H.R. Schroeder

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF FONTENELLE DAM SPILLWAY SEEDSKADEE PROJECT, WYOMING

Hydraulic Laboratory Report No. Hyd-486

DIVISION OF ENGINEERING LABORATORIES

OFFICE OF ASSISTANT COMMISSIONER AND CHIEF ENGINEER
DENVER, COLORADO

February 19, 1962

HYD 486

CONTENTS

CONTENTS--Continued

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Office of Assistant Commissioner
and Chief Engineer
Division of Engineering
Laboratories
Hydraulics Branch
Structures and Equipment Section
Denver, Colorado
February 19, 1962

Laboratory Report No. Hyd-486
Compiled by: T. J. Rhone
Checked by: W. E. Wagner
Reviewed by: J. W. Ball
Submitted by: H. M. Martin

Subject: Hydraulic model studies of Fontenelle Dam spillway--
Seedskadee Project, Wyoming

SUMMARY

Hydraulic model studies of the Fontenelle Dam spillway, Figure 3, described herein were performed on a 1:30 scale model, Figure 5. The model included a portion of the reservoir area, the double side-channel spillway, the spillway chute, the stilling basin, and a section of the channel downstream from the stilling basin.

The preliminary design, with minor modifications, was found to be satisfactory in all respects. There was some asymmetry in the flow in the spillway, Figure 8, that carried downstream into the spillway chute and overtopped the right training wall about halfway down the chute. Although this asymmetry was corrected by placing a deflector wall on the floor of the spillway basin, Figure 11A, the expense of the wall could not be justified due to the infrequency of operation at the maximum discharge. An 18-inch-wide coping strip along the top of the chute sidewalls, Figure 17, was found to be sufficient to prevent most of the overtopping. The flow in the spillway chute was equally distributed across the chute by the time it reached the stilling basin. The stilling action in the basin was very good at all discharges for both low and high tail water conditions, Figures 21, 22, 23, and 24. Erosion tests showed that without riprap protection there would be some bed scour at the end of the stilling basin, Figure 25, and the side slopes would be destroyed by the wave action. When the channel was riprapped, Figure 25, there was no bed scour and the side slopes remained intact.

Dynamic pressure measurements were obtained on the stilling basin sidewalls. These measurements revealed that impact pressures greater than hydrostatic should be considered in the structural design of the stilling basin training walls.

The discharge capacity calibration of the spillway showed that the maximum discharge, 20,000 second-feet, would be attained at reservoir elevation 6512.9, Figure 26. Pressure measurements of the spillway crest showed that the lowest observed pressure was about 1.5 feet of water below atmospheric, Figure 15.

INTRODUCTION

Fontenelle Dam is the principal feature of the Seedskadee Project, a participating project of the Colorado River Storage Project. It is located in southwestern Wyoming on the Green River, 50 miles northwest of Rock Springs, Wyoming, Figure 1.

The dam is an earth and gravel structure approximately 5,000 feet long at the crest and will rise about 127 feet above the riverbed.

The principal hydraulic features are the spillway and the river outlet works. The spillway is located in the right abutment and the river outlet works are located near the center of the embankment, Figure 2. The spillway, designed for a maximum discharge of 20,000 cubic feet per second, is an uncontrolled double side-channel spillway with a crest length of about 300.0 feet, Figure 3. Flow from the spillway passes through a 400-foot-long diverging rectangular chute and into a stilling basin. From the stilling basin, the flow passes through an excavated channel into the Green River.

The river outlet works, Figure 4, designed for a maximum discharge of 18,700 second-feet, includes an intake structure, three 11.0-foot-diameter conduits from the intake structure to a gate chamber, three 8-foot 6-inch by 11-foot 0-inch fixed-wheel gates located just upstream from three 8-foot 6-inch by 11-foot 0-inch top-seal radial regulating gates, three 14-foot-diameter horseshoe conduits from the gate chamber to the stilling basin chute, the chute, a stilling basin, and an excavated channel extending from the stilling basin to the natural river channel.

The model studies described herein were concerned with the spillway. The studies were made to investigate flow conditions in the spillway approach area, in the double side-channel spillway and in the spillway chute; the effectiveness of the stilling basin; and the flow in the excavated channel. The model was also used to determine the discharge capacity of the spillway crest.

THE MODEL

The model, built to a geometrical scale of 1:30, included the double side-channel spillway with sufficient surrounding approach area in the reservoir to develop representative approach flow conditions, the spillway chute, the stilling basin, and the excavated channel leading to the river, Figure 5.

The spillway crest was formed in mortar screeded to sheet metal templates. The surrounding topography was formed by mortar and metal lath placed on wood templates supported by wooden ribs. The spillway chute, stilling basin, chute blocks, and dentated end sill were made of wood treated to resist swelling. The excavated channel was formed in river sand to facilitate scour testing.

A rock baffle along one end of the headbox served to smooth out the flow, so as to introduce the flow into the model reservoir in as uniform a manner as possible. Discharges in the model were measured using calibrated venturi meters permanently installed in the laboratory. The reservoir water surface elevation was measured by means of a hook gage in a transparent plastic stilling well. The inlet of the well was located in the canal about 4 feet (model) from the spillway crest, Figure 5, well upstream from the influence of drawdown in the spillway. Tail water elevations were controlled by an adjustable tailgate at the downstream end of the model; the tail water elevation was measured on a staff gage located near the center of the channel about 2 feet upstream from the tailgate.

Pressure measurements were made on the crest by means of piezometers connected to open-tube glass manometers. Pressure measurements on the stilling basin sidewalls and on the chute downstream from the underdrain portal were made by means of piezometers connected to electronic pressure cells which actuated a direct writing oscillograph.

THE INVESTIGATION

The investigation was concerned with flow conditions in the double side-channel spillway, in the sloping chute between the spillway and the stilling basin, in the stilling basin, and in the river channel downstream from the stilling basin.

Spillway Crest Studies

The spillway is a double side-channel spillway in a U-shape with a crest length of 300.0 feet, Figure 3. For convenience in describing the investigations, the area within the U-shaped crest will be referred to as the spillway basin. At the upstream end, Station 1+33.00, the spillway basin is 68.81 feet wide from the crest axis on one side to the crest axis on the other side; at the toe of the crest the basin is 30.00 feet wide. At the downstream end, Station 2+37.00, the spillway basin is 79.85 feet wide from crest axis to crest axis; at the toe of the crest the basin is 40 feet wide. Between Stations 1+33.00 and 2+37.00 the floor of the basin is on a 0.01 slope, Figure 6. At Station 2+37.00, a curved pier on either side of the spillway basin directs the flow into a rectangular channel 40 feet wide leading to the spillway chute, Figure 3.

The approach to the spillway is a broad excavated berm at elevation 6503.00; the spillway crest is at elevation 6506.00.

Preliminary Design. --The model was operated with the original layout to determine the flow conditions over a complete range of discharges. For 5,000 second-feet the flow appearance was excellent. The flow from the reservoir approached the spillway in a smooth, well-distributed pattern. The flow from both sides of the crest came together in the center of the spillway basin and formed a small standing wave. There was only a slight drawdown of the flow around the piers. By the time the flow passed through the rectangular passage downstream from the piers, the standing wave had smoothed out and the flow was symmetrical, Figure 7.

When the discharge was increased to 10,000 second-feet, the appearance of the flow approaching and passing over the crest was still excellent. However, the standing wave formed by the intersection of the flows from opposite sides of the spillway was slightly to the right of center, indicating that more flow was passing over the left side of the spillway than over the right side, Figure 7. There was still only a slight drawdown in the water surface adjacent to the piers. The slight drawdown smoothed out as the flow passed through the rectangular section.

For 15,000 second-feet, the flow in the approach was excellent. The standing wave in the spillway basin, caused by the intersection of the flows from the opposite sides of the spillway, had become even more off center toward the right, Figure 8. The standing wave was higher at the upstream end of the basin; however, it did not completely submerge the crest. With the 15,000-second-foot discharge, there was some impingement of the flow against the face of the piers at the downstream end of the spillway basin. The water surface was consistently within 2 to 3 feet of the top of the piers with considerable splash overtopping the piers. However, the drawdown around the piers was still negligible. The impingement against the piers and the concentration of the flow on the right side resulted in unsymmetrical flow conditions in the rectangular passage.

With the maximum discharge 20,000 second-feet, the flow conditions in the spillway approach still had an excellent appearance. In the spillway basin, the boil, or standing wave, was still off center to the right and it submerged the crest at the upstream end, Figure 8. The impingement of the flow against the pier faces was more pronounced and the tops of the piers were saturated from the frequent overtopping. The flow around the piers had a noticeable drawdown at this discharge.

Figure 9 shows the water surface profiles in the spillway basin at the test discharges.

The concentration of the flow on the right side, the impingement against the piers and the drawdown around the piers combined to form extremely rough flow conditions in the rectangular passage.

First Modification.--To prevent the offcenter boil from forming, a training wall was installed on the floor of the spillway basin. The first wall was 4 feet high, 4 feet wide, and extended from the upstream end of the basin downstream to Station 2+37.00, Figure 10. The right side of the wall was along the basin centerline.

The wall moved the boil to the center of the basin for flows up to 15,000 second-feet. However, at the maximum discharge the boil was still over to the right side, Figure 11A.

Second Modification.--The height of the wall was increased to 8 feet between the upstream end of the basin and Station 1+87.00. Also, the downstream radius of the side piers was increased from 10 to 15 feet to provide a more gradual change in direction, Figure 12.

The higher deflector wall in the basin corrected the uneven flow concentration and improved the flow distribution in the rectangular passage. However, the surface flow moving downstream in the basin still impinged against the faces of the side piers and the drawdown around the piers formed a depressed water surface adjacent to the walls at the upper end of the passage and a high water surface at the lower end, Figure 11B. This flow condition carried down into the sloping chute and the side walls were overtopped about halfway down the chute.

Third Modification.--To reduce the amount of drawdown and the effect of the flow impingement on the face of the piers, the piers were further streamlined as shown in Figure 13.

Neither of the two extreme streamlinings of the piers improved the flow conditions, Figure 14, so it was decided to use the original pier design. In addition, it was decided that, due to the infrequent operation of the spillway at or near the maximum discharge, the expense of the deflector wall in the spillway basin could not be justified and the wall was not included in the final design.

Pressures on Spillway Crest.--Three rows of seven piezometers were installed on the spillway crest for obtaining pressure measurements along the crest profile. One row of piezometers was located in the upstream left corner of the spillway; one row on the left side at about Station 1+93.0; and the third row on the right side at about Station 1+90.0, Figure 15. Pressure measurements were obtained for discharges of 5,000, 10,000, 15,000, and 20,000 second-feet. The measurements indicated that for the first three discharges slightly subatmospheric pressures would occur on the crest. However, the lowest pressure measured was only about 1.5 feet of water below atmospheric. At the 20,000-second-foot discharge, the crest was submerged and all of the pressures were well above atmospheric. The pressure profiles representing the pressure measurements are shown on Figure 15.

Flow Distribution on Berm.--To determine the flow distribution on the berm surrounding the spillway, the depth and velocity of flow were measured at several points around the outside of the spillway about 15 feet from the edge of the crest, Figure 16. The flow velocities at each station were measured at elevation 6506.0; the flow depths above the berm elevation, 6503.0, were recorded. The data were obtained for the maximum discharge, 20,000 second-feet, and for 10,000 second-feet.

The measurements showed that the depths were 3 to 12 percent greater and velocities 8 to 30 percent lower on the right side of the spillway than on the left side. The depths and velocities of flow at each measuring station are shown on Figure 16.

Sloping Chute Studies

Downstream from the rectangular channel at the end of the spillway basin the flow enters a rectangular sloping chute leading to a stilling basin, Figure 3. The chute diverges from a width of

40 feet at the upstream end, Station 3+15.00, to a width of 52 feet at the downstream end, Station 6+36.75. For the first 12.75 feet, the floor of the chute is on a 0.002 slope, then an 80-foot-long vertical curve changes the bottom slope to 0.46. The total drop from the upstream to the downstream end of the chute is 123.84 feet.

Preliminary Design.--In the preliminary design, the flow conditions at the upstream end of the chute were satisfactory for discharges up to 10,000 second-feet. At 15,000 second-feet, some buildup of the flow occurred on the right side of the chute. At this discharge, the flow concentration was not great enough to overtop the training wall. By the time the flow had passed over the vertical curve it was evenly distributed across the full width of the chute. However, at 20,000 second-feet the flow concentration was sufficient to overtop the training wall a short distance downstream from the vertical curve. Downstream from this point the high velocity flow redistributed itself and the depth of flow was comparatively uniform across the chute at the upstream end of the stilling basin. Water surfaces profiles along both training walls and at three transverse sections along the chute are shown in Figure 17.

The proposed deflector wall in the spillway basin had provided good flow distribution in the spillway chute. When it was decided not to use this modification, a method of preventing the flow from overtopping the right training wall was sought. The method that was adopted consisted of an overhanging coping strip 12 inches deep and 18 inches wide placed at the top of both training walls. The strip extended the full length of the chute, Figure 17. This coping strip contained most of the flow that rose along the right wall with only occasional splashing going over the wall at the maximum discharge.

Underdrain Deflector.--The system of drains under the spillway chute empties into a central gallery beneath the chute. Drainage water leaves this gallery through an 18-inch-diameter concrete pipe that empties onto the spillway chute at Station 5+23.50, Figure 3. To deflect the spillway flow away from the opening in the chute, a tapered deflector is located immediately upstream from the opening. In cross section, the deflector is an arc of a 24-inch-radius circle; the deflector starts at the surface of the chute at Station 5+14.50 and rises to 0.90 foot above the floor at its downstream end, Station 5+23.50.

To study the pressure conditions in the vicinity of the drainage outlet, the deflector, without the recess in the floor where the drainage pipe exits, was installed in the model, Figure 18.

Two piezometers were installed in the chute floor downstream from the deflector; one piezometer was at the left downstream corner of the deflector; and the second was in a direct line 6.25 feet further downstream. Measurements indicated that the pressure at the downstream piezometer would be above atmospheric for all discharges, ranging from about 3.12 feet of water at a discharge of 5,000 second-feet to about 8.75 feet of water at the maximum discharge. The upstream piezometer registered subatmospheric pressures at all flows; the pressure varied from a negative 10 feet of water below atmospheric when the discharge was 5,000 second-feet to a negative 13 feet below atmospheric at 10,000 second-feet and rose to a minus 11 feet of water at the maximum discharge, Figure 18.

Because these pressures were well above the cavitation range, damage to the concrete surface is considered unlikely. However, during extended periods of spillway operation subatmospheric pressure at the downstream end of the chute block would cause a partial vacuum in the drain gallery which could cause piping or other damage in the individual drain lines. Therefore, it was recommended that an air vent be provided at the downstream end of the deflector to relieve the subatmospheric pressures.

Stilling Basin Studies

The spillway stilling basin is a rectangular, hydraulic jump basin 143 feet long by 52 feet wide. The floor of the basin is at elevation 6354.0 and the tops of the training walls are at elevation 6408.0. Chute blocks are used at the upstream end of the basin and a dentated end sill is located at the downstream end, Figure 19. The six chute blocks equally spaced across the basin at the toe of the chute are 4 feet 4 inches wide, 4 feet high, and 8 feet 8-1/2 inches long; the top edges of the blocks are streamlined with elliptical curves. The dentated end sill has a 2:1 slope on the upstream and downstream faces. Four 9-foot-high dentils are spaced on the upstream face of the sill. The two dentils on either side of the centerline are 7 feet 6 inches wide; the two dentils adjacent to the walls are 7 feet 3 inches wide. The upstream edges of each dentil are streamlined with a 12-inch-radius quarter circle, Figure 19.

Downstream from the stilling basin, the riprapped channel bed slopes upward on a 5:1 slope to elevation 6394.0, Figures 2 and 5. The bottom width diverges from 52.0 feet at the stilling basin to 212 feet at the top of the slope. At the top of the bottom slope, the right side of the channel curves to the left in a 400-foot-radius circle toward the original riverbed and the left bank of the channel merges into the river channel. The sides of the channel are formed on a 2:1 slope. The effectiveness of the stilling basin was evaluated for four discharges, representing the full range of possible operating conditions. The discharges were 5,000, 10,000, 15,000, and 20,000 second-feet. For each discharge, the performance of the stilling basin was evaluated with the tail water elevation set to represent the condition for the degraded channel with the spillway only operating, lower curve of Figure 20, and for the existing channel with the spillway and outlet works operating, top curve of Figure 20. The criteria used to evaluate the stilling basin performance were (1) the general appearance of the hydraulic jump, (2) the magnitude of the wave action in the channel downstream from the basin, and (3) the amount of bank erosion and channel bed scour after an extended period of operation at the maximum discharge.

Evaluation of Stilling Basin.--At a discharge of 5,000 second-feet, the entering flow was evenly distributed across the basin. The action in the hydraulic jump was excellent and was confined to the upstream end of the basin, Figure 21. Downstream from the end of the basin, the flow was tranquil. The waves in the channel about 100 feet downstream from the end of the basin were only about 0.5 foot high and were not choppy.

At a discharge of 10,000 second-feet, the entering flow was also evenly distributed across the basin. The hydraulic jump extended to within about 30 feet of the end of the basin and provided excellent energy dissipation, Figure 22. The flow in the downstream channel was smooth; the maximum waves being only about 1.2 feet high. At 15,000 second-feet, the flow entering the stilling basin was rougher than for the two previous discharges but was still equally distributed across the basin. The hydraulic jump was very effective in dissipating the energy, Figure 23. The jump occupied the full length of the basin with the boil at the end of the jump occasionally moving about 15 feet downstream from the end of the basin. The flow in the channel beyond the end of the jump was fairly smooth with the maximum wave height being about 2.4 feet. The frequency of the maximum waves was such that they caused very little damage to the channel banks. There was no scour of the channel bottom.

At the maximum discharge, 20,000 second-feet, the flow entering the basin was extremely rough but was well distributed across the basin. The jump in the basin was very rough, with considerable splashing and surging that frequently overtopped the training walls along the full length of the basin, Figure 24. The boil at the end of the jump extended about 15 to 30 feet beyond the end of the basin. About 100 feet downstream from the end of the basin, the waves had a maximum height of about 4.5 feet and occurred frequently. The choppy water surface in the downstream channel rapidly destroyed the sand side slopes.

At the end of 8 hours of model operation (2 hours at each of the four test discharges), the channel bed had eroded only a small amount at both corners of the stilling basin, Figure 25. The maximum depth of erosion was 4 feet at the right corner and 2 feet at the left corner. The channel bed at the top of the slope had degraded about 4 feet during the 8-hour test period.

The overall performance of the stilling basin was considered excellent with the exception of the excessive splashing at the maximum discharge. Since about half of the basin extended into the channel and was surrounded by water, the flow overtopping the walls would not be harmful except possibly contributing to the wave action in the channel. However, at the upstream end of the basin any flow overtopping the walls would fall on the backfill and could conceivably remove much of the material. To reduce the amount of overtopping, coping strips, similar to these added to the sidewalls of the chute, were also placed at the top of the stilling basin walls.

Riprap.--At the conclusion of the stilling basin evaluation tests, a protective layer of riprap was placed in the excavated channel downstream from the basin. The riprap covered the upward sloping bottom of the channel, the side slopes on the right and left sides, and the flat area on the left side as shown in Figures 3 and 25. The model riprap consisted of 1/4- to 3/4-inch gravel representing 7.5- to 24-inch prototype rocks.

After the riprap had been placed, the model was operated for about 16 hours at the maximum discharge with both the high and low tail water conditions. Inspection at the end of this period showed that there was no erosion at the end of the basin and the riverbed had not degraded as it had when the channel was formed in river sand. However, on the right bank, about 60 feet downstream from the basin, the wave action had moved some of the riprap and had started to erode the side slopes. The riprap in this area was replaced with 3/4- to 1-1/4-inch gravel representing 24- to 36-inch prototype rock. At the conclusion of another 16-hour test run at the maximum discharge, the riprap was intact throughout the excavated channel. Therefore, it was recommended that the larger size riprap be placed along the right side of the channel.

Sweepout Test.--To determine the possibility of the hydraulic jump sweeping out of the basin the model was operated at the maximum discharge, 20,000 second-feet, and the tail water gradually lowered. The tail water could only be lowered to elevation 6396.0, 4.6 feet below the minimum design elevation, at which point the riprapped channel bed became the control rather than the tailgate. At this elevation the toe of the jump was approximately 20 feet upstream from the end of the sloping chute and the chute blocks were never exposed.

Since the tail water could not be lowered further, the discharge was increased to 28,500 second-feet to provide a more severe operating condition. At this discharge the model tail water elevation was approximately 6397.0, 5 feet below the minimum elevation for this flow. The toe of the jump moved down to the end of the sloping chute and the chute blocks were intermittently uncovered.

Based on these tests, it was determined that the stilling basin had at least 5 feet of depth as a margin of safety against sweep-out.

Pressure Investigations.--When the spillway is operating, the water surface level inside the stilling basin is generally lower than the tail water level in the channel. Since the end of the basin projects into the tail water pool, there is a pressure differential on the training walls. In addition, dynamic forces produced by the hydraulic jump action create intermittent pressure surges on the inside of the walls. To aid in the structural design of the training walls, these forces were evaluated in the model. Pressure measurements were made on the training walls of the stilling basin to determine the magnitude of the pressure on each side of the wall, the pressure differential on the wall, and the extent of the pressure fluctuation.

A total of 12 piezometers were installed along the inside surface of the left wall at Stations 7+03.25, 7+18.25, 7+38.75, 7+48.75, and 7+74.75, Figure 19. At the upstream station, the piezometer was at elevation 6357.5. At the three middle stations, the piezometers were at elevations 6357.5, 6370.0, and 6385.0. At the downstream station, the piezometers were at elevations 6370.0 and 6385.0. Four piezometers were also installed in the right wall; one piezometer was installed at Station 7+03.25, elevation 6357.5; three piezometers were installed at Station 7+18.25 at elevations 6357.5, 6370.0, and 6385.0.

The piezometer leads were connected to pressure cells sensitive to instantaneous pressure fluctuations. Pressure fluctuation and magnitude were converted in an electronic circuit to signals which activated a direct writing oscillograph. The trace produced on the oscillograph chart thus became a measurement of the frequency and amplitude of the dynamic pressure at the piezometer. These data were obtained for the maximum discharge, 20,000 second-feet, at two tail water elevations, 6400.7 and 6404.4. During the pressure tests with the high tail water elevation, water surface profiles on the inside of the basin training walls were measured by mechanical means to aid in interpreting and analyzing the pressure measurements. These profiles are shown on Figure 17.

The pressure tests indicated considerable difference between the hydrostatic pressures as determined from the water surface profiles and the dynamic pressures measured by the pressure cells. The maximum dynamic pressures were usually either very close to the hydrostatic pressures or 25 to 55 percent higher. The two upstream piezometers in the top row showed maximum dynamic pressures that were about 30 percent lower than the maximum hydrostatic pressure. However, the minimum dynamic pressures were consistently 30 to 90 percent lower than the minimum hydrostatic pressures.

The highest dynamic pressure was 87.5 feet of water while the highest hydrostatic pressure was 57.0 feet. The minimum dynamic pressure was 0.8 feet of water below atmospheric while the minimum hydrostatic pressure was 10 feet of water above atmospheric.

The results of the pressure tests are shown in Table 1 and also on Figure 19.

Table 1

COMPARISON OF DYNAMIC AND HYDROSTATIC PRESSURES ON STILLING BASIN SIDEWALLS

Discharge = 20,000 cfs

Tail Water Elevation = 6404.4

Piezometer			Height of water surface above piezometer, in feet		Dynamic pressure in feet of water above piezometer	
No.	Station	Elevation	Maximum	Minimum	Maximum	Minimum
1	7+74.75	6385.0	26.0	18.5	49.5	5.9
2	7+74.75	6370.0	41.0	33.5	38.8	12.4
3	7+48.75	6385.0	29.5	13.5	24.9	3.3
4	7+48.75	6370.0	44.5	28.5	42.2	10.7
5	7+48.75	6357.5	57.0	21.0	87.5	14.3
6	7+38.75	6385.0	29.5	10.0	19.0	2.2
7	7+38.75	6370.0	44.5	25.0	57.8	1.6
8	7+38.75	6357.5	57.0	47.5	64.7	25.4
9	7+18.25	6385.0	27.0	10.0	18.0	1.2
9R*	7+18.25	6385.0	26.0	10.0	23.8	1.3
10	7+18.25	6370.0	42.0	25.0	59.5	1.6
10R*	7+18.25	6370.0	41.0	25.0	52.9	-0.8
11	7+18.25	6357.5	54.5	37.5	49.2	20.7
11R*	7+18.25	6357.5	53.5	37.5	68.8	14.5
12	7+03.25	6357.5	54.5	35.5	62.1	4.5
12R*	7+03.25	6357.5	54.5	35.0	63.1	21.1

*Indicates piezometers in right wall. All other piezometers were in left wall.

As previously mentioned, four piezometers were placed in the right wall opposite the piezometers at Stations 7+03.25 and 7+18.25 in the left wall. The pressures on the right wall were obtained under the same conditions as the tests described above. The maximum pressures on the right wall were usually higher than on the left wall. The minimum pressures were approximately the same on both walls. This seemed to indicate that there was some asymmetry of flow in the basin due to a slightly greater concentration on the right side. Since this asymmetry was not apparent visually and the differences in pressures were not excessive, no corrective measures were tried.

Tests were made in which all six piezometers at Station 7+18.25 were recorded simultaneously. This was done to determine whether any excessive lateral motion of the jump caused the higher pressures on the right wall in the stilling basin. The measurements showed that the pressure highs and lows occurred at approximately the same instant on both sides of the basin.

Discharge Capacity Calibration

The discharge capacity of the uncontrolled double side-channel spillway was obtained for four different approach conditions as a part of the model studies. The first condition was with the spillway approach berm formed in smooth concrete to elevation 6503.0. The second condition was with the berm on the right side of the spillway lowered to elevation 6500.5. The third condition was with all of the surrounding berm lowered to elevation 6500.5. The final condition was with the berm covered with riprap, the top of the riprap being at elevation 6503.0. The model riprap was composed of 1/4-inch gravel, representing 7- to 8-inch rock in the prototype. The discharge capacity curves for the four conditions are shown on Figure 26.

The different approach conditions had only a minor effect on the discharge capacity. With the berm represented in smooth concrete at elevation 6503.3 and with all of the berm lowered to elevation 6500.5, the maximum discharge of 20,000 second-feet was attained at reservoir elevation 6512.85. With only the right side of the berm lowered to elevation 6500.5, the maximum discharge occurred at reservoir elevation 6512.80. With the berm covered with riprap, the maximum discharge occurred with the reservoir at elevation 6512.9.

The approach condition with the berm covered with riprap represents the prototype condition and, therefore, the top curve on Figure 26 should be used to obtain the prototype discharge capacity.

At a discharge of approximately 15,000 second-feet, the crest of the upstream end of the spillway begins to submerge; by the time the flow reaches 20,000 second-feet the upstream end of the spillway is completely submerged. In the model the submergence causes a surge in the reservoir water surface. At 15,000 second-feet the change in elevation amounted to about 0.08 foot (prototype); at 20,000 second-feet the difference increased to about 0.16 foot. In the prototype, the effect of the crest submergence would probably be reflected in a change in discharge rather than in a change in reservoir elevation. At a given reservoir elevation, the discharge would fluctuate between the amount shown on Figure 26 and about 300 to 500 second-feet less than the amount shown.

Extreme Operating Condition

A test was made to determine the performance of the structure under the extreme operating condition when the reservoir level was at or near the crest of the dam embankment, elevation 6519.0. At this reservoir elevation the discharge through the spillway was 28,500 second-feet, Figure 26.

With this extreme operating condition, the water surface in the spillway approach was very choppy with waves about 1 foot high; however, this possibly could have been caused by the drawdown across the rock baffle in the model and would not necessarily represent the prototype condition. The appearance of the flow in the spillway basin was remarkably good considering the amount of water being discharged. The crest was completely submerged over its full length and consequently the shifting control that caused the reservoir level to fluctuate at the 20,000-second-foot discharge was not present.

The flow was more evenly distributed in the sloping chute than it had been at the 20,000-second-foot discharge and did not overtop the sidewalls to any greater extent than it had at the smaller flow.

The hydraulic jump in the stilling basin was extremely rough for both high and low tail water conditions. However, the energy dissipation was very good and the flow appearance in the downstream channel was satisfactory. When the tail water was lowered to elevation 6397.0, about 5 feet below the minimum elevation for this flow, the toe of the jump moved down to the end of the sloping chute, uncovering the chute blocks, but gave no indication of sweeping out of the basin.

Although it is unlikely that this operating condition will ever occur in the prototype, the model tests indicated that the spillway, sloping chute, and stilling basin would operate satisfactorily at greater than maximum discharges.

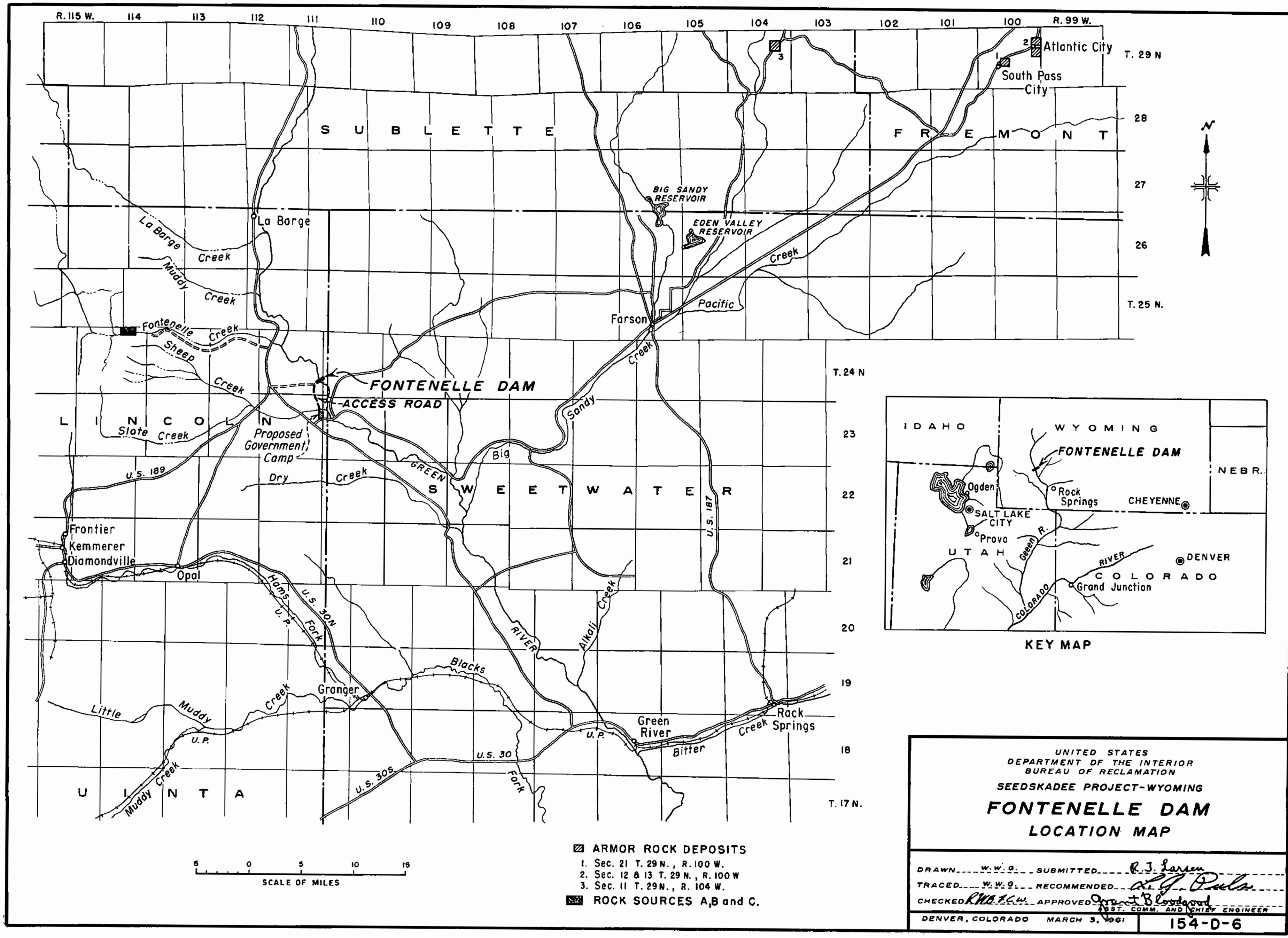
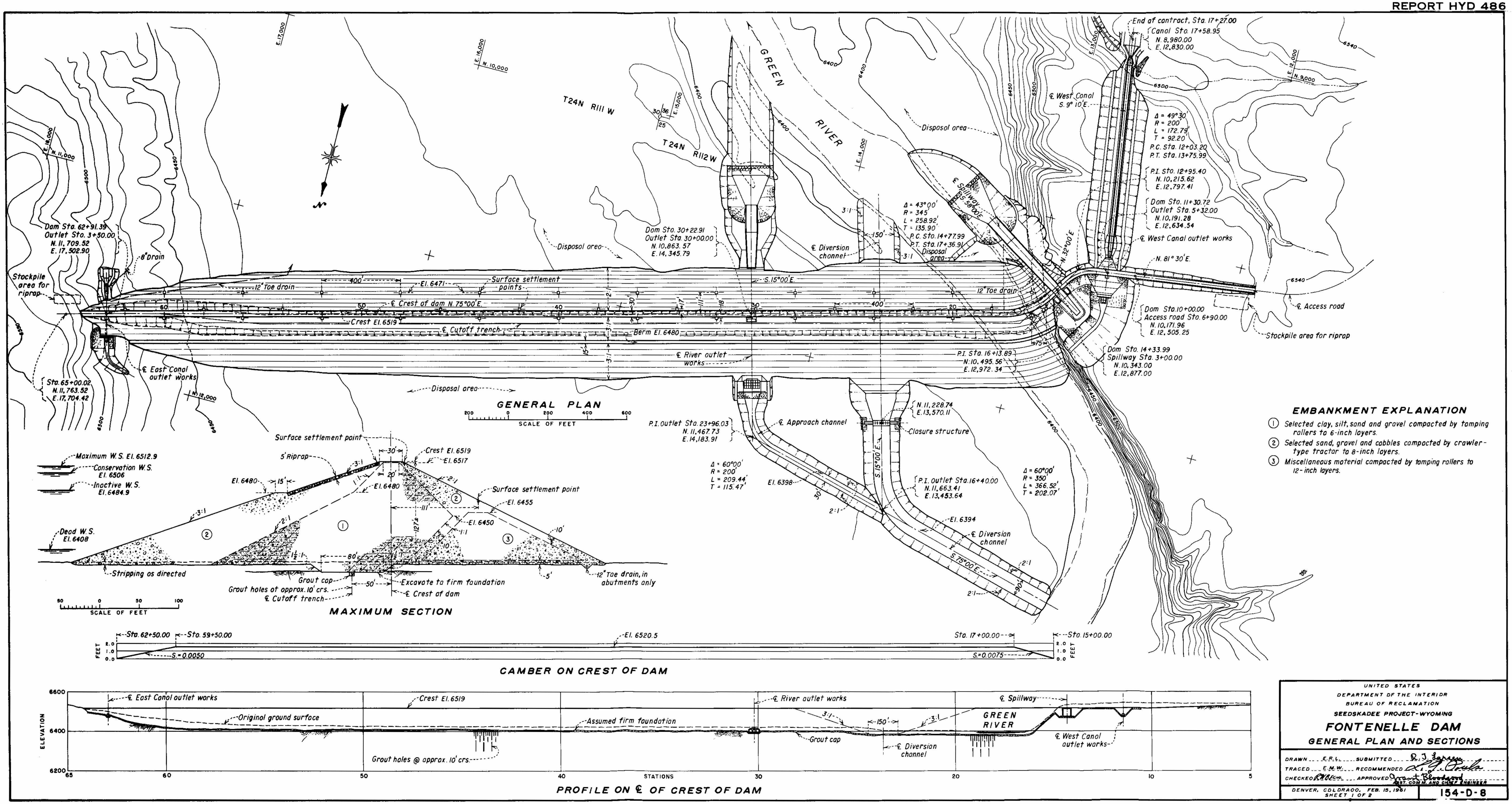


FIGURE NO. 1
REPORT HYD 486

FIGURE NO. 2
REPORT HYD 486



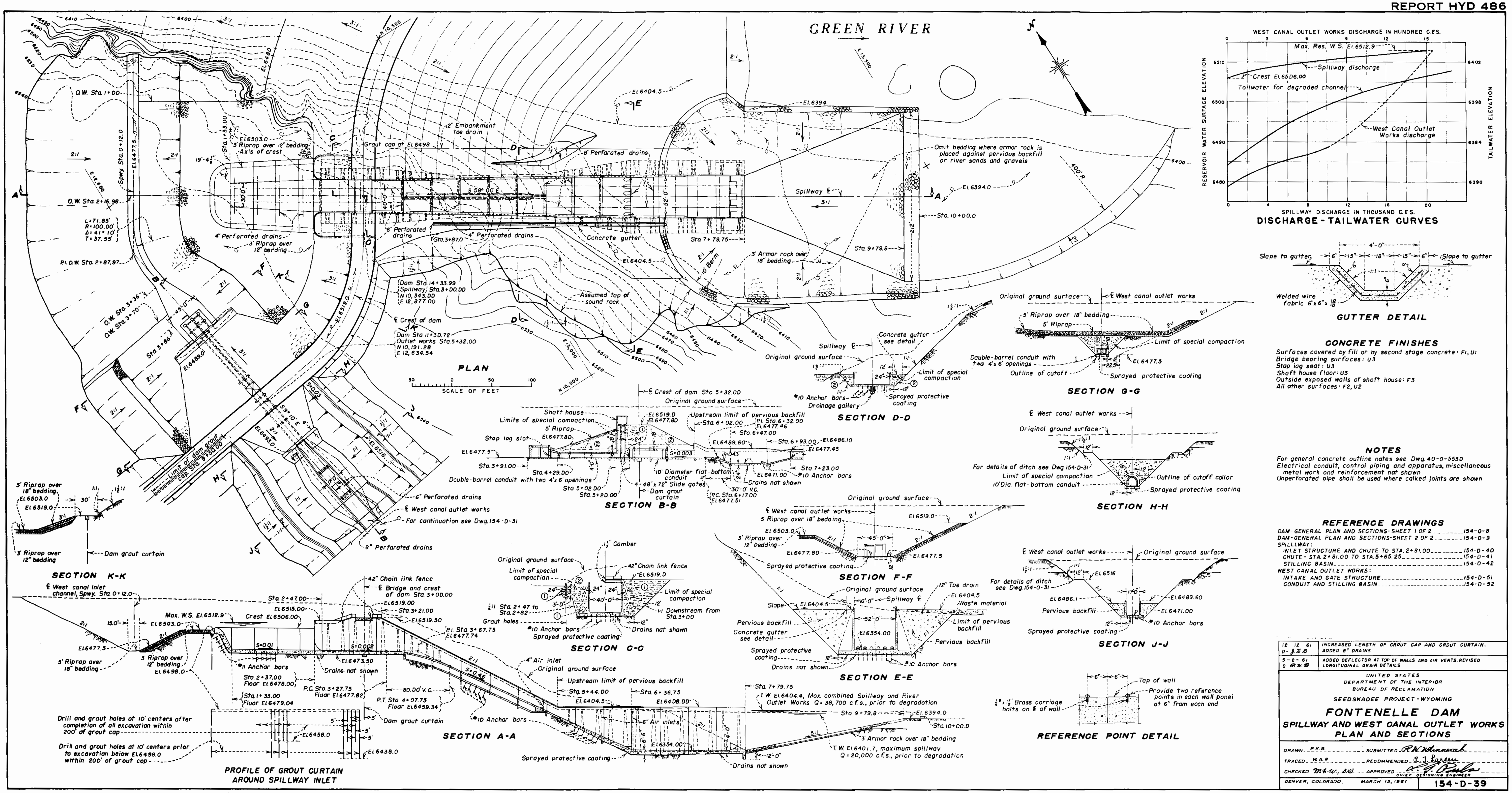
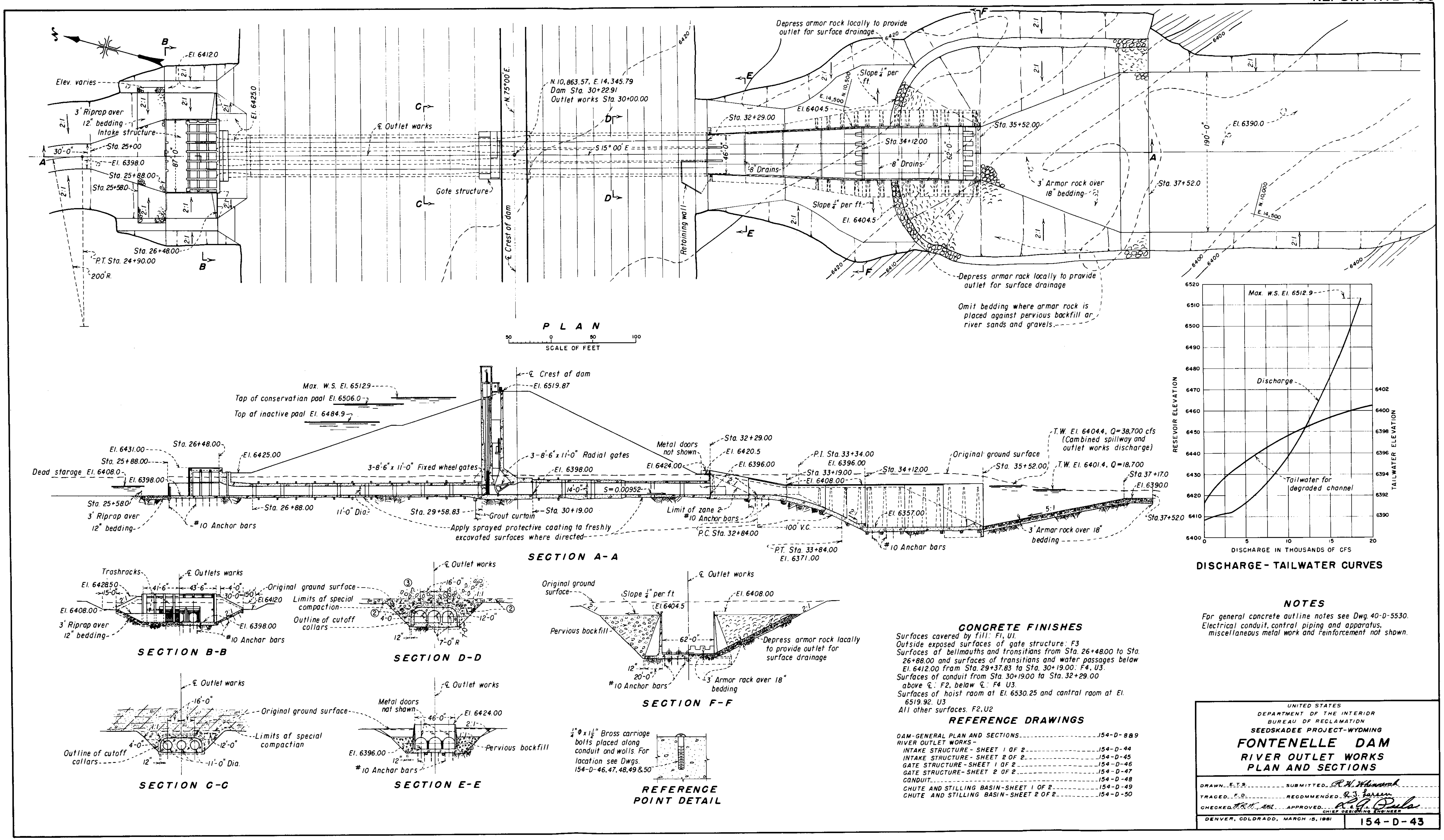
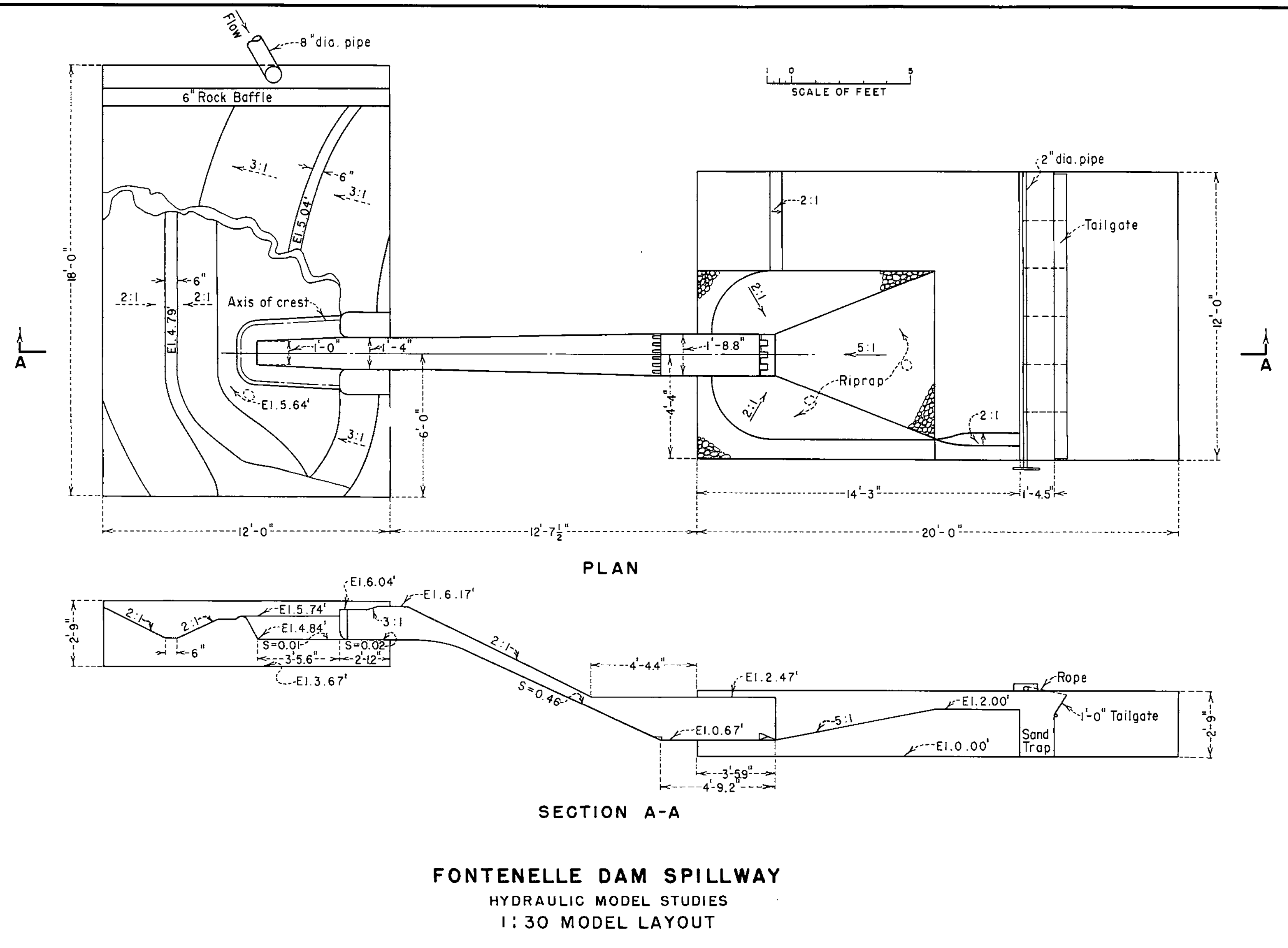

GREEN RIVER
PLAN
SCALE OF FEET
WEST CANAL OUTLET WORKS DISCHARGE IN HUNDRED C.F.S.
SPILLWAY DISCHARGE IN THOUSAND C.F.S.
RESERVOIR WATER SURFACE ELEVATION
TAILWATER ELEVATION
Max. Res. W.S. El. 6512.9
Spillway discharge
Crest El. 6506.00
Tailwater for degraded channel
West Canal Outlet Works discharge
DISCHARGE - TAILWATER CURVES
GUTTER DETAIL
CONCRETE FINISHES
Surfaces covered by fill or by second stage concrete: F1, U1
Bridge bearing surfaces: U3
Stop log seat: U3
Shaft house floor: U3
Outside exposed walls of shaft house: F3
All other surfaces: F2, U2
NOTES
For general concrete outline notes see Dwg. 40-0-5530
Electrical conduit, control piping and apparatus, miscellaneous metal work and reinforcement not shown
Unperforated pipe shall be used where calked joints are shown
REFERENCE DRAWINGS
DAM - GENERAL PLAN AND SECTIONS - SHEET 1 OF 2 ..... 154-D-8
DAM - GENERAL PLAN AND SECTIONS - SHEET 2 OF 2 ..... 154-D-9
SPILLWAY:
INLET STRUCTURE AND CHUTE TO STA. 2+81.00 ..... 154-D-40
CHUTE - STA. 2+81.00 TO STA. 5+65.25 ..... 154-D-41
STILLING BASIN 154-D-42
WEST CANAL OUTLET WORKS:
INTAKE AND GATE STRUCTURE 154-D-51
CONDUIT AND STILLING BASIN 154-D-52
SECTION K-K
SECTION B-B
SECTION D-D
SECTION G-G
SECTION H-H
SECTION C-C
SECTION F-F
SECTION E-E
SECTION J-J
SECTION A-A
PROFILE OF GROUT CURTAIN AROUND SPILLWAY INLET
REFERENCE POINT DETAIL
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SEEDSKADEE PROJECT - WYOMING
FONTENELLE DAM
SPILLWAY AND WEST CANAL OUTLET WORKS
PLAN AND SECTIONS
DENVER, COLORADO. MARCH 15, 1961
154-D-39

FIGURE NO. 4
REPORT HYD 486





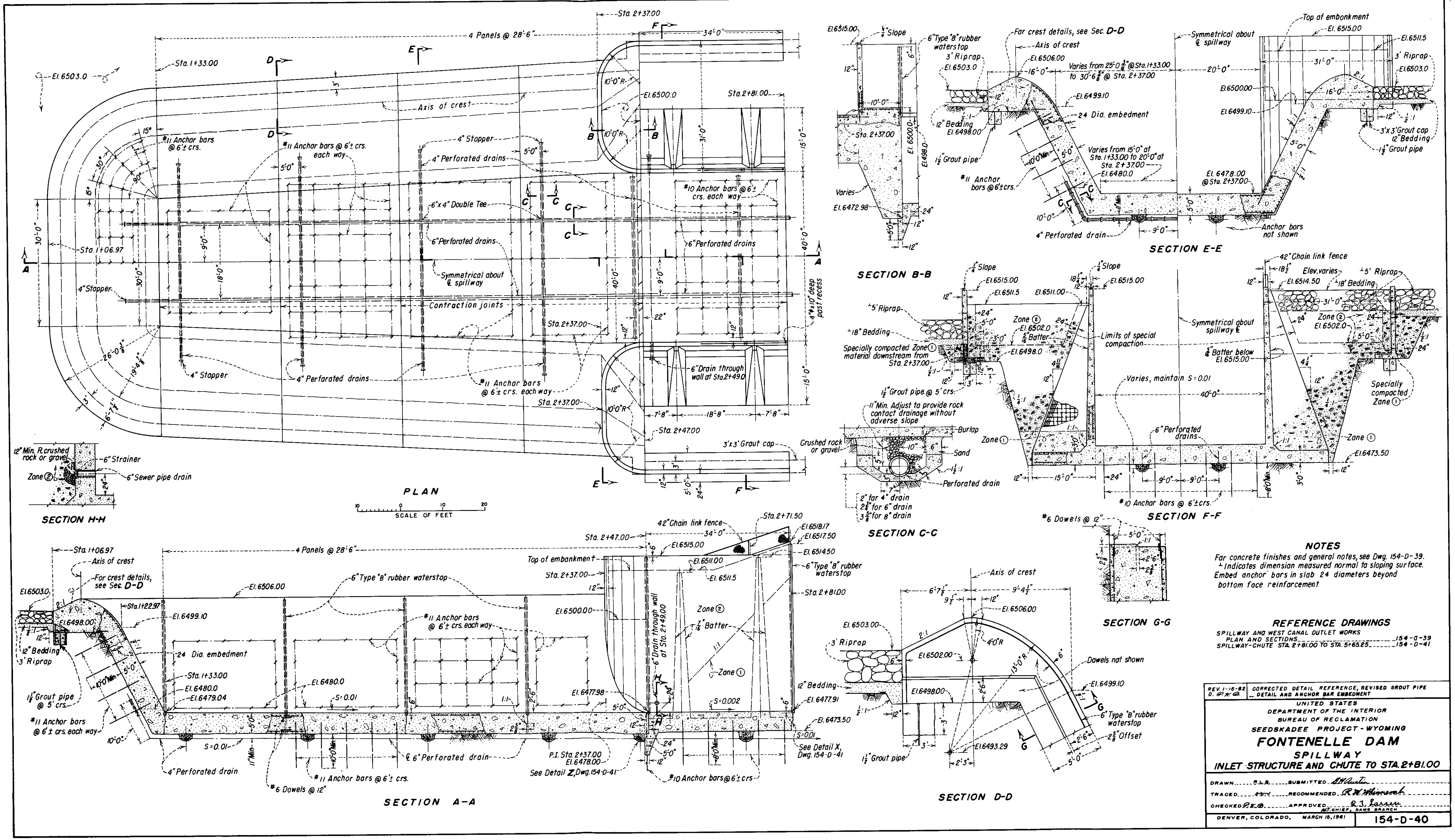
FONTENELLE DAM SPILLWAY

HYDRAULIC MODEL STUDIES

1:30 MODEL LAYOUT

FIGURE 5
REPORT HYD. 486

FIGURE NO. 6
REPORT HYD 486



Q ≐ 5,000 cfs

Q = 10,000 cfs

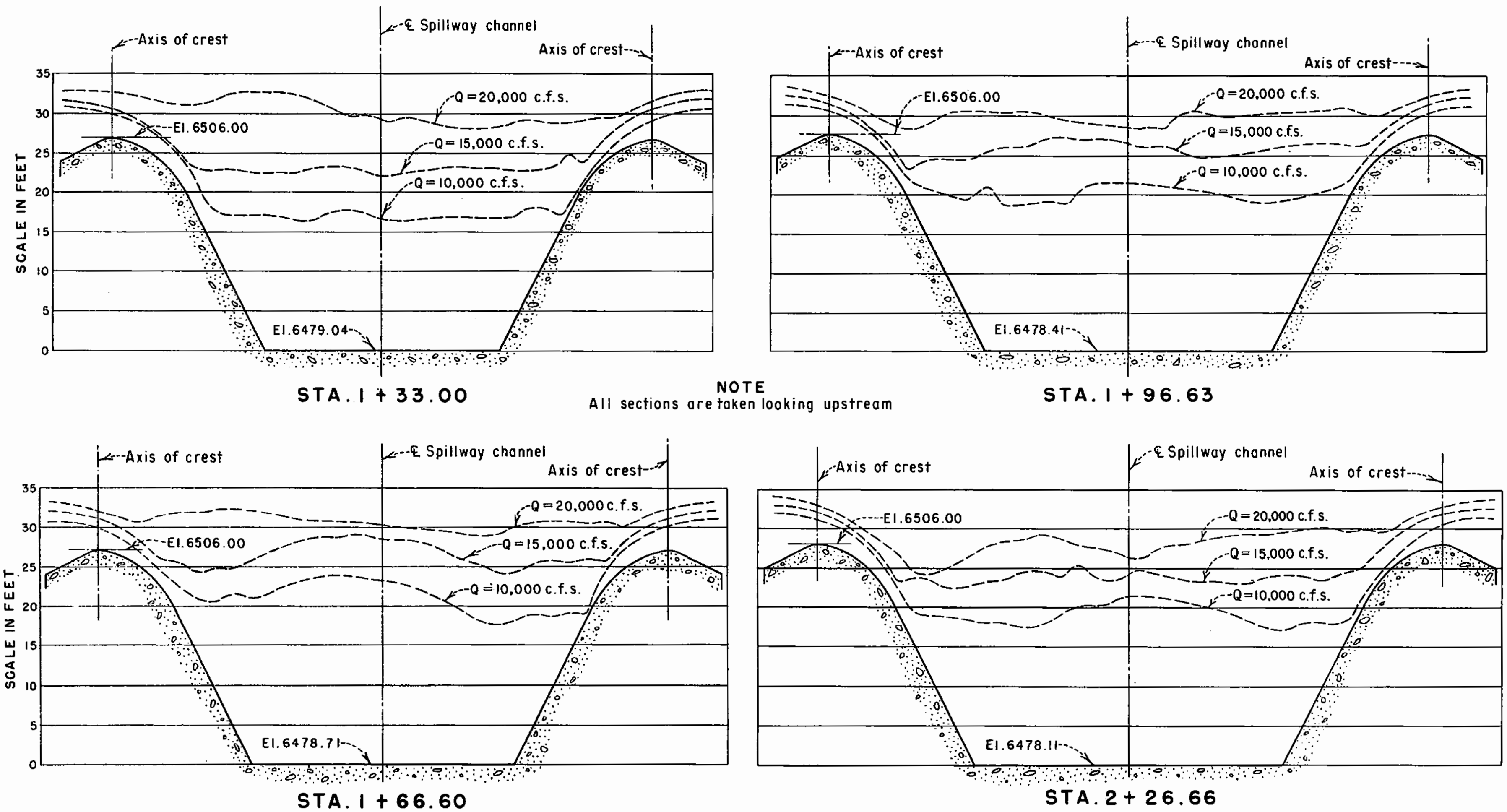
FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
1:30 Scale Model
Low Flows in Spillway Basin
Preliminary Design

Figure 8
Hyd 486

Q = 15,000 cfs

Q = 20,000 cfs

FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
1:30 Scale Model
Large Flows in Spillway Basin
Preliminary Design



FONTENELLE DAM SPILLWAY

HYDRAULIC MODEL STUDIES

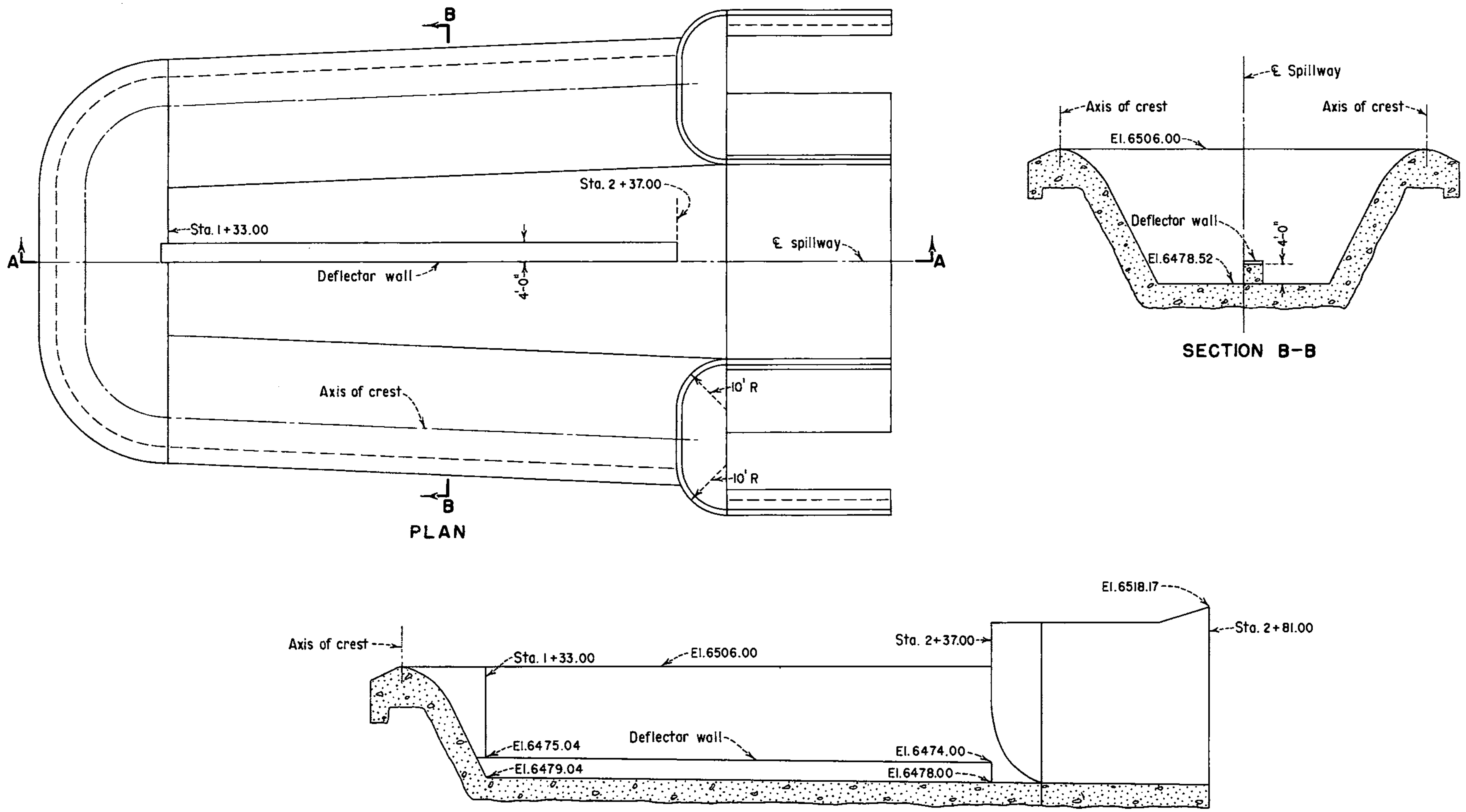
1:30 MODEL

WATER SURFACE PROFILES IN SPILLWAY BASIN

639

FIGURE 9
REPORT HYD. 488

FIGURE 10
REPORT HYD 486



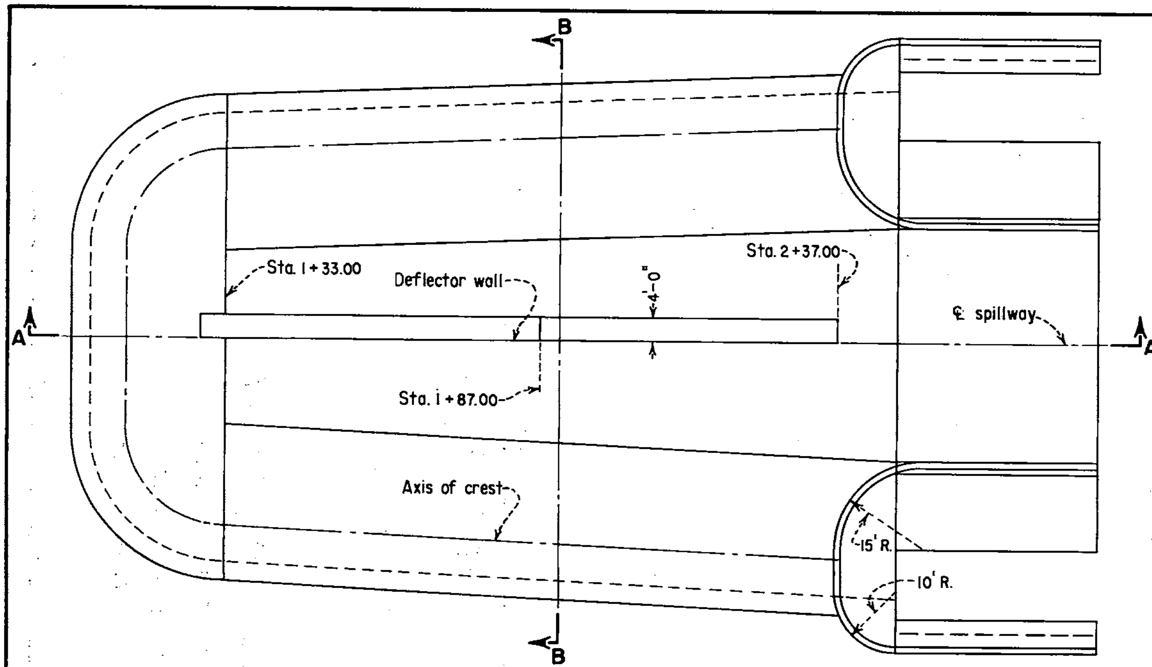
FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
1:30 MODEL
SPILLWAY BASIN STUDIES
MODIFICATION No. 1

639

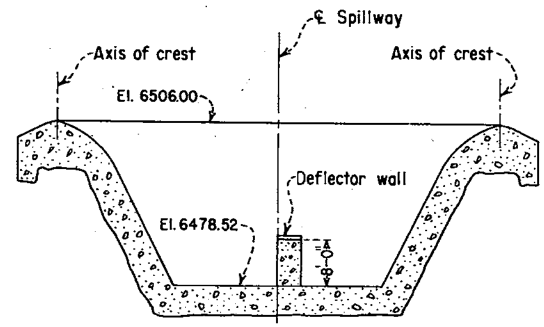
A. 4- by 4-foot Deflector Wall on Basin Floor
First Modification

B. Stepped Deflector Wall on Basin Floor
15-foot Radius on Side Piers
Second Modification

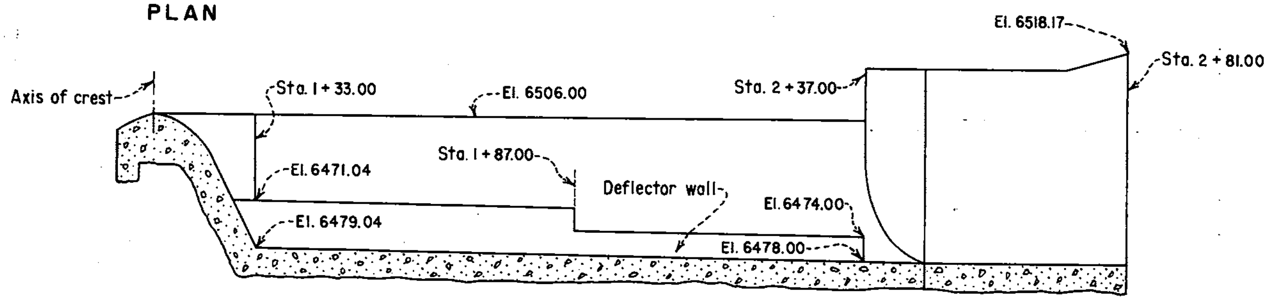
FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
1:30 Scale Model
20,000 cfs Flow in Modified Spillway Basin



PLAN



SECTION B-B



SECTION A-A

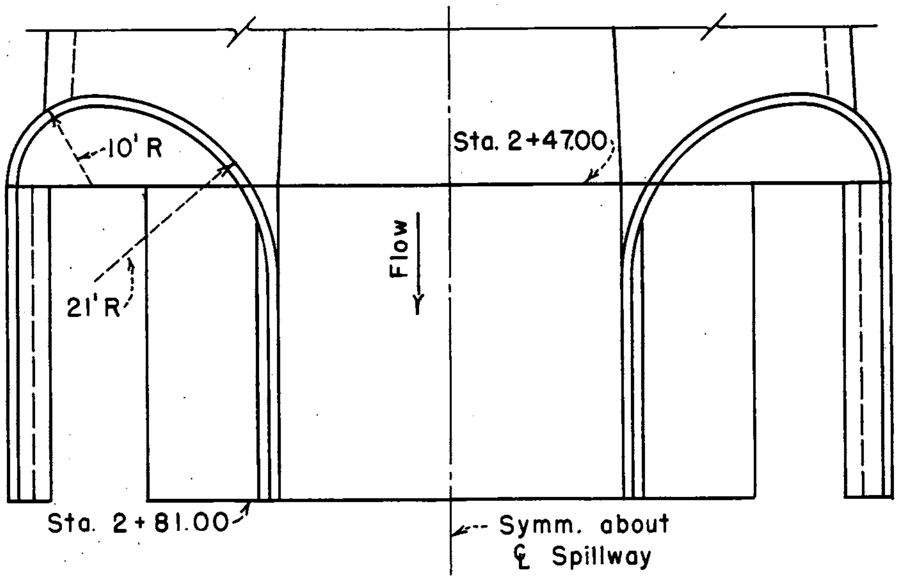
FONTENELLE DAM SPILLWAY

HYDRAULIC MODEL STUDIES

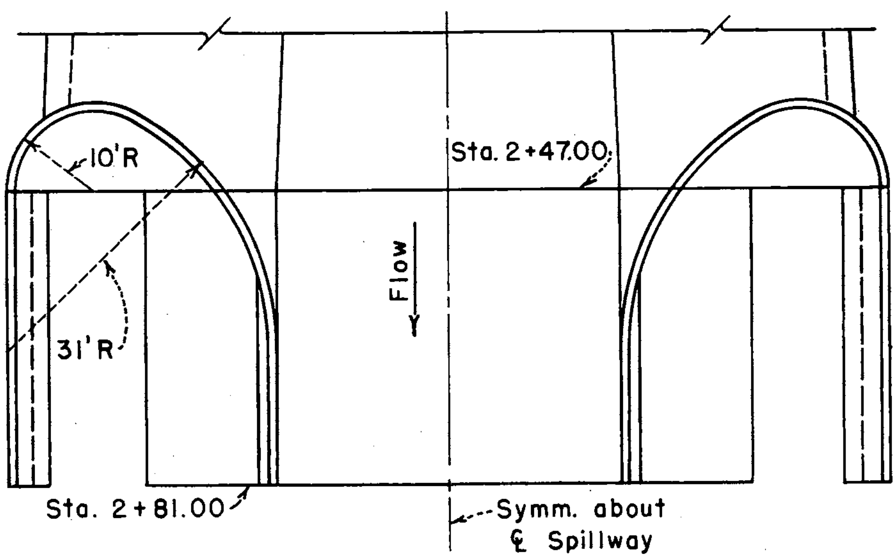
1 : 30 MODEL

SPILLWAY BASIN STUDIES
MODIFICATION No. 2

639



PIER MODIFICATION NO. 2



PIER MODIFICATION NO. 3

FONTENELLE DAM SPILLWAY

HYDRAULIC MODEL STUDIES

SPILLWAY BASIN STUDIES

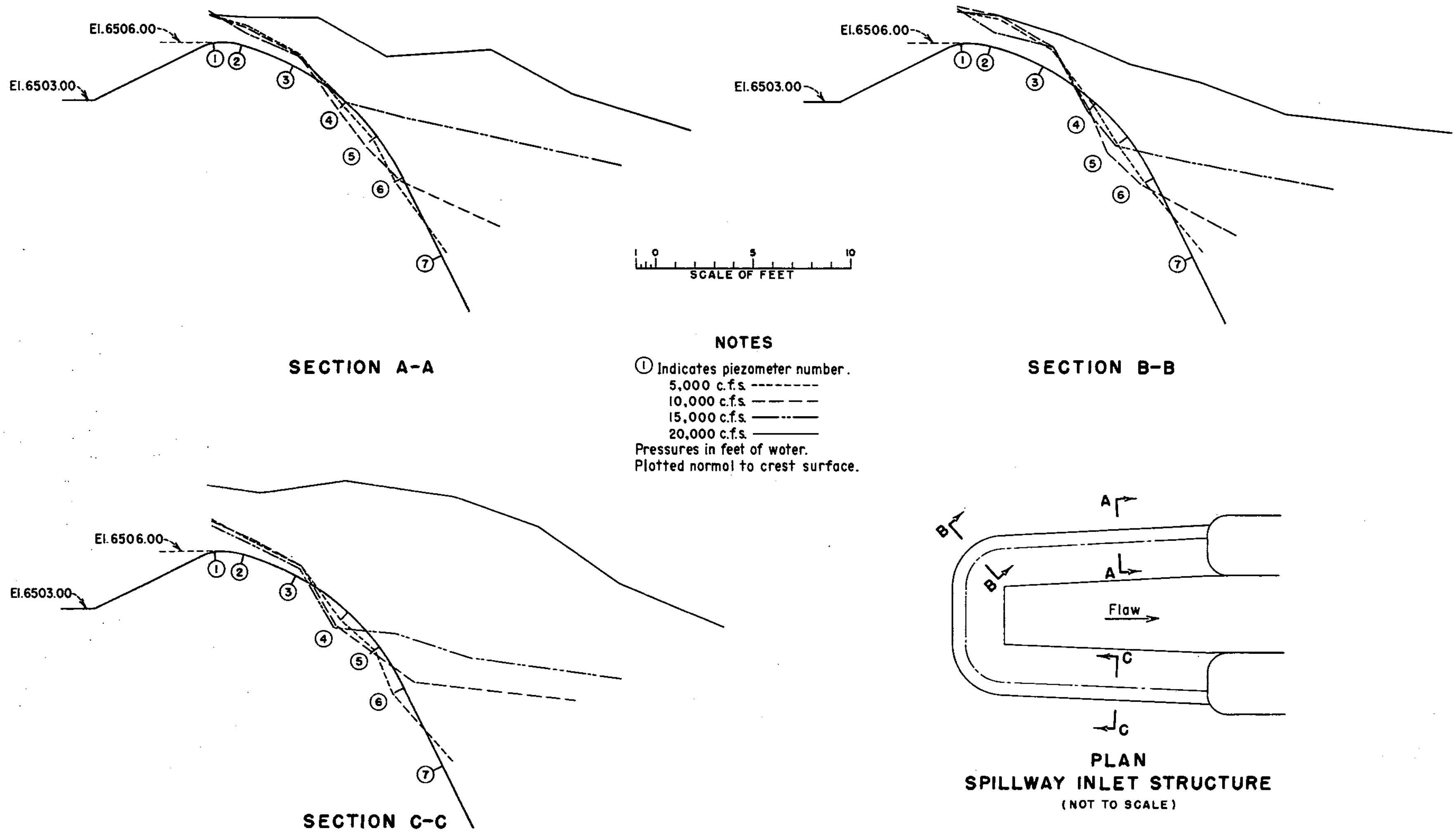
PIER MODIFICATIONS 2 AND 3

A. 21-foot Radius on Side Piers

B. 31-foot Radius on Side Piers

Q = 20, 000 cfs

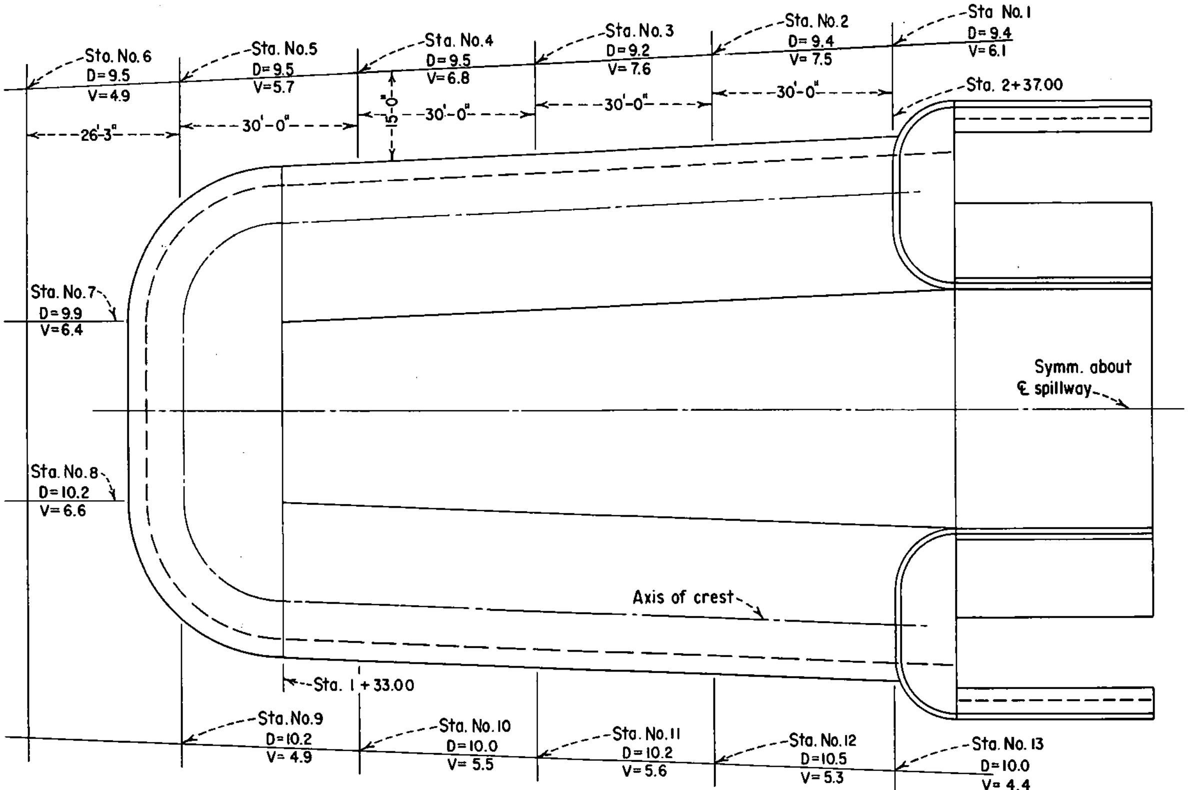
FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
1:30 Model
Flow in Spillway Basin, Pier Modifications 2 and 3



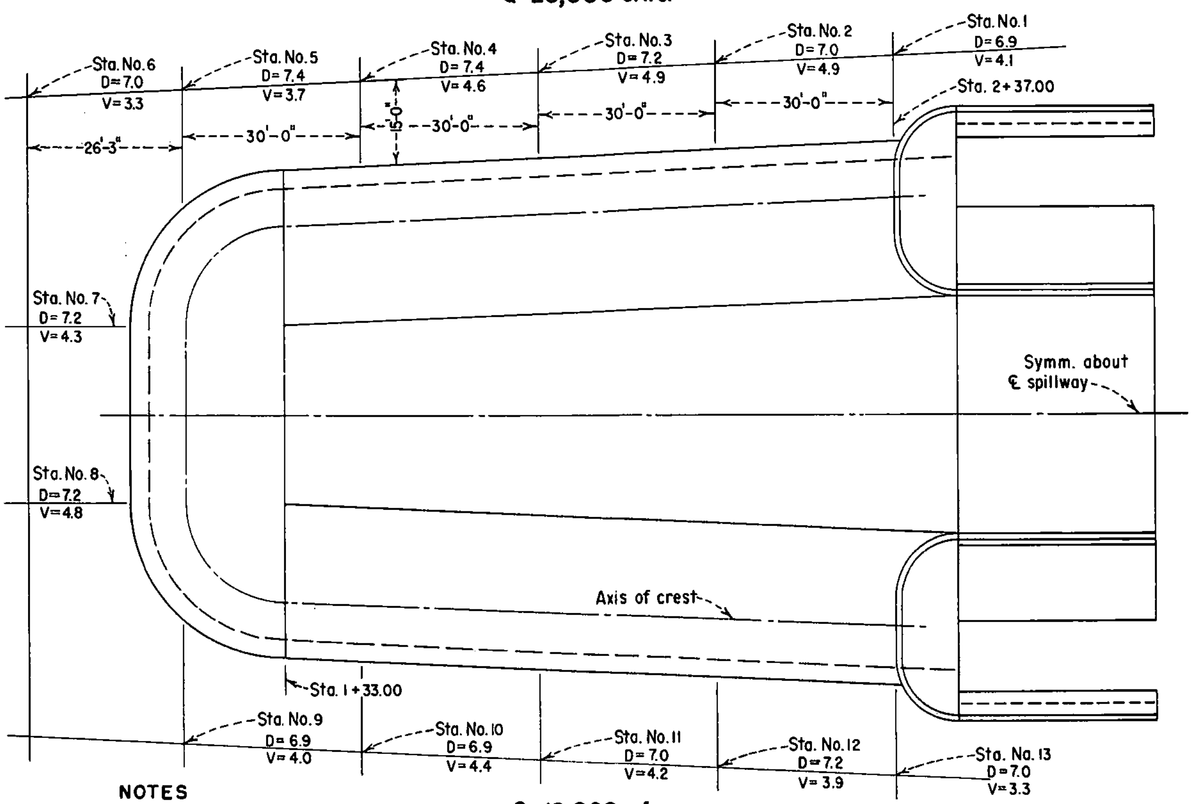
FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
1:30 MODEL
CREST PRESSURES

FIGURE 15
REPORT HYD. 486

639



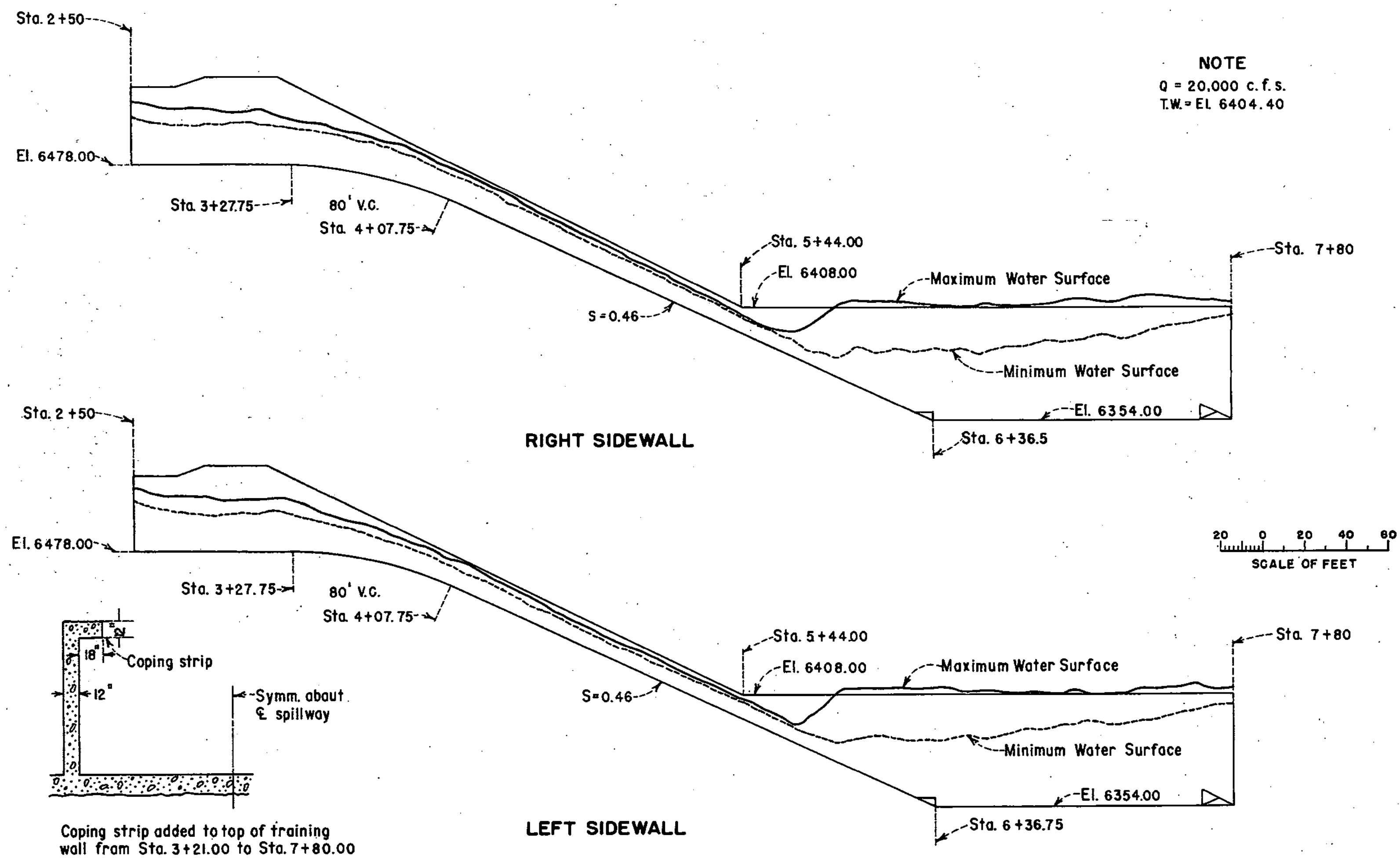
Q=20,000 c.f.s.



NOTES
V= Velocity of water in feet per second.
D= Water depth in feet.

Q=10,000 c.f.s.

FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
1:30 MODEL
FLOW DEPTH AND VELOCITY ON SPILLWAY BERM



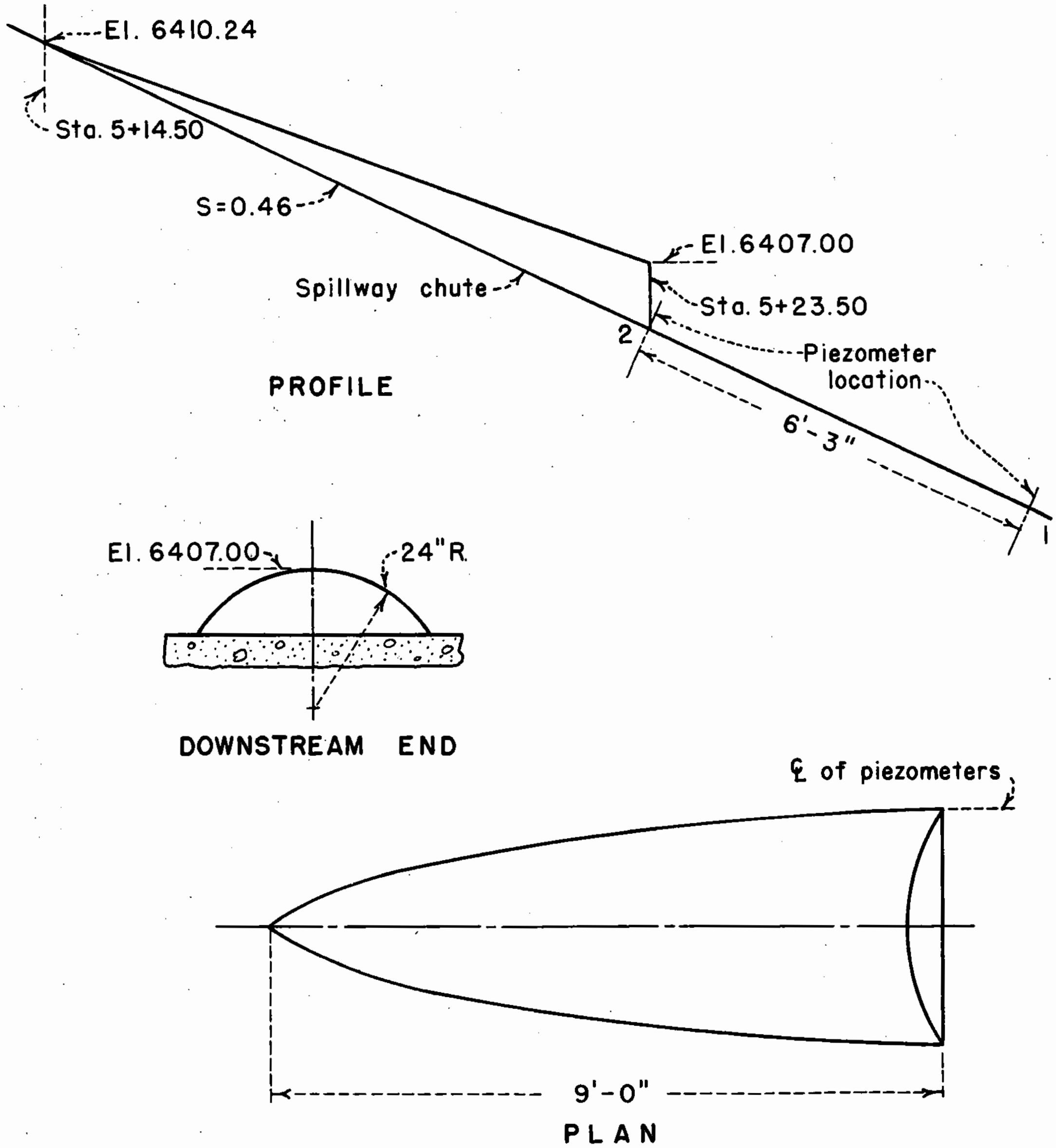
FONTENELLE DAM SPILLWAY

HYDRAULIC MODEL STUDIES

1:30 MODEL

WATER SURFACE PROFILES IN SPILLWAY CHUTE AND STILLING BASIN

639



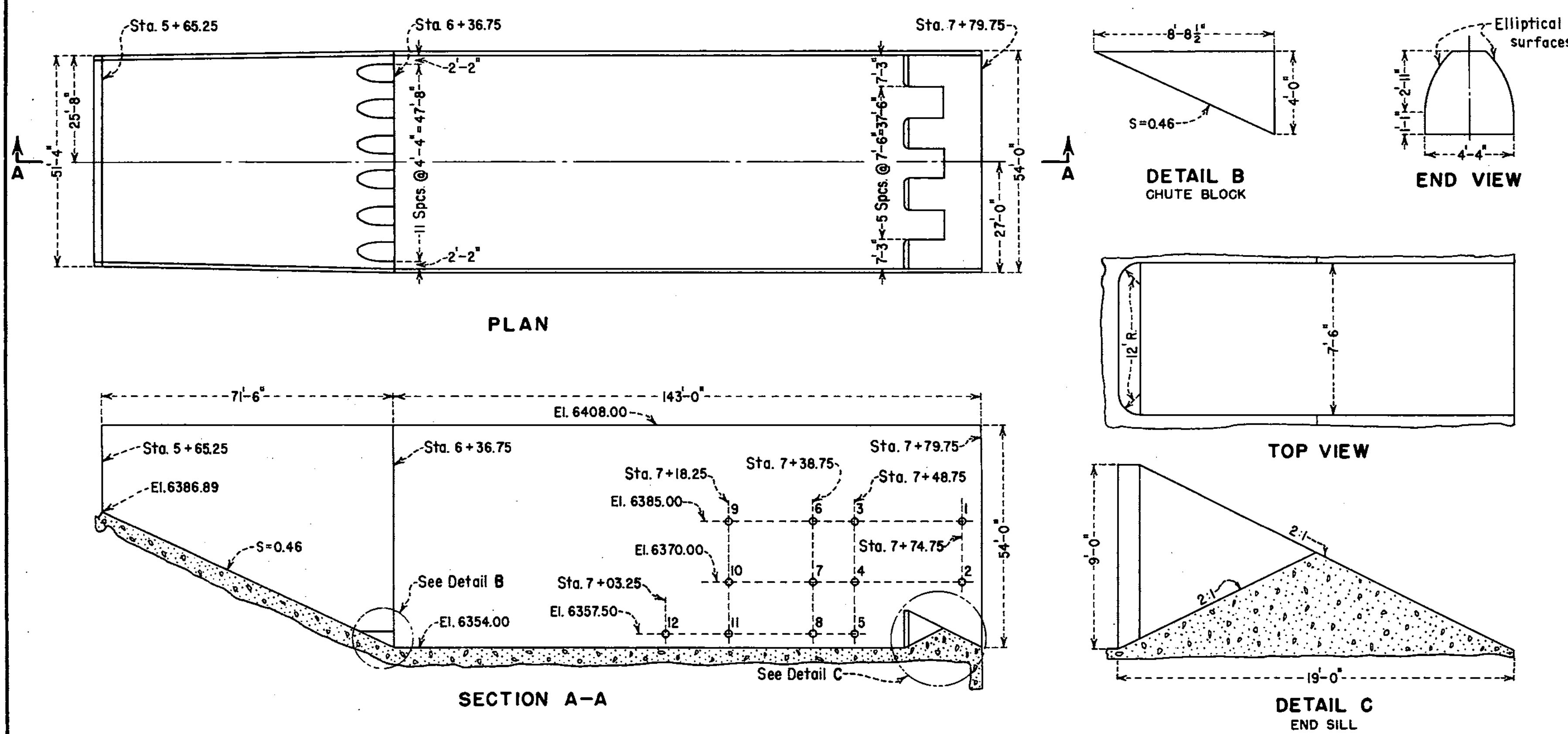
PRESSURES AT EYEBROW DEFLECTOR

PIEZ. NO.	Q = 5,000 C.F.S.	Q = 10,000 C.F.S.	Q = 15,000 C.F.S.	Q = 20,000 C.F.S.
1	3.12'	5.31'	6.56'	7.35'
2	-10.00'	-12.94'	-12.81'	-10.94'

FONTENELLE DAM SPILLWAY

HYDRAULIC MODEL STUDIES

DRAIN DEFLECTOR ON SPILLWAY CHUTE



NOTES

Numbers 1 thru 12 indicate piezometers located in the left wall of the stilling basin.
Numbers 9R thru 12R indicate piezometers located in the right wall at the same stations as similar numbers in left wall.

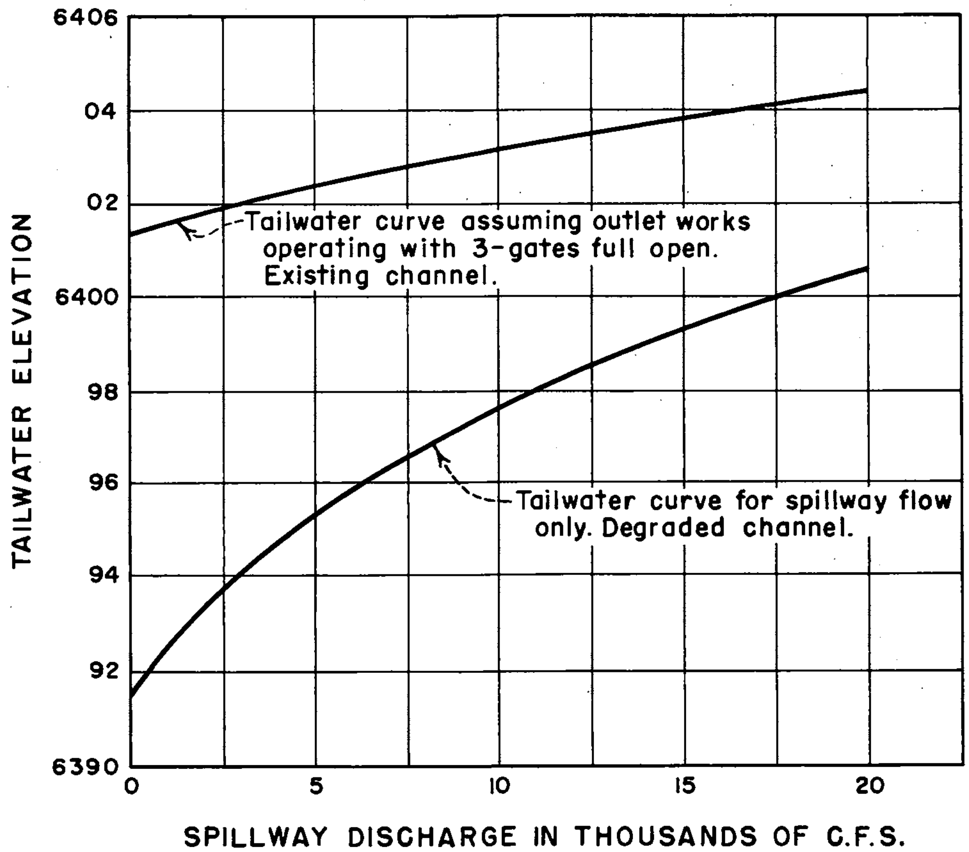
PRESSURE ON STILLING BASIN SIDEWALLS

PIEZ. NO. →		1	2	3	4	5	6	7	8	9	10	11	12	9R	10R	11R	12R
Q=20,000 c.f.s.	MAX.	49.5	38.8	24.9	42.2	87.5	19.0	56.8	64.7	18.0	59.5	49.2	62.1	23.8	52.9	68.8	63.1
T.W.=6404.4	MIN.	5.9	12.4	3.3	10.7	14.3	2.2	1.6	25.4	1.2	1.6	20.7	4.5	1.3	-0.8	14.5	21.1
Q=20,000 c.f.s.	MAX.	30.2	37.6	22.8	44.3	69.2	18.4	48.4	56.9	16.2	52.0	46.2	57.6	19.0	47.8	54.4	64.3
T.W.=6400.6	MIN.	11.6	10.0	-2.7	0.1	6.5	0.2	0.7	18.8	-5.4	2.8	13.2	0.6	-1.1	3.1	7.9	0.7

FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
1:30 MODEL
STILLING BASIN STUDIES
SIDEWALL PRESSURES AND PIEZOMETER LOCATIONS

639

FIGURE 19
REPORT HYD 486



FONTENELLE DAM SPILLWAY

HYDRAULIC MODEL STUDIES

TAILWATER ELEVATION CURVES

Q = 5,000 cfs, T.W. = el. 6395.20

Q = 5,000 cfs, T.W. = el. 6402.30

FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
1:30 Model
5,000 cfs Flow in Final Stilling Basin

Figure 21
Hyd 486

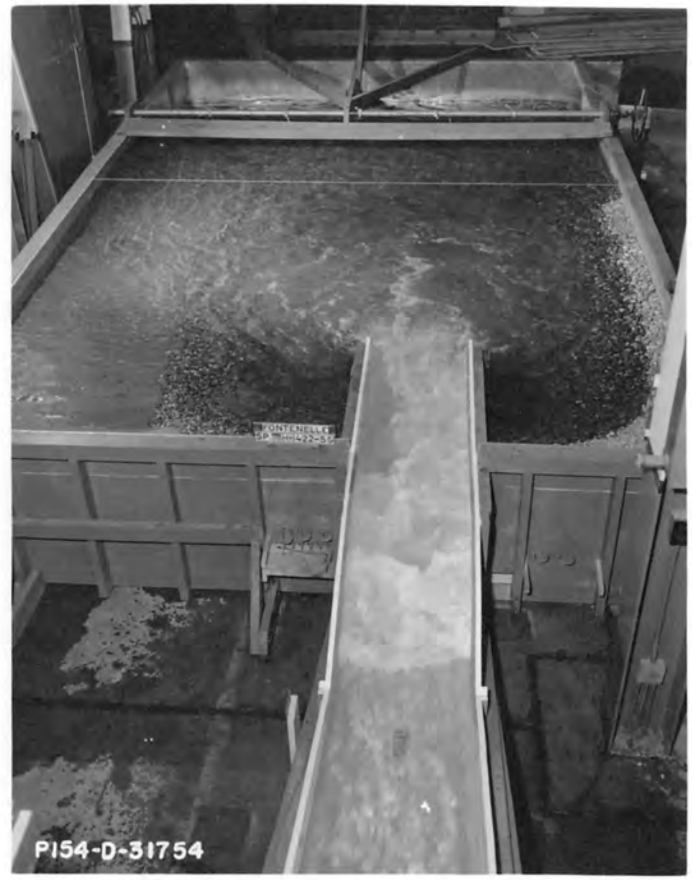
Q = 10,000 cfs, T.W. = el. 6397.60

Q = 10,000 cfs, T.W. = el. 6403.10

FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
1:30 Scale Model
10,000 cfs Flow in Final Stilling Basin

Figure 22
Hyd 486

Q = 15,000 cfs, T.W. = el. 6399.70



Q = 15,000 cfs, T.W. = 6403.80

FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
1:30 Scale Model
15,000 cfs Flow in Final Stilling Basin

Q = 20,000 cfs, T.W. = 6400.55

Q = 20,000 cfs, T.W. = 6404.40

FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
1:30 Scale Model
20,000 cfs Flow in Final Stilling Basin

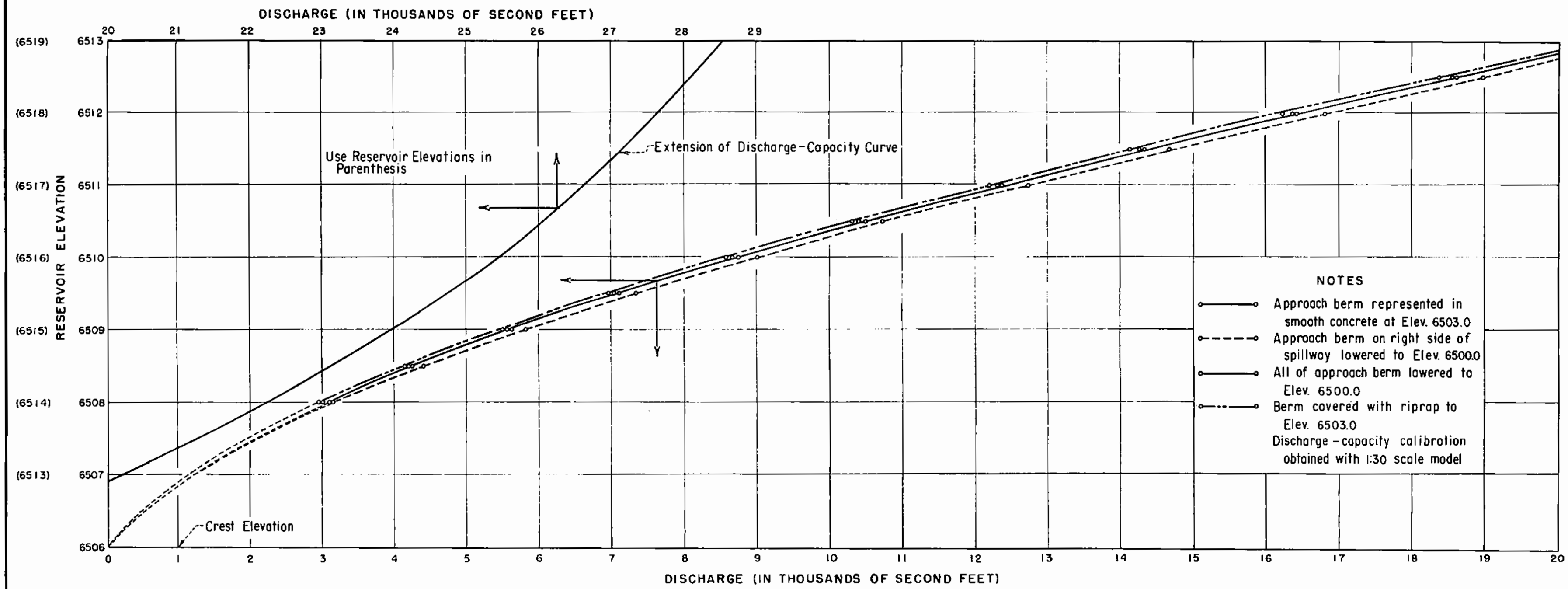
Channel before erosion test

Erosion after 8 hours

Riprap in downstream channel

FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
1:30 Model
Channel Bed Scour and Protection

Figure 25
Hyd 486



* Reservoir elevations 6506 to 6513 are for discharges from 0 to 20,000 second-feet.
Reservoir elevations (6513) to (6519) are for discharges from 20,000 to 28,500 second-feet.

FONTENELLE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
DISCHARGE – CAPACITY

639

FIGURE 26
REPORT HYD. 486