UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF THE FONTENELLE DAM OUTLET WORKS SEEDSKADEE PROJECT, WYOMING

Hydraulic Laboratory Report No. Hyd. 487

Division of Engineering Laboratories

OFFICE OF ASSISTANT COMMISSIONER AND CHIEF ENGINEER
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Subject: Hydraulic model studies of the Fontenelle Dam Outlet Works--Seedskadee Project, Wyoming

PURPOSE

The studies were conducted to develop a satisfactory design for the outlet works conduits, the stilling basin, and a proposed pressure conduit and Y-branch to be installed in the right conduit of the outlet works.

CONCLUSIONS

1. Flow through the gate control structure and the horseshoe conduits was satisfactory for all discharges (Figure 8).

2. The addition of tapered piers between adjacent tunnels at the portals improved the flow appearance on the outlet works chute (Figures 8C and 10).

3. The flow distribution on the outlet works chute was satisfactory for all operating combinations of the three tunnels (Figure 11).

4. The stilling basin provided good energy dissipation for all discharges (Figures 13, 14, and 15), although there was considerable splashing and spray at the larger discharges. An 18-inch-wide coping strip, (Figure 9), added to the top of the basin walls, reduces tendencies for surges and waves to overtop the training walls.

5. Single operation of either outside conduit at maximum discharge (6,233 cfs) and high tailwater conditions should be avoided. This operation causes large waves to form and splash over the sides of the basin (Figure 15).

6. Dynamic pressure measurements made on the stilling basin side walls indicated pressure fluctuations both higher and lower than the static pressure based on the water surface profiles. These pressure fluctuations should be considered in the structural design of the walls.

7. The head discharge capacity of the top seal radial gates was adequate for expected releases (Figure 16).

8. Head losses in the preliminary and recommended Y-branches were measured and loss coefficients versus the ratio of the flow in the penstock lateral to the total flow entering the Y-branch were determined, Figures 17 and 28. The head loss with the recommended Y-branch was as much as 56 percent less than with the preliminary Y-branch.

9. Under normal or expected operating conditions the pressures for both the preliminary and recommended Y-branches were satisfactory. However, under certain operating conditions extreme subatmospheric pressures were encountered, Figures 17 and 28. To prevent these poor pressure conditions the gate size was reduced from 8 feet 6 inches by 9 feet to 8 feet 6 inches square and a stop was placed on the stem to prevent the gate from operating at openings greater than 8 feet.

10. Best operation in the spillway chute was obtained with the penstock outlet structure downstream from the Y-branch placed horizontal and turned 5° to the left, Figures 25, 26, and 27. However, satisfactory flow conditions on the chute also were obtained with the structure tilted downward 5° and turned 5° to the left, Figures 18, and 19, and with the structure horizontal and turned 3° to the left, Figures 23 and 24. Based on structural considerations, the recommended outlet structure was tilted downward 5° 34' and turned 4° to the left, Figure 28.

11. Flow from the recommended penstock outlet structure was equally distributed across the stilling basin chute; however, it was necessary to place short, curved training walls at the downstream end of the structure to prevent objectionable fins from forming along the chute walls (Figures 29, 30, and 31).

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, 24 miles downstream from LaBarge, Wyoming (Figure 1).

The dam is an earth and gravel structure approximately 6,000 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 is located near the center of the embankment (Figure 2). The spillway, designed for a maximum discharge of 20,000 cfs, is an uncontrolled double side channel spillway with a crest length of about 310 feet. 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 (Figure 3). Hydraulic model studies of the spillway are discussed in Report Hyd. 486.

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

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

Ultimately it is planned to construct a hydroelectric plant as a part of the project. A 10-foot-diameter pressure conduit will be placed in the right horseshoe conduit of the outlet works, (Figure 5). Water to the powerplant will be supplied through a 10-foot-diameter penstock lateral which branches off the main conduit about 20 feet upstream from the tunnel portal. The main conduit will terminate at a slide-gate-controlled turnout structure at the upstream end of the stilling basin chute.

As a part of the hydraulic investigations, the Y-branch and the penstock outlet structure were installed in the model to determine the losses in the Y-branch and the hydraulic characteristics of the outlet structure.

THE MODEL

The model was built to a geometrical scale ratio of 1:24.7 so that parts of a recently tested model could be reused. Represented in the model were the three 11.0-foot-diameter conduits, the gate chamber and radial gates, the horseshoe conduits, the stilling basin, and the excavated channel (Figure 6).

The intake structure was not included in the model; water was distributed to the three circular conduits through a baffled manifold connected directly to the laboratory water supply system. To assure smooth flow in the conduits, four-vane flow straighteners were placed at the upstream end of each conduit.

The circular conduits upstream from the gate chamber were represented in the model by 6.2-inch-diameter sheet metal pipes. The gate chamber and horseshoe conduits were fabricated from transparent plastic. The radial gates were made of galvanized sheet metal. The gate chamber piers, the stilling basin, and the stilling basin chute blocks and dentated end sill were made of wood treated to resist swelling. The downstream river channel was formed in sand with a median diameter of approximately 0.8 mm, with 90 percent between the No. 8 and 200 Tyler standard screens.

Discharges in the model were measured using calibrated Venturi meters permanently installed in the laboratory. Pressure heads in the circular conduits were measured by means of piezometers placed in the invert of each conduit and located one-conduit diameter upstream from the gate chamber transition.

The piezometer leads from each of the conduits were connected to a separate open-tube glass manometer. Tailwater elevations, measured on a staff gage located in the center of the channel, were controlled by an adjustable tailgate at the downstream end of the model.

Since the model did not include the reservoir area or the outlet works intake structure, the corresponding pressure head in the conduits was determined by computing the hydraulic losses from the reservoir water surface to the pressure-measuring station, 1-diameter upstream from the gate chamber transition. The hydraulic losses included the entrance loss at the intake structure and the pipe friction loss between the intake structure and the piezometer ring. The entrance loss (h_e) was determined from $h_e = K \frac{v^2}{2g}$ where K = constant = 0.1 and v = the mean velocity in the conduit. The relatively high K value was used because gate slots were placed in the bellmouth entrance. The friction loss was computed from $h_f = f \frac{\ell}{d} \frac{v^2}{2g}$ where:

$f = 0.012$ = friction factor
$\ell = 242$ feet = conduit length
$d = 12.76$ feet = conduit diameter
$v = \frac{Q}{A}$ for the circular conduit. (Mean velocity.)

Although the diameter of the circular conduits in the prototype structure is 11.00 feet, the 6.2-inch-diameter pipe used in the model scales up to 12.76 feet; therefore, the 12.76-foot diameter was used in determining the model velocity head and corresponding pressure head for test purposes.

THE INVESTIGATION

During the investigation, test discharges were based on two reservoir elevations and two tailwater elevation curves. The two reservoir elevations were the maximum reservoir elevation 6512.9, and the maximum conservation pool, elevation 6506.0. At the maximum reservoir elevation the spillway discharges 20,000 second-feet which flows into the same channel as the outlet works. Therefore, the tailwater elevation is governed by the combined flow of the spillway and the outlet works. For the 6506.0 reservoir elevation, only the outlet works is operating and the tailwater elevation is based accordingly. In addition to the above, the downstream channel conditions also govern the tailwater elevation. After the first few years of operation the discharge channel is expected to degrade several feet, thus lowering the tailwater elevation. The two tailwater curves used in these tests are shown on Figure 7. The top curve is for the maximum reservoir elevation and assumes existing channel conditions. The bottom curve is for the lower reservoir elevation and assumes degraded channel conditions.

Flow in Gate Control Structure and Horseshoe Conduits

The gate control structure in each of the three conduits consists of an 8-foot-6-inch by 11-foot rectangular passage, containing a fixed-wheel slide gate followed by a top-seal radial gate (Figure 4). The radial gates will be used for flow regulation; the fixed-wheel gates will be used for emergency closure if the radial gates need repair. Neither the fixed-wheel gates nor the gate slots were reproduced in the model.

Downstream from the radial gates, the sidewalls in each passage diverged to increase the channel width to 14 feet. The roof of this diverging section was also transitional to form the semicircular crown of each of the downstream horseshoe conduits. Each horseshoe conduit extended from the end of this transition at Station 30+19.00, downstream to Station 32+29.00.

Flow passed through the gate chamber, transition, and horseshoe conduits in a satisfactory manner for all discharges. When the radial gates were partially closed, the water surface was smooth and the flow was well distributed at the start of the horseshoe conduits. When the gates were fully open, the flow pattern was equally good and the gate trunnions were above the water surface at all discharges.

At the maximum discharge a fin formed along the sidewalls at the upstream end of the conduits but the fins had no tendency to fold over and fill the conduit (Figure 8A). Aside from the fins, the water surface in the horseshoe conduits was steady with no apparent undulations.

Outlet Works Chute

The chute between the horseshoe conduits and the stilling basin was a rectangular open channel diverging from 46 feet wide at the tunnel portals to 62 feet wide at the stilling basin. The chute bottom was horizontal for the first 55.00 feet with a vertical curve for the next 100 feet and 2:1 slope for the final 28 feet (Figures 4 and 9).

The performance of the chute was excellent in all respects during symmetrical tunnel operation. The water jets spread as they emerged from the three conduits, and only a small surface fin formed where the flows from adjacent tunnels met, (Figure 8B). Although these fins extended only a short distance and caused no adverse flow conditions in the stilling basin, it was decided to eliminate the fins since they were unsightly and a source of spray. To eliminate the fins, tapered piers were installed between the tunnels at the tunnel portals (Figure 10). The piers were 19 feet long and tapered from 2 feet wide at the tunnel portals to 1 foot wide at the downstream end. The piers reduced the fins to a negligible size (Figure 8C), and allowed the flow to spread across the chute when releases were made through one or two tunnels.

With all combinations of one or two tunnels operating, the flow distribution in the chute was sufficiently good that no adverse eddies formed in the stilling basin. Figure 11 shows the flow conditions in the chute for various combinations of one and two operating conduits.

Stilling Basin Studies

The stilling basin studies were concerned with developing an effective stilling basin that would provide good energy dissipation with a minimum amount of bank and channel bed scour for all of the expected combinations of discharge and tailwater elevation.

Tailwater elevation at the outlet works stilling basin will be governed not only by the discharge from the outlet works but also by the flow from the service spillway since the flow from both the spillway and the outlet works enter the same channel. When the reservoir elevation is higher than the service spillway crest (elevation 6506.0), the tailwater elevation is determined by the combined flows of the spillway and outlet works; when the reservoir elevation is below 6506.0, the tailwater elevation is determined by the outlet works discharge only.

Channel conditions in the Green River also have a bearing on tailwater elevations. Two tailwater elevation curves are shown on Figure 7; the upper curve is for existing tailwater conditions and the lower curve is for a degraded channel. Preliminary tests showed that the high tailwater elevations provided the most severe operating conditions.

Preliminary Stilling Basin. The preliminary stilling basin was 140 feet long and 62 feet wide. The floor was at elevation 6357.0, and the top of the training wall was at elevation 6408.0. Chute blocks were used at the upstream end of the basin, and a dentated sill was placed at the downstream end, Figure 9. The five chute blocks, equally spaced across the basin at the toe of the chute, were 4 feet high and 5.75 feet wide; the top edges of the blocks were streamlined with elliptical curves. The dentated end sill had a 2:1 slope on the upstream and downstream faces. Four dentils equally spaced on the upstream face of the sill were 9 feet high and 5.75 feet wide. The dentils adjacent to the wall on either side were 5 feet 1-1/2 inches wide. The upstream edges of each dentil were streamlined with a 12-inch radius quarter circle (Figure 9).

Downstream from the stilling basin, the riprapped channel bed sloped upward on a 5:1 slope to elevation 6390. The bottom width diverged from 62 feet at the basin to 190 feet at the top of the slope. The sides of the channel were formed on a 2:1 slope. The prototype riprap was represented in the model with 1/4- to 3/4-inch gravel.

The effectiveness of the stilling basin was evaluated for maximum discharge at both tailwater conditions. 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 or riprap movement, during or after a reasonable period of operation, usually about 1 hour.

For the maximum discharge and the high tailwater condition, the flow appearance was very good. The surface of the hydraulic jump was rough with considerable surging but the jump was entirely contained within the basin (Figure 13). At the toe of the jump there was considerable splashing and some water overtopped the training walls. The maximum wave heights, measured at the end of the basin to the right of the training wall, were about 3.6 feet, with the average height being about 1.4 feet. The riprap on the floor and sides of the channel was not disturbed during this operating condition.

With the maximum discharge and low tailwater, the jump was somewhat rougher and extended a short distance downstream beyond the end of the basin (Figure 13). The action at the toe of the jump did not splash as much water over the top of the training walls as observed with the higher tailwater. A small amount of riprap on the left side slope about 70 feet downstream from the end of the basin moved during this test but the quantity was not measurable. The wave heights for this tailwater condition averaged about 1.2 feet with a maximum of about 2.5 feet.

Tests were also made with various combinations of conduits operating to determine if the asymmetrical flow would cause inadequate energy dissipation in the stilling basin. For most combinations, the stilling basin was satisfactory; the stilling action was confined within the basin at all times, and the flow beyond the end of the basin was smooth (Figure 14).

The tests with unsymmetrical operation indicated one operating condition that should be avoided if possible. When either outside conduit was operated alone at maximum capacity, 6,233 cfs, with the high tailwater elevation 6403.0, the jet did not spread across the chute and failed to penetrate the tailwater. Consequently, the jet pushed the tailwater downstream and formed a large wave that splashed over the side of the training wall (Figure 15). Excessive splash probably will cause erosion of the backfill on the outside of the wall. A similar action also occurred when the center and one outside conduit were in operation but the magnitude of the waves was much less; this action did not occur with only the center conduit operating.

Recommended Stilling Basin. The preliminary stilling basin was satisfactory in providing good energy dissipation and no major modifications were recommended. However, to eliminate some of the splashing that overtopped the training walls at the maximum discharge, a coping strip 18 inches wide and 12 inches deep was installed at the top of both walls between Stations 32+74.0 and 35+05.33 (Figure 9, Detail C). Although the coping strip did not prevent splashing from overtopping the walls, the amount of splash was greatly reduced.

Sweepout Test. A test was performed to further evaluate the stilling basin by determining whether the hydraulic jump would sweep from the basin when the tailwater was below the minimum design elevation. The tailwater was gradually lowered, with the maximum discharge of 18,700 second-feet passing through the basin. Because of model limitations, the tailwater could only be lowered to elevation 6391 or about 9 feet below the minimum design elevation. At this tailwater level, the toe of the jump was 25 feet upstream from the end of the chute and the chute blocks remained covered at all times. This test indicated that the basin could be safely operated at maximum discharge at a tailwater elevation at least 9 feet lower than the minimum predicted tailwater.

Pressures on Training Walls. The stilling basin sidewalls extend out into the tailwater pool. The water behind the walls will not be in motion and will stand at about the elevation of the downstream tailwater, producing a relatively constant force on the outside of the training walls. When the outlet works is operated, the water surface inside the basin will be generally lower than the downstream tailwater elevation, producing a differential pressure on the walls. In addition, dynamic forces produced by the hydraulic jump action create intermittent pressures that vary above the static pressures on the inside of the walls. To aid in the structural design of the walls, these forces were evaluated in the model by measuring the magnitude of the pressures, the pressure differential, and the extent of pressure fluctuations on the training walls.

Initially, piezometers were installed on the inside surface of the right training wall at Stations 34+92.00, 35+17.00 and 35+42.00 to measure the dynamic forces on the walls. At the first two stations piezometers were located at elevations 6360, 6370, 6380, and 6390; piezometers at the downstream station were located at elevations 6370, 6380, 6390. Preliminary pressure measurements indicated that large pressure fluctuations might occur in the vicinity of the piezometer at Station 34+92.00, elevation 6370, at the contraction joint at Station 35+05.33, and near the piezometer at Station 35+17.00, elevation 6390. To obtain more data at these locations, additional piezometers were installed at Station 34+82.00, elevations 6370 and 6375; at Station 35+04.50, elevations 6370, 6375, 6385, and 6395; at Station 35+17.00, elevations 6385 and 6395; and at Station 35+29.5, elevations 6385 and 6395. A total of 22 piezometers were installed in the right wall (Figure 9).

The piezometers were connected to pressure cells sensitive to instantaneous pressure fluctuations. Magnitude and frequency of pressure fluctuations 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 pressures at the piezometer. Water surface profiles within the basin (Figure 12) and pressure measurements were obtained with all three conduits operating at maximum capacity, with various combinations of two conduits operating at capacity, and with each conduit operating singly at capacity. For each combination, both high and low tailwater elevations were used. However, only the conditions when all three conduits were in operation have been tabulated since this operation seemed to result in the greatest pressure variation.

The results of the pressure tests are shown in Table 1 and on Figure 9. It is recommended that the minimum dynamic pressures be used to compute the pressure differential and forces on the sidewalls.

Discharge Capacity

The head-discharge capacity of the structure with the flow controlled by the radial gates was obtained as part of the model studies. The measurements were made with the three gates equally opened in 2-foot increments, commencing with the gates raised 1 foot. The calibration procedure was to set carefully the gate opening at the desired increment, increase the discharge through the model until the pressure head in the circular conduits 1-diameter upstream from the transition was equivalent to elevation 6430.0, and to measure the quantity of flow. This procedure was repeated for 10-foot increments in pressure head up to elevation 6500.0. Discharges versus total energy heads thus obtained for each gate opening are shown in Figure 16. To use these curves to determine prototype discharges, it will be necessary to install in each conduit of the prototype structure a pressure measuring piezometer in the same relative location described for the model, and a gage suitable for use by an operator should be provided. Once the relationship between headwater elevation and piezometer pressure head has been established as a result of prototype operation, the ordinate of Figure 16 may be changed to show the relationship of discharge to headwater elevations.

Powerplant Turnout

A 10-foot-diameter pressure conduit and Y-branch will be installed in the right outlet works tunnel, Figure 5. One leg of the Y-branch, the penstock lateral, will supply water to the powerplant. The other leg will lead to a slide-gate-controlled river outlet. Flow from the penstock outlet structure will discharge onto the outlet chute and into the stilling basin similar to flow from the existing horseshoe tunnel, Figure 5.

To investigate the flow conditions in the Y-branch and at the exit of the penstock outlet structure, the 10-foot-diameter pressure conduit, the Y-branch, the penstock outlet structure, and the 30° elbow and a short length of conduit downstream from the bend were installed in the model, Figure 17.

Three items of concern were studied: (1) The pressure conditions near the intersection lines of the penstock lateral, (2) the head loss

Table 1

COMPARISON OF DYNAMIC AND HYDROSTATIC PRESSURES
ON STILLING BASIN SIDEWALLS
Discharge = 18,700 cfs

Piezometer			Tailwater elevation 6404.6				Tailwater elevation 6400.8			
			Hydrostatic Pressure*		Dynamic Pressure*		Hydrostatic Pressure*		Dynamic Pressure*	
No.	Station	Elevation	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.
1	35+04.5	6395	11.5	3.5	14.1	3.8	9.0	0	9.2	0.8
2	35+17.0	6395	15.0	4.5	16.8	5.7	6.5	1.0	10.9	3.0
3	35+29.5	6395	15.0	5.0	11.6	5.2	11.5	1.0	8.7	0
4	34+92.0	6390	17.5	6.0	18.4	7.5	13.0	4.0	11.0	4.3
5	35+17.0	6390	20.0	9.5	20.5	10.4	11.5	6.0	21.7	9.4
6	35+42.0	6390	20.0	12.0	13.2	6.1	17.0	7.5	12.8	4.8
7	35+04.5	6385	21.5	13.5	19.5	11.1	19.0	10.0	14.0	5.6
8	35+17.0	6385	25.0	14.5	20.6	11.2	16.5	11.0	17.1	9.7
9	35+29.5	6385	25.0	15.0	21.3	12.3	21.5	11.0	15.0	6.4
10	34+92.0	6380	27.5	16.0	28.4	12.0	23.0	14.0	27.4	3.1
11	35+17.0	6380	30.0	19.5	32.8	18.3	21.5	16.0	29.9	17.5
12	35+42.0	6380	30.0	22.0	26.3	19.9	27.0	17.5	24.3	16.9
13	34+82.0	6375	32.0	20.0	35.8	24.8	28.0	17.0	34.0	16.4
14	34+92.0	6375	32.5	21.0	31.9	18.1	28.0	19.0	35.3	20.7
15	35+04.5	6375	31.5	23.5	31.9	18.1	39.0	20.0	33.9	11.7
16	34+82.0	6370	37.0	25.0	36.0	24.6	33.0	22.0	38.2	16.8
17	34+92.0	6370	37.5	26.0	34.2	20.8	33.0	24.0	35.6	14.2
18	35+04.5	6370	36.5	28.5	31.7	20.8	34.0	25.0	27.3	16.8
19	35+17.0	6370	40.0	29.5	39.1	24.0	31.5	26.0	36.2	16.9
20	35+42.0	6370	40.0	32.0	42.8	24.8	37.0	27.5	40.6	24.0
21	34+92.0	6360	47.5	36.0	50.8	30.0	43.0	32.0	49.1	28.6
22	35+17.0	6360	50.0	39.5	49.3	32.5	41.5	36.0	47.8	26.8

*Pressures are in feet of water above piezometer.

through the Y-branch and elbow, and (3) the dispersion of the flow downstream from the penstock outlet structure.

Preliminary Y-Branch

The preliminary Y-branch consisted of a 10-foot-diameter penstock lateral taking off to the right from the main conduit at an angle of 60°. At the end of the 60° turn the lateral was turned to the left by a 30° elbow, and then continued toward the powerplant in a straight line. Flow in the penstock is not expected to exceed 1,700 second-feet. However, it is possible that both the penstock outlet and the powerplant will operate simultaneously, but more generally one of them will operate singly. In any case, the total flow entering the Y-branch is not expected to exceed 4,734 second-feet.

Head Loss. The amount of head loss in the penstock lateral of the Y-branch was determined by subtracting the total head (velocity head plus pressure head) measured four pipe diameters downstream from the P. T. of the bend, from the total head measured one pipe diameter upstream from the Y-branch. A loss coefficient, K, was then determined by dividing this head loss by the velocity head of the flow entering the Y-branch. The loss coefficient was then related to the ratio of the penstock discharge to the total discharge entering the Y-branch. This relationship, shown on Figure 17, indicated that for the 60° turnout K will have a minimum value of about 0.49 when the discharge ratio is about 0.60. When the total flow is diverted through the penstock, K will equal about 0.70.

The combined loss coefficient, K, for the 60° turnout and 30° elbow varied between a minimum value of about 0.66 when the discharge ratio was 0.54 to a value of about 0.85 when the total flow was diverted through the penstock lateral.

Pressures. Average pressures, measured with water manometers, were obtained (1) with 1,200 and 4,734 second-feet passing through the penstock outlet structure and no flow through the penstock, and (2) with no flow through the outlet and with seven different discharges varying from 1,200 to 4,150 second-feet through the penstock. Piezometers were placed in critical low-pressure regions of the Y-branch, as shown on Figure 17.

The average pressure measurements indicated that when the total flow passed through the penstock outlet no subatmospheric pressures occurred in the Y-branch at any discharge. Also, when the quantity of flow diverted into the penstock was 2, 835 second-feet or less, the pressures were above atmospheric. However, with 4, 150 second-feet diverted into the penstock, all piezometers along the upstream intersection line between the penstock lateral and the main conduit indicated subatmospheric pressures near vapor pressure. Since the maximum flow through the penstock is limited to 1, 650 second-feet, preliminary pressure measurements indicated the preliminary Y-branch to be satisfactory. The average pressures obtained at the different discharges are shown on Figure 17.

Preliminary Penstock Outlet

The preliminary gate-control structure downstream from the Y-branch is a rectangular passage 8. 5 feet wide by 9. 0 feet high containing two high-pressure slide gates in tandem. The upstream gate is an emergency gate and was not reproduced in the model. The downstream slide gate was represented by a square-bottomed plate. In the preliminary design the structure was pointed directly downstream with the same centerline as the original horseshoe tunnel. The portal of the penstock river outlet structure is at Station 32+50, 21 feet further downstream than the portal of the horseshoe conduit, Figure 5.

The invert of the pressure conduit upstream from the penstock outlet structure is 2 feet above the chute floor. To lower the invert of the penstock outlet structure 2 feet and at the same time spread the jet emerging from the structure, the transition between the circular conduit and the rectangular penstock outlet was designed so that the gate section was tilted downward at a 5° angle.

The flow distribution on the chute and the stilling basin performance were investigated in the initial tests for the following operating conditions. Discharges of 1, 200, 2, 400, and the maximum capacity of 4, 734 second-feet through the penstock outlet structure with: (1) no flow through the two horseshoe tunnels, (2) maximum capacity through the center horseshoe tunnel, and (3) maximum capacity through the center and left horseshoe tunnels.

When the penstock outlet was operating by itself, the flow spread to the left a small amount but the right edge of the flow impinged against the chute wall forming a fin of water. For the 1, 200- and 2, 400-second-foot discharges the fin was not objectionable but for the maximum discharge the fin overtopped the training wall (Figure 18).

When either one of the two horseshoe conduits was operating in conjunction with the 1, 200- and 2, 400-second-foot discharges from the penstock outlet, the flow seemed insignificant; however, a large fin formed where the flow from the conduits intersected the penstock outlet flow (Figure 19). The flow from the conduits rose over the top of the penstock outlet flow, causing a very rough water surface on the chute and an unsymmetrical jump in the stilling basin.

These preliminary tests indicated that design modifications of the penstock outlet structure were necessary to improve the flow distribution on the chute and to reduce the height of waves and fins along the right training wall.

First Modification. The penstock outlet structure was modified by turning the gate section 5° to the left (Figure 20A).

This change was accomplished in the model by placing a wedge between the circular-to-rectangular transition and the gate section.

With the penstock outlet structure operating by itself, the flow was well distributed across the chute and stilling basin for all test discharges (Figure 21).

The height of the fin on the right wall was greatly reduced and even at the maximum discharge it rose only about halfway to the top of the wall. With either one or both horseshoe conduits operating in conjunction with the outlet, the flows impinged causing a high fin to form (Figure 22). Although this fin was unsightly and caused some disturbance on the chute, it was well contained within the sidewalls and caused no undue splashing outside the confines of the basin. The jump was unsymmetrical when either the center or left conduit was operating with the spillway outlet structure but the poor flow distribution was fairly well smoothed out by the time it left the stilling basin.

Although this modified spillway outlet structure provided satisfactory operation it was felt that more even flow distribution might be obtained if the spillway outlet structure discharged horizontally onto the chute. In addition, the 5° turn to the left involved some structural design difficulties that should be avoided; therefore, the testing was continued with the structure discharging horizontally onto the chute and with less deflection to the left.

Second Modification. The circular-to-rectangular transition was modified so that the gate section was placed horizontal and pointed directly downstream (Figure 20B).

The second modification was only a slight improvement over the preliminary design. A fin rose high on the right wall and large waves overtopped the wall at the maximum discharge (Figure 23A).

Third Modification. For the third modification, the horizontal gate section was turned 2° toward the left (Figure 24A).

With the outlet structure turned 2° to the left, the fin still formed on the right wall but it had moved about 15 feet further downstream and was about 3 feet lower than that observed with the second modification. However, at the maximum discharge, the flow entering the stilling basin still created large waves that frequently overtopped the basin walls although not to the extent observed with the previous designs (Figure 23B).

Fourth Modification. The amount of turn-in was increased from 2° to 3° for the fourth modification (Figure 24B).

With the 3° turn-in the height and location of the fin was about the same as it was with the 2° turn-in. However, the flow spread sufficiently across the chute and forced the tailwater downstream, preventing the formation of large waves that overtopped the right wall in the earlier designs. Some splashing occasionally went over the wall but it was a comparatively minor amount (Figure 23C). There was some flow concentration along the right side of the stilling basin but no eddies or swirling action formed in the basin and the flow leaving the basin was very smooth.

Fifth Modification. The penstock outlet structure was turned 5° toward the left for the fifth modification (Figure 25).

With the 5° turn-in, the jet was well distributed across the basin at the maximum discharge of 4,734 second-feet and the fin along the right wall was reduced to negligible proportions (Figure 26). The wave action caused by the jet striking the tailwater was entirely eliminated.

The jet was less evenly distributed across the basin for the lower discharges of 1,200 and 2,400 second-feet. However, the flow concentrated near the center of the basin and any resulting asymmetry in the hydraulic jump was confined to the upper end of the basin (Figure 27).

When the penstock outlet structure was operated in conjunction with either one or both horseshoe conduits, a large fin formed where the penstock outlet flow joined the flow from the horseshoe conduits. However, the fin did not affect the flow distribution across the chute or the action of the hydraulic jump, and the splashing caused by the fin falling back on the high velocity jet was well contained within the sidewalls of the basin.

Recommendations for Penstock Outlet Structure and Y-branch

Three of the six penstock outlet structure arrangements that were investigated in the model provided acceptable flow conditions on the chute and in the stilling basin. In preferential order these were:

(1) with the structure horizontal and turned 5° toward the left (Fifth Modification); (2) with the structure tilted downward 5° and turned 5° toward the left (First Modification); and (3) with the structure horizontal and turned 3° toward the left (Fourth Modification). The selection of the arrangement for prototype construction was governed by the limited length available to install the circular-to-rectangular transition upstream from the penstock outlet structure and yet turn the gate section to the left and lower the flow to the chute floor. In addition, sufficient space was needed between the downstream gate frame and the pier to provide a structurally sound design.

The studies also indicated that the head loss in the penstock lateral might be reduced. This could be accomplished by streamlining the 60° Y-branch and replacing the 30° elbow with one of less curvature.

Recommended Y-Branch

The Y-branch was streamlined by substituting a series of mitered turns rather than an abrupt turn of 60° in the turnout, and the 30° elbow was replaced with a 20° mitered elbow, Figure 28.

Head Loss. Head loss coefficients for the streamlined Y-branch were obtained in the same manner as for the preliminary Y-branch. The coefficients, related to the ratio of the penstock discharge to the total discharge entering the Y-branch, are shown on Figure 28. The tests showed that the minimum loss coefficient, K, will be about 0.29 when the discharge ratio is about 0.57. When the total flow was diverted through the penstock, K, will equal about 0.47. The streamlined Y-branch reduced the head loss by about 45 percent when all the flow was diverted through the penstock and as much as 56 percent when part of the flow was diverted.

Pressures. Nine piezometers were placed in the recommended Y-branch. Three near the crotch and six on the upstream side of the penstock lateral, Figure 28. The pressures were obtained by means of water manometers connected to the piezometers, which gave an average pressure and did not show the maximum or minimum fluctuations.

The piezometers on the upstream side of the penstock lateral indicated that the pressures would be near atmospheric or above for all of the expected operating conditions. The pressures measured by the piezometers in the crotch were above atmospheric as long as there was flow in the penstock lateral, Runs 2 and 3 in the table of Figure 28. However, when there was no flow through the penstock and maximum discharge through the outlet, two of the three piezometers in the crotch indicated subatmospheric pressures of about 18.0 feet of water, Run 1 in the table in Figure 28.

When the control gate at the downstream end of the penstock outlet was closed 0.2 foot, the crotch pressures were raised to about 5.0 feet of water below atmospheric, and with the gate closed 0.4 foot the pressures were near atmospheric. (Runs 4 and 5 in the table in Figure 28.)

These changes in gate opening reduced the maximum discharge by 300 and 500 second-feet, respectively.

Instantaneous dynamic pressure measurements were obtained at the crotch piezometers and at the first piezometer on the upstream side of the penstock lateral. The measurements were made with the gate full open and closed 0.2 foot. The average instantaneous pressures compared favorably with the values obtained from the water manometer measurements. However, the extremes of the instantaneous fluctuations indicated that at Piezometers No. 8 and 9 severe subatmospheric pressures in the vapor pressure range would occur a large percentage of the time, even with the gate closed 0.2 foot. These pressures have been tabulated in Table 2.

It was felt that a smaller gate operated at 100 percent opening would raise the subatmospheric pressures in the Y-branch to an acceptable degree and would reduce the discharge less than restricting the opening of a larger gate. Therefore, it was decided to reduce the height of the gate by 6 inches.

With the 8-foot 6-inch square gate fully open the maximum discharge capacity was about 4,400 second-feet, or 334 second-feet less than the capacity of the larger gate. The piezometers in the crotch indicated average water manometer pressures equivalent to about 8 feet of water below atmospheric, (Run 5 in the table on Figure 28).

Instantaneous dynamic pressure measurements were made with the small gate full open and closed 0.25, 0.50, 1.0, and 2.0 feet. There was no flow through the penstock turnout. In general, the average instantaneous pressures compared favorably with the values obtained from the water manometer measurements. However, the minimum instantaneous readings indicated that severe subatmospheric pressures at Piezometers No. 8 and 9 would still occur a large percentage of the time with the gate full open and closed 0.25 foot. With the gate closed 0.5 foot, instantaneous minimum pressures equivalent to about 18 to 19 feet of water below atmospheric were recorded. With the 1.0-foot closure, the minimum pressures were only about 3 to 4 feet of water below atmospheric and with the 3-foot closure they were about 26 to 30 feet of water above atmospheric.

Since it was believed that pressures of the magnitude of 17 feet of water below atmospheric could be tolerated for short periods, it was recommended that a stop be placed on the gate stem so that the gate could not be opened more than 8.0 feet. (Assuming the 8.5- by 8.5-foot gate will be installed.) With the gate closed 0.5 foot the discharge capacity was reduced by 834 second-feet, to 3,900 second-feet.

Instantaneous maximum dynamic pressures were also obtained during these tests. The maximum pressures varied from 10 feet of water higher than the minimum pressure at Piezometer No. 1, to 86 feet of water higher than the minimum pressure at Piezometer No. 8. Thus, the fluctuations on the oscillograph recordings indicated that there could be an instantaneous change from the maximum to the minimum pressure equivalent to about 86 feet of water. The average frequency of fluctuations was about 5 cycles per second. The maximum and minimum pressures for the various operating conditions are shown in Table 2.

Recommended Penstock Outlet

On the basis of the laboratory investigations and other criteria, it was decided to tilt the penstock outlet structure downward 5° 30' and turn it to the left 4° 0', Figure 28. The emergency gate also was removed from the design which shortened the length of the gate structure by 8 feet.

Pressures. The turning and tilting of the penstock outlet structure were accomplished by a compound miter cut in the circular pipe upstream from the transition. This method of construction resulted in some abrupt changes in alinement that could conceivably cause low-pressure conditions along the flow boundaries. To investigate this possibility, 12 piezometers were placed in critical areas in the transition and gate structure, Figure 28.

Manometer pressure measurements indicated that no objectionable pressures occurred in the transition at any discharge, with either the 8.5- by 9.0-foot gate or the 8.5- by 8.5-foot gate. With the maximum discharge (using the large gate) of 4,734 second-feet going through the penstock outlet and no flow through the penstock lateral, the pressures varied from 1.5 feet of water below atmospheric to 19.2 feet of water above atmospheric. With divided flow of 1,600 second-feet through the penstock lateral and 3,134 second-feet through the penstock outlet (using the large gate), the pressures varied from 31.4 to 44.1 feet of water above atmospheric. The pressures with the maximum discharge through the small gate varied from

Table 2

INSTANTANEOUS PRESSURES IN Y-BRANCH CROTCH

*Piezometer No.		8.5- by 9.0-foot gate			8.5- by 8.5-foot gate			
		Full open	Closed 0.2 ft	Full open	Closed 0.25 ft	Closed 0.5 ft	Closed 1.0 ft	Closed 2.0 ft
1	Maximum	18.9	28.8	23.8	33.6	38.6	53.4	68.3
	Minimum	-5.8	0.1	4.1	16.3	26.3	36.2	58.4
7	Maximum	82.1	88.4	79.1	80.7	82.2	63.7	94.5
	Minimum	14.3	20.4	17.4	41.2	39.0	51.3	69.8
8	Maximum	33.8	56.0	36.3	48.2	41.2	56.0	68.4
	Minimum	VP**	-30.4	VP	VP	-18.1	-3.3	26.4
9	Maximum	32.5	37.4	39.9	42.3	49.7	59.6	69.5
	Minimum	VP**	VP	VP	VP	-19.4	-4.6	30.0

*For location of piezometers see Figure 28.
**Indicates vapor pressure.

2.0 feet of water below atmospheric to 18.9 feet of water above atmospheric. The pressure readings for these, and other flow conditions have been tabulated on Figure 28.

Walls Downstream from Gate. Flow from the penstock outlet gate discharges onto the stilling basin chute. The gate structure had been tipped downward and turned to the left to distribute the flow across the chute so that it would enter the stilling basin at uniform depth. With the length of the gate section reduced by 8 feet, the structure terminated upstream from the end of the center piers, Figure 29. The flow emerging from the gate was not in contact with the center pier or the right wall of the chute, and consequently, the flow spread and impinged against the pier and the wall, causing large fins to rise along these surfaces. On the left side, the fin was not harmful but was unsightly and would be a source of excessive spray. On the right side the fin frequently overtopped the wall and could possibly damage the fill on the outside of the wall. To eliminate these fins, it was decided to install training walls downstream from the gate structure.

Several different combinations of walls were investigated; these included diverging and straight walls on the left side extending to the end of the pier, and diverging, converging, and straight walls of various lengths on the right side. The best performance on the left side was obtained with a wall that diverged 5° from the side of the gate. This wall prevented the fin from forming and permitted the jet to diverge and spread across the chute. However, it was feared that the abrupt 5° deflection at the end of the gate structure might cause subatmospheric pressures to occur downstream from the change in alinement so the divergence was accomplished by means of a 47.3-foot-radius curve, Figure 29. The performance with the curved wall was very good, Figure 30, and it was accepted for prototype installation.

Six piezometers were placed along the wall to measure the pressures. One row of three piezometers was 1 foot above the floor, and a second row of three piezometers was 3 feet above the floor, Figure 29. The pressures were determined for 100, 50, and 25 percent gate openings at maximum reservoir elevation and are tabulated in Figure 29. With the 100 percent gate opening, the lowest observed pressure was about 5.1 feet of water below atmospheric, recorded at the downstream piezometer in the top row. The lowest pressures with the 50 percent gate opening, was about 6 feet of water below atmospheric, recorded at the downstream piezometer in the bottom row. With the 25 percent gate opening, the lowest observed pressure was about 1.0 feet of water below atmospheric.

Several wall arrangements were tested on the right side of the penstock outlet. A wall about 35 feet long that diverged from the edge of the gate structure to the sidewall of the chute reduced the fin to negligible proportions. A shorter diverging wall had very little effect on the jet. A straight wall, or a continuation of the gate structure sidewall, only served to move the fin farther downstream on the chute. A wall at the end of the gate structure that converged 5° into the flow practically eliminated the fin. To make a symmetrical structure, the convergence was accomplished by means of a curved wall approximately parallel to the left wall, Figure 29. The curved converging wall reduced the fin to negligible proportions, and the flow entering the stilling basin was equally distributed, Figure 30.

When the outlet was operated in conjunction with either the center or left conduit or with both conduits, the flow distribution was satisfactory on the chute and in the stilling basin, Figure 31.

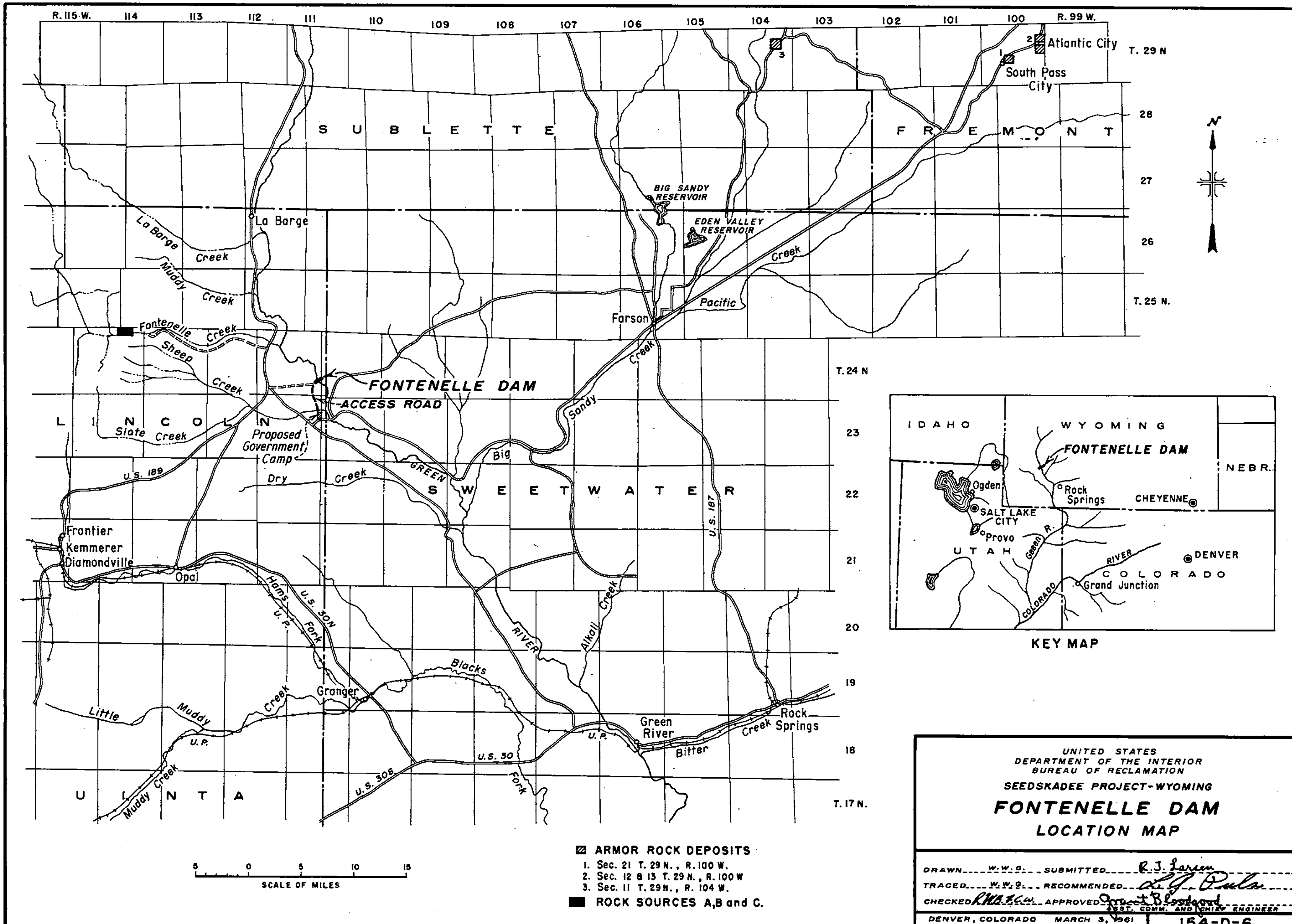


FIGURE 1
REPORT HYD 487

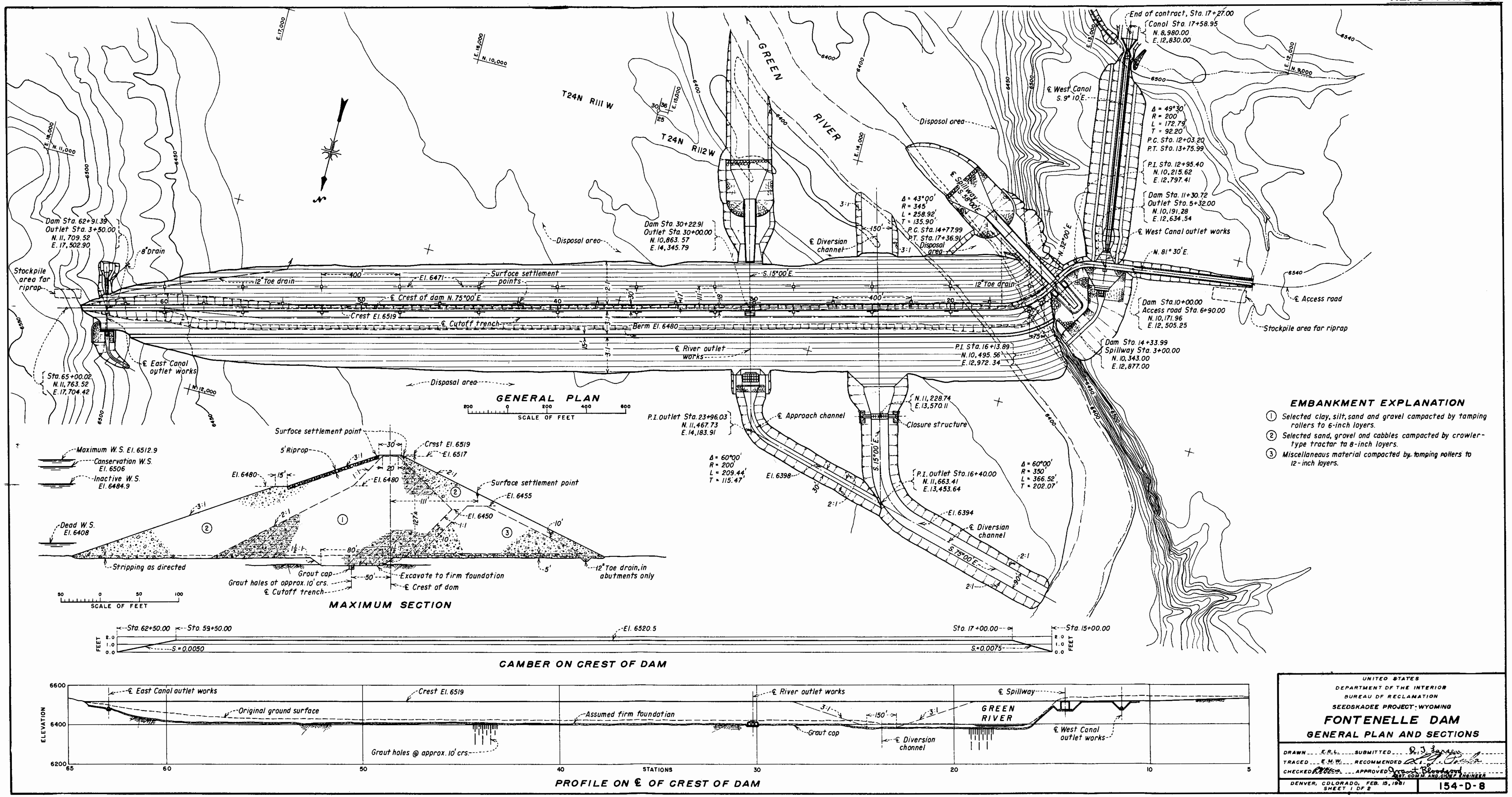
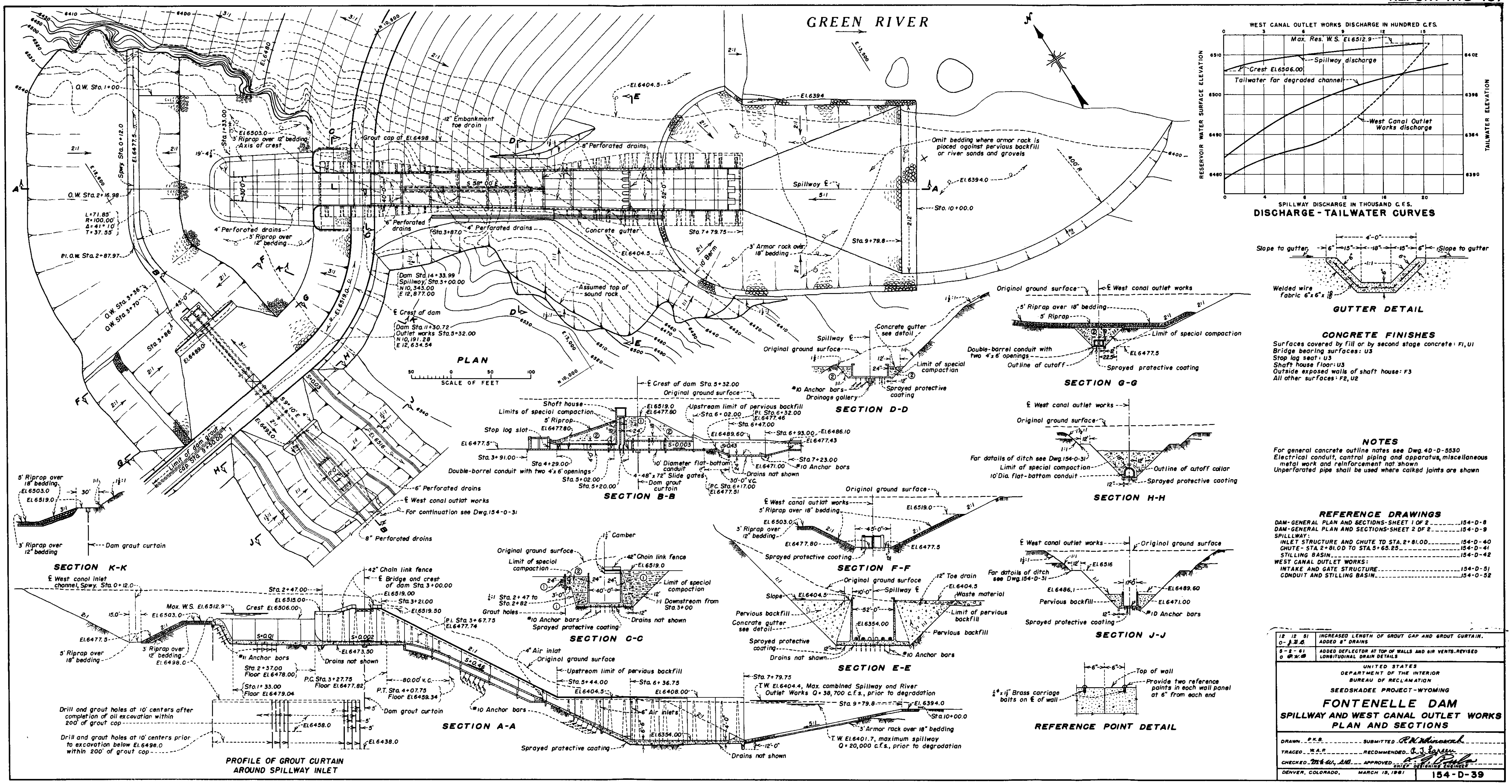
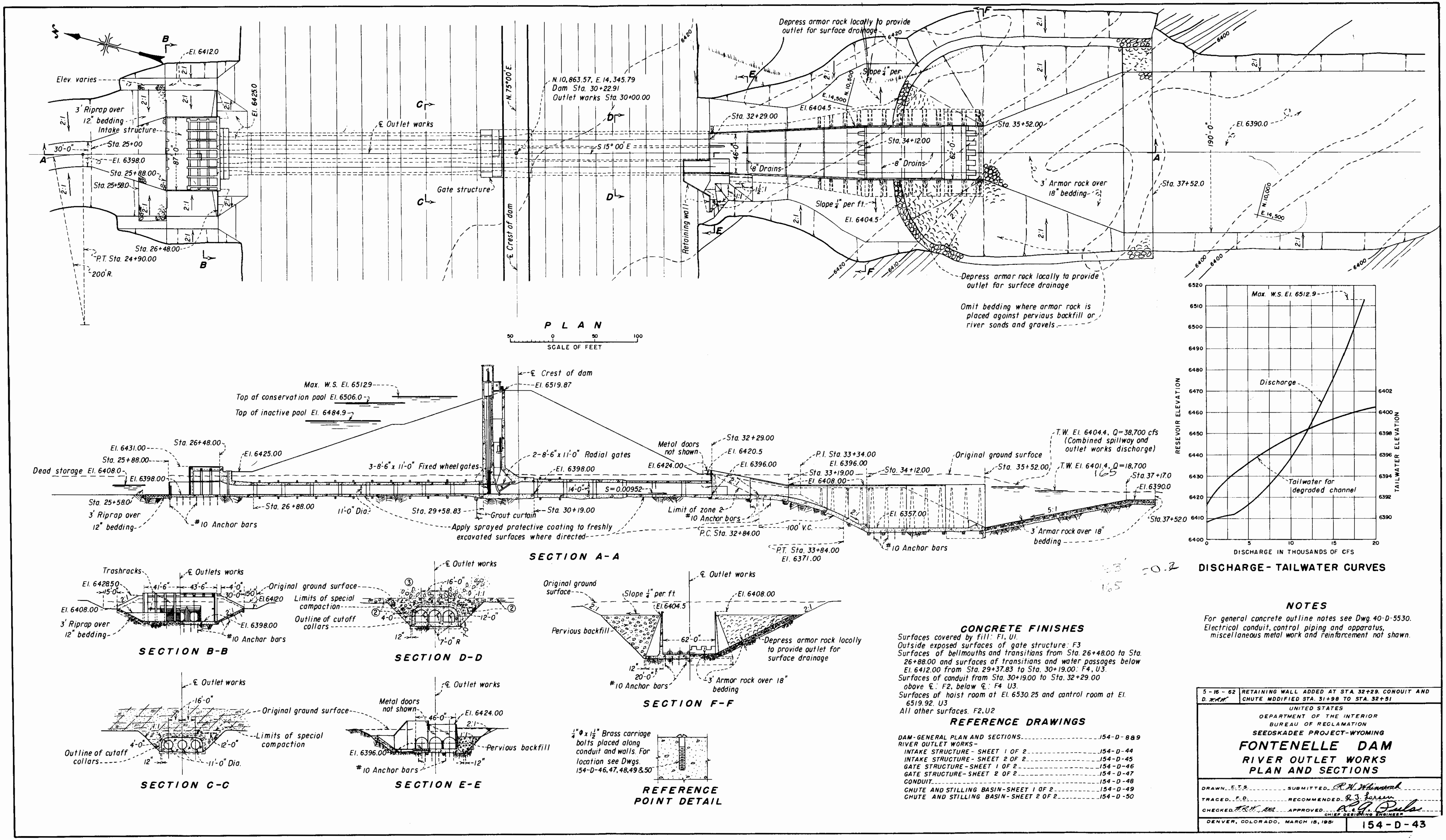
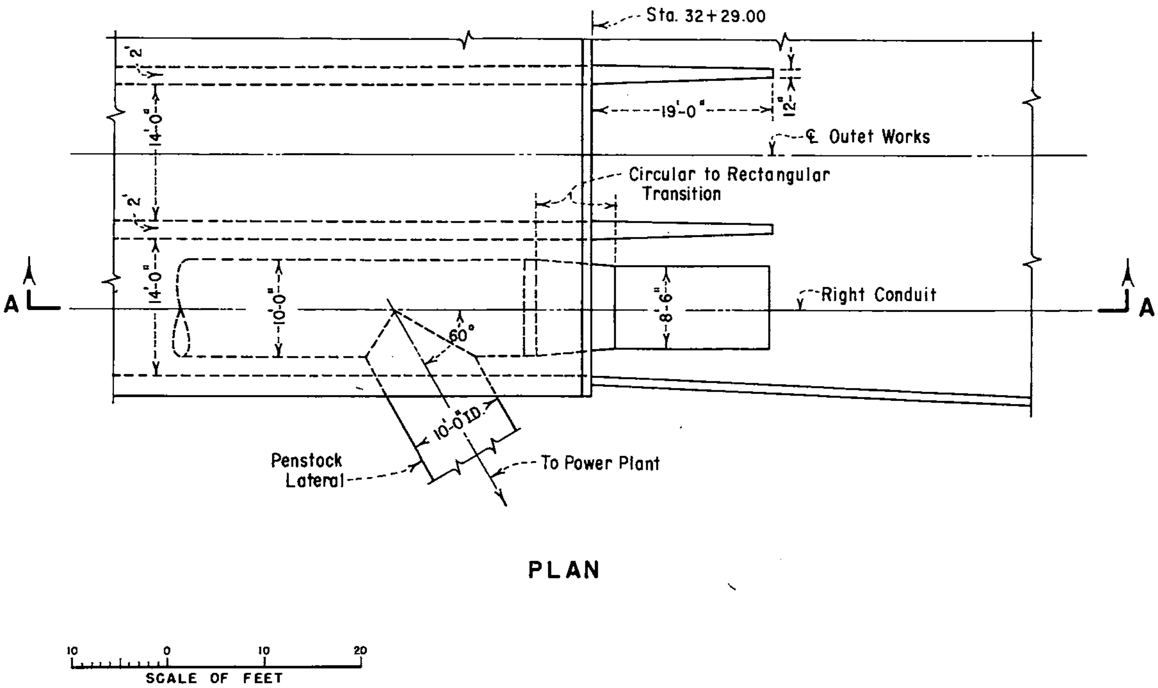

GENERAL PLAN
SCALE OF FEET
GREEN RIVER
T24N R111W
T24N R112W
Disposal area
Stockpile area for riprap
12" Toe drain
Surface settlement points
Crest of dam N. 75°00' E.
Crest El. 6519
Cutoff trench
Berm El. 6480
River outlet works
East Canal outlet works
West Canal outlet works
Approach channel
Closure structure
Diversion channel
Spillway
Access road
Dam Sta. 62+91.39
Outlet Sta. 3+50.00
N. 11,709.52
E. 17,502.90
Sta. 65+00.02
N. 11,763.52
E. 17,704.42
Dam Sta. 30+22.91
Outlet Sta. 30+00.00
N. 10,863.57
E. 14,345.79
End of contract, Sta. 17+27.00
Canal Sta. 17+58.95
N. 8,980.00
E. 12,830.00
Dam Sta. 11+30.72
Outlet Sta. 5+32.00
N. 10,191.28
E. 12,634.54
Dam Sta. 10+00.00
Access road Sta. 6+90.00
N. 10,171.96
E. 12,505.25
Dam Sta. 14+33.99
Spillway Sta. 3+00.00
N. 10,343.00
E. 12,877.00
P.I. Sta. 16+13.89
N. 10,495.56
E. 12,972.34
P.I. Outlet Sta. 23+96.03
N. 11,467.73
E. 14,183.91
P.I. Outlet Sta. 16+40.00
N. 11,663.41
E. 13,453.64
EMBANKMENT EXPLANATION
(1) Selected clay, silt, sand and gravel compacted by tamping rollers to 6-inch layers.
(2) Selected sand, gravel and cobbles compacted by crawler-type tractor to 8-inch layers.
(3) Miscellaneous material compacted by tamping rollers to 12-inch layers.
MAXIMUM SECTION
Maximum W.S. El. 6512.9
Conservation W.S. El. 6506
Inactive W.S. El. 6484.9
Dead W.S. El. 6408
Surface settlement point
5' Riprap
Crest El. 6519
Stripping as directed
Grout cap
Grout holes at approx. 10' crs.
Cutoff trench
Excavate to firm foundation
Crest of dam
12" Toe drain, in abutments only
SCALE OF FEET
CAMBER ON CREST OF DAM
Sta. 62+50.00
Sta. 59+50.00
El. 6520.5
Sta. 17+00.00
Sta. 15+00.00
S=0.0050
S=0.0075
PROFILE ON ℄ OF CREST OF DAM
Original ground surface
Assumed firm foundation
Grout holes @ approx. 10' crs.
Grout cap
ELEVATION
STATIONS
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SEEDSKADEE PROJECT-WYOMING
FONTENELLE DAM
GENERAL PLAN AND SECTIONS
DENVER, COLORADO, FEB. 15, 1961
SHEET 1 OF 2
154-D-8

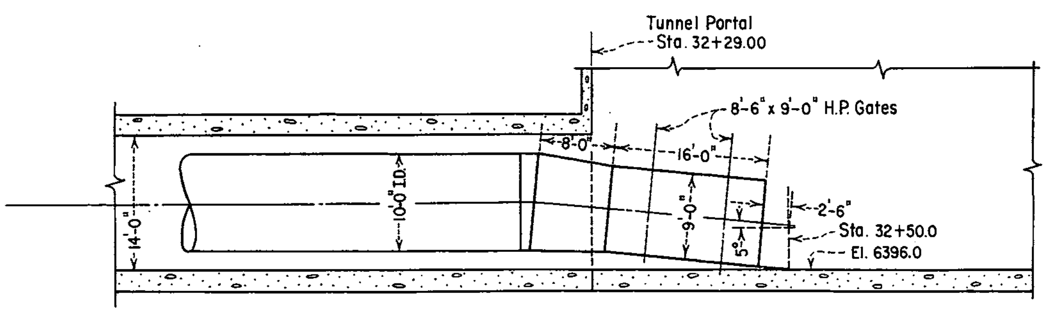
FIGURE 3
REPORT HYD 487




PLAN
SCALE OF FEET
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F
REFERENCE POINT DETAIL
℄ Outlet works
Intake structure
Gate structure
Retaining wall
℄ Crest of dam
N. 75°00' E.
S 15° 00' E
N. 10,863.57, E. 14,345.79
Dam Sta. 30+22.91
Outlet works Sta. 30+00.00
Sta. 25+00
Sta. 25+88.00
Sta. 25+58.0
Sta. 26+48.00
P.T. Sta. 24+90.00
200' R.
Sta. 32+29.00
Sta. 34+12.00
Sta. 35+52.00
Sta. 37+52.0
El. 6412.0
El. 6425.0
El. 6398.0
El. 6404.5
El. 6390.0
Elev. varies
3' Riprap over 12" bedding
8" Drains
3' Armor rock over 18" bedding
Slope 1/4" per ft.
Depress armor rock locally to provide outlet for surface drainage
Omit bedding where armor rock is placed against pervious backfill or river sands and gravels
Max. W.S. El. 6512.9
Top of conservation pool El. 6506.0
Top of inactive pool El. 6484.9
Dead storage El. 6408.0
El. 6519.87
3-8'-6" x 11'-0" Fixed wheel gates
2-8'-6" x 11'-0" Radial gates
Metal doors not shown
S = 0.00952
11'-0" Dia.
Grout curtain
#10 Anchor bars
Limit of zone 2
Apply sprayed protective coating to freshly excavated surfaces where directed
Original ground surface
T.W. El. 6404.4, Q=38,700 cfs (Combined spillway and outlet works discharge)
T.W. El. 6401.4, Q=18,700
P.I. Sta. 33+34.00 El. 6396.00
P.C. Sta. 32+84.00
P.T. Sta. 33+84.00 El. 6371.00
100' V.C.
El. 6357.00
Trashracks
Limits of special compaction
Outline of cutoff collars
Pervious backfill
1/4"Φ x 1 1/2" Brass carriage bolts placed along conduit and walls. For location see Dwgs. 154-D-46, 47, 48, 49 & 50
DISCHARGE - TAILWATER CURVES
RESERVOIR ELEVATION
TAILWATER ELEVATION
DISCHARGE IN THOUSANDS OF CFS
Discharge
Tailwater for degraded channel
CONCRETE FINISHES
Surfaces covered by fill: F1, U1.
Outside exposed surfaces of gate structure: F3
Surfaces of bellmouths and transitions from Sta. 26+48.00 to Sta. 26+88.00 and surfaces of transitions and water passages below El. 6412.00 from Sta. 29+37.83 to Sta. 30+19.00: F4, U3.
Surfaces of conduit from Sta. 30+19.00 to Sta. 32+29.00 above ℄: F2, below ℄: F4 U3.
Surfaces of hoist room at El. 6530.25 and control room at El. 6519.92. U3
All other surfaces. F2, U2
REFERENCE DRAWINGS
DAM-GENERAL PLAN AND SECTIONS 154-D-889
RIVER OUTLET WORKS-
INTAKE STRUCTURE - SHEET 1 OF 2 154-D-44
INTAKE STRUCTURE - SHEET 2 OF 2 154-D-45
GATE STRUCTURE - SHEET 1 OF 2 154-D-46
GATE STRUCTURE - SHEET 2 OF 2 154-D-47
CONDUIT 154-D-48
CHUTE AND STILLING BASIN - SHEET 1 OF 2 154-D-49
CHUTE AND STILLING BASIN - SHEET 2 OF 2 154-D-50
NOTES
For general concrete outline notes see Dwg. 40-D-5530.
Electrical conduit, control piping and apparatus, miscellaneous metal work and reinforcement not shown.
5-16-62 RETAINING WALL ADDED AT STA. 32+29. CONDUIT AND CHUTE MODIFIED STA. 31+98 TO STA. 32+51
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SEEDSKADEE PROJECT-WYOMING
FONTENELLE DAM
RIVER OUTLET WORKS
PLAN AND SECTIONS
DRAWN E.T.S.
TRACED F.D.
SUBMITTED
RECOMMENDED
APPROVED
CHIEF DESIGNING ENGINEER
DENVER, COLORADO, MARCH 18, 1961
154-D-43

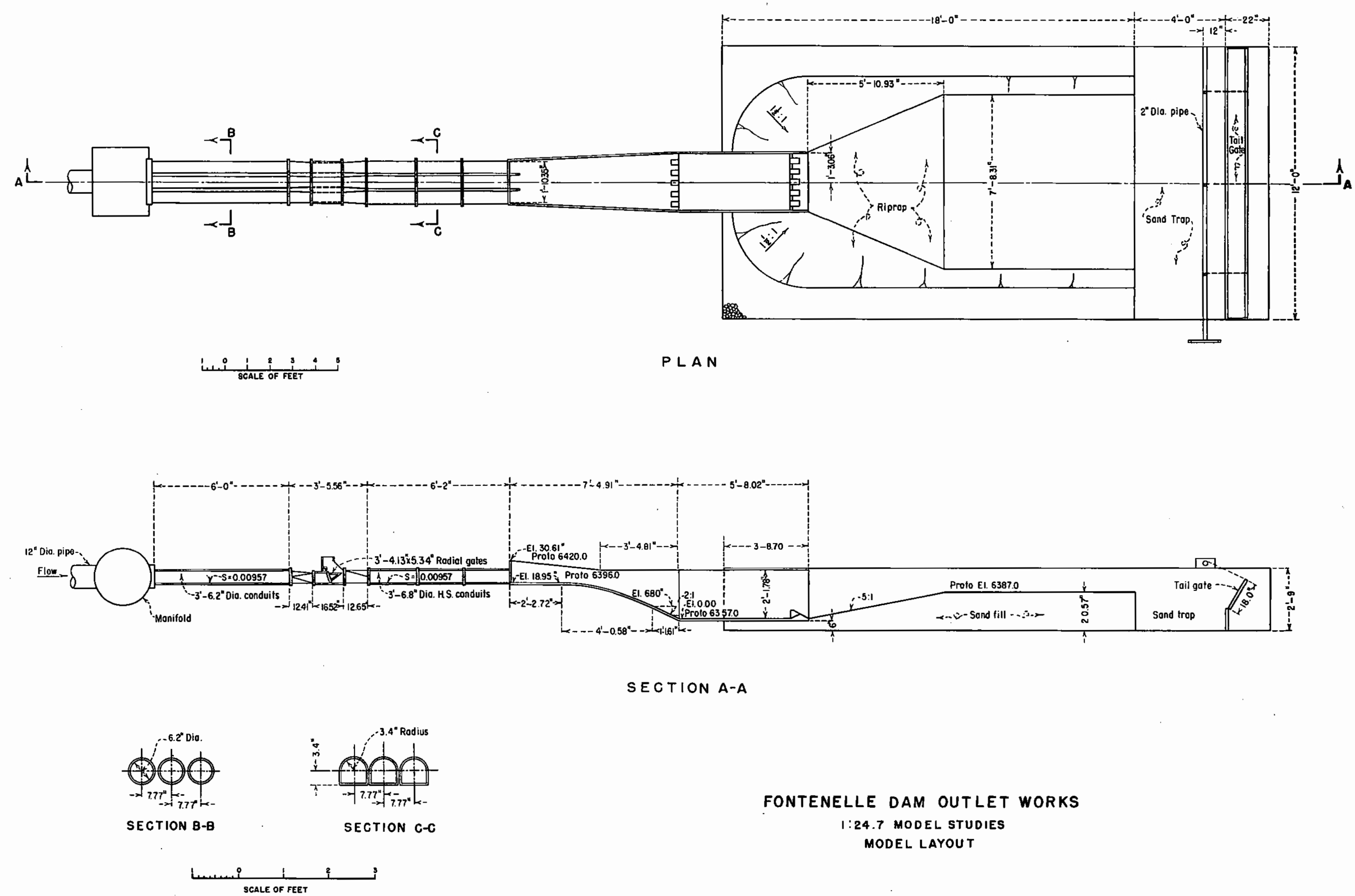


PLAN



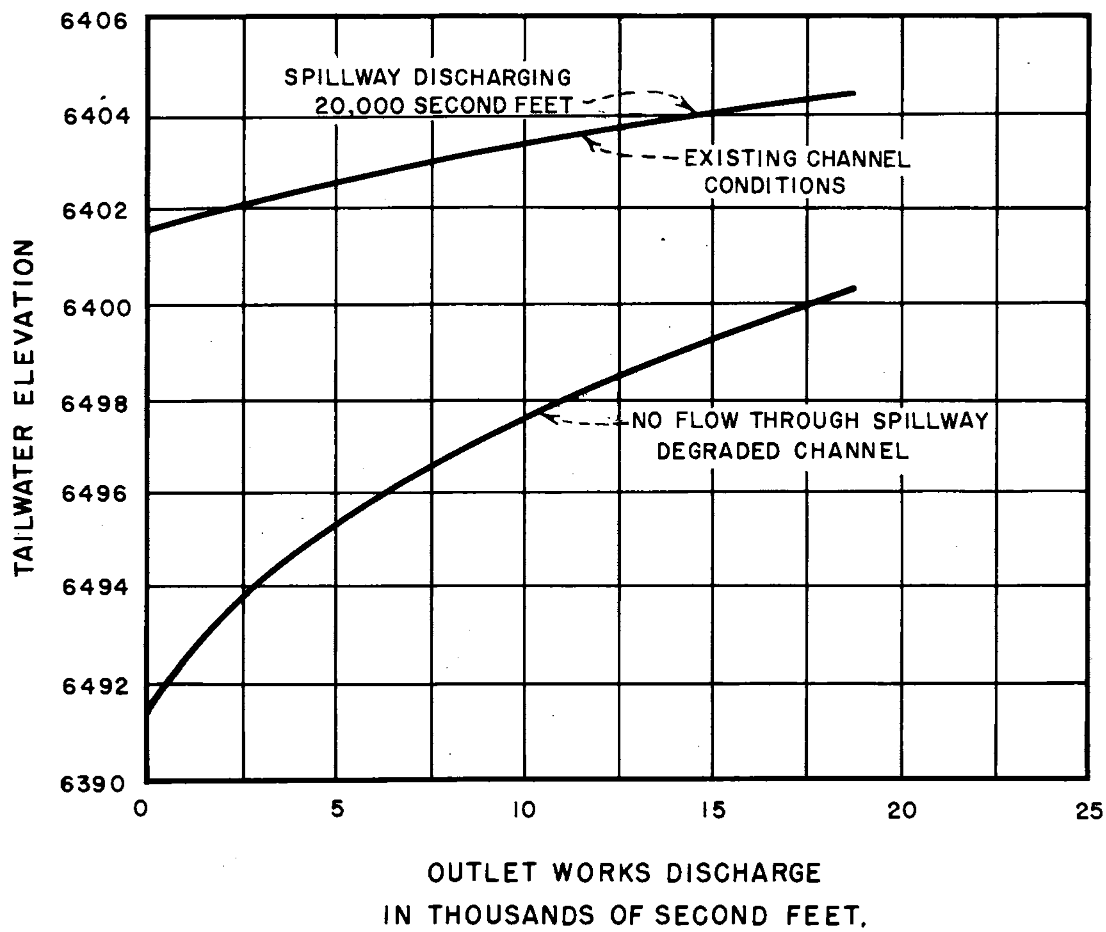
SECTION A-A

FONTENELLE DAM OUTLET WORKS
1:24.7 MODEL STUDIES
Y-BRANCH AND OUTLET STRUCTURE
PRELIMINARY DESIGN



FONTENELLE DAM OUTLET WORKS
1:24.7 MODEL STUDIES
MODEL LAYOUT

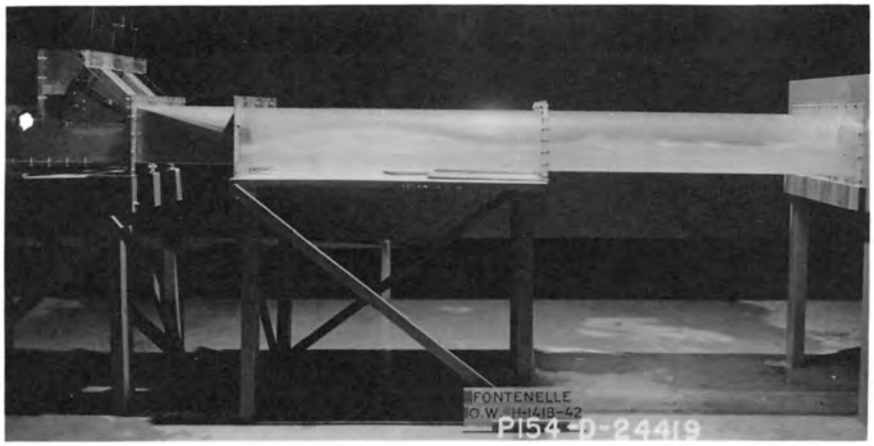
FIGURE 6
REPORT HYD. 481



FONTENELLE DAM OUTLET WORKS

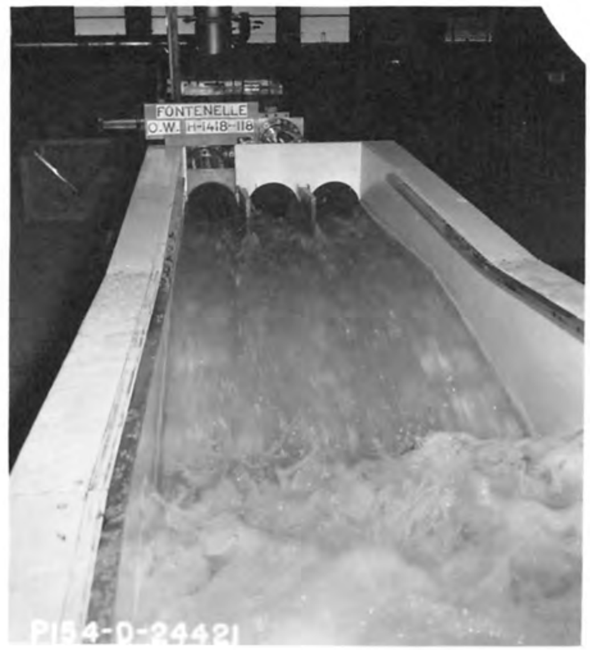
1:24.7 MODEL STUDIES

TAILWATER ELEVATION CURVES



A. Flow in tunnel. Radial gates open 9 feet. Fin along side wall was highest at this opening.

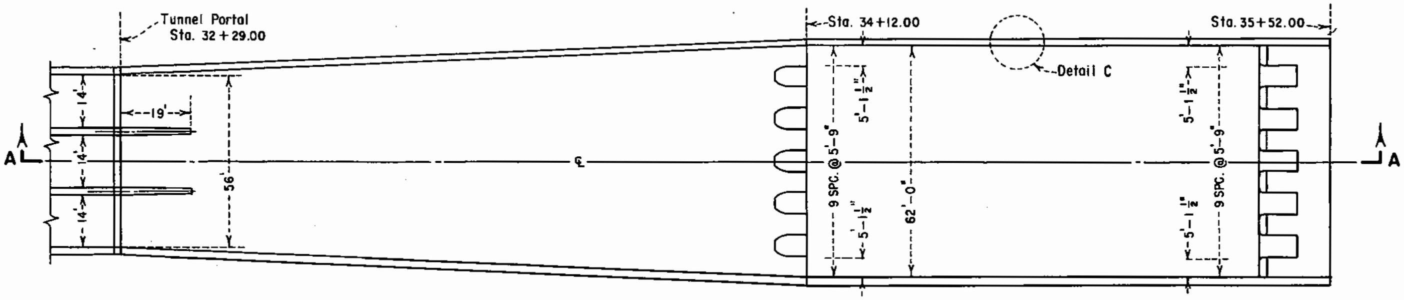
B. No piers at tunnel portals.



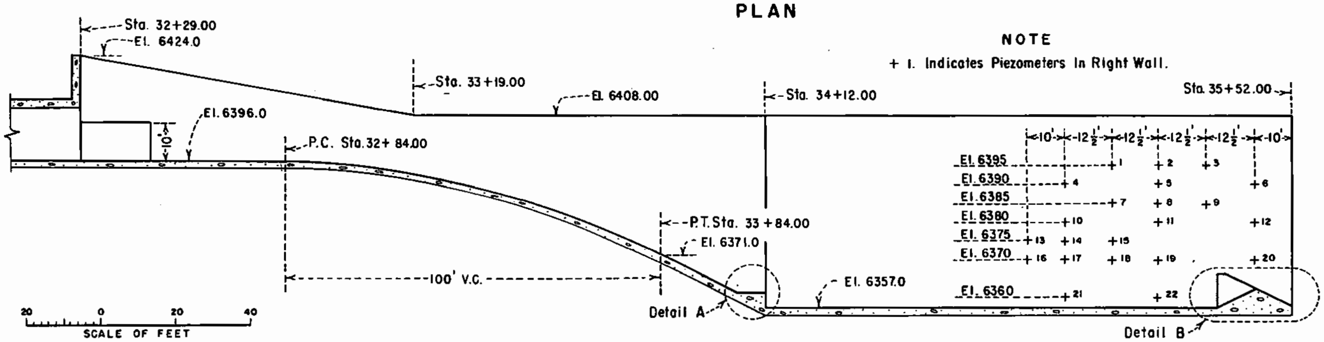
C. Tapered piers installed at tunnel portals.

FONTENELLE DAM OUTLET WORKS

1:24.7 Model Studies
Discharge = 6,233 cfs per Tunnel
Flow in Tunnel and at Tunnel Portal



PLAN



NOTE

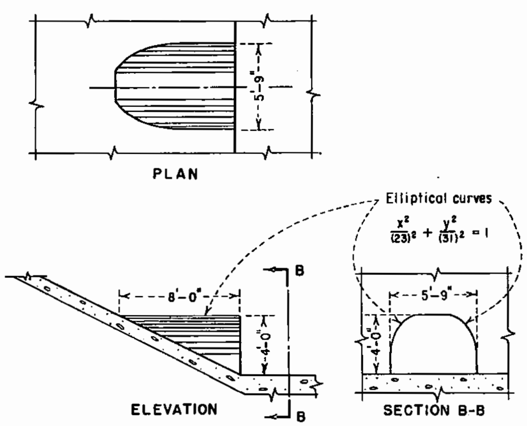
+ 1. Indicates Piezometers In Right Wall.

SECTION A-A

PRESSURES ON RIGHT WALL

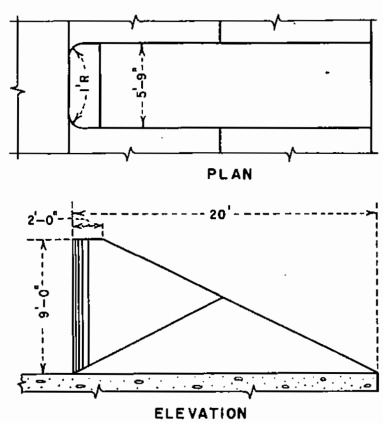
IN FEET OF WATER

PIEZ.	CENTER AND RIGHT Q = 12,467 cfs.				THREE CONDUITS Q = 18,700 cfs			
	T.W. 6398.5		T.W. 6403.8		T.W. 6400.8		T.W. 6404.6	
	Max.	Min.	Max.	Min.	Max.	Min	Max.	Min.
1	7.7	1.5	11.9	6.7	9.2	0.8	14.1	3.8
2	8.4	0.5	13.4	5.3	10.9	3.0	16.8	5.7
3	5.3	-0.7	11.2	5.8	8.7	0	11.6	5.2
4	10.7	3.8	16.4	9.8	11.0	-4.3	18.4	7.5
5	20.5	10.8	27.9	10.6	21.7	9.4	20.5	10.4
6	10.3	6.1	13.2	7.1	12.8	4.8	13.2	6.1
7	13.1	6.6	18.8	12.6	14.0	5.6	19.5	11.1
8	15.4	6.0	20.3	12.2	17.1	9.7	20.6	11.2
9	15.3	8.6	18.0	10.6	15.0	6.4	21.3	12.3
10	23.2	15.0	26.4	17.7	27.4	3.1	28.4	12.0
11	25.9	15.6	29.4	21.0	29.9	17.5	32.8	18.3
12	22.1	15.9	24.1	20.4	24.3	16.9	26.3	19.9
13	29.0	15.4	34.0	22.4	34.0	16.4	31.5	18.7
14	31.3	20.2	38.7	22.7	35.3	20.7	35.8	24.8
15	28.9	16.1	31.6	22.8	33.9	11.7	31.9	18.1
16	31.5	19.4	36.5	28.8	38.2	16.8	36.0	24.6
17	33.9	16.4	35.9	22.8	29.7	14.2	34.2	20.8
18	32.2	18.6	34.4	25.8	27.3	16.8	31.7	20.8
19	35.7	22.1	36.9	26.5	36.2	16.9	39.1	24.0
20	33.9	22.1	38.9	27.0	40.6	24.0	42.8	24.8
21	46.8	25.6	48.3	35.7	49.1	28.6	50.8	30.0
22	41.3	32.6	44.5	37.8	47.8	26.8	49.3	32.5



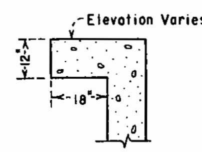
DETAIL A

CHUTE BLOCKS



DETAIL B

END SILL



DETAIL C

Coping strip placed on top of training walls between stations 32 + 74.00 and 35 + 05.33

FONTENELLE DAM OUTLET WORKS

1:24.7 MODEL STUDIES

STILLING BASIN DETAILS

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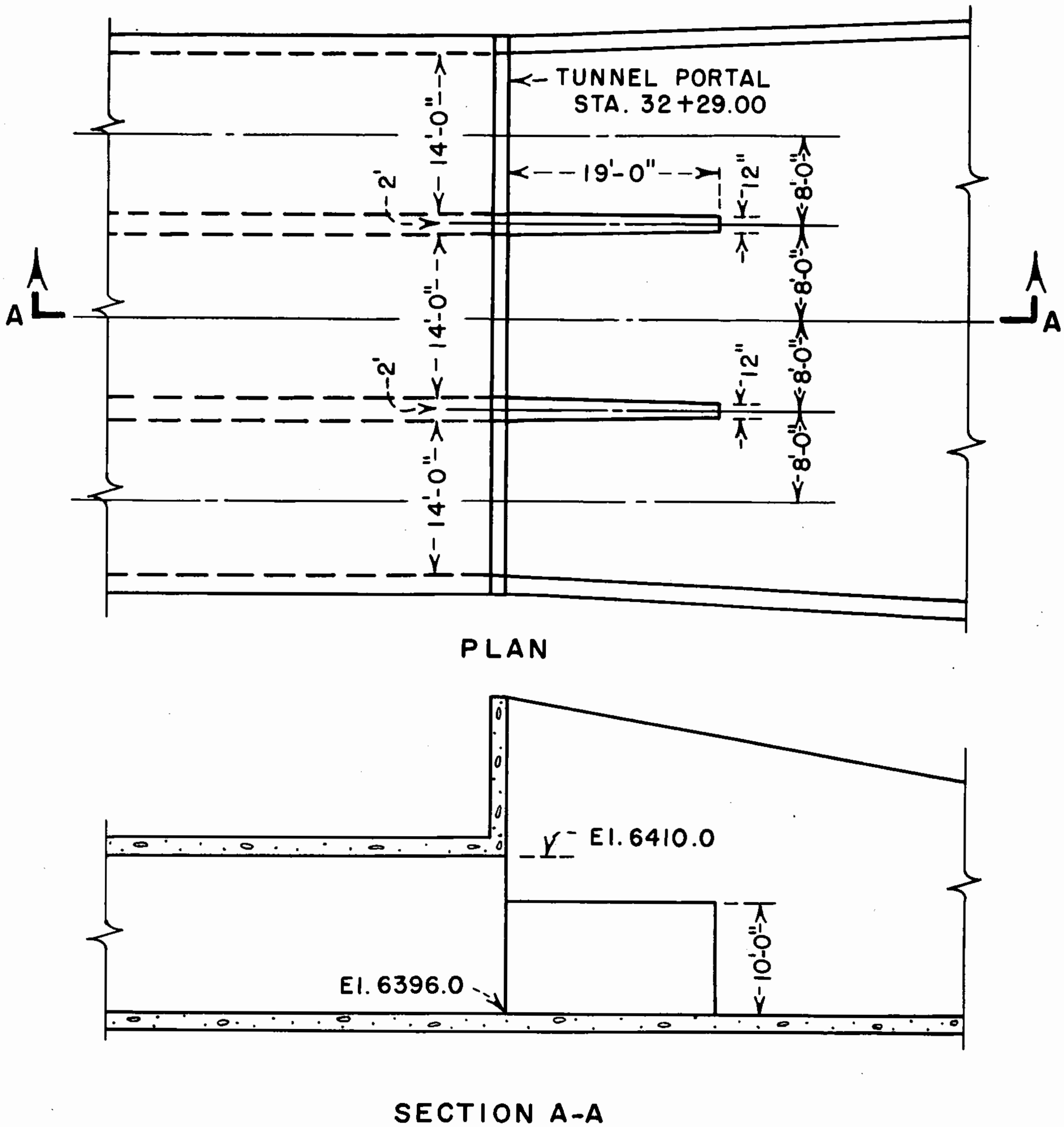
FIGURE 9
REPORT HYD. 487



FONTENELLE DAM OUTLET WORKS

1:24.7 MODEL STUDIES

PIERS AT TUNNEL PORTALS

Report Hyd. 487
Figure 11

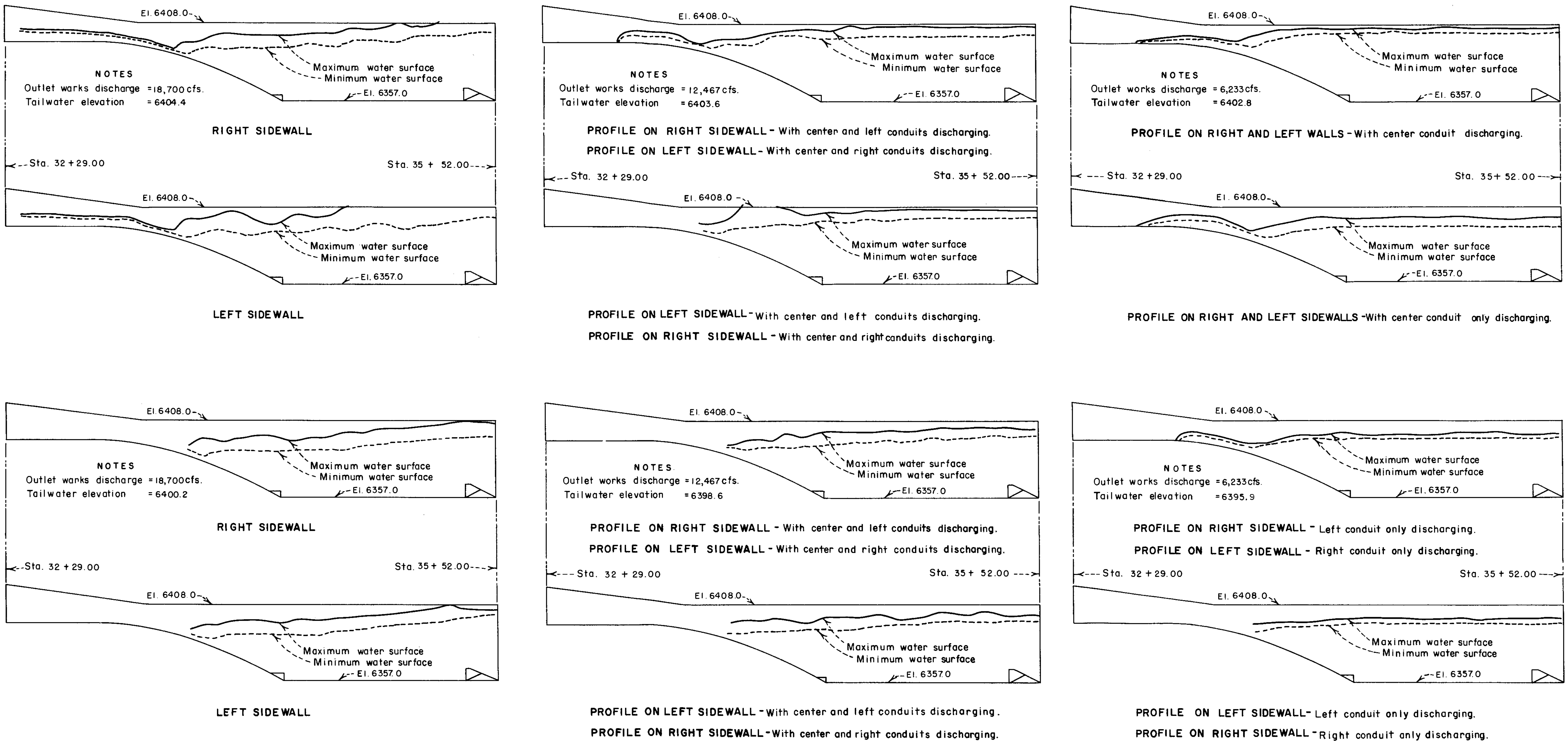
Left tunnel only operating
Discharge = 6,233 cfs

Center tunnel only operating
Discharge = 6,233 cfs

Center and left tunnels operating
Discharge = 12,467 cfs

FONTENELLE DAM OUTLET WORKS

1:24.7 Model Studies
Unsymmetrical operation with
piers installed at tunnel portals



FONTENELLE DAM OUTLET WORKS
1:24.7 MODEL STUDIES
WATER SURFACE PROFILES

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Tailwater Elevation = 6404.3

Tailwater Elevation = 6400.4

FONTENELLE DAM OUTLET WORKS

1:24.7 Model Studies
Flow in Stilling Basin
Discharge = 18,700 cfs

Figure 14
Report Hyd. 487

Center Conduit
Discharging 6, 233 cfs
Tailwater Elevation = 6395. 9

Left Conduit
Discharging 6, 233 cfs
Tailwater Elevation = 6395. 9

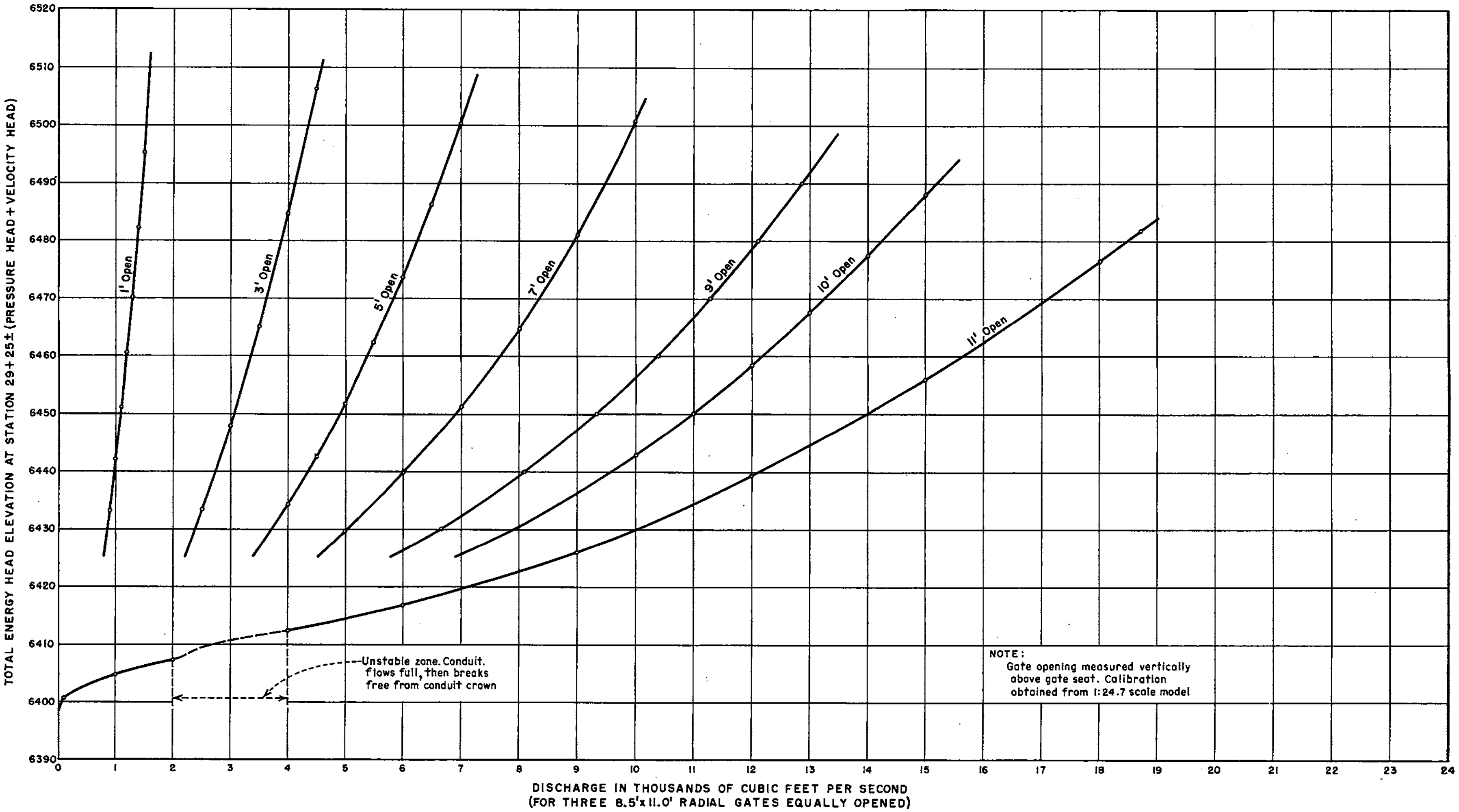
Center and Left Conduits
Discharging 12, 467 cfs
Tailwater Elevation = 6398. 5

FONTENELLE DAM OUTLET WORKS

1:24. 7 Model Studies
Basin Performance with Unsymmetrical Operation

FONTENELLE DAM OUTLET WORKS

1:24.7 Model Studies
Basin Performance with Left Conduit Only
Operating at 6,233 cfs
High Tailwater Elevation = 6402.8



FONTENELLE DAM OUTLET WORKS
DISCHARGE CAPACITY

639

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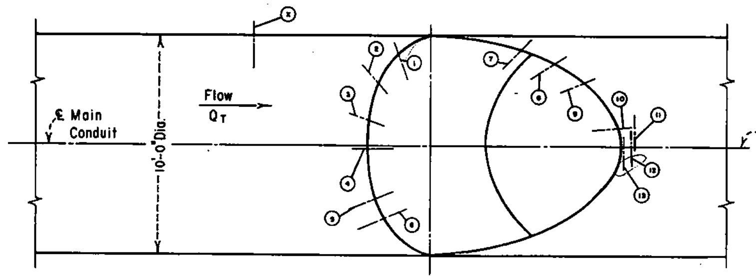
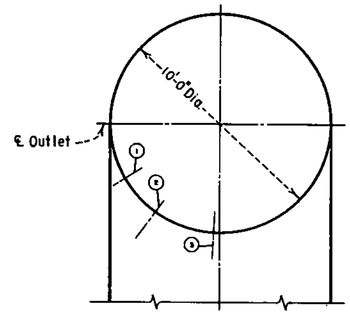
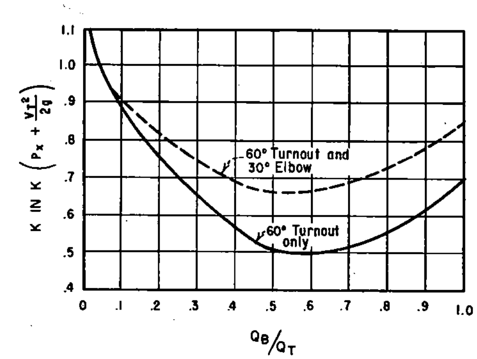


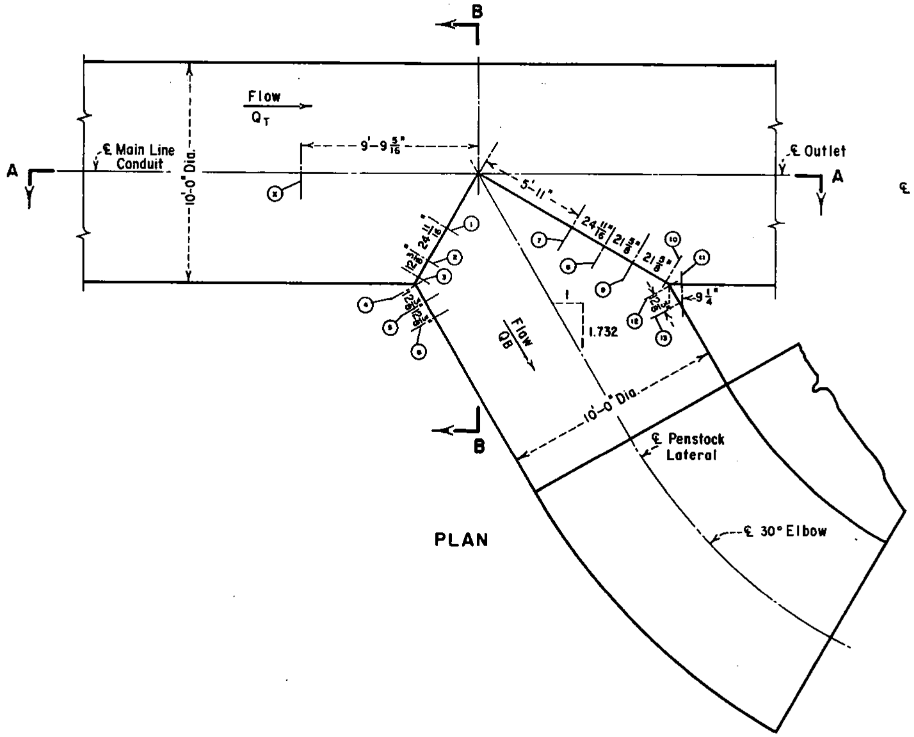
SECTION A–A



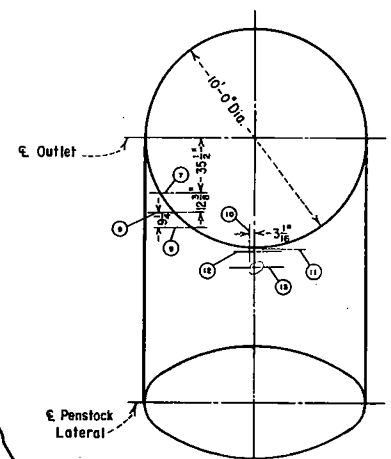
SECTION B–B



HEAD LOSS IN Y—BRANCH



PLAN



END VIEW

PRESSURE IN FEET OF WATER

PIEZOMETER NUMBER	PENSTOCK				BY-PASS	
	1350 cfs.	1650cfs.	2835cfs.	4150cfs	1200cfs.	4734cfs.
X	102.5	97.8	74.4	34.1	101.0	30.9
1	81.0	79.3	32.1	V.P.*	68.1	16.1
2	89.2	81.5	36.6	V.P.	75.3	17.8
3	90.4	82.7	38.0	V.P.	80.2	19.0
4	91.1	83.2	38.8	V.P.	82.5	19.5
5	90.6	83.0	37.8	V.P.	82.5	19.5
6	90.6	82.7	37.5	V.P.	81.7	19.3
7	92.4	87.7	58.3	8.9	60.5	14.3
8	94.6	90.4	66.0	26.4	64.6	15.3
9	97.8	93.4	76.6	50.1	68.0	16.1
10	101.3	93.8	86.0	68.7	78.2	18.5
11	101.3	98.6	87.5	71.4	80.3	20.3
12	90.4	83.7	39.3	V.P.	86.3	19.0
13	89.9	82.0	35.8	V.P.	80.3	19.0

* DENOTES VAPOR PRESSURE

FONTENELLE DAM OUTLET WORKS
1:24.7 MODEL STUDIES
PRESSURE AND HEAD LOSS IN PRELIMINARY Y-BRANCH

FIGURE 17.
REPORT HYD. 487

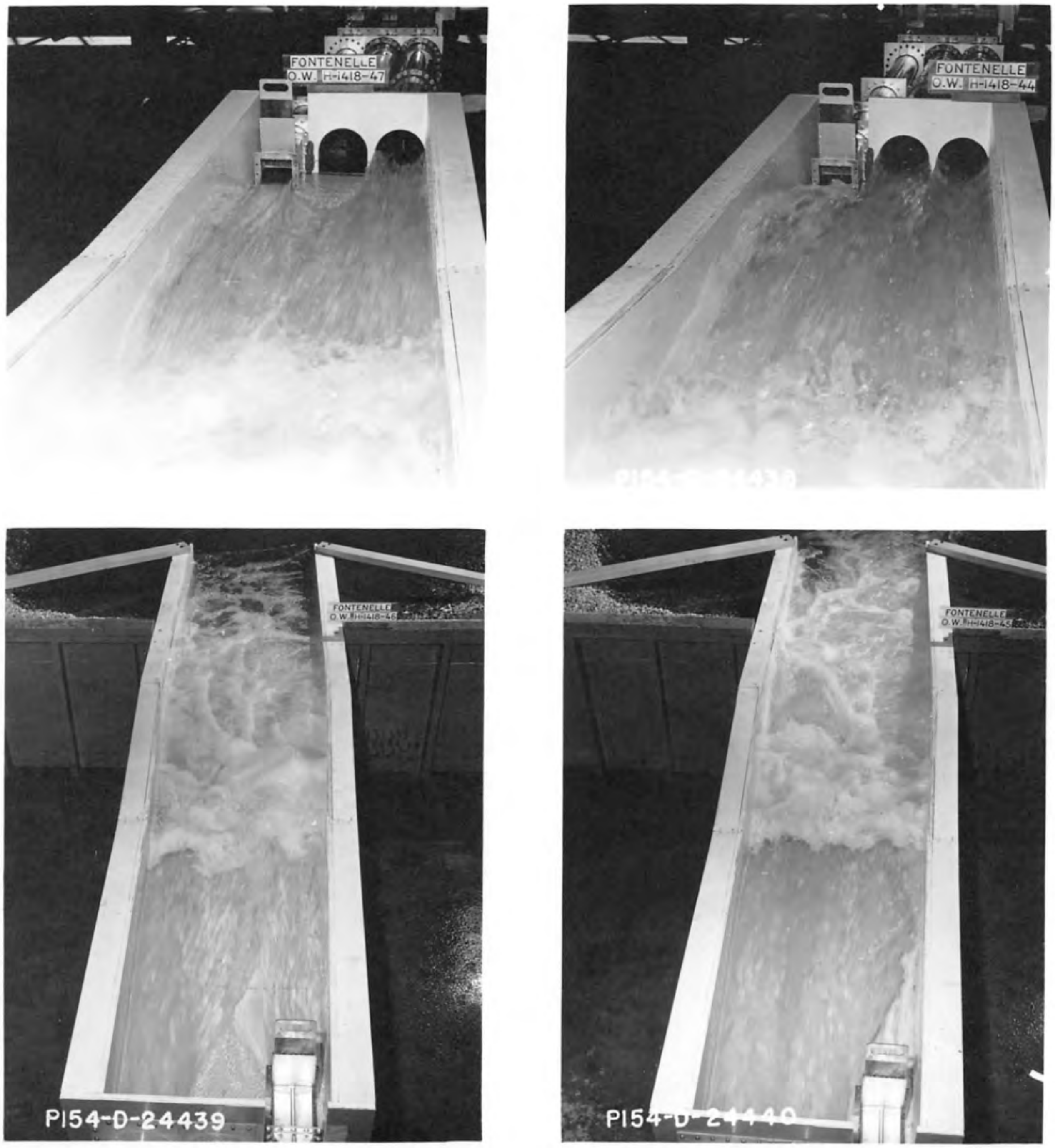
Discharge = 1,200 second-feet
Tailwater Elevation = 6401.9

Discharge = 2,400 second-feet
Tailwater Elevation = 6402.1

Discharge = 4,734 second-feet
Tailwater Elevation = 6402.3

FONTENELLE DAM OUTLET WORKS

1:24.7 Model Studies
Preliminary Basin Operation
Flow Through 8.5- x 9.0-Foot Penstock Outlet

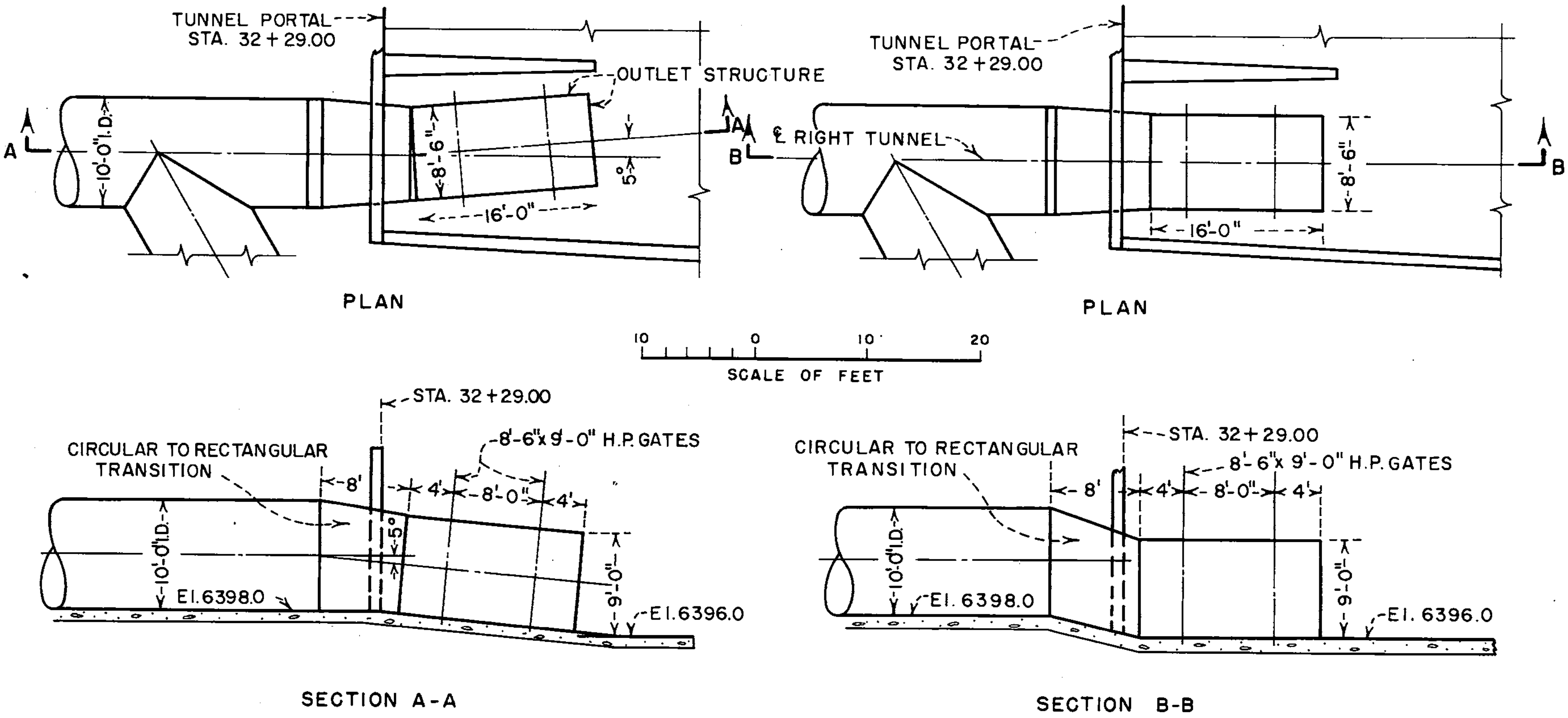


Penstock Outlet Discharge = 2,400 cfs
Left Conduit Discharge = 6,233 cfs
Center Conduit Closed
Tailwater Elevation = 6402.8

Penstock Outlet Discharge = 2,400 cfs
Center and Left Conduits
Discharge = 12,467 cfs
Tailwater Elevation = 6403.6

FONTENELLE DAM OUTLET WORKS

1:24.7 Model Studies
Basin Operation
Combined Flows Through Preliminary Penstock Outlet
and Horseshoe Conduit



A. FIRST MODIFICATION

B. SECOND MODIFICATION

FONTENELLE DAM OUTLET WORKS

1:24.7 MODEL STUDIES

OUTLET STRUCTURE MODIFICATIONS

FIGURE 20.
REPORT HYD. 487

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Discharge = 1, 200 cfs
Tailwater Elevation = 6401. 9

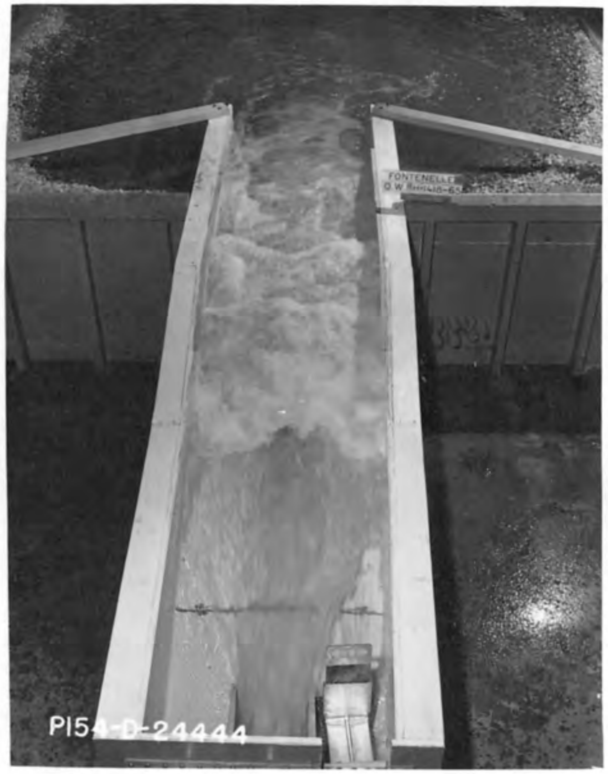
Discharge = 2, 400 cfs
Tailwater Elevation = 6402. 1

Discharge = 4, 734 cfs
Tailwater Elevation = 6402. 3

FONTENELLE DAM OUTLET WORKS

1:24. 7 Model Studies
Basin Operation
Penstock Outlet Turned Left 5° - Tipped Down 5°
(First Modification)

Penstock Outlet Discharge = 2,400 cfs
Left Conduit Discharge = 6,233 cfs
Center Conduit Closed
Tailwater Elevation = 6402.8



Penstock Outlet Discharge = 2,400 cfs
Left Conduit Closed
Center Conduit Discharge = 6,233 cfs
Tailwater Elevation = 6402.8

Penstock Outlet Discharge = 2,400 cfs
Center and Left Conduits Open
Discharge = 12,467 cfs
Tailwater Elevation = 6403.6

FONTENELLE DAM OUTLET WORKS

1:24.7 Model Studies
Basin Operation Combined Flows
Penstock Outlet Turned Left 5° - Tipped Down 5°
(First Modification)

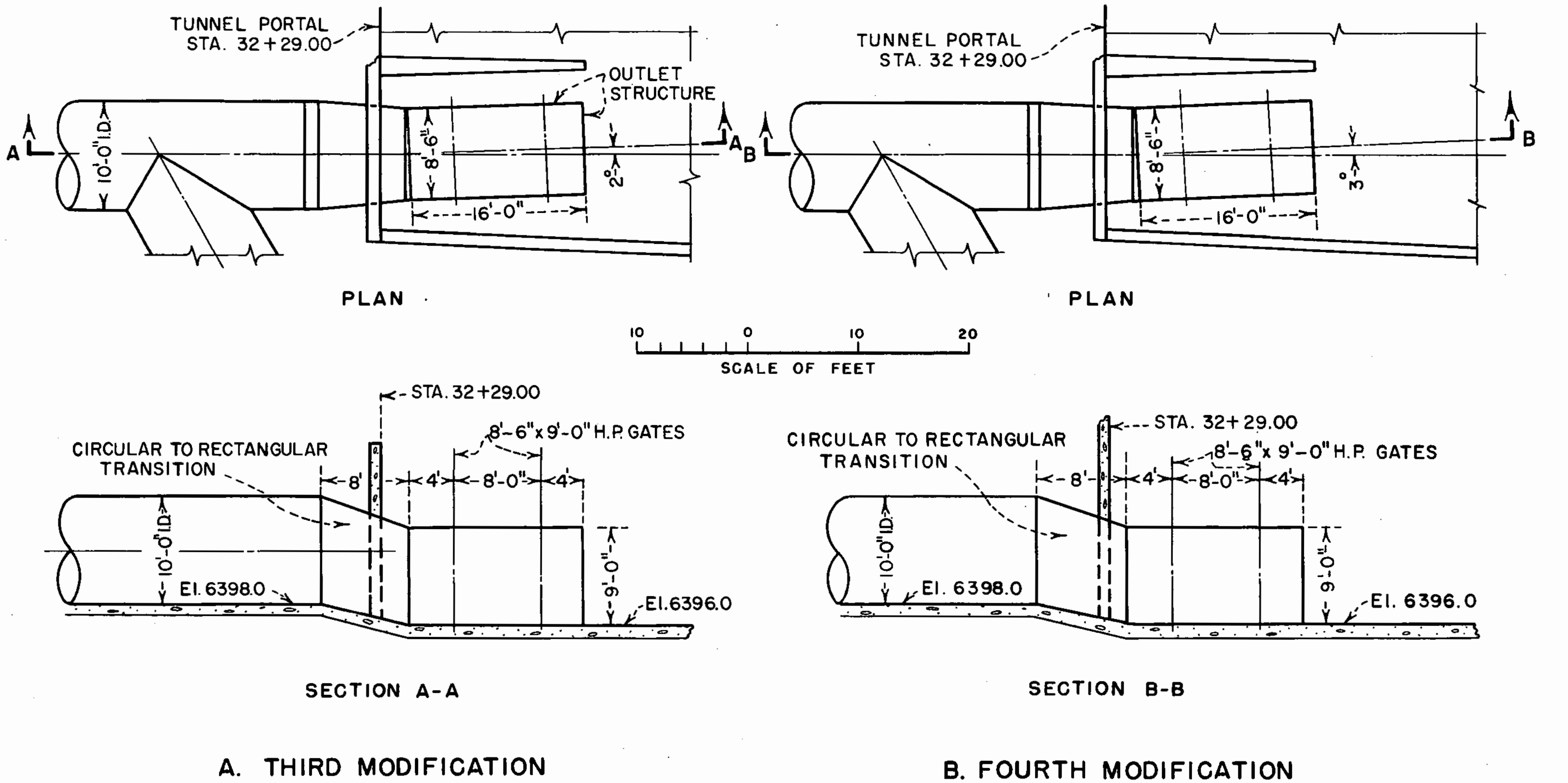
A. Penstock Outlet Horizontal and Pointed Downstream (Second Modification)
Discharge = 4,734 cfs
Tailwater Elevation = 6402.3

B. Penstock Outlet Horizontal and Turned 2° to the Left (Third Modification)
Discharge = 4,734 cfs
Tailwater Elevation = 6402.3

C. Penstock Outlet Horizontal and Turned 3° to the Left (Fourth Modification)
Discharge = 4,734 cfs
Tailwater Elevation = 6402.3

FONTENELLE DAM OUTLET WORKS

1:24.7 Model Studies
Basin Operation with
Different Modification of Penstock Outlet

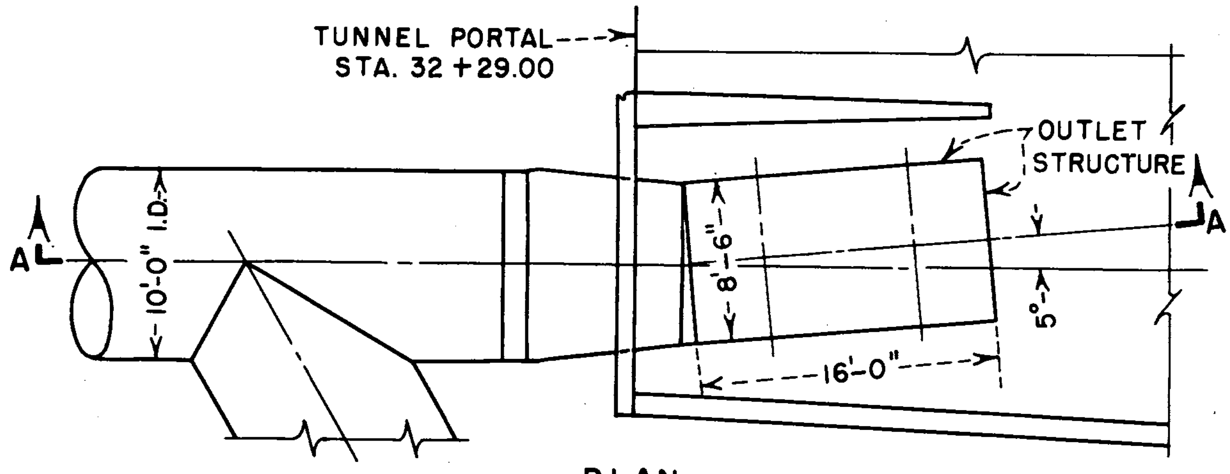


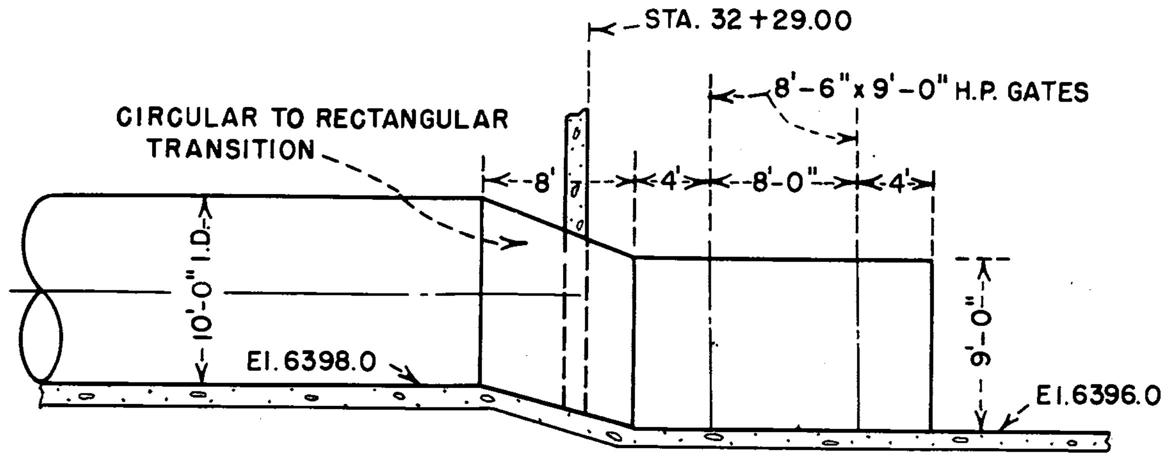
FONTENELLE DAM OUTLET WORKS

1:24.7 MODEL STUDIES

OUTLET STRUCTURE MODIFICATIONS

FIGURE 24.
REPORT HYD. 487

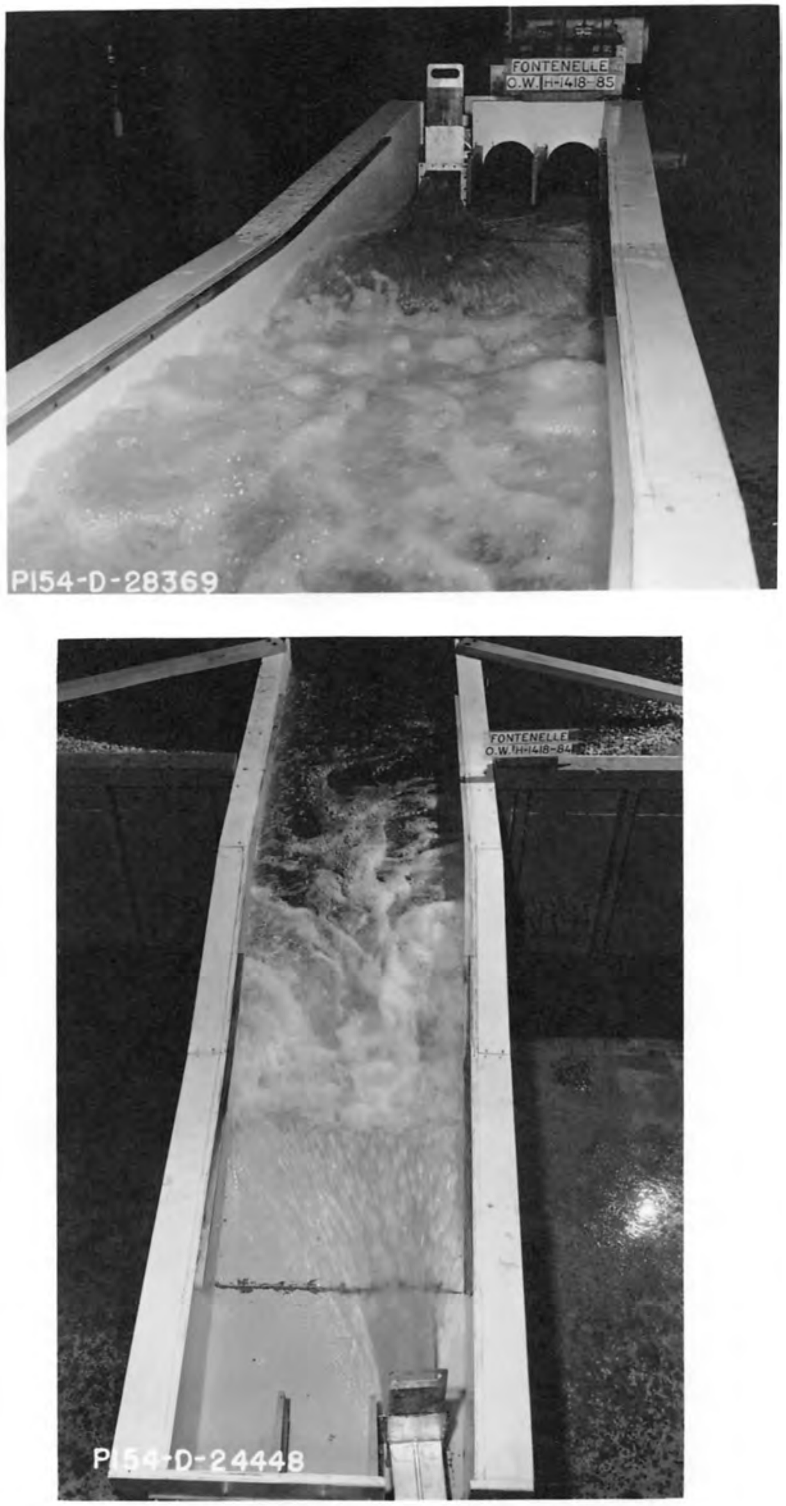




FONTENELLE DAM OUTLET WORKS

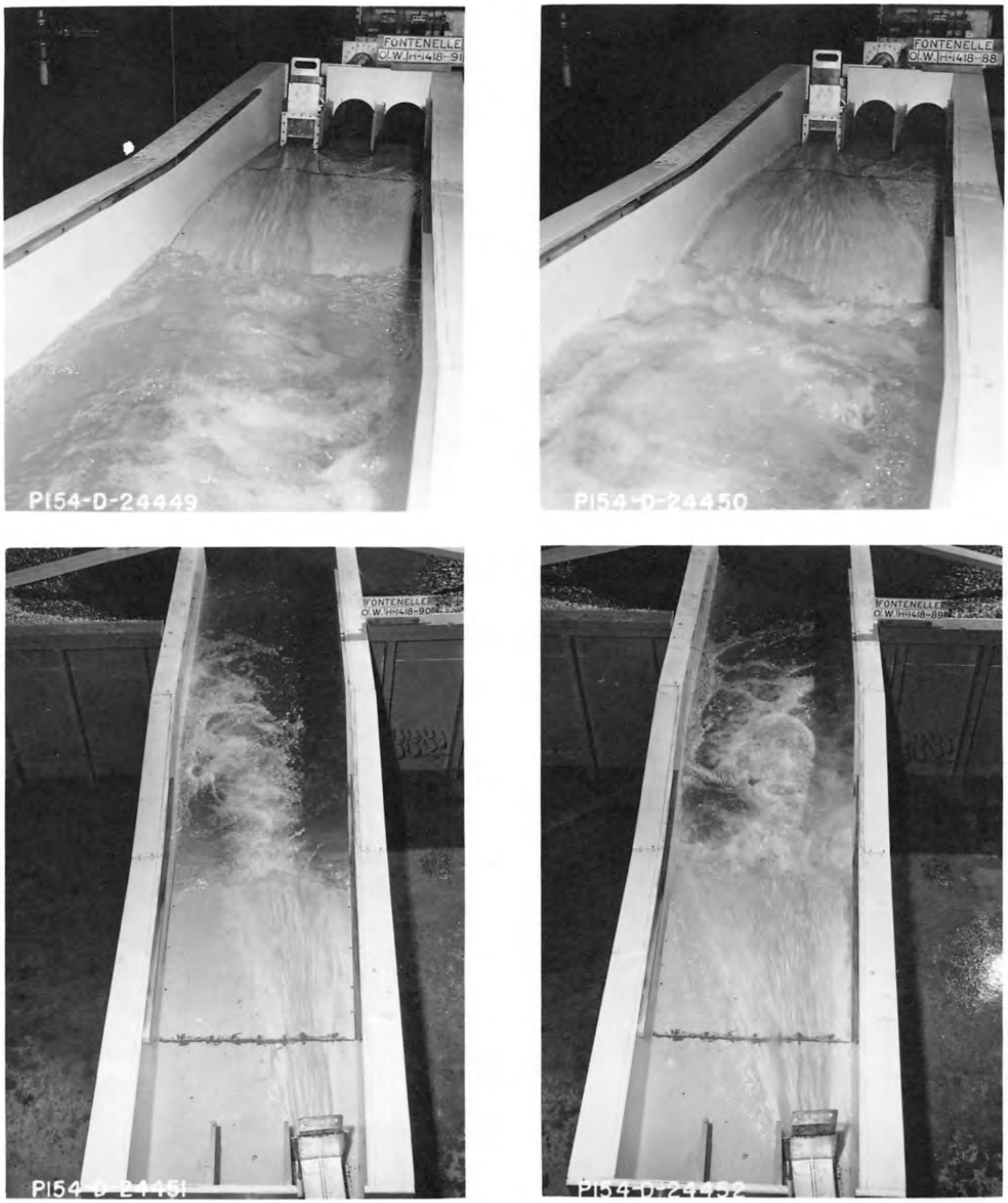
1:24.7 MODEL STUDIES

OUTLET STRUCTURE - FIFTH MODIFICATION



FONTENELLE DAM OUTLET WORKS

1:24.7 Model Studies
Basin Operation - Penstock Outlet Horizontal and Turned
5° to the Left (Fifth Modification)
Discharge = 4,734 cfs, Tailwater Elevation = 6402.3



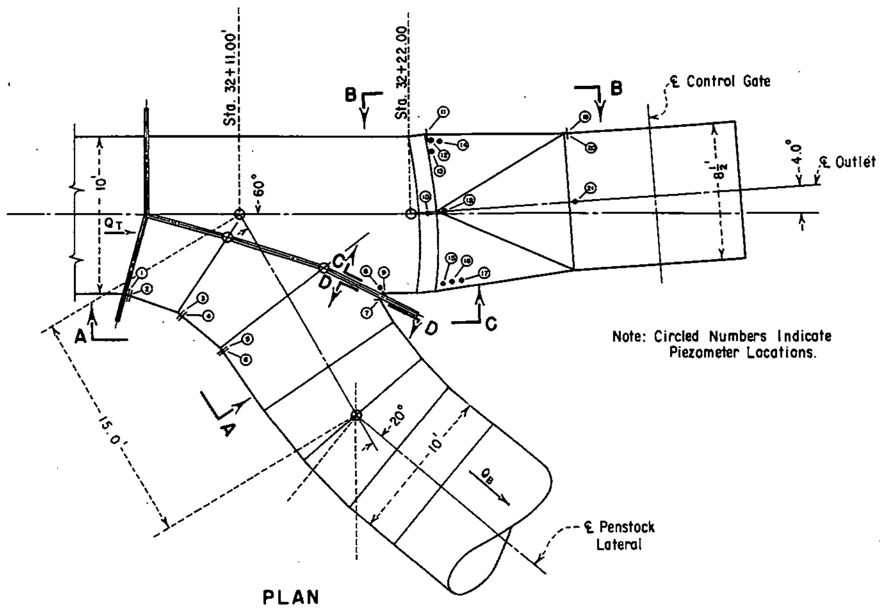
Discharge = 1,200 cfs

Discharge = 2,400 cfs

FONTENELLE DAM OUTLET WORKS

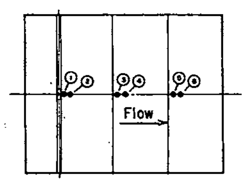
1:24.7 Model Studies
Basin Operation - Penstock Outlet Horizontal and Turned
5° to the Left (Fifth Modification)
Discharges of 1,200 and 2,400 cfs

water manometers

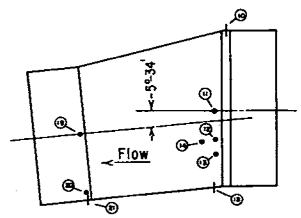


PLAN

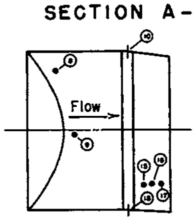
Note: Circled Numbers Indicate Piezometer Locations.



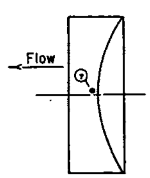
SECTION A-A



SECTION B-B



SECTION C-C



SECTION D-D

PIEZOMETER NUMBER	AVERAGE PRESSURE IN FEET OF WATER				
	RUN No. 1	RUN No. 2	RUN No. 3	RUN No. 4	RUN No. 5
1	-1.0	0.2	82.5	11.4	7.9
2	12.4	23.2	87.2	23.5	19.8
3	21.7	42.7	85.5	32.1	27.9
4	22.2	42.7	87.7	32.4	28.4
5	23.0	43.2	89.4	33.1	29.1
6	23.0	43.2	90.0	33.1	29.9
7	34.6	83.8	84.0	45.0	41.5
8	-17.5	53.2	98.1	-4.4	-7.7
9	-17.5	52.2	103.9	-5.2	-8.9
10	19.2	37.7		—	—
11	9.1	38.9		19.2	16.1
12	-0.8	32.2		9.9	5.7
13	5.2	34.2		16.1	10.1
14	11.9	40.0		22.5	17.3
15	3.2	35.2		14.1	9.9
16	9.9	40.8		20.8	16.1
17	13.3	44.1		24.2	18.8
18	2.5	37.9		13.6	8.9
19	-1.5	31.4		10.4	-2.0
20	12.4	41.5		23.2	15.1
21	14.0	38.0		—	

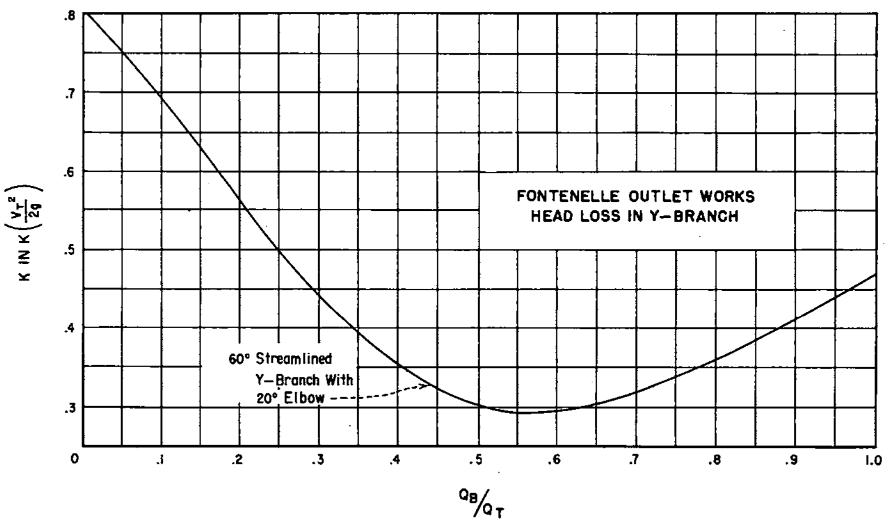
RUN No. 1 – $8\frac{1}{2}' \times 9'$ GATE FULL OPEN, OUTLET FLOW = 4734 CFS. NO FLOW IN PENSTOCK.

RUN No. 2 – $8\frac{1}{2}' \times 9'$ GATE, OUTLET FLOW = 3134 CFS. PENSTOCK FLOW = 1600 CFS.

RUN No. 3 – OUTLET CLOSED, FLOW IN PENSTOCK = 1600 CFS.

RUN No. 4 – $8\frac{1}{2}' \times 9'$ GATE CLOSED 0.2 FT, OUTLET FLOW = 4436 CFS, NO FLOW IN PENSTOCK.

RUN No. 5 – $8\frac{1}{2}' \times 9'$ GATE - FULL OPEN, OUTLET FLOW = 4600 CFS, NO FLOW IN PENSTOCK.



SCALE OF FEET

FONTENELLE DAM OUTLET WORKS

1:24.7 MODEL STUDIES

PRESSURE AND HEAD LOSS IN RECOMMENDED Y-BRANCH AND OUTLET STRUCTURE

FIGURE 28
REPORT HYD. 487

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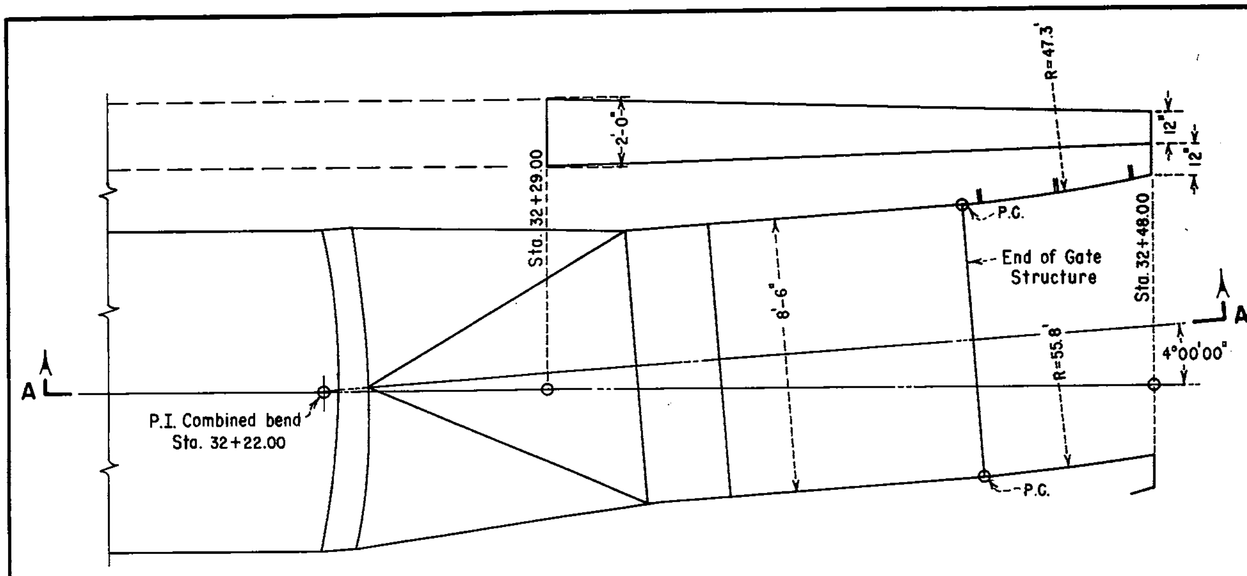
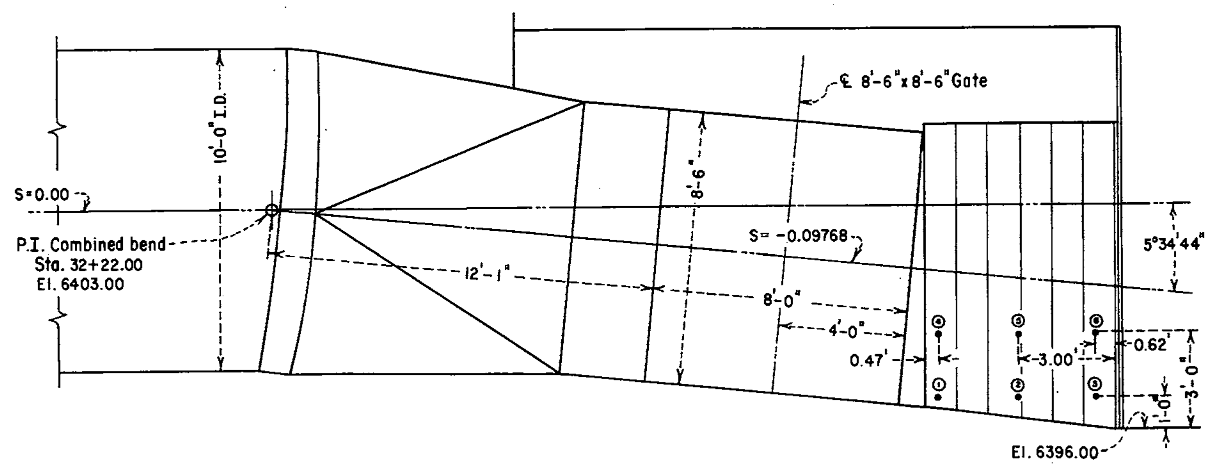


PLAN



SECTION A–A

WALL PRESSURES (FEET OF WATER)			
PIEZOMETER	100% OPEN	50% OPEN	25% OPEN
①	3.9	4.9	3.3
②	-0.5	-1.0	1.0
③	-3.5	-6.1	-0.2
④	-0.1	6.0	-0.2
⑤	-1.4	4.3	-0.4
⑥	-5.1	-1.5	-1.0

FONTENELLE DAM OUTLET WORKS
1:24.7 MODEL STUDIES
RECOMMENDED RIVER OUTLET STRUCTURE
WALL PRESSURES

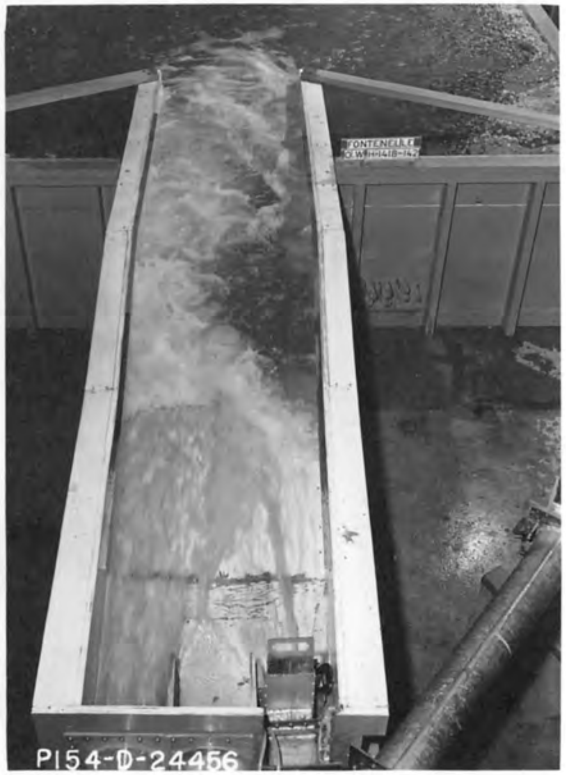
Discharge = 1,200 cfs

Discharge = 2,400 cfs

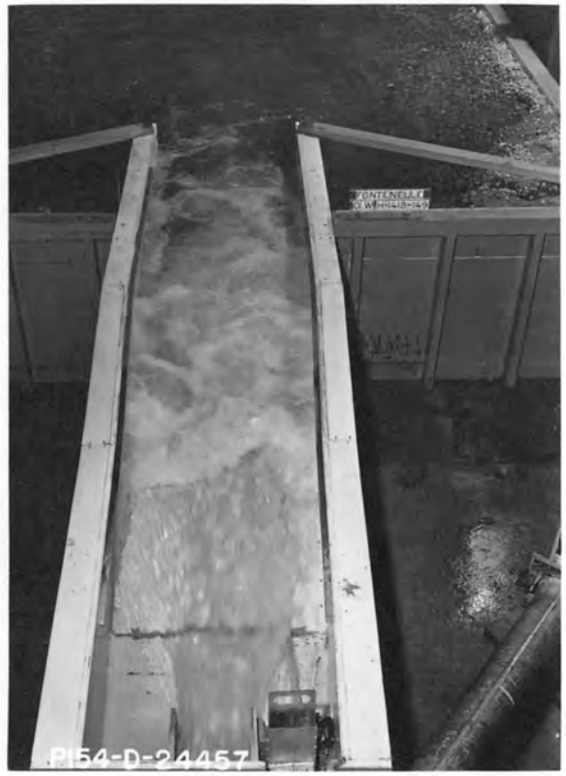
Discharge = 4,400 cfs

FONTENELLE DAM OUTLET WORKS

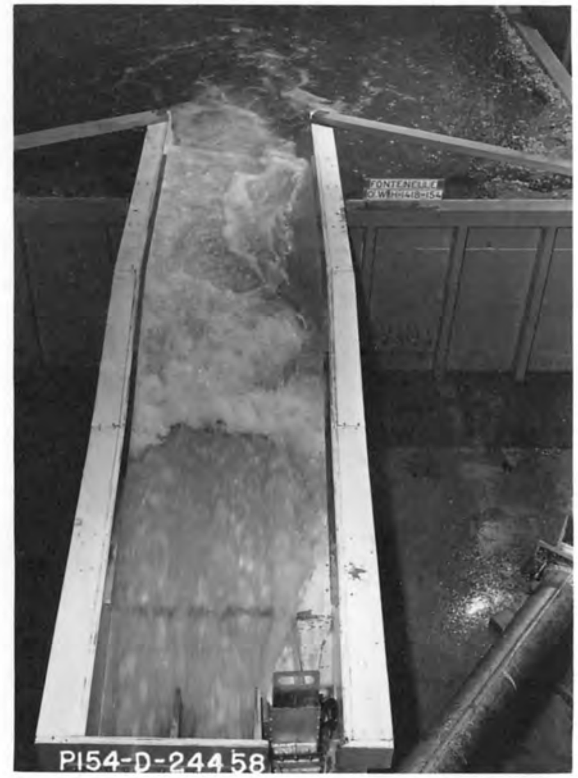
1:24.7 Model Studies
Basin Operation
Recommended Penstock Outlet



Penstock Outlet Discharge = 2, 400 cfs
Left Conduit Discharge = 6, 233 cfs
Center Conduit Closed
Tailwater Elevation = 6402. 8



Penstock Outlet Discharge = 2, 400 cfs
Left Conduit Closed
Center Conduit Discharge = 6, 233 cfs
Tailwater Elevation = 6402. 8



Penstock Outlet Discharge = 2, 400 cfs
Center and Left Conduits Open
Discharge = 12, 467 cfs
Tailwater Elevation = 6403. 6

FONTENELLE DAM OUTLET WORKS

1:24. 7 Model Studies
Basin Operation Combined Flows
Recommended Penstock Outlet

Figure 31
Report Hyd. 487