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# HYDRAULIC MODEL STUDY OF BUFFALO BILL DAM AND SPILLWAY REHABILITATION

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16. ABSTRACT

A new, increased inflow design flood and increased power generation requirements for 75-year-old Buffalo Bill Dam prompted modifications to the existing dam, spillway, and powerplant. A hydraulic model study investigated the proposed designs for the spillway and dam rehabilitation. The spillway approach channel configuration was determined and gate opening discharge curves were developed for 20.5- by 28-foot top-seal radial gates operating under a maximum head of 115 feet. The spillway tunnel geometry was changed to a modified horseshoe shape and realigned to reduce the potential for erosion in the riverbed downstream. A broad crest shape was developed for passing discharge over the top of the dam.

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# HYDRAULIC MODEL STUDY OF BUFFALO BILL DAM AND SPILLWAY REHABILITATION

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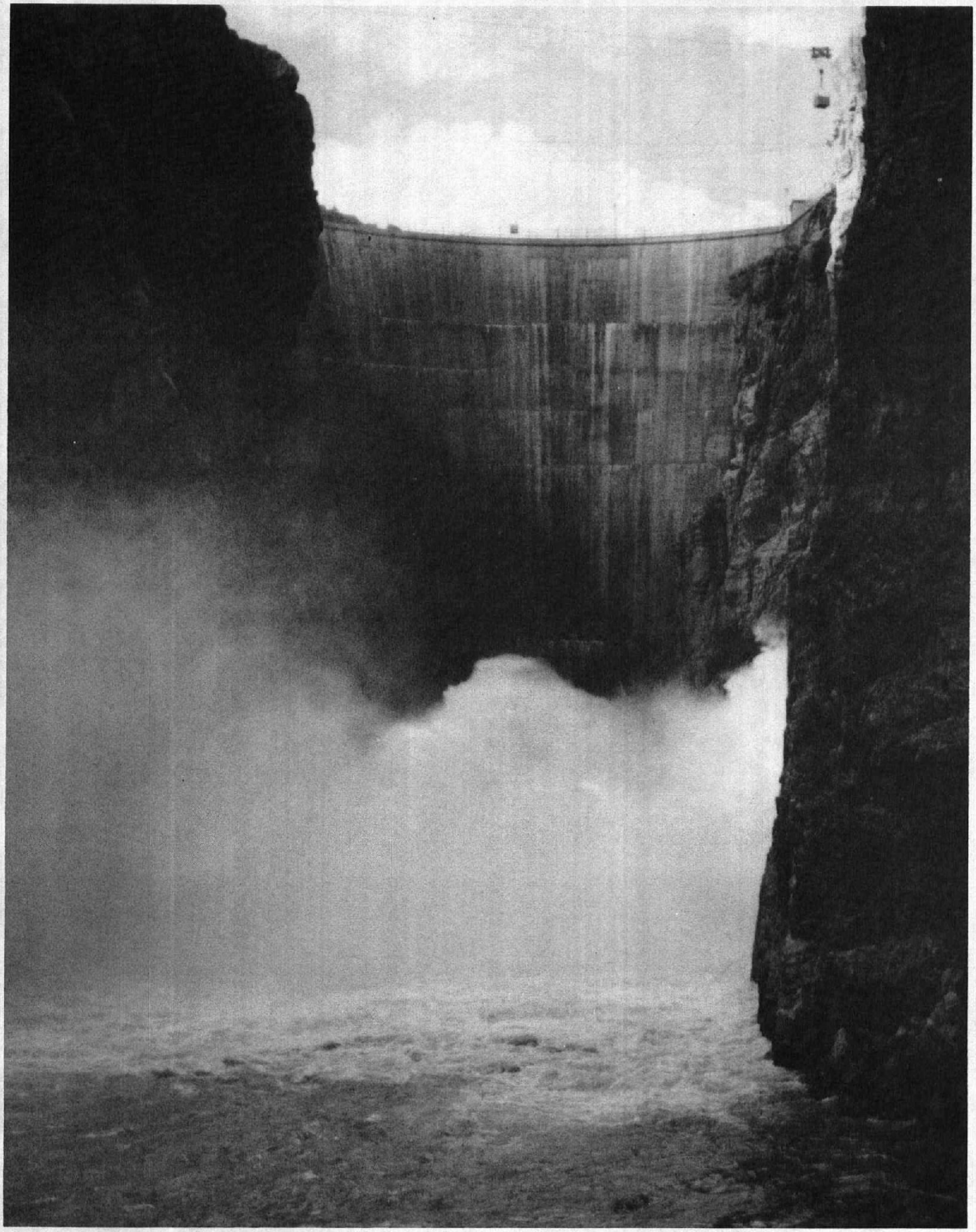

Frontispiece. – Buffalo Bill Dam spillway and Shoshone Powerplant.

## CONTENTS



## FIGURES

## PURPOSE

The model study was performed to investigate proposed designs for the rehabilitation of Buffalo Bill Dam and spillway. The study investigated:

- Tunnel spillway approach conditions and gate chamber location
- Capacity of the spillway tunnel controlled by two top-seal radial gates
- Spillway tunnel geometry and flow conditions
- Energy dissipation characteristics of the river channel
- Dam overtopping

The model study results contributed significantly to the final designs.

## INTRODUCTION

The potential for developing the area of the Shoshone Project was first seen by Col. William F. "Buffalo Bill" Cody. About 1900, a canal was built to transport water from the Shoshone River. In 1903, the Bureau of Reclamation was commissioned to develop the project. A major part of the project was Buffalo Bill Dam, located 7 miles west of Cody, Wyoming, on the Shoshone River. It was completed in 1910, when it was known as Shoshone Dam. The 325-foot-high concrete arch dam was one of the first high concrete arch dams built in the United States (fig. 1). The reservoir impounded by the dam provides irrigation, hydropower, and recreation facilities. The spillway, an uncontrolled concrete side channel weir upstream of the left abutment, discharges into a 21-foot-diameter unlined rock tunnel. Currently, the spillway capacity is 18,000 $ft^3/s$ at reservoir elevation 5369.0 feet. The 3,130-$ft^3/s$-capacity outlet works tunnels through the left abutment are controlled by two 4- by 5-foot rectangular high-pressure gates that discharge immediately downstream of the dam. The concrete-lined Shoshone canyon diversion conduit located near the right abutment supplies water to the Heart Mountain Powerplant and Irrigation District. The Shoshone Powerplant is located about 500 feet downstream of the dam on the left side of the canyon. This powerplant has a capacity of about 6,000 kilowatts and is supplied by penstocks through the base of the dam.

The increased PMF (probable maximum flood) for Buffalo Bill Dam requires that the overflow structures of the dam pass 120,000 $ft^3/s$ at reservoir elevation 5410.0. The magnitude of this increase makes it uneconomical to enlarge the existing spillway to pass the entire flood. As a result, the spillway was designed for 80,000 $ft^3/s$, leaving 40,000 $ft^3/s$ to pass over the dam.

The model study investigated proposed modifications for the tunnel spillway and the dam. The spillway modifications included adding a gate chamber with two top-seal radial gates controlling flow through an enlarged concrete-lined tunnel in the left abutment at the same location as the existing spillway (fig. 2). The dam will be raised 25 feet and a road constructed on top. The required spillway capacity at reservoir elevation 5410.0 is 80,000 $ft^3/s$. The final design for the spillway and dam modifications is shown on figure 3. The outlet works and Shoshone Powerplant will be rehabilitated, and an additional powerplant (Buffalo Bill) will be built on the right side of the river channel downstream from the Shoshone Powerplant.

## RESULTS AND RECOMMENDATIONS

The following is a summary of the model study results and recommendations. Each recommendation is discussed further in the appropriate section of this report.

1. The spillway gate chamber will be located downstream of the existing free overflow side channel intake at elevation 5360.0. Optimum hydraulic performance can be obtained by excavating the portion of the existing intake immediately in front of the chamber to elevation 5295.0 (fig. 11). This excavation will provide better flow distribution through the gates and tunnel and prevent impingement on the gate trunnions.

2. The recommended location for the gate chamber is station 2+00, flush with the existing steep canyon wall.

3. The width of both top-seal radial gates was increased 2 feet over the feasibility design width of 18.5 feet, producing 20.5-foot-wide by 28-foot-high gates. This gate size allowed passage of the required maximum tunnel spillway discharge of 80,000 $ft^3/s$ at reservoir elevation 5400.0. At the maximum reservoir elevation, 5410.0, 85,000 $ft^3/s$ was passed through the tunnel. Discharge curves for the recommended gate and approach channel excavation were developed for 10 to 100 percent gate openings (fig. 12).

4. An air vent is not required downstream of the gates.

5. Flow conditions through the initial 41-foot circular tunnel were poor. Significant improvement was achieved by changing the tunnel shape to a modified horseshoe and lengthening the upstream transition to 120 feet while maintaining the same flow area. The center pier will extend 75 feet

into the transition, tapering from 14 feet wide at the gate chamber to 3 feet wide at the downstream end (fig. 4). The gate chamber and transition section should be entirely lined. Preferably, the full length of the downstream tunnel should be lined; however, the semicircular tunnel crown above the springline may remain unlined downstream from the transition if the lining is uneconomical. This should have little impact on the flow conditions in the tunnel.

6. The end of the spillway tunnel was realigned to prevent the concentrated high-velocity jet exiting the tunnel from impinging on the steep cliff that supports the dam access road. The end of the tunnel should be realigned by shifting the centerline at least 4.8° toward the river channel on the right, starting at station 5+00 (fig. 3).

7. The most severe river channel erosion will occur at the base of the cliff downstream from the excavated spillway channel. The principal deposition will be just upstream and in front of the existing Shoshone Powerplant (fig. 20).

8. The newly raised dam will be at elevation 5395.0. The broad crest shape, shown on figure 21, is recommended for the top of the dam. This shape, with a crest length of 190 feet, will pass 38,800 $ft^3/s$ at reservoir elevation 5410.0 (fig. 22). Overtopping flow impinged on the canyon walls and at the toe of the dam. The tailwater elevation of 5181.0 feet produced by a spillway discharge of 77,000 $ft^3/s$ before dam overtopping occurs is necessary to protect the toe of the dam.

## HYDRAULIC MODEL AND TEST PROCEDURES

The 1:42.783 hydraulic model, which was designed by Froude relationships, is shown on figure 5. The model scale was chosen so that the 41-foot-diameter circular tunnel would correspond to clear plastic pipe available for use in the model. Included in the model were the reservoir to approximately 813 feet upstream, the gate chamber (initially housing two 18.5- by 28-foot top-seal radial gates), and an 82-foot-long rectangular to circular transition leading to the 41-foot-diameter tunnel. Also modeled were 750 feet of the downstream river channel topography and a nonoperable representation of the Shoshone Powerplant on the left bank. The full height and shape of the dam, including the 25-foot-height increase and 190-foot-long crest, were also modeled.

The reservoir was modeled inside a 19- by 14.25-foot head box that provided appropriate approach flow to both the spillway intake and the dam. Flows through the 0.96-foot-diameter model tunnel were initially controlled by 0.43- by 0.65-foot sheet metal gates. The transition from the gate chamber to the circular tunnel was 1.92 feet long. The side channel spillway intake area was modeled using Styrofoam, and the tunnel was modeled with clear plastic so the flow could be easily seen. The downstream river channel was modeled using concrete for the steep canyon walls and geometrically sized movable material for the streambed. The existing dam was modeled with concrete, but the extension representing its height increase was modeled with polyurethane foam. The maximum discharge, 120,000 $ft^3/s$, was represented in the model by 10.02 $ft^3/s$.

The model was systematically tested from upstream to downstream, modifying as needed each upstream feature that might affect the downstream hydraulic performance. Testing proceeded in the following order:

1. Approach channel excavation
2. Gate chamber placement
3. Increasing gate size to attain required capacity
4. Determining discharge capacity of the tunnel
5. Investigation of tunnel transition, shape, and lining
6. Investigation of river channel erosion
7. Investigation of dam overtopping

Model studies were conducted from October 1984 to September 1985.

## SPILLWAY INVESTIGATION

The required spillway discharge capacity is 80,000 $ft^3/s$ at reservoir elevation 5410.0. To pass the increased discharge, the existing unlined spillway, which had a capacity of only 18,000 $ft^3/s$, required significant modifications, including enlargement, lining, and control by two top-seal radial gates. The gate chamber will be located at the upstream end of the existing tunnel, approximately 60 feet downstream from the crest of the existing side channel weir intake. The proposed gate chamber invert is at elevation 5295.0, and the existing side channel weir crest is at elevation 5360.0.

### Approach Channel

The extent of excavation for the existing side channel weir and the optimum location for the gate chamber were investigated. Tests were conducted to optimize flow conditions while attempting to minimize excavation cost. The excavations that were tested and the initial location proposed for the gate chamber are shown on figure 6. Three excavation options were investigated in the following order:

1. Keeping the entire channel weir crest at elevation 5360.0 and excavating the downstream side immediately upstream of the gate chamber to a ¼:1 slope.

2. Excavating the portion of the weir immediately upstream of the gate chamber to elevation 5320.0 with a ¼:1 downstream slope. This was the proposed design (fig. 2).

3. Completely removing the weir immediately upstream of the gate chamber, leaving a flat approach channel at invert elevation 5295.0.

Flow conditions were investigated by injecting dye at various locations and at various reservoir elevations upstream of the gate chamber and by observing flow conditions in the tunnel.

Tests were conducted with the entire side channel weir at the original elevation of 5360.0, 65 feet above the channel invert. The downstream face of the portion in front of the gate chamber was excavated to a ¼:1 slope (fig. 6). The approach channel between the weir and the gate chamber was cut vertically to form the left and right boundaries (fig. 7). In the prototype, these walls will be nearly vertical with 1-foot-wide berms every 20 feet for construction purposes (fig. 4, sec. F-F). This option was investigated to reduce reservoir drawdown during construction and to decrease the cost associated with excavation of the existing side channel topography.

Spillway operation with the weir at elevation 5360.0 produced poor flow conditions. Dye traces revealed a nonuniform flow distribution that was dependent on the reservoir elevation. Most of the flow from the reservoir surface entered the left bay; flow closer to the elevation of the side channel weir entered the right bay. Most of the total flow entered the right bay.

Turbulence in the approach channel was caused by flow over the high existing side channel weir. With small gate openings or low discharges, a stagnant area developed on the downstream side of the weir. As the discharge increased, flow over the weir impinged on the invert at elevation 5295.0, upstream of the gates. A component of this flow was deflected upward for gate openings exceeding 50 percent and impinged on the gate trunnions. This caused concern for the loading on the gate trunnions (fig. 8).

When the discharge was measured with the gates fully open, the required maximum discharge was not achieved. The discharge measurements and approach flow conditions were used as a basis for comparison with later approach topography modifications.

Additional excavation would provide more direct access to the gate chamber and, hopefully, improve flow conditions and increase discharge capacity. A section of the side channel weir immediately upstream of the gate chamber was excavated from elevation 5360.0 down to elevation 5320.0. This formed a berm as wide as and directly in front of the gate chamber. The remainder of the approach channel area was not modified (fig. 9).

Investigation of flow conditions upstream of the gates again showed areas of low velocity at the base of the berm near the invert. The flow distribution remained uneven, and more flow entered the right bay than the left. Most of the flow from around the right corner and over the berm entered the right bay, whereas most of the flow over the original side channel weir to the left of the approach channel entered the left bay. The effects of the uneven flow distribution were also observed in the downstream tunnel where flow impinged on the trunnions of the right gate during maximum discharge. Even though impingement did not occur until the gates were fully open, concern remained for the structural integrity of the gate trunnions. A set of gate opening discharge curves were developed for this approach configuration and tunnel design (fig. 10). The required tunnel capacity (80,000 ft$^3$/s) was not attained although flow conditions were improved somewhat over those with the entire weir at elevation 5360.0.

Optimum hydraulic performance was obtained by excavating the existing side channel weir directly in front of the gates down to elevation 5295.0 for the full width of the gate chamber (figs. 3 and 11). This excavation and vertical walls on both sides of the gate chamber approach channel allowed the flow to travel directly to the gates. This recommended approach tunnel excavation produced more uniform flow conditions upstream of and through the tunnel, no impingement on the gate trunnions, and increased discharge capacity.

**Gate Chamber Location**

The proposed gate chamber location is shown on figures 2 and 6. Note that the upper right corner of the chamber protruded into the reservoir beyond the edge of the steep topography. Persistent vortexes formed upstream of the chamber that were not diminished by the approach channel improvements. Vortexes were caused by flow around the corner of the chamber during large discharges at the normal reservoir elevation of 5393.5 and above. To alleviate this, the chamber was moved to station 2+00, 26.75 feet downstream from the original location. This placed the upper right corner of the chamber flush with the canyon wall and reduced or eliminated vortexes. The recommended gate chamber location is shown on figure 3. This location still provided access across the top of the chamber to the reservoir area upstream.

### Tunnel Capacity

The proposed tunnel design consisted of two 18.5- by 28-foot top-seal radial gates upstream of an 82-foot-long rectangular to circular transition and a 41-foot-diameter tunnel. The recommended approach channel configuration required removal of the entire side channel weir immediately upstream of the gates. It was not certain whether cost and reservoir elevation constraints would allow moving the weir from elevation 5320.0 to the invert. Therefore, discharge curves were developed for equal gate openings of 10 to 100 percent and for the side channel weir at elevation 5320.0 upstream of the gates (fig. 10). This design passed a maximum spillway discharge of 73,000 $ft^3/s$ at reservoir elevation 5410.0, 8.75 percent less than required. Previous tests had shown that removing the upstream berm entirely increased the discharge only slightly; therefore, a larger gate area was needed to pass 80,000 $ft^3/s$. Discussions with mechanical designers about the structural design of the gates led to the decision to increase the discharge capacity by making the gates wider rather than higher.

A top-seal radial gate discharge coefficient of 0.87 was calculated from model tests with the original gates fully open at maximum reservoir elevation 5410.0. This coefficient was then used to calculate the gate width required to pass the 80,000 $ft^3/s$ maximum discharge. The gate width was determined from a modified version of the orifice equation:

$$Q = CA\sqrt{2g\left(H - \frac{G}{2}\right)}$$

where:

- $Q$ = maximum discharge = 80,000 $ft^3/s$,
- $C$ = discharge coefficient = 0.87,
- $G$ = maximum gate opening = 28 feet,
- $A$ = area of gate opening = (Number of gates)$GW$ = 2(28)$W$ = 56$W$, where $W$ = gate width,
- $H$ = maximum total head above tunnel invert = 115 feet, and
- $g$ = gravitational constant = 32.2 $ft/s^2$.

Solving for the gate width, the equation becomes:

$$W = \frac{Q}{56C\sqrt{2g\,(H-14)}} = 20.36 \text{ feet}$$

The gate widths were increased to 20.5 feet with a maximum center pier width of 8 feet. This maintained the outside width of the gate chamber and decreased the center pier width. For structural design purposes the center pier width was later increased to 14 feet, with the same gate width. These gate dimensions and the approach channel invert at elevation 5295.0 allowed passage of the required maximum discharge, 80,000 ft$^3$/s, at reservoir elevation 5400.0, and 85,000 ft$^3$/s at reservoir elevation 5410.0. Discharge curves for the two gates open equally from 10 to 100 percent are shown on figure 12.

**Tunnel Geometry**

The original design consisted of an 82-foot-long transition from the 49- by 33-foot gate chamber to the 41-foot-diameter tunnel. Just downstream of the gate seat the tunnel invert began a 10-percent slope (fig. 6). Operation of the tunnel spillway revealed adequate flow area in the tunnel to pass the maximum discharge but poor flow conditions. During low discharges, the flow separated from the tunnel surface on both sides of the invert centerline at the beginning of the circular tunnel. As the discharge increased, separation continued up the sides of the tunnel. Fins developed and produced unacceptable flow conditions when this separated flow returned to the tunnel surface farther downstream. The fins rose up both sides of the tunnel and approached the tunnel crown at maximum discharge (fig. 13).

The poor flow conditions prompted modification of the tunnel geometry. The separation from the tunnel surface was caused by the shape changing from rectangular to circular in too short a distance. The first recommendation was to lengthen the transition to 120 feet, approximately 3 tunnel diameters. However, because of the high velocity of the flow, lengthening the transition alone might not have solved the problem. Therefore, the downstream tunnel shape was also changed, to a modified horseshoe with a 35- by 24-foot rectangular bottom and a 17.5-foot radius crown at station 4+15.33. Less shape transition is required to change to a smaller rectangle on the lower half and from a rectangle to a semicircle on the upper half of the tunnel. At the end of the transition, station 4+15.33, the same area as the 41-foot-diameter tunnel was duplicated by the modified horseshoe-shaped lined tunnel. The crown was unlined and formed by a 19-foot radius for the remainder of the downstream tunnel (fig. 4).

A free water surface was maintained throughout the transition and downstream tunnel, and a small fin formed downstream of the center pier. Flow conditions in the tunnel were greatly improved over the previous design (fig. 14). The horseshoe-shaped tunnel should require less excavation because of its shape and position with respect to the original tunnel alignment.

The design provided for an air vent in the gate chamber for aeration downstream of the gates. The air vent was modeled, but did not draw air for any flow rate. In the prototype, air will be adequately supplied from the free surface in the downstream tunnel; therefore, an air vent is not necessary.

**Tunnel Lining**

The existing 21-foot-dimater tunnel spillway is unlined and has a maximum capacity of 18,000 ft³/s at reservoir elevation 5369.0. Figure 15 shows the spillway tunnel discharging approximately 6,000 ft³/s. The rough surface of the unlined tunnel causes turbulence, aeration, and dispersion of the jet.

The question of whether to line the proposed larger-capacity tunnel arose because the existing unlined tunnel has produced acceptable flow patterns and the new tunnel construction was assumed to be less expensive if it were not lined. The gate chamber and transition section will be fully lined.

The model study investigated the flow patterns and capacity of a lined tunnel. The modified horseshoe-shaped tunnel will be required to pass significantly larger discharges at higher velocities than the existing unlined tunnel. These high-velocity flows will increase the possibility of erosion in an unlined tunnel. Because the increased surface roughness of an unlined tunnel would require a significantly larger tunnel to pass the required discharge, and because better flow conditions will exist with a lined tunnel, the modified horseshoe shape should be lined at least to the springline, 24 feet above the invert. If it is more economical, the crown section may be left unlined because flows will only reach that depth during the PMF.

**Unbalanced Gate Operations**

Balanced gate operation is recommended. Unbalanced operation should be used only during a gate malfunction. Nevertheless, unbalanced operations were investigated to determine the flow conditions in the tunnel. The pier between the gates has been structurally designed to withstand the loading produced by fully opening only one gate under maximum reservoir head. Unequal gate openings produced fins in the tunnel downstream of the center pier. Because the tunnel might be lined only to the springline, uneven operation might cause erosion to the unprotected surface above the lining.

To prevent flows above the lining, neither gate should be opened more than 75 percent during single gate operation. With one gate 75 percent open, additional flow should be passed through the other gate until it is 75 percent open. Further increases must be made under balanced operation. Operation of the left gate alone will cause the jet to impinge on the canyon wall downstream of the tunnel, possibly causing erosion damage.

### Downstream Tunnel Alignment

The updated discharge and head requirements, 80,000 $ft^3/s$ and 115 feet, respectively, will produce significantly higher velocities through the proposed spillway tunnel. Flow through the lined spillway tunnel will disperse much less than flow through the existing unlined tunnel. The proposed alignment of the tunnel caused the concentrated high-velocity jet to exit from the tunnel and impinge on the steep cliff downstream from the excavated spillway channel (fig. 16). The dam access road located atop the cliff could be endangered by the jet impinging on the base of the cliff and undermining or fracturing the rock. Enlarging the tunnel along the existing alignment will require that this channel be widened toward the left, exposing more of the downstream cliff to the impact of the jet (fig. 17).

Several attempts were made in the model to divert the spillway jet away from the cliff and toward the river channel. Sheet metal plates were attached to the invert and left wall inside the end of the tunnel. The bottom plate was used to form a ramp to lift the jet. The tests indicated that the jet did not require lifting; therefore, the bottom plate was removed. The left sidewall plate was angled to redirect low- to medium-range discharges toward the river channel; however, too great a sidewall angle was required at the end of the tunnel to redirect larger discharges.

Next, tunnel realignment was investigated. The model tunnel was constructed of several clear plastic sections. The last two sections were joined at approximately station 5+34, about 119 feet downstream from the transition section. A flexible joint was installed there to permit easy realignment of the end of the tunnel. Small discharges were easily deflected by slightly moving the end of the tunnel toward the river channel. The jet produced by the maximum discharge required a minimum deflection angle of 4.8° with respect to the tunnel centerline. This alignment, which deflected the jet away from the cliff and into the river channel at the base of the cliff (fig. 18), should reduce the excavation needed for the steep canyon wall along the left side of the spillway channel. The rock outcropping on the right side of the spillway channel (fig. 16) may require minor excavation.

This deflection angle, 4.8°, should be reproduced in the prototype by a long-radius horizontal curve. The station chosen for the realignment in the model was not critical and need not be duplicated in the prototype.

**Downstream River Channel Erosion**

Approximately 750 feet of the existing river channel downstream of the dam was modeled. The lowest elevation in the model represented an actual bedrock elevation of approximately 5070.0 feet. The river bottom profile was set in the model based on data for reach 1 available from a 1983 tailwater study. This profile was used as the reference for evaluating river channel erosion or deposition. The riverbed material used in the model was geometrically sized to represent 5- to 10-foot rock in the prototype. The general areas of erosion and deposition patterns were determined by the model; however, the quantity of erosion was not correctly modeled.

The following test procedure was used to investigate river channel erosion and deposition patterns with the recommended spillway tunnel alignment. Gate openings of 10, 20, 30, 50, 100 percent and the PMF (gates 100 percent open and dam overtopped) were tested consecutively. Each gate opening was tested to represent 12 hours of prototype operation with an additional hour of operation under the PMF for a cumulative total of 61 hours. The tailwater elevation was set before each release (fig. 19). Erosion and deposition patterns were noted at the end of each release period and recorded on a map of the river channel before continuing to the next discharge.

The excavated spillway channel and location of the jet impact area for most of the discharges is shown on figure 18. The jet from a 10-percent gate opening impinged on the excavated spillway channel, then flowed down into the river channel. All larger discharges cleared the spillway channel and impacted at the base of the cliff where most of the erosion occurred.

The rock outcropping on the right side of the river channel across from the powerplant provided a natural barrier for containing the high-velocity spillway jet. The erosion process gradually removed the smaller material from the impact area at the base of the cliff, forming an armored plunge pool that aided the stilling action. The riverbed surrounding the plunge pool sloped upward to the original bed elevation and higher where deposition occurred. A portion of the flow exiting the plunge pool area eroded a channel along the right riverbank that began opposite the plunge pool.

Discharges from 10-percent spillway gate openings eroded material that eventually was deposited upstream of the powerplant. There is evidence of this deposition pattern from operation of the existing spillway. Discharges from increased gate openings eroded more material from the plunge pool area and deposited this material upstream of and in front of the powerplant. General erosion and deposition patterns are shown on figure 20 for 20-percent gate openings. These general areas of erosion increased as the discharge increased.

The discharges when the gates were open more than 50 percent produced high-velocity flows downstream from the plunge pool. These flows impinged on the rock outcropping opposite the powerplant, rebounded across the river channel, and impacted on the Shoshone Powerplant retaining wall. Erosion then began in front of the powerplant and continued as the discharge increased. According to the tailwater curve, the tailwater created by discharges greater than 22,000 $ft^3/s$ will entirely submerge the powerplant.

The original river channel profile at the toe of the dam was undisturbed by the spillway flows, and no excessively high waves occurred. However, flow overtopping the dam did erode material from this area. A small amount of this material was deposited in a berm at the upstream end of the spillway channel, but most was moved downstream by the spillway flows.

## DAM OVERTOPPING

The PMF required passage of 120,000 $ft^3/s$ at reservoir elevation 5410.0. This discharge requirement was met by passing 80,000 $ft^3/s$ through the redesigned tunnel spillway and 40,000 $ft^3/s$ over the top of the dam. The existing dam will be raised 25 feet to elevation 5395.0. Increasing the dam height will provide more storage for irrigation water and more head for passing spillway tunnel flows and for producing power. Flow will pass over the dam only when the tunnel spillway discharge capacity at reservoir elevation 5395.0 is exceeded. Two different shapes for the top of the dam were investigated: a 10-foot-wide broad crest with a rounded upstream edge and a sharp crest with a 1:1 downstream slope (fig. 21).

### Outlet Works Gate Chamber

The new outlet works gate chamber will be located at elevation 5129.0 on the centerline of the downstream face of the dam. The 30-inch jet-flow outlet works gate has been designed to operate submerged and will do so most of the time. Pressures were measured on the face of the dam at the gate chamber location to determine the possible magnitude of the forces that the chamber may have to withstand. Pressures were measured during the PMF, which is maximum spillway discharge and maximum overtopping. The results showed average pressures only 1 foot higher than the hydrostatic pressure produced by the tailwater elevation of 5197.0 at 120,000 $ft^3/s$.

Pressure fluctuations, which did not exceed 12 feet, were caused by the roller that formed between the overtopping jet impacting in the river channel and the face of the dam. Impact pressures in the river channel at the toe of the dam were not severe because of the tailwater (53 ft) that had been developed by maximum spillway discharge before overtopping.

### Broad Crest

The initial proposed design consisted of a 10-foot-wide broad crest with a rounded upstream edge along the full 190-foot length of the dam. A chamber inside the 25-foot-high addition to the top of the dam would provide access across the dam during overtopping. The discharge curve developed for this crest showed passage of 38,800 ft$^3$/s ($C = 3.52$) at maximum reservoir elevation 5410.0 (fig. 22). For discharges less than approximately 15,000 ft$^3$/s, the nappe clung to the face of the dam. The nappe sprung free and impacted in the river channel away from the toe of the dam as the discharge increased (fig. 23). Acceptable flow conditions were achieved with the broad crest shape; however, a structural design change required adding piers to the top of the dam to support a road instead of the chamber access. The piers reduced the effective crest length; therefore, a more efficient crest shape was investigated to increase the discharge over the remaining crest length.

### Sharp Crest

Five 3-foot-wide piers with rounded upstream noses were added to the top of the dam to support the road. This reduced the total crest length from 190 to 175 feet and produced a 170.5-foot effective crest length after subtracting pier and abutment losses.[1] Therefore, to attain the required discharge, the broad crest shape was changed to a more efficient sharp crest shape.

The sharp crest shape passed 40,000 ft$^3$/s at reservoir elevation 5410.0 (fig. 24). Drawdown of the water surface upstream of the dam at reservoir elevation 5410.0 allowed flow under the road between piers, but water impacting the piers surged over the road. Although the unit discharge was significantly higher with the sharp crest shape, flow conditions were more desirable with the broad crest shape. For the sharp crest shape, the nappe for low flows clung to the crest and flowed down the dam face. As the discharge increased, the jet sprang free and impinged on the canyon walls along the abutments closer to the toe of the dam than with the broad crest (fig. 25). The amount of impingement on the abutments was relatively insignificant, but was greater than that with the broad crest shape. The sharp crest shape did not aerate as easily as the broad crest shape but should aerate adequately in the prototype because of lesser surface tension effects.

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*Design of Small Dams*, U.S. Department of the Interior, Bureau of Reclamation, p. 373, GPO, Washington, D.C., 1977.

**Crest Selection**

The broad crest shape was selected for the top of the dam based on the ease of construction and the better flow conditions over the dam. The roadway and piers will be constructed on the broad crest shape even though they were not modeled. The reduction in discharge over the dam caused by the addition of the piers should be adequately recovered from the additional 5,000 $ft^3/s$ passed by the spillway tunnel at elevation 5410.0.

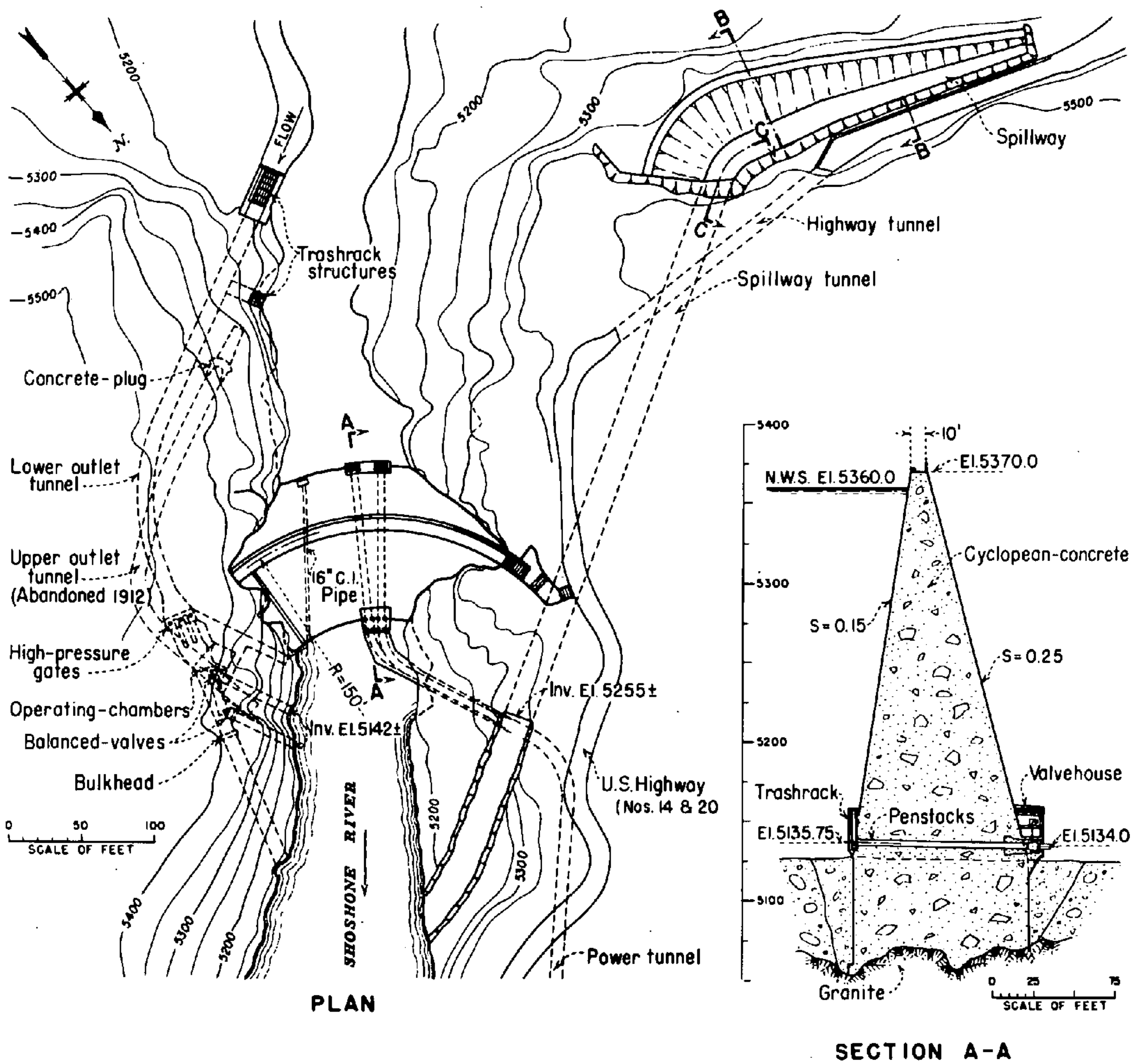



Figure 1. – Existing hydraulic structures at Buffalo Bill Dam.

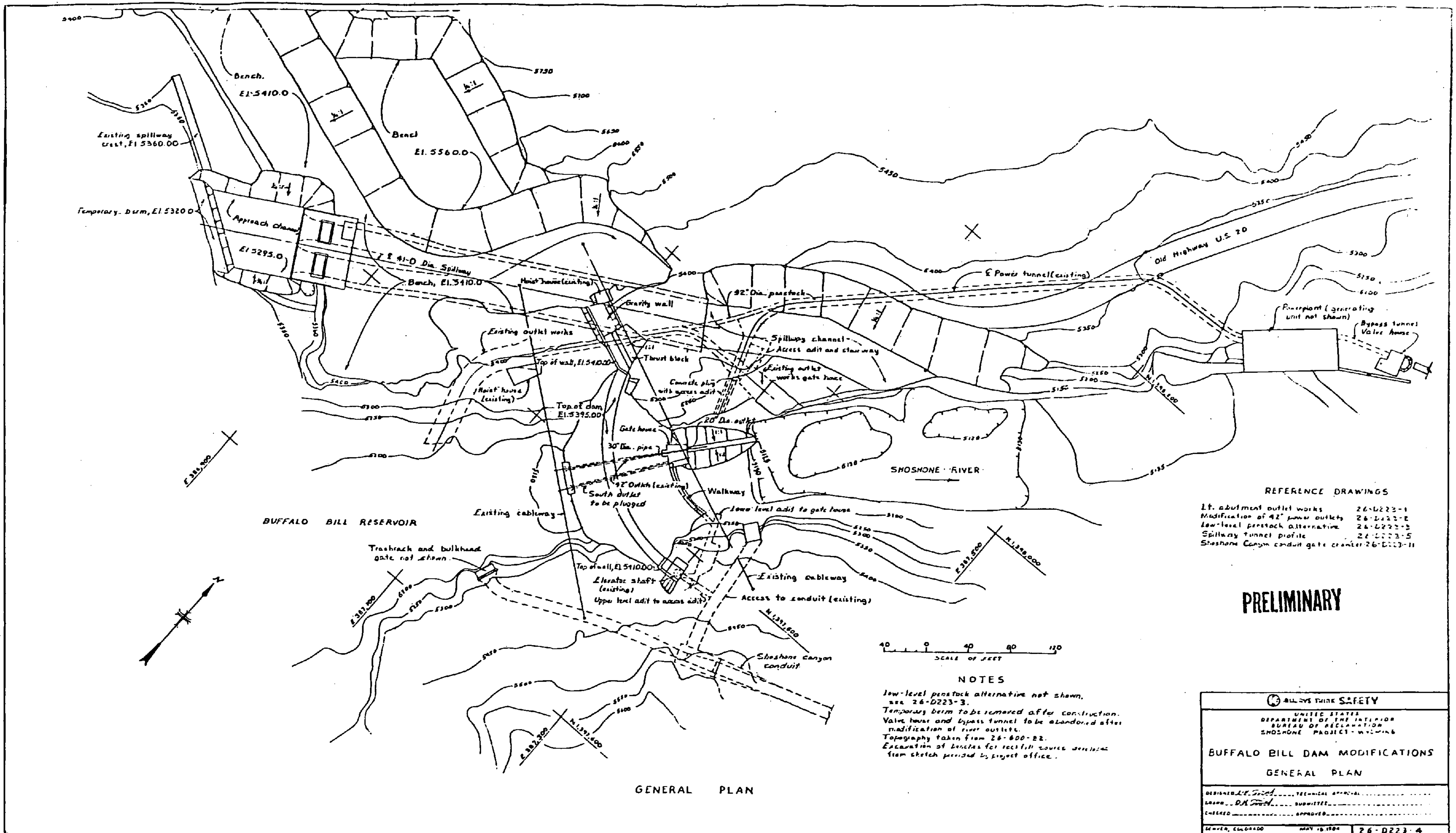



Figure 2. – General plan of proposed spillway and dam modifications at Buffalo Bill Dam.

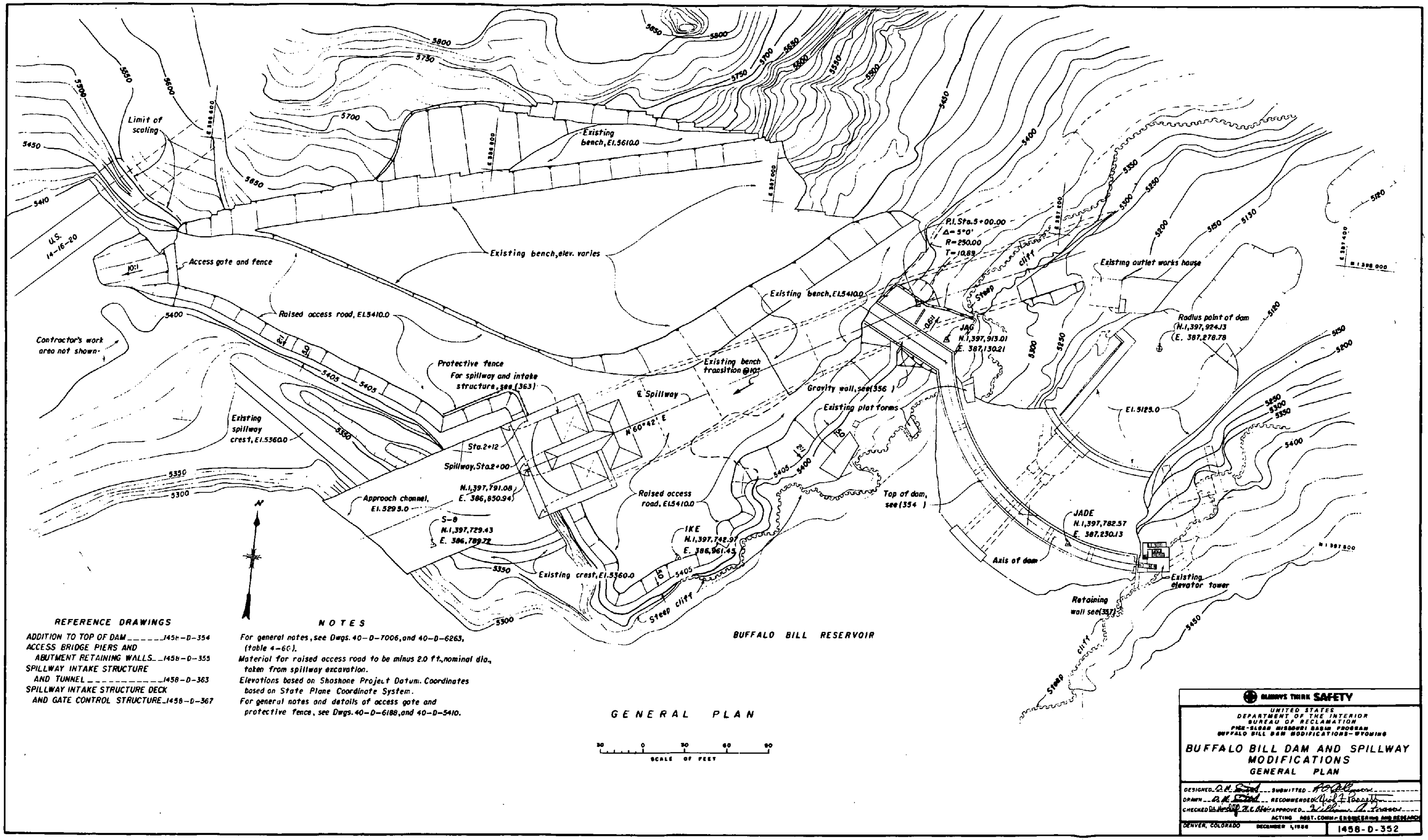



Figure 3. – General plan of final spillway and dam modifications at Buffalo Bill Dam.

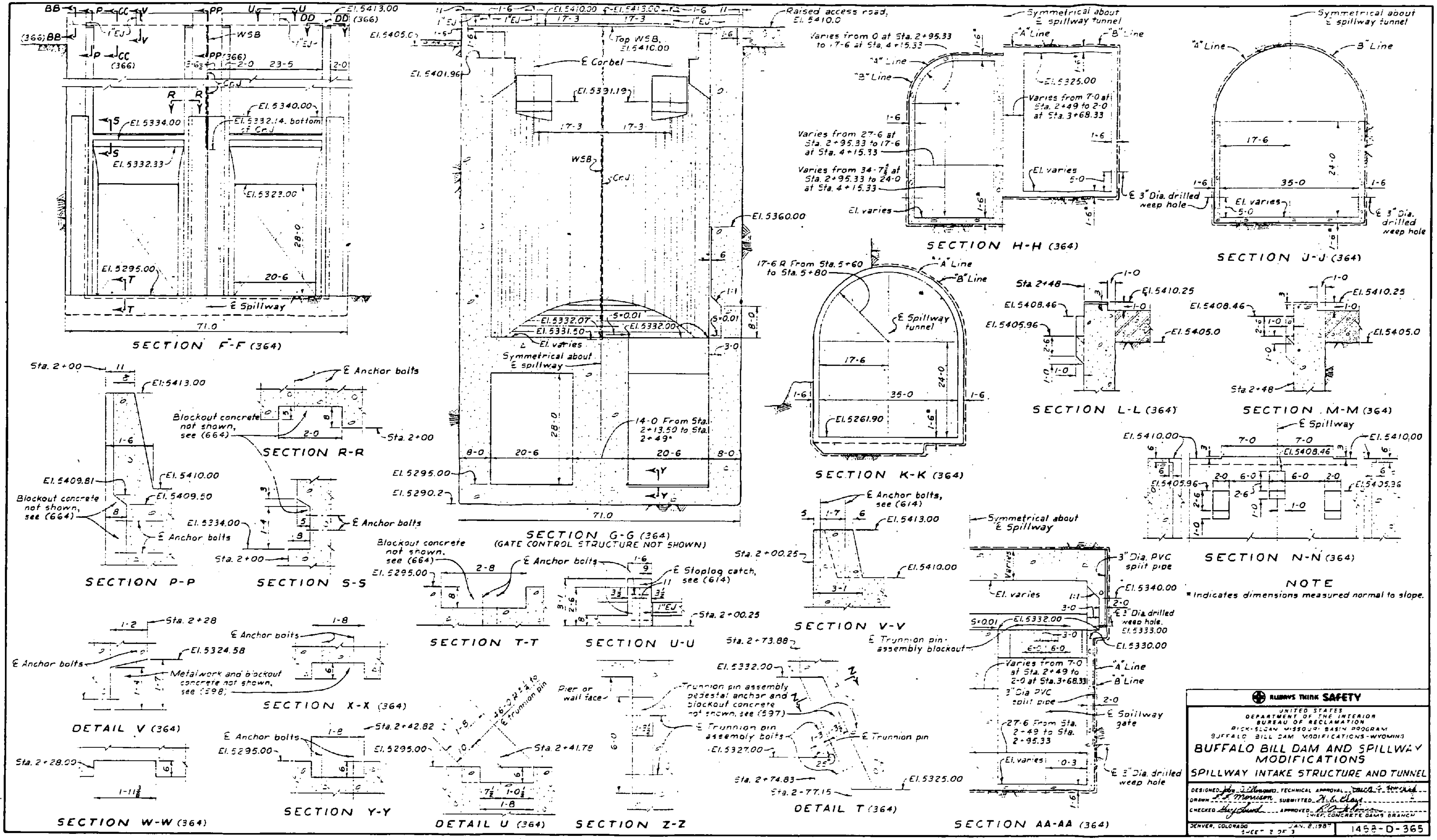



Figure 4. – Final modifications for spillway intake structure. (Sheet 1 of 2).

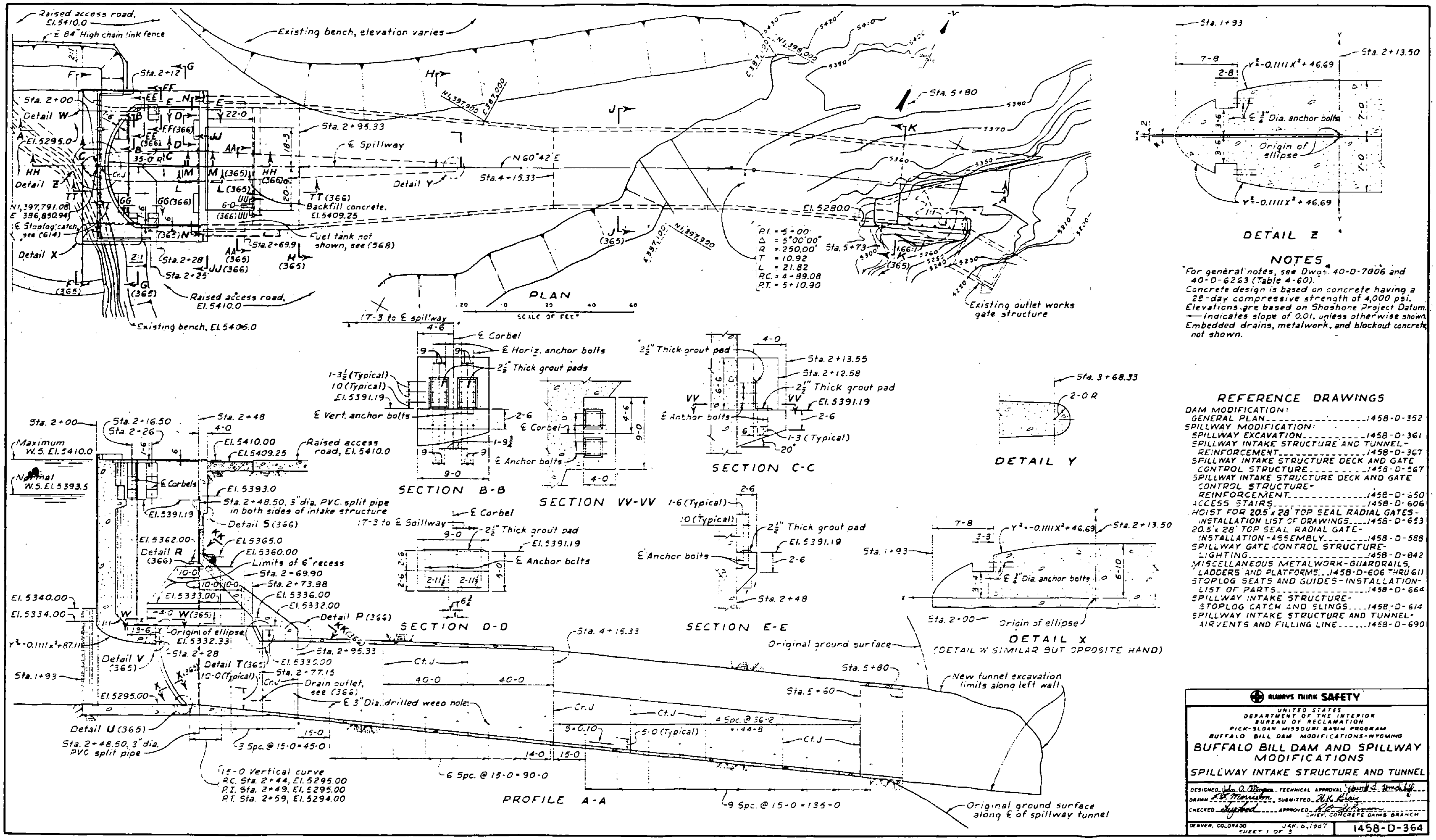
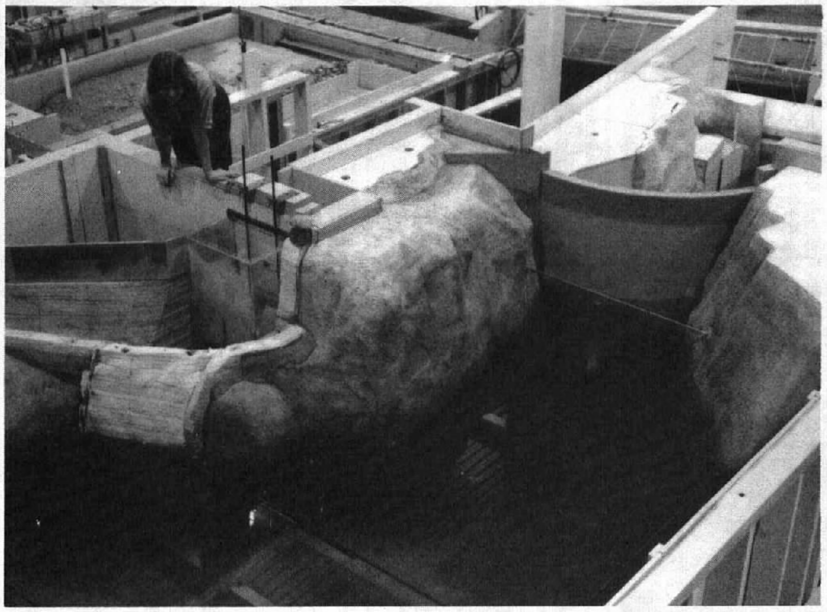
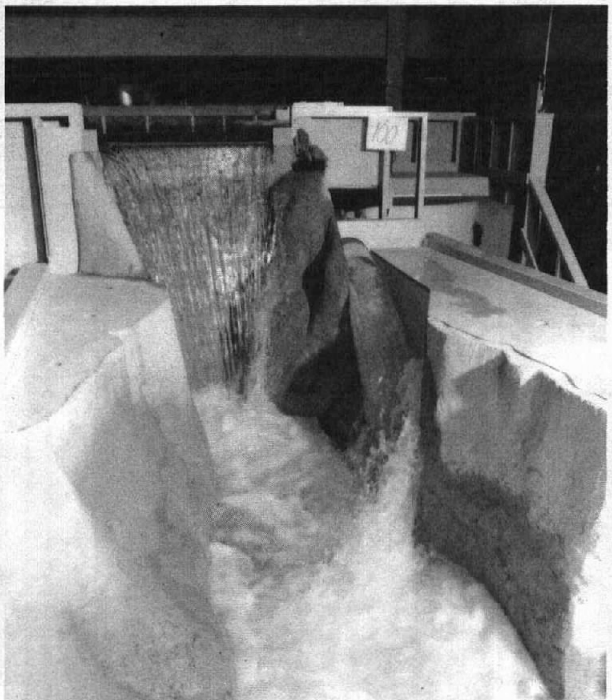



Figure 4. – Final modifications for spillway intake structure. (Sheet 2 of 2).



(a) View of the tunnel intake, dam, and reservoir in the 1:42.783 scale model of Buffalo Bill Dam.




(b) View of the downstream channel in the 1:42.783 scale model of Buffalo Bill Dam.

Figure 5. – Overall view of 1:42.783 scale model of Buffalo Bill Dam.

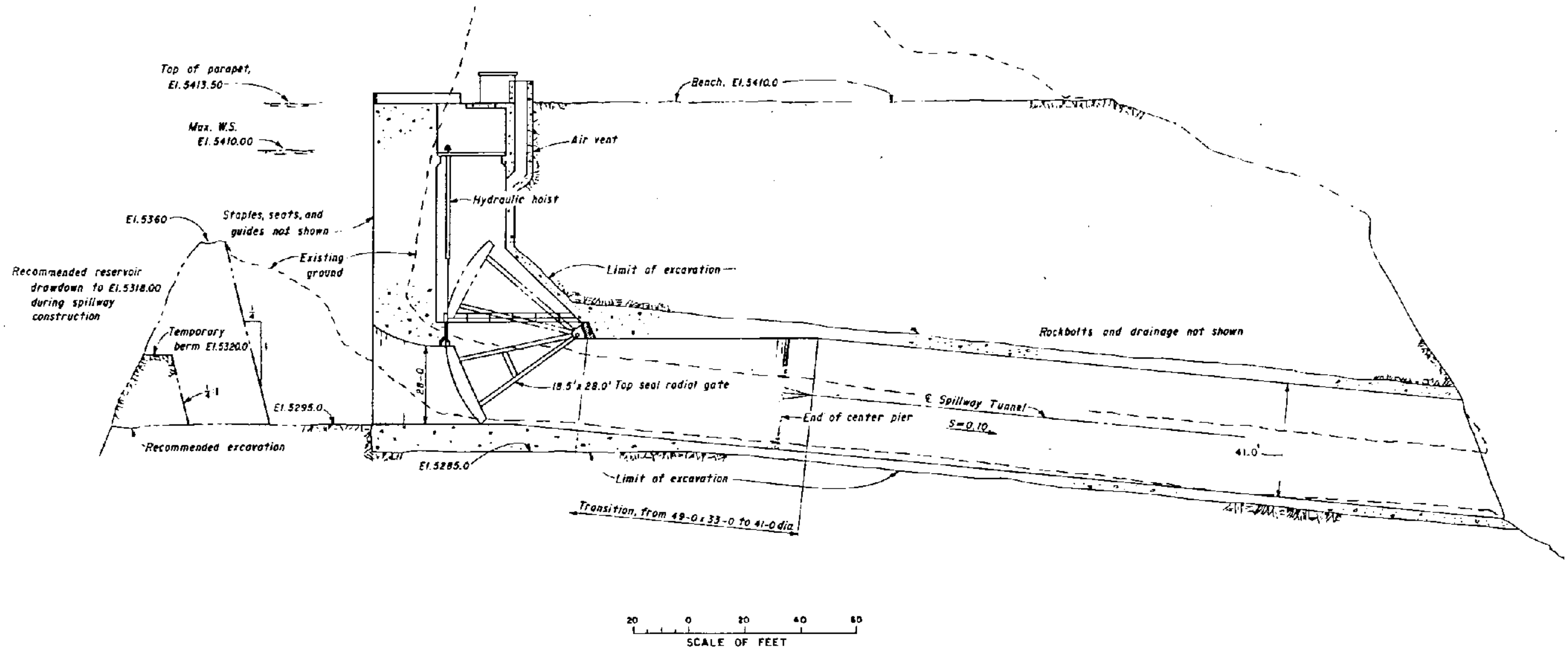



Figure 6. – Proposed gate chamber location and tested excavations.

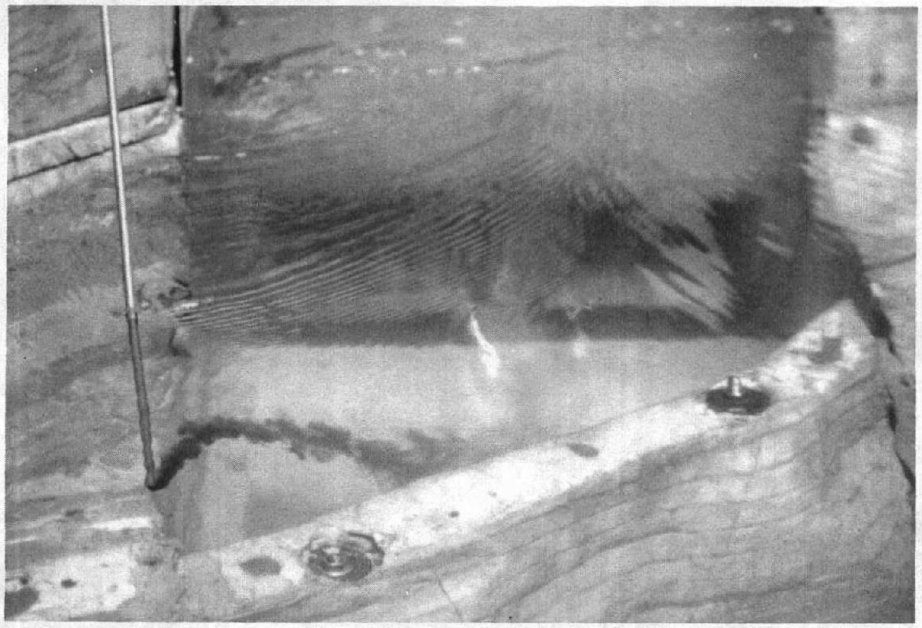


Figure 7. – Gate chamber approach channel with existing side channel weir at elevation 5360.0.

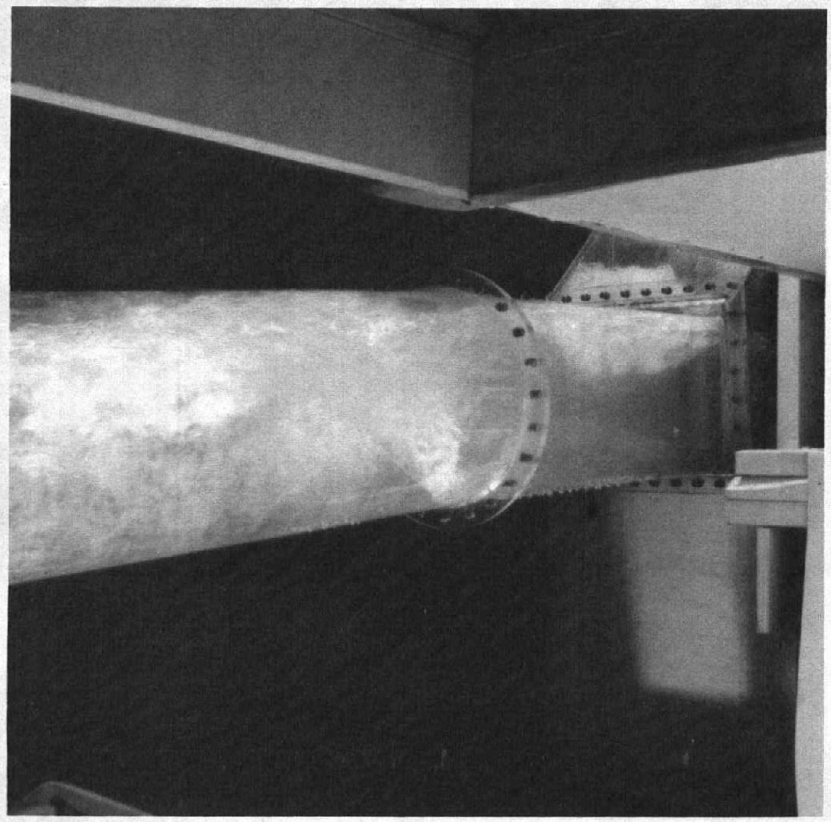


Figure 8. – Impingement on gate trunnions with existing side channel weir at elevation 5360.0.

Figure 9. – Gate chamber approach channel with berm at elevation 5320.0.

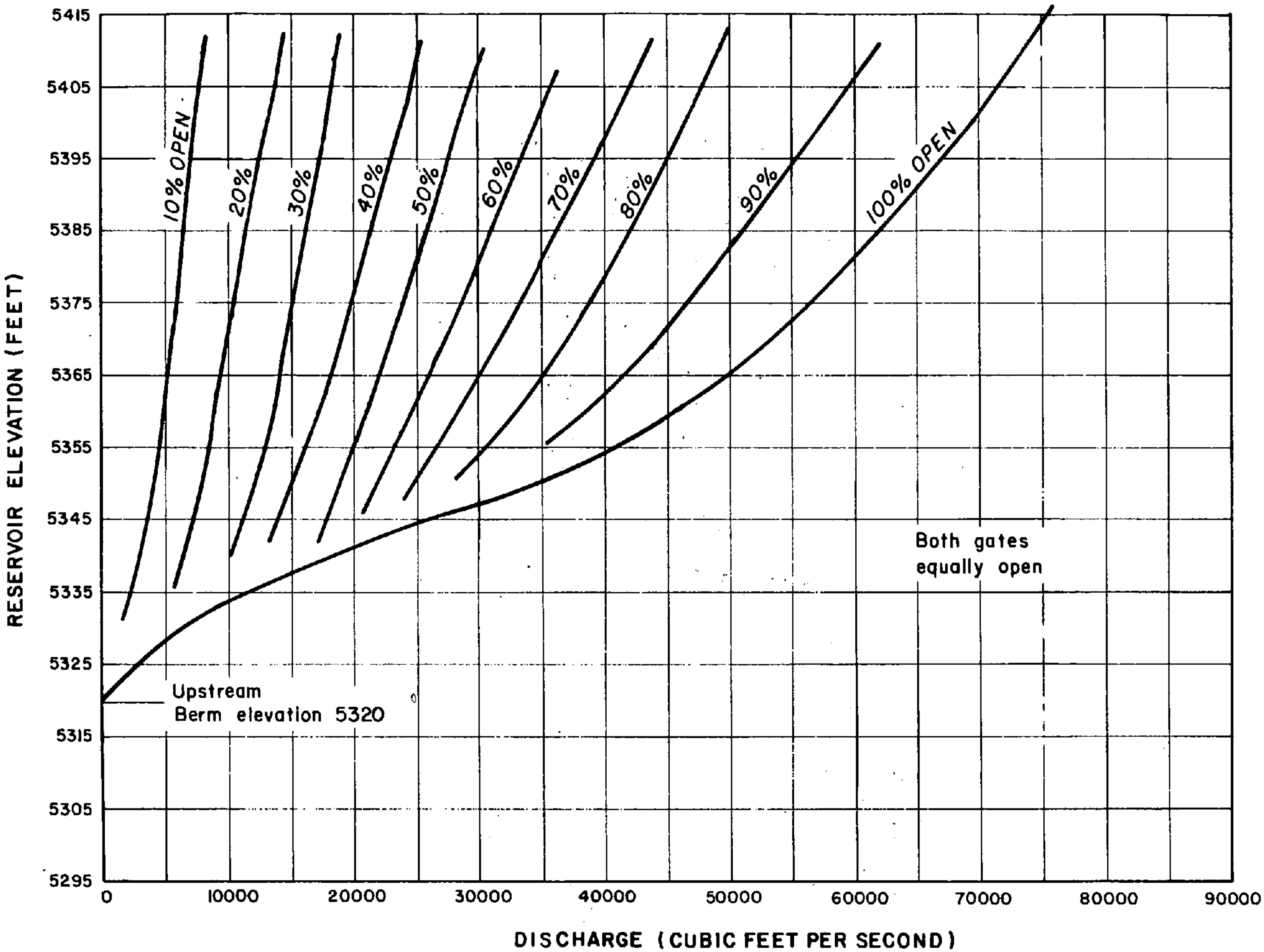



Figure 10. – Discharge curves for original gate and tunnel design.

Figure 11. – Recommended approach channel excavation.

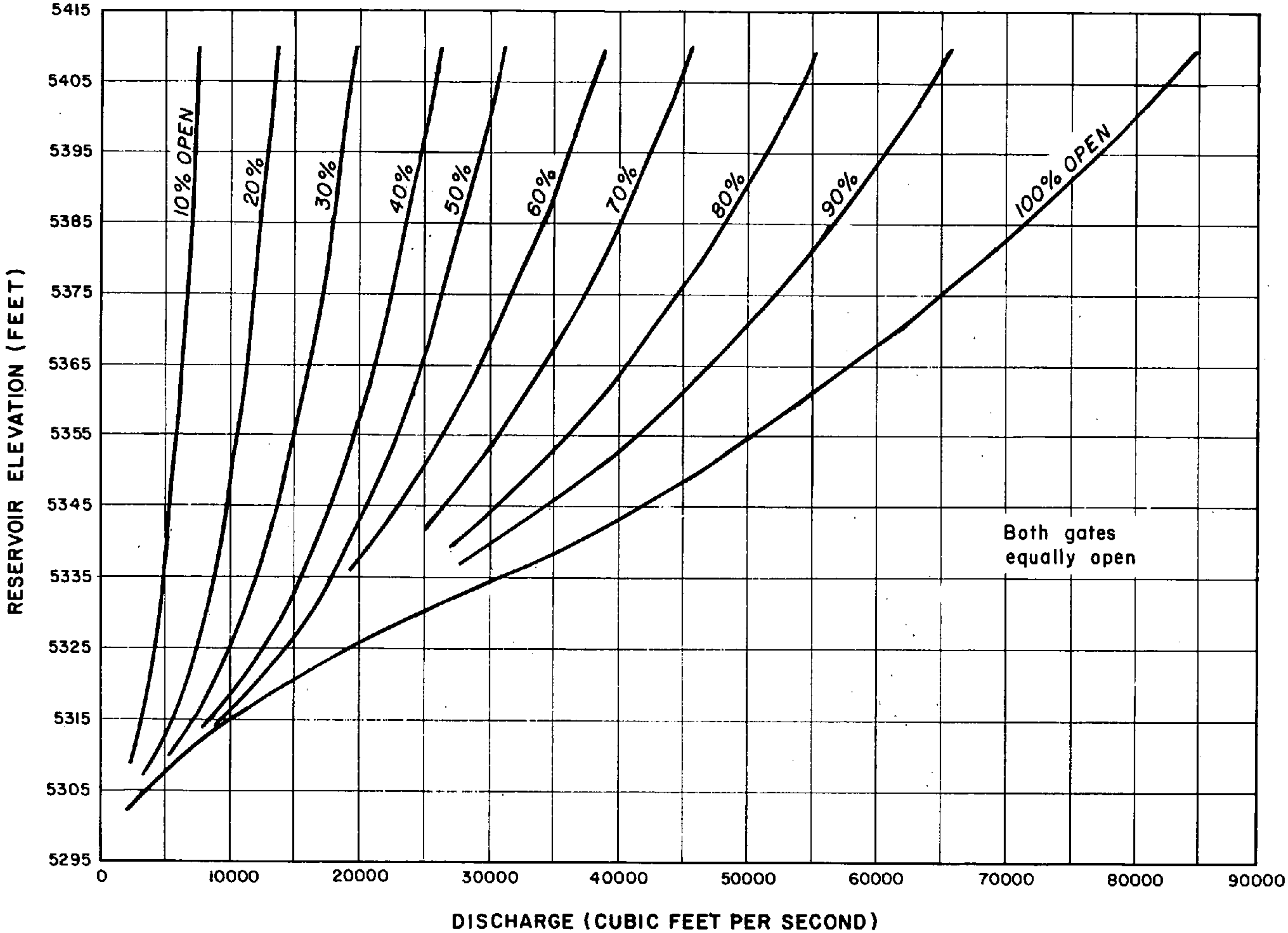



Figure 12. – Discharge curves for final gate and tunnel design.

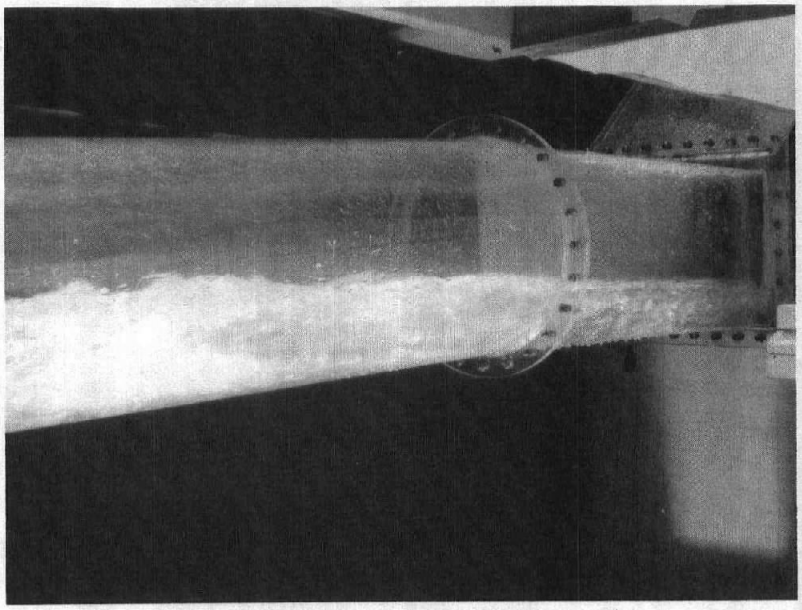


Figure 13a. – Original tunnel design, gates 10 percent open.

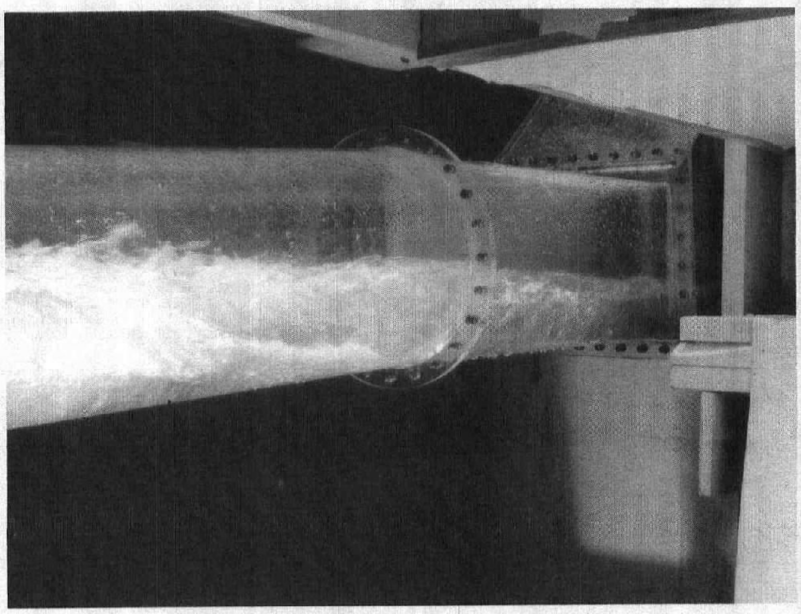


Figure 13b. – Original tunnel design, gates 50 percent open.

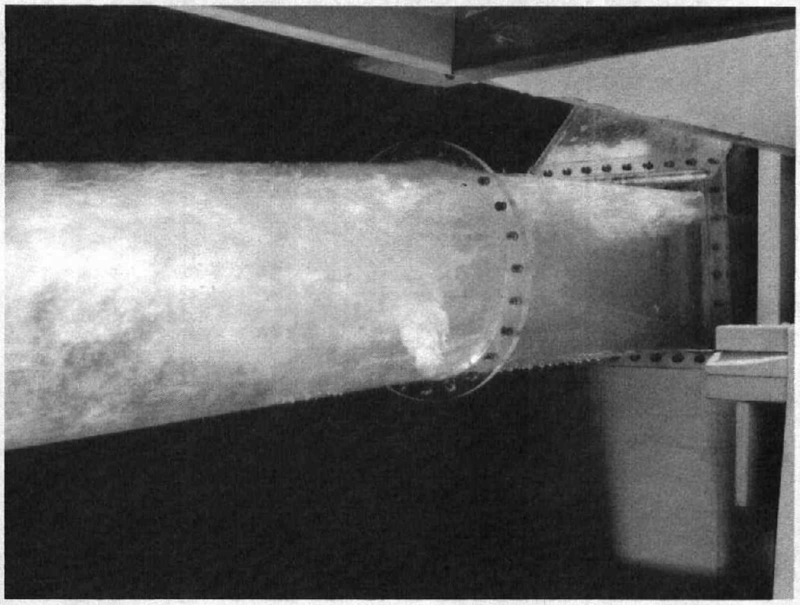


Figure 13c. – Original tunnel design, gates 100 percent open.

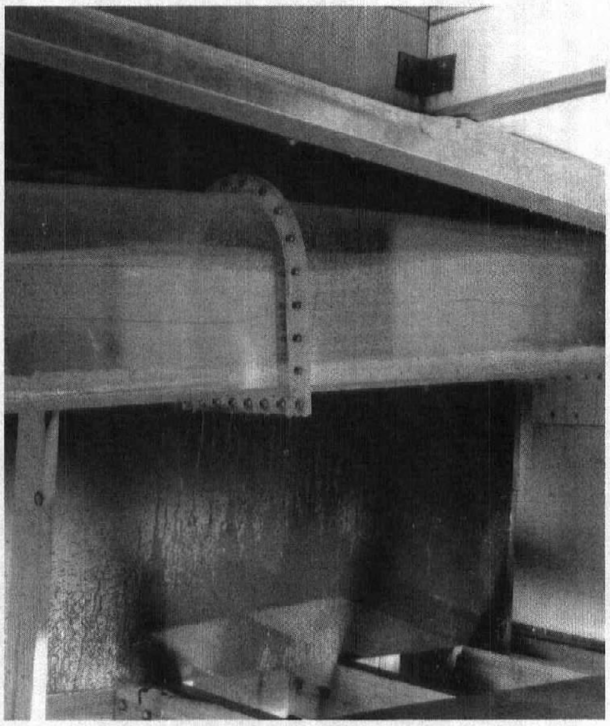


Figure 14a. – Final tunnel shape, gates 10 percent open.

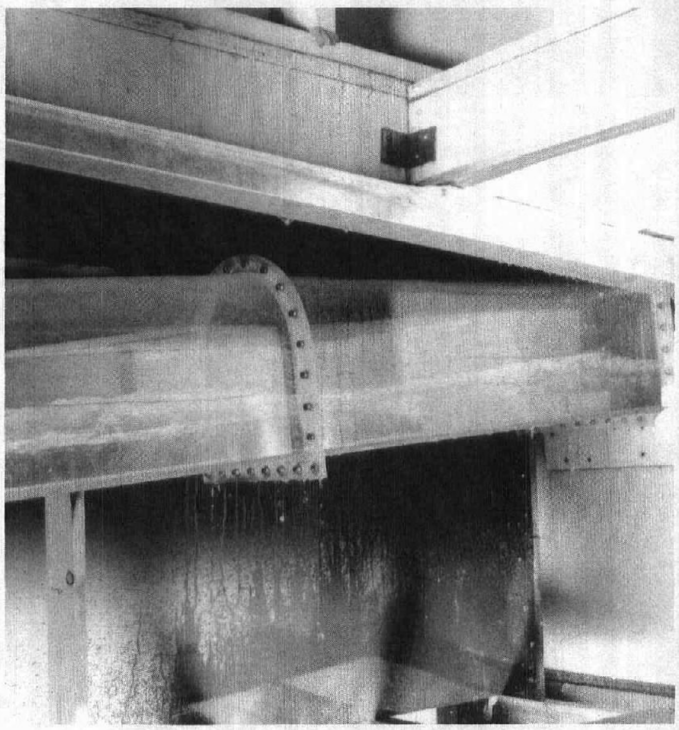


Figure 14b. – Final tunnel shape, gates 50 percent open.

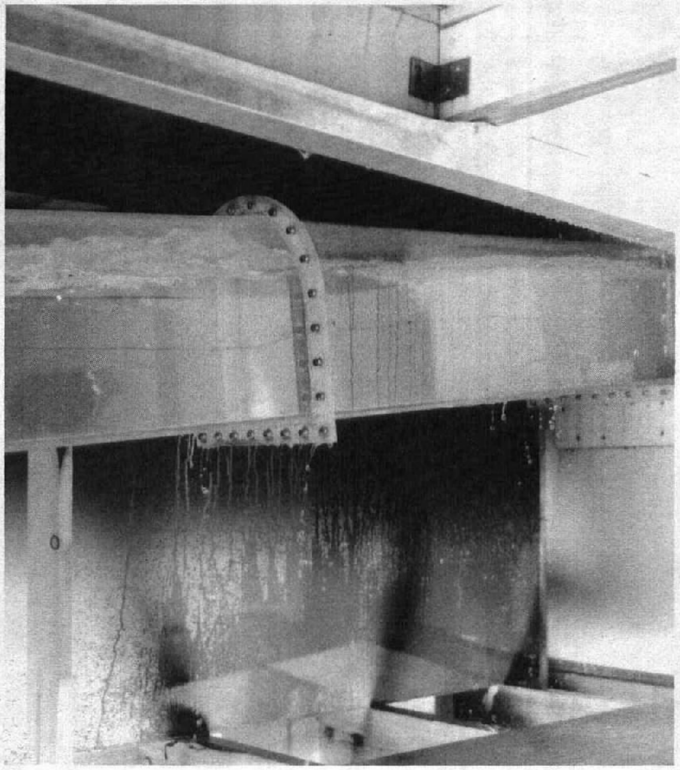


Figure 14c. – Final tunnel shape, gates 100 percent open.

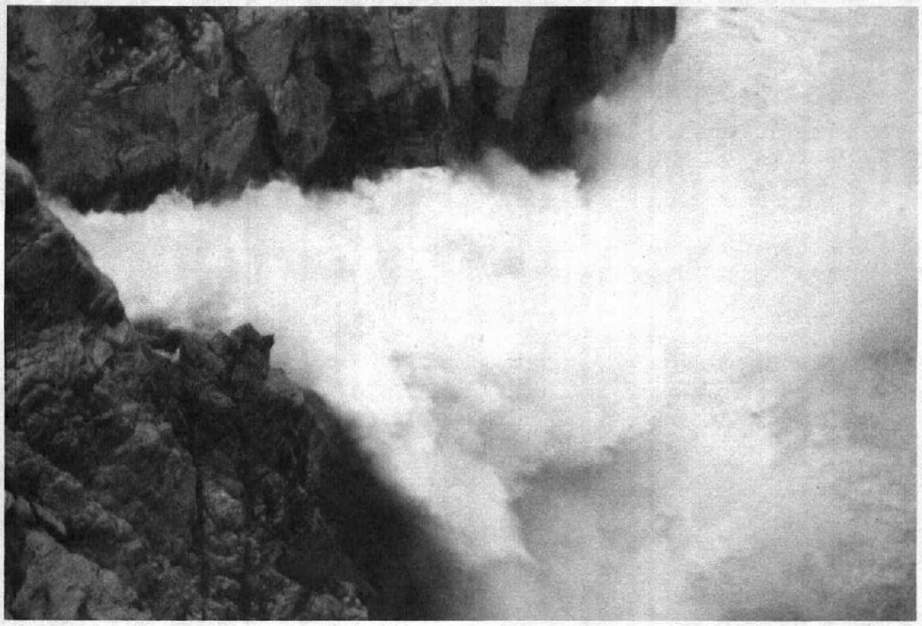


Figure 15. – Existing spillway tunnel discharging approximately 6,000 $ft^3/s$.

Figure 16. – Excavated channel downstream of existing spillway tunnel.

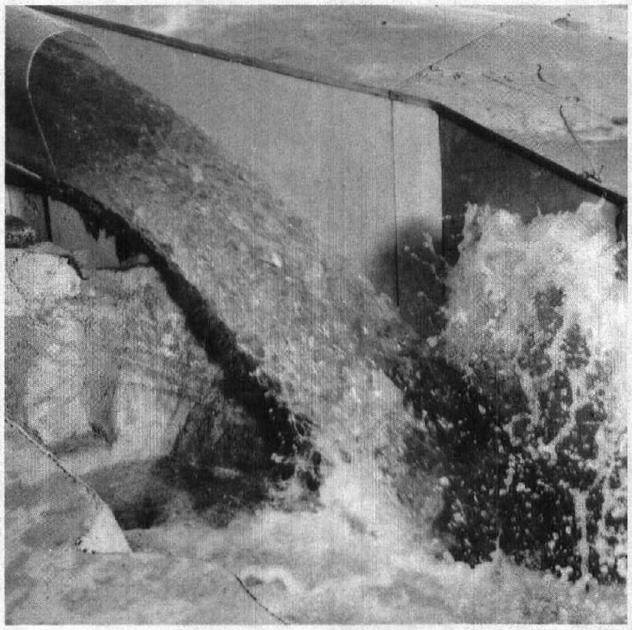


Figure 17. – Spillway jet impinging on canyon wall before realignment of the end of the tunnel, $Q$ = 75,000 ft³/s.

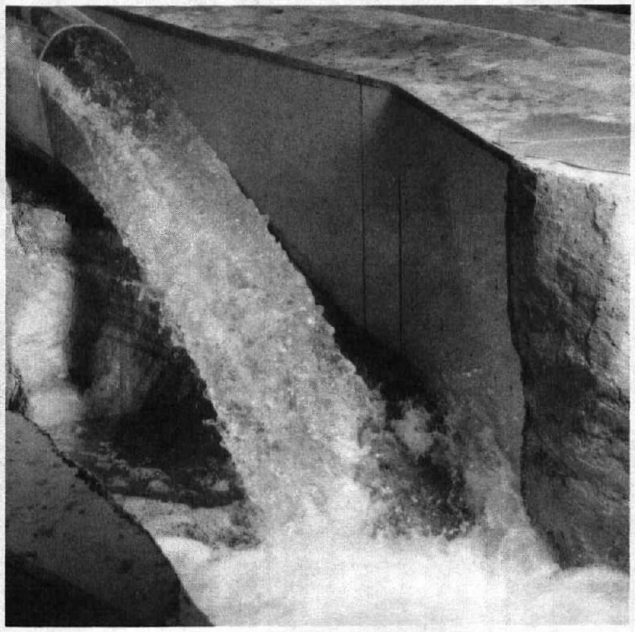


Figure 18. – Spillway jet impinging at the base of the cliff after realignment of the tunnel, $Q$ = 75,000 ft³/s.

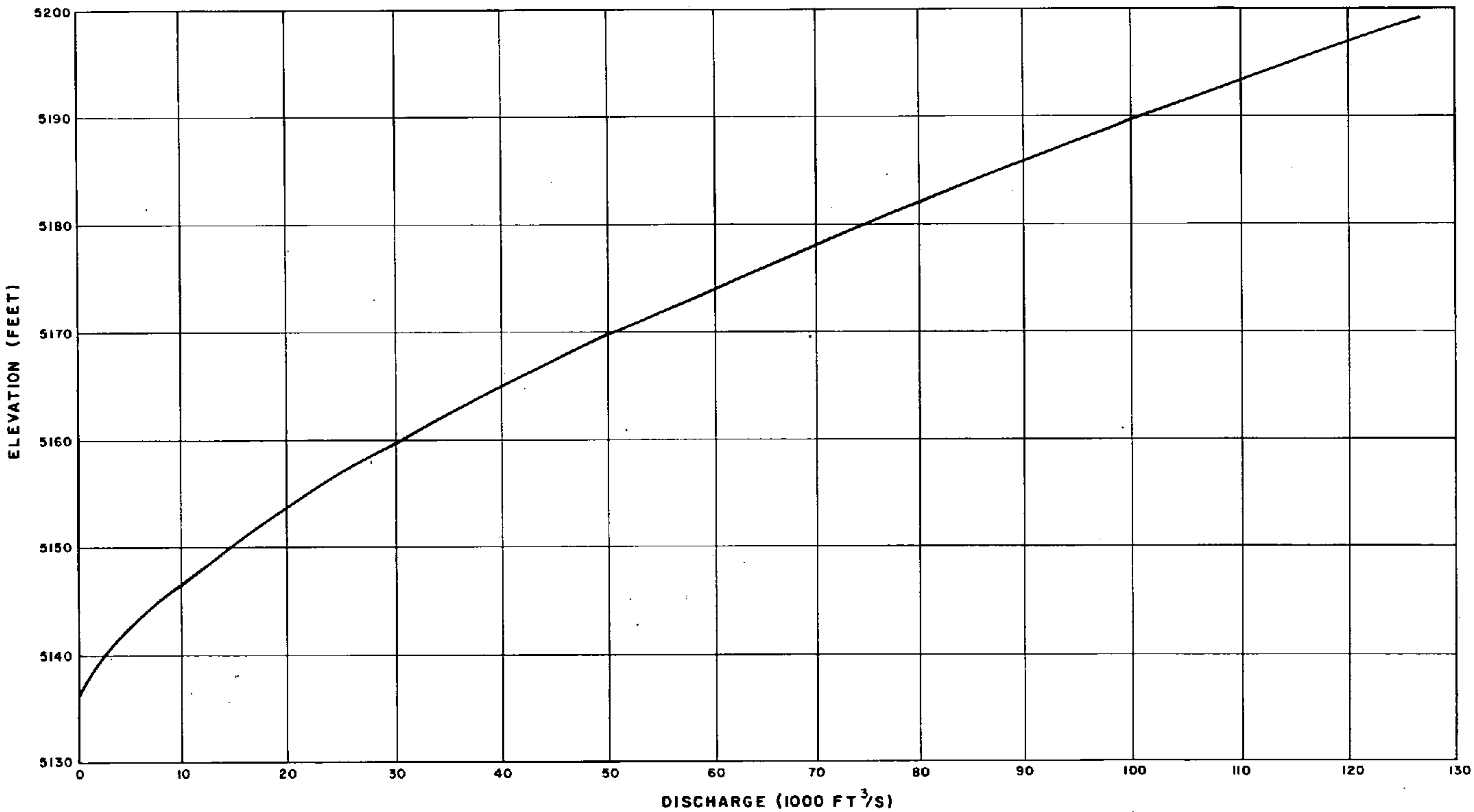



Figure 19. – Tailwater rating curve for the river reach downstream of the dam.

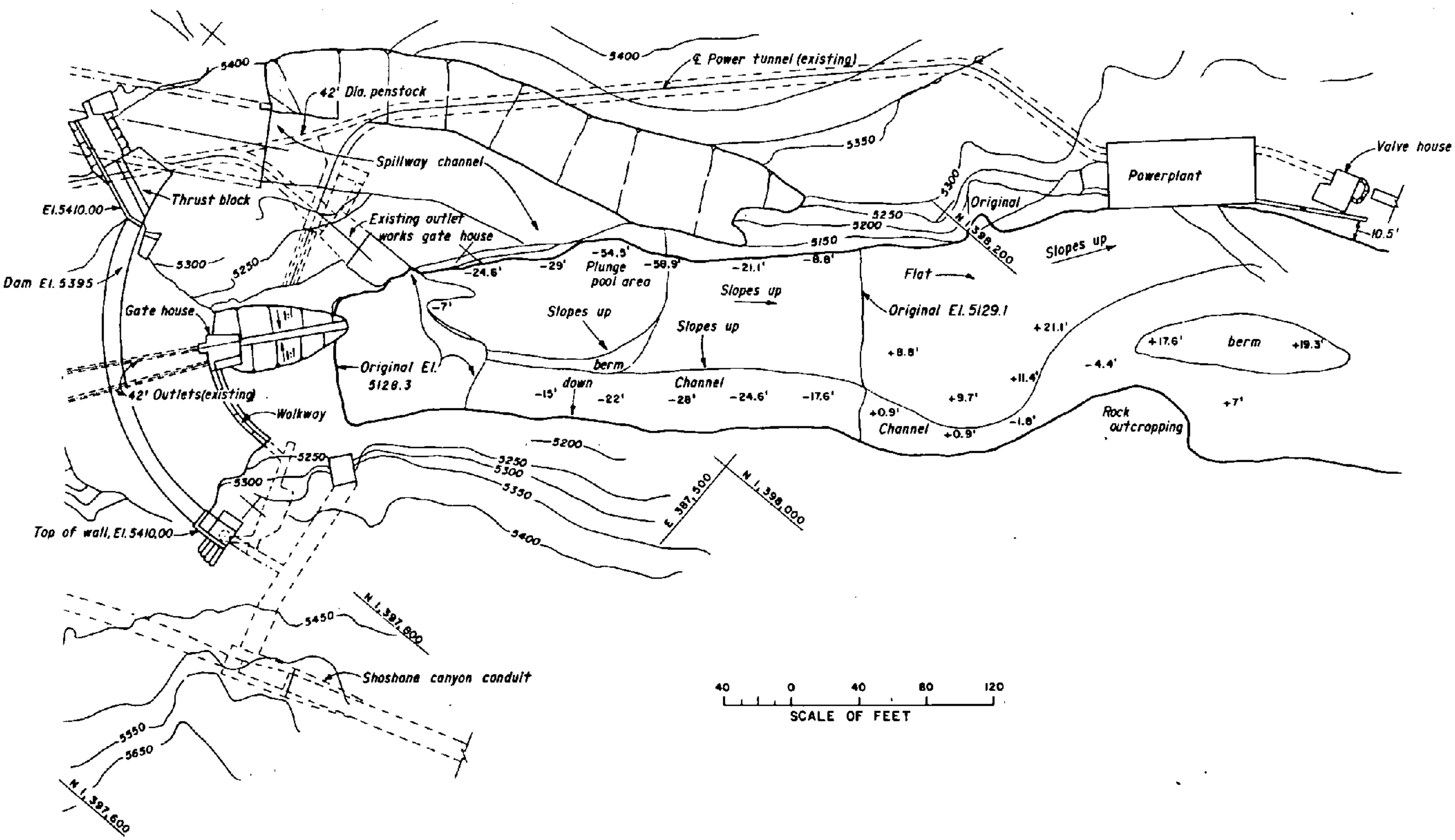



Figure 20. – Erosion and deposition patterns for a 20-percent spillway gate opening.

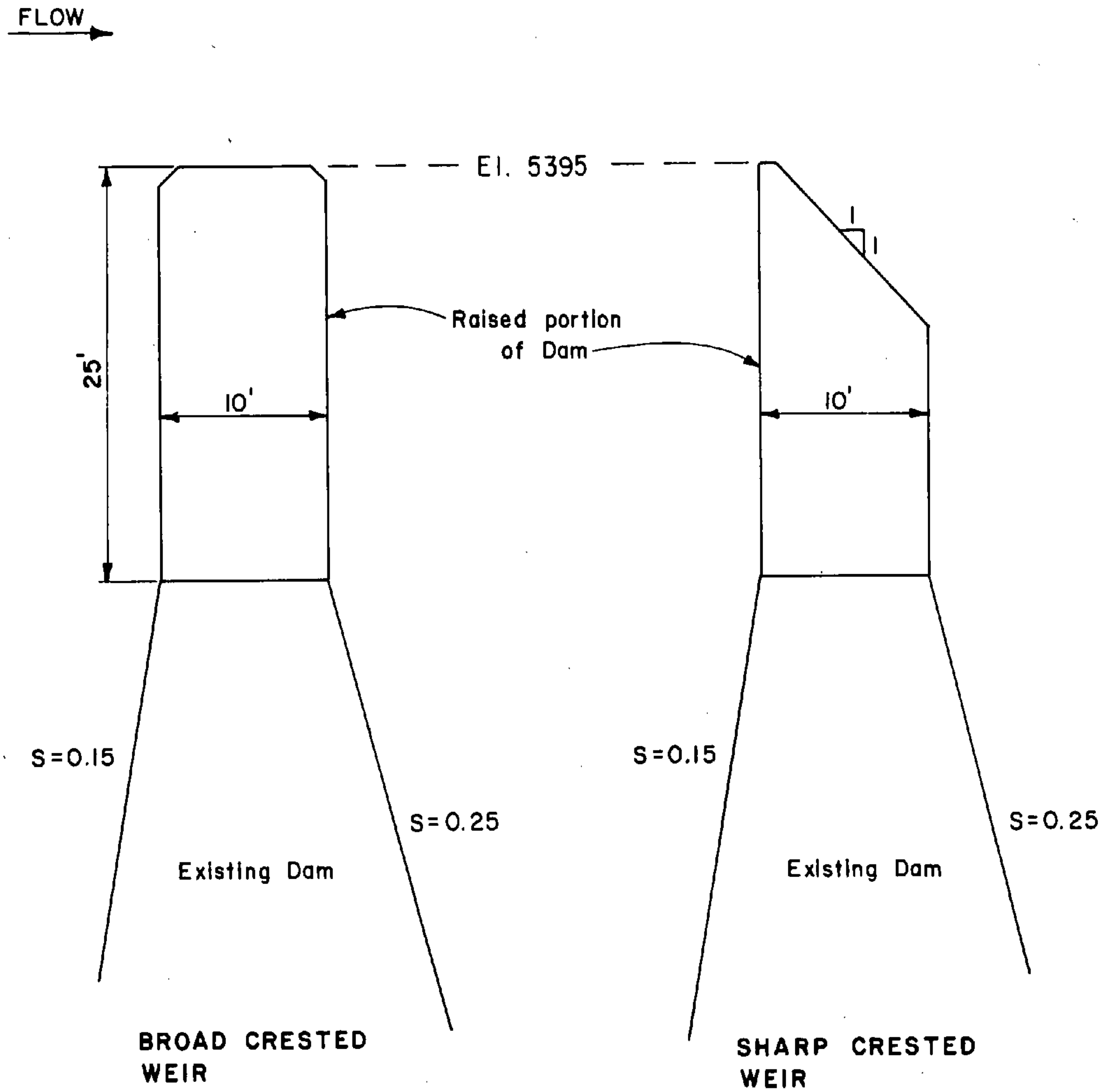



Figure 21. – Shapes tested for the top of the dam.

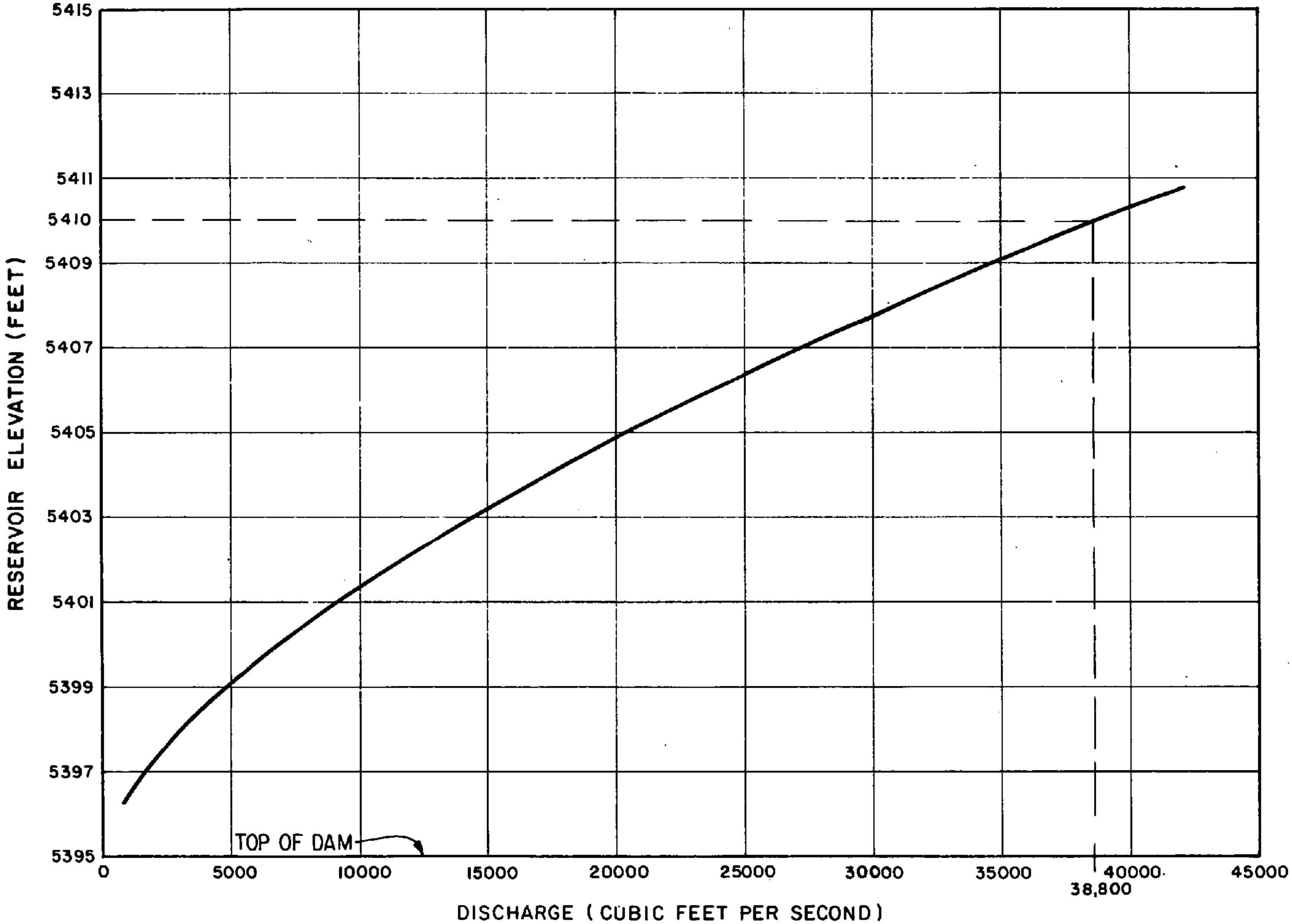



Figure 22. – Discharge curve for dam overtopping with the broad crest shape.

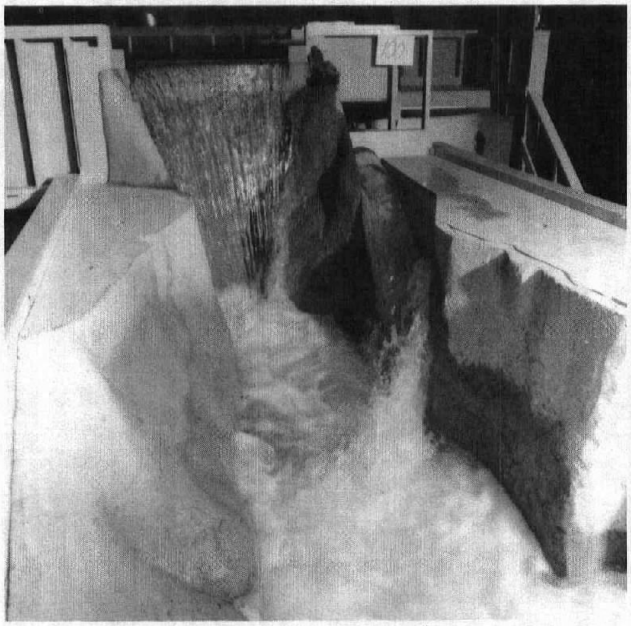


Figure 23. – Flow overtopping the broad crest shape.

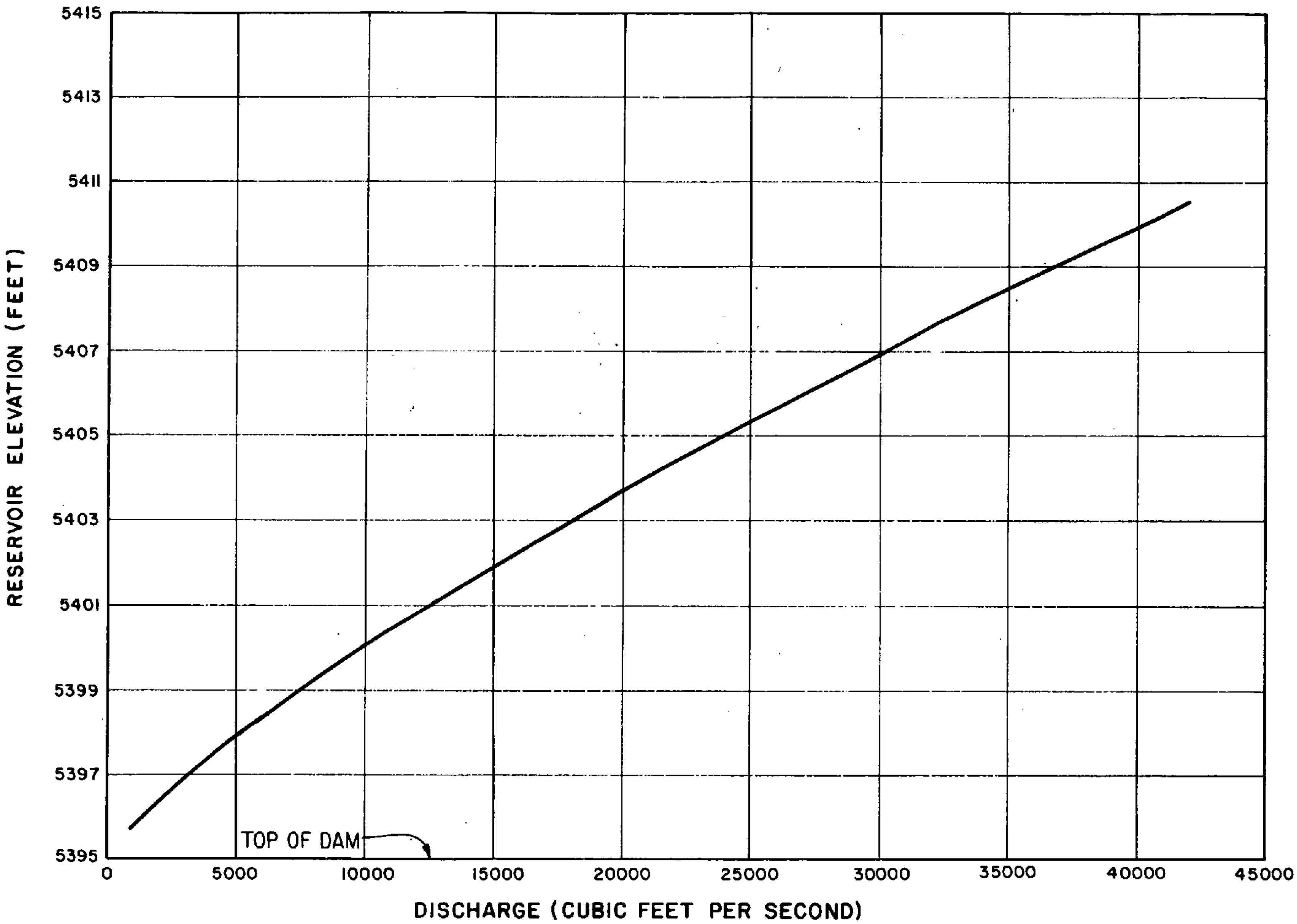



Figure 24. – Discharge curve for dam overtopping with the sharp crest shape.

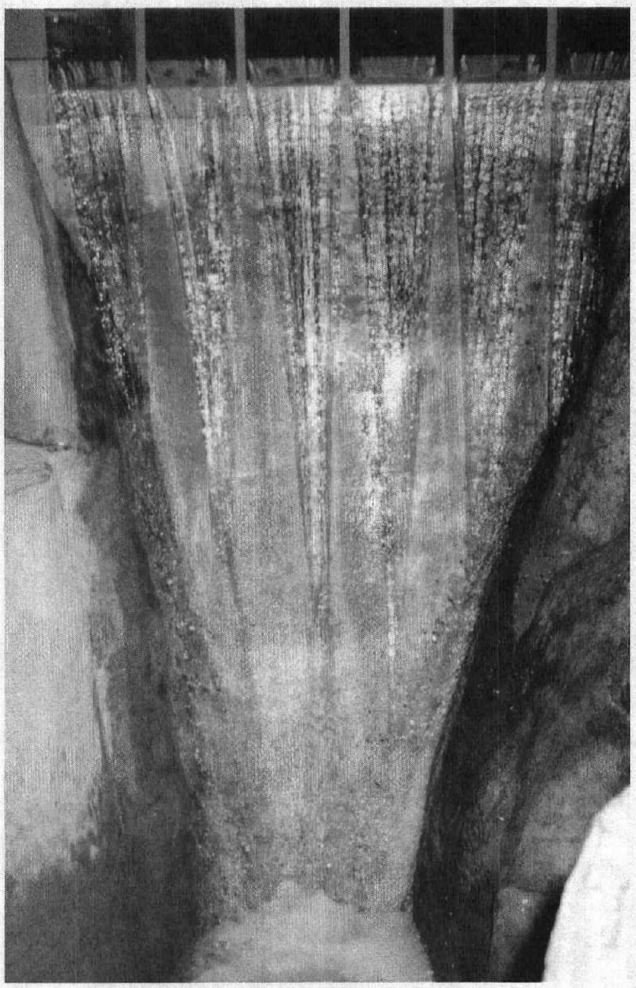


Figure 25. – Flow overtopping the sharp crest shape.



GPO 854-522

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*The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.*

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*Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.*

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