING GORGE DAM

1:36 SCALE MODEL

PRESSURE AT END OF BUCKET

Figure 39
Report Hyd-531

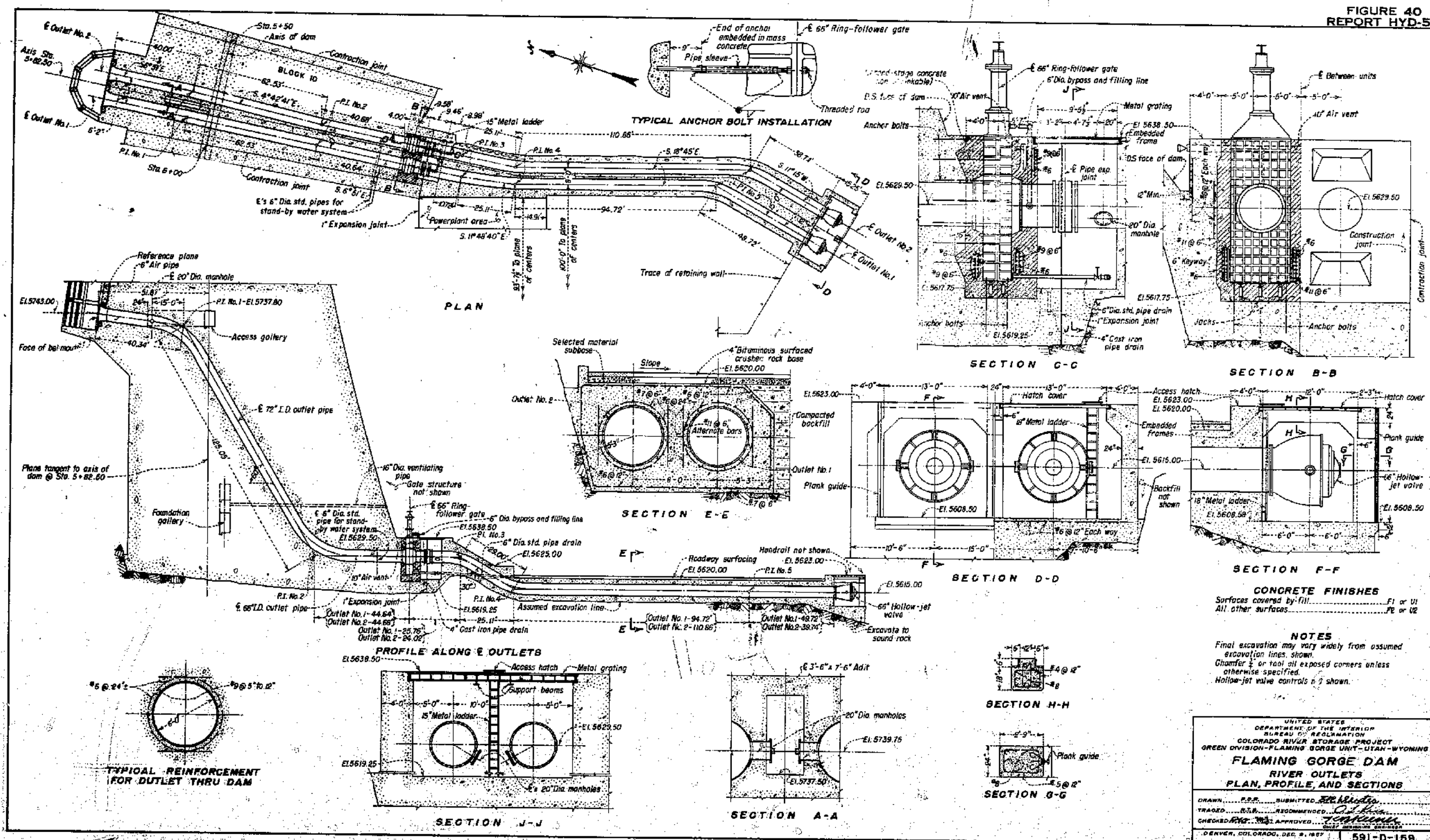
Pumping Plant access road (left river bank). Molded in weak concrete mixture.

Erosion after 10 hours model operation at flows of 5,000 to 15,000 cfs

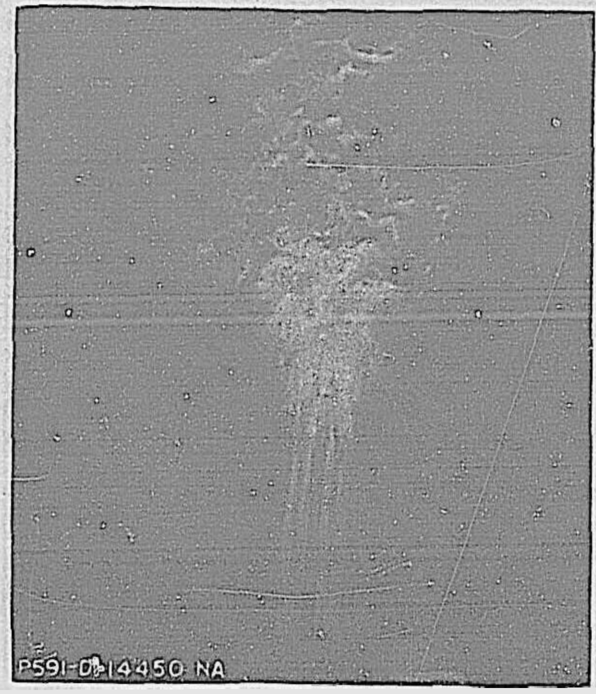
FLAMING GORGE DAM
1:36 Scale Model

Pumping Plant Access Road

FIGURE 40
REPORT HYD-531



Preliminary alinement

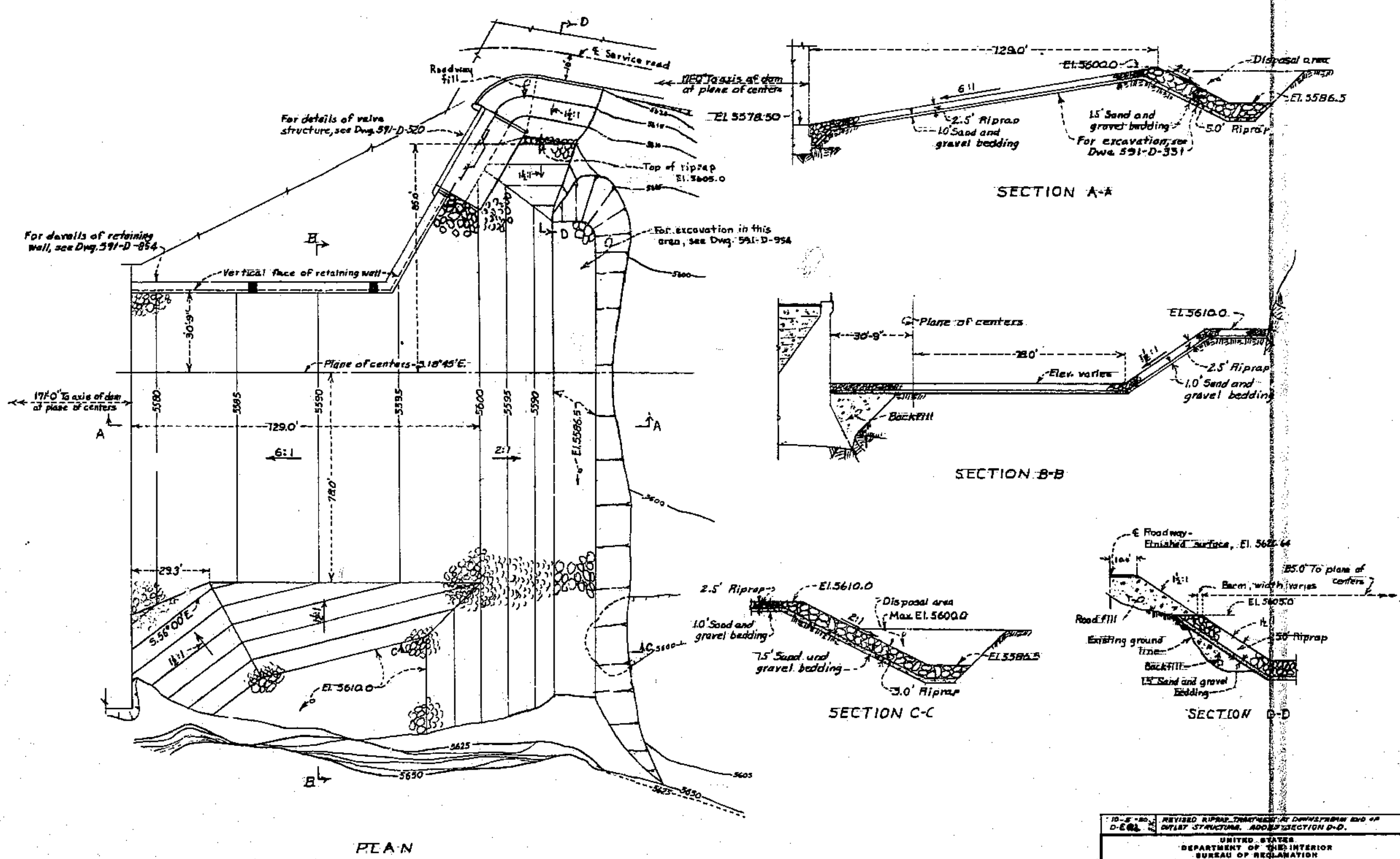


Both valves turned 5° to right

Erosion pattern after 2 hours model operation at a discharge of 1,000 cfs per valve. Preliminary valve alinement.

FLAMING GORGE DAM
1:36 Scale Model

Valve alinement and Scheme No. 1 Riprap Protection



REVISED RIPRAP TREATMENT AT DOWNSTREAM END OF OUTLET STRUCTURE. ADDED SECTION D-D.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
GREEN DIV.-FLAMING GORGE UNIT - UTAH-WYOMING

FLAMING GORGE DAM

RIPRAP FOR TAILRACE

DRAWN E.A.L. SUBMITTED
TRACED RECOMMENDED
CHECKED APPROVED
DENVER, COLORADO, SEPT. 1, 1959

591-D-778

Riverbed erosion after 2-1/2 hours operation (model time). Discharge of 1,000 cfs per valve. Reservoir elevation 6045.

River outlets discharging 2,000 cfs per valve. Reservoir elevation 6045.

FLAMING GORGE DAM
1:36 Scale Model

Riprap Evaluation for 1,000 cfs Discharge
Scheme No. 2

Erosion after 2-1/2 hours at 1,000 cfs per valve and 1-1/2 hours at 2,000 cfs per valve. Reservoir elevation 6045

Erosion after 2-1/2 hours at 1,000 cfs per valve and 5-1/2 hours at 2,000 cfs per valve. Reservoir elevation 6045

FLAMING GORGE DAM
1:36 Scale Model

Riprap Evaluation for 2,000 cfs Discharge
Scheme No. 2

ABSTRACT

Hydraulic model studies were conducted to insure satisfactory performance of the spillway approach channel, spillway crest, tunnel, and flip bucket and to determine the flow characteristics in the downstream river channel when the spillway and/or the outlet works were operating. The spillway approach channel was considerably modified to improve flow conditions. A combination circular pier and angled approach wall on the right side of the spillway entrance reduced the drawdown and improved flow conditions in the right spillway bay. An outward sloping nose on the spillway center pier reduced the water surface contraction in the left spillway bay. The center pier was extended downstream and tapered to eliminate a large fin of water in the tunnel. Pressure measurements and water surface profiles indicated that the spillway ogee and the transition from the rectangular gate section to the circular tunnel were satisfactory. The discharge capacity of the spillway was greater than anticipated, permitting the crest elevation to be raised 1 foot. Flow in the sloping tunnel, vertical bend, and horizontal tunnel were excellent with symmetrical gate operation. Five angles of lift were investigated for the flip bucket at the tunnel portal; a 15° angle provided optimum flow conditions in the river channel. Pressures up to 65 feet of water were measured on the invert of the flip bucket. The minimum pressure observed was 8 feet below atmospheric. The tailwater elevation at the powerplant might be lowered as much as 12.5 feet when the spillway is operating at maximum discharge. The extent of riprap protection needed downstream from the powerplant to prevent damage by the river outlet discharges was determined.

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Hyd-531
Rhone, Thomas J.
HYDRAULIC MODEL STUDIES OF FLAMING GORGE DAM SPILLWAY AND OUTLET WORKS, COLORADO RIVER STORAGE PROJECT, UTAH
Laboratory Report, Bureau of Reclamation, Denver, 21 pp., 44 Figures, 1 Table, 1 Reference, 1964

DESCRIPTORS--*Flip buckets/ outlet works/ air demand/ *tunnels/ discharge coefficients/ *hydraulic models/ jets/ pipe bends/ vortices/ water pressures/ afterbays/ bank protection/ open channel flow/ spillway crests/ piezometers/ *riprap/ negative pressures/ drawdown/ water surface profiles/

IDENTIFIER--Spillway profiles/ spillway approaches/ flow contractions/ entrance transitions/ stream lining/ tunnel spillways/ spillway piers/

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