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HYDRAULIC MODEL STUDIES OF CRYSTAL DAM SPILLWAY AND OUTLET WORKS COLORADO RIVER STORAGE PROJECT

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16. ABSTRACT

Hydraulic model studies were made to determine the flow characteristics of the Crystal Dam spillway, plunge pool, and outlet works. The initial flip bucket spillway was tested, and the bucket exit modified by extending the 15-foot (4.57-meter) radius beyond the bucket invert to a 4:1 tangent at the bucket lip. The modified bucket was required to adequately flip the spillway jet into the plunge pool. A 15-foot (4.57-meter) high deflector wall was placed at the downstream end of the excavated rock plunge pool to deflect the high energy spillway jet from the 3 to 1 riprapped slope leading to the downstream river channel. A vortex appearing in the preliminary horizontal bellmouth transition design from the intake towers to the 54-inch (1,371-mm) outlet works conduits was eliminated by raising the floor of the intake tower closer to the invert of the bellmouth. This design was later improved by using one intake tower for both outlet conduits placed at the same elevation. Discharge coefficients are given for both the spillway and the submerged 48-inch (1,219-mm) jet-flow gates.

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by
P.H. Burgi
S. Fujimoto

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Hydraulics Branch
Division of General Research
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Denver, Colorado

UNITED STATES DEPARTMENT OF THE INTERIOR * BUREAU OF RECLAMATION

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PURPOSE

This investigation verifies the hydraulic design of the Crystal Dam spillway plunge pool, and outlet works. The coefficient of discharge of the outlet works was determined for the spillway and for the submerged jet-flow gates.

RESULTS

1. The recommended spillway, with a 15-foot (4.57-meter) radius bucket extended beyond the bucket invert to a 4:1 tangent at the bucket lip, operates satisfactorily for all discharges. The 4 to 1 slope on the spillway bucket lip is required to adequately flip the spillway jet into the plunge pool.

2. Because of the proximity of the spillway to the upstream right abutment, flow over the right side of the spillway is somewhat rough for discharges greater than 30,000 cfs (850 meter^3/sec). The recommended design shows a considerable improvement over the initial design, Figures 7 and 8.

3. The eliptical pier developed during the model study provides excellent flow conditions around the pier and along the spillway training walls, Figure 10.

4. To protect the 3 to 1 riprap slope at the downstream end of the plunge pool, a 15-foot (4.57-meter) high deflector wall with a 1:4 batter is needed on the floor of the plunge pool, Figure 16. The riprap slope should start 2 feet (0.61-meter) below the top of the wall.

5. A spillway discharge of 42,350 cfs (1,171 meters^3/sec) is attained at the design head of 16 feet (4.88 meters), Figure 4.

6. At design discharge the spillway jet impinges on the plunge pool floor approximately 278 feet (84.73 meters) from the axis of the dam.

7. The preliminary location of the bellmouth transition from the vertical intake tower to the outlet conduit resulted in a violent vortex in the bellmouth entrance, Figure 21. This vortex was eliminated by raising the floor of the intake tower closer to the bellmouth invert, Figure 19.

8. The recommended single-intake tower system operates satisfactorily.

9. A minimum submergence of 13 feet (3.96 meters) at 60 percent gate opening is required to protect the jet-flow gates from cavitation damage. This depth will also provide adequate energy dissipation for the submerged jet to prevent extreme water surface disturbance in the plunge pool.

APPLICATION

In general, results of this investigation apply to the structure studied. However, designs were developed which may be applicable to similar structures.

INTRODUCTION

Crystal Dam, Figure 1, is on the Gunnison River 6 miles (9.65 km) downstream from Morrow Point Dam and 14 miles (22.53 km) east of Montrose, Colo. Construction of Crystal Dam will complete the Curecanti Unit of the Colorado River Storage Project.

The dam ~~and reservoir~~ will serve in a regulating capacity for Morrow Point and Blue Mesa Dams, releasing a relatively constant discharge downstream. The 164.1-foot (50.02-meter) long flip-bucket spillway will have a design capacity of 41,350 cfs (1,171 meters^3/sec). Two 48-inch (1,219-mm) jet-flow gates will control river releases through the outlet works. The powerplant will have one 28,000-kilowatt generating unit.

MODEL INVESTIGATIONS

The Spillway and Plunge Pool Model

The spillway model, constructed to a scale of 1:36, included 400 feet (122 meters) of the upstream reservoir, the concrete arch dam, and 800 feet (244 meters) of the downstream river channel, Figure 2. The flip-bucket spillway was constructed of high-density (6 lb/cu foot) polyurethane and milled and sanded to the desired profile, Figure 3A. Twenty piezometers were installed in the model spillway along two radial lines to measure pressures on the spillway flow surface. The canyon topography was constructed in the model from 25-foot (7.62-meter) interval field contours. The scaled contours were cut from wood, placed in the model, and covered with metal lath. The lath was covered with approximately 3/4 inch (19 mm) of cement mortar, Figure 3B. Three pipes representing the penstock and two intake towers for the outlet works were connected between the reservoir head box and the powerplant in the tailbox. A tailgate assembly and sand trap were used to control the downstream tailwater elevation and collect eroded pea gravel. Water was supplied to the

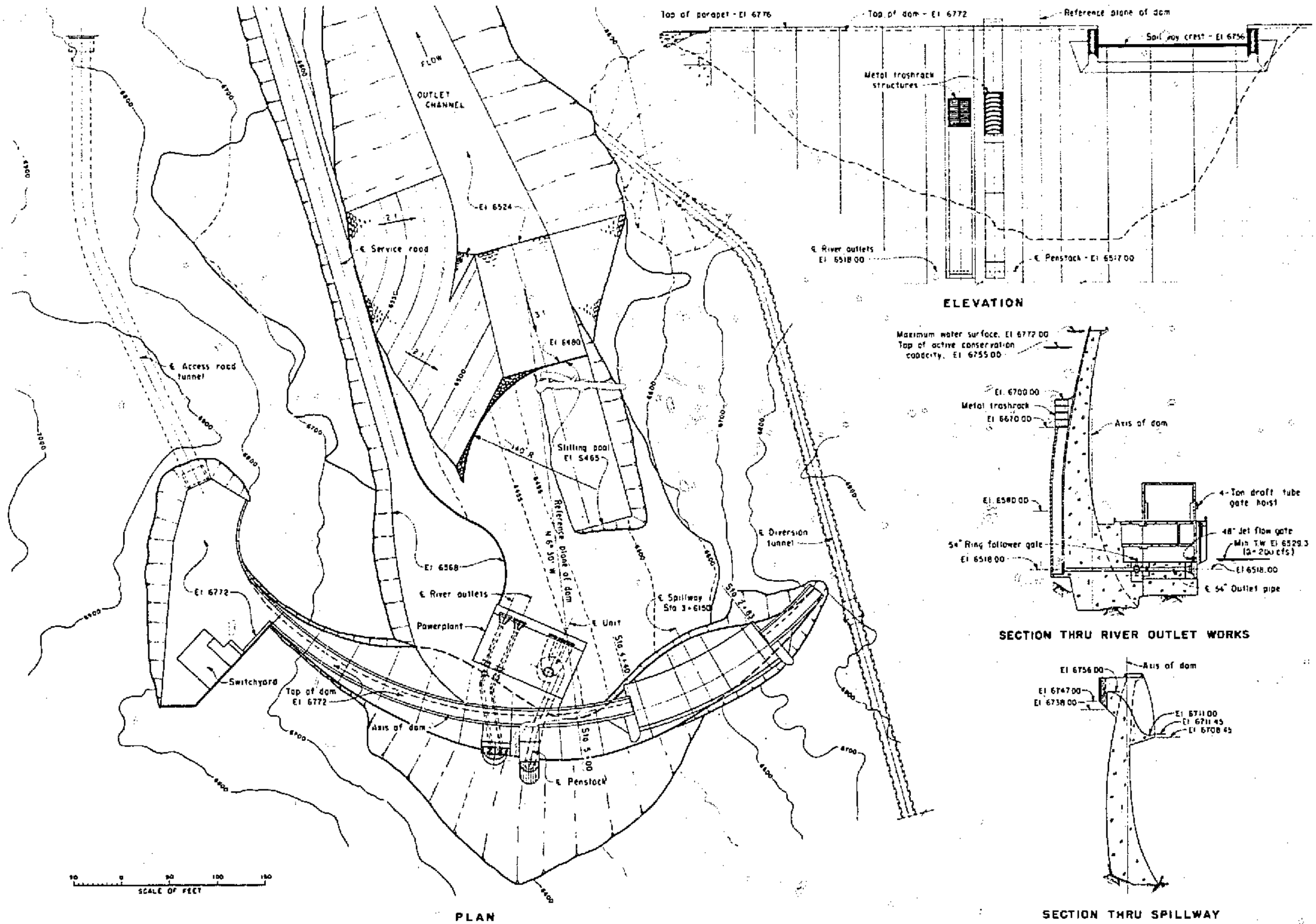


Figure 1. Crystal Dam features.

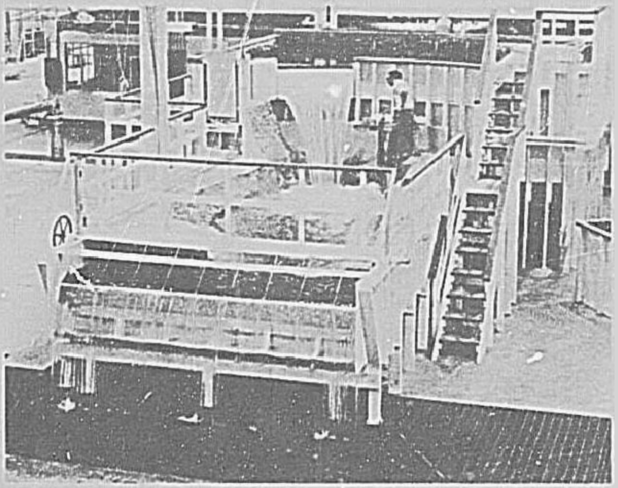
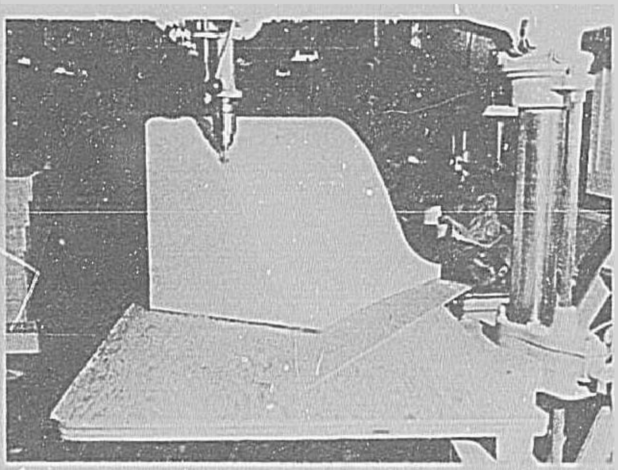
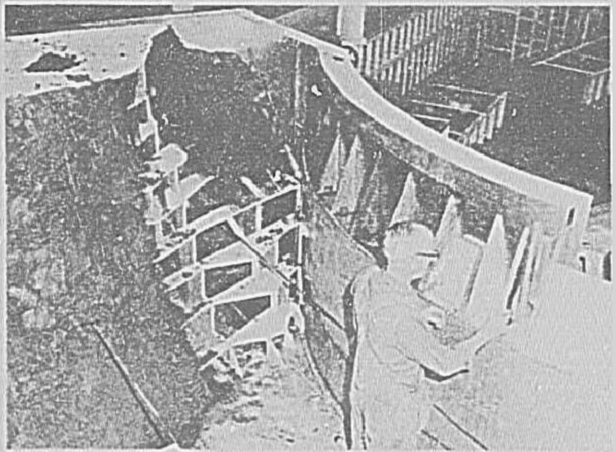


Figure 2. 1 to 36–scale spillway model. Photo P622-D-74343



A. View of polyurethane spillway in construction. Photo P622-D-74322



B. Construction of the model dam and reservoir topography in the head box. Photo P622-D-74323

Figure 3. Model construction.

model through the permanent laboratory system and was measured by one of a bank of Venturi meters installed in the laboratory.

Spillway Studies

The spillway profile.—The spillway profile was designed to minimize adverse stresses in the thin-arch dam during construction and operation and to maximize the discharge coefficient. The spillway profile was described by the equation:

$$x^2 = 24y$$

where

x = horizontal distance from the crest, and
y = vertical distance below the crest.

which terminated in a 15-foot (4.57-meter) radius, horizontal exit bucket. Figure 4 shows the head-discharge curve for the recommended spillway profile. The design head of 16 feet (4.88 meters) yields a spillway discharge of 41,350 cfs (1,171 meters^3/sec). The design discharge coefficient is 3.92.

The spillway profile was designed for a partial vacuum crest at the design head of 16 feet (4.88 meters). Piezometric pressures were measured on the spillway profile along two radial lines. One line was near the center of the spillway and the other line was 27 feet (8.23 meters) from the left training wall. Pressure measurements along the two lines were essentially equal. Figure 5 describes the water manometer pressures for several discharges and the maximum water surface profiles in the center and along the training walls. The minimum pressure of minus 2.6 feet (0.79 meter) was recorded at piezometer 2 for the design discharge of 41,350 cfs (1,171 meters^3/sec). Flow depths along the spillway centerline profile were measured for several discharges less than design and are presented in Figure 6.

Reservoir approach conditions.—The initial spillway location produced a very poor spillway flow condition for discharges above 30,000 cfs (850 meters^3/sec), Figure 7A. Figure 7B shows the disturbance to the reservoir water surface on the right one-third of the spillway approach. The protrusion of the canyon wall in the foreground of the photograph prevented the flow from uniformly approaching the spillway, as indicated by the flow lines of confetti.

To improve the reservoir approach the protrusion was cut back along a 150-foot (45.7-meter) radius, tangent to a line passing along the right training wall. The cut extended to a horizontal bench 10 feet (3.05 meters)

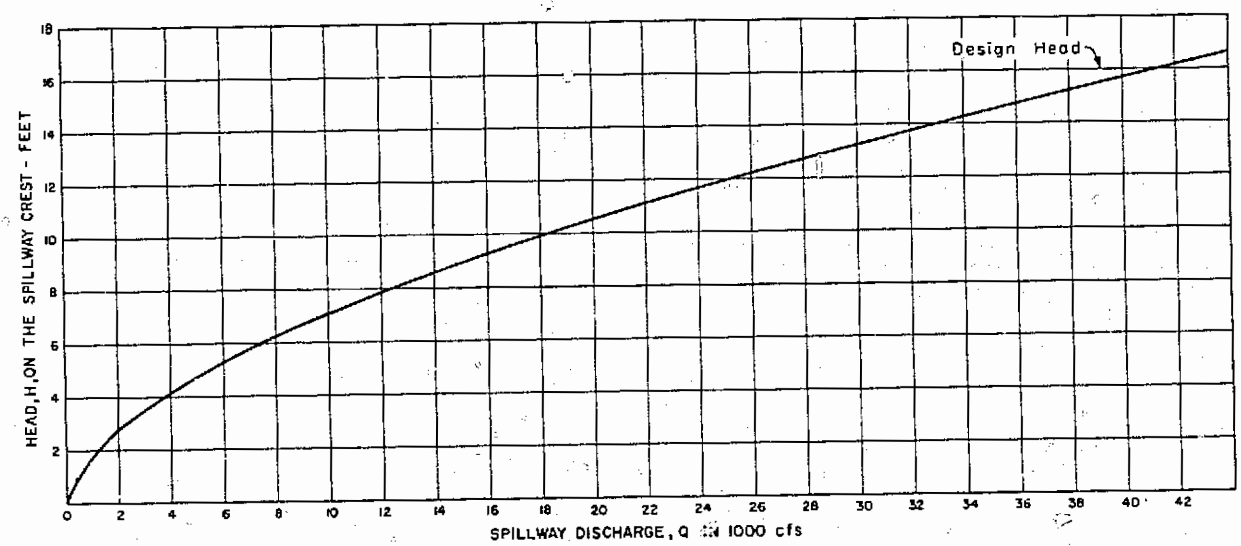


Figure 4. Spillway discharge versus head on spillway crest. 1:36—scale model.

below the spillway crest elevation. Figures 8A and 8B illustrate the improved flow conditions on the spillway and along the reservoir approach.

Later in the studies the axis of the dam was realined and the spillway was moved 30 feet (9.14 meters) toward the center of the dam and away from the right abutment. This new alinement should provide good flow distribution without removing the protrustion described above.

Pier nose design.—Several pier designs were tested in an attempt to reduce the abrupt water surface drawdown around the pier. These designs are shown in Figure 9. The preliminary pier design turned the flow too abruptly and caused a very pronounced drawdown around the pier, resulting in a very rough flow condition along the training wall as the flow accelerated over the spillway, Figure 10A. The recommended elliptical pier was developed based on work by Rouve*. For a river intake, Rouve found the following limits yielded optimum flow conditions near the pier.

$$0.85B \leq L \leq 0.95B$$

$$0.65B \leq B_2 \leq 0.72B$$

B, B_2, and L are shown in Figure 9.

Equations for the four radii, R_1, R_2, R_3, and R_4, Figure 9, which best approximate Rouve's ellipse are:

$$R_1 = \left[(L-B_2) + \frac{\sqrt{L^2+B_2^{\,2}} - (L-B_2)}{2}\right]\left[\frac{\sqrt{L^2+B_2^{\,2}}}{B_2}\right]$$

$$R_2 = \left[\frac{\sqrt{L^2+B_2^{\,2}} - (L-B_2)}{2}\right]\left[\frac{\sqrt{L^2+B_2^{\,2}}}{L}\right]$$

$$R_3 = \left[(B_1-L_1) + \frac{\sqrt{L_1^{\,2}+B_1^{\,2}} - (B_1-L_1)}{2}\right]\left[\frac{\sqrt{L_1^{\,2}+B_1^{\,2}}}{L_1}\right]$$

$$R_4 + \left[\frac{\sqrt{L_1^{\,2}+B_1^{\,2}} - (B_1-L_1)}{2}\right]\left[\frac{\sqrt{L_1^{\,2}+B_1^{\,2}}}{B_1}\right]$$

where

$$L_1 = \frac{B_1^{\,2}}{R_2}$$

To minimize adverse stresses in the thin arch, the pier width, B, was selected as 10 feet (3.05 meters). Based on the previous limits, the dimensions L and B_2 were selected as 9 feet (2.74 meters) and 6.8 feet (2.07 meters), respectively. Figure 10B shows the improved flow along the training wall with the elliptical pier.

Flip bucket.—The preliminary spillway profile, with the horizontal exit from the bucket lip, did not flip the spillway discharge an adequate distance from the dam

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*Rouve, Dr. Von Ing. Gerhard, Der Krafthaustrennpfeiler, Stromung sverhaltnisse an gekrummten Wanden, Januar 1958.

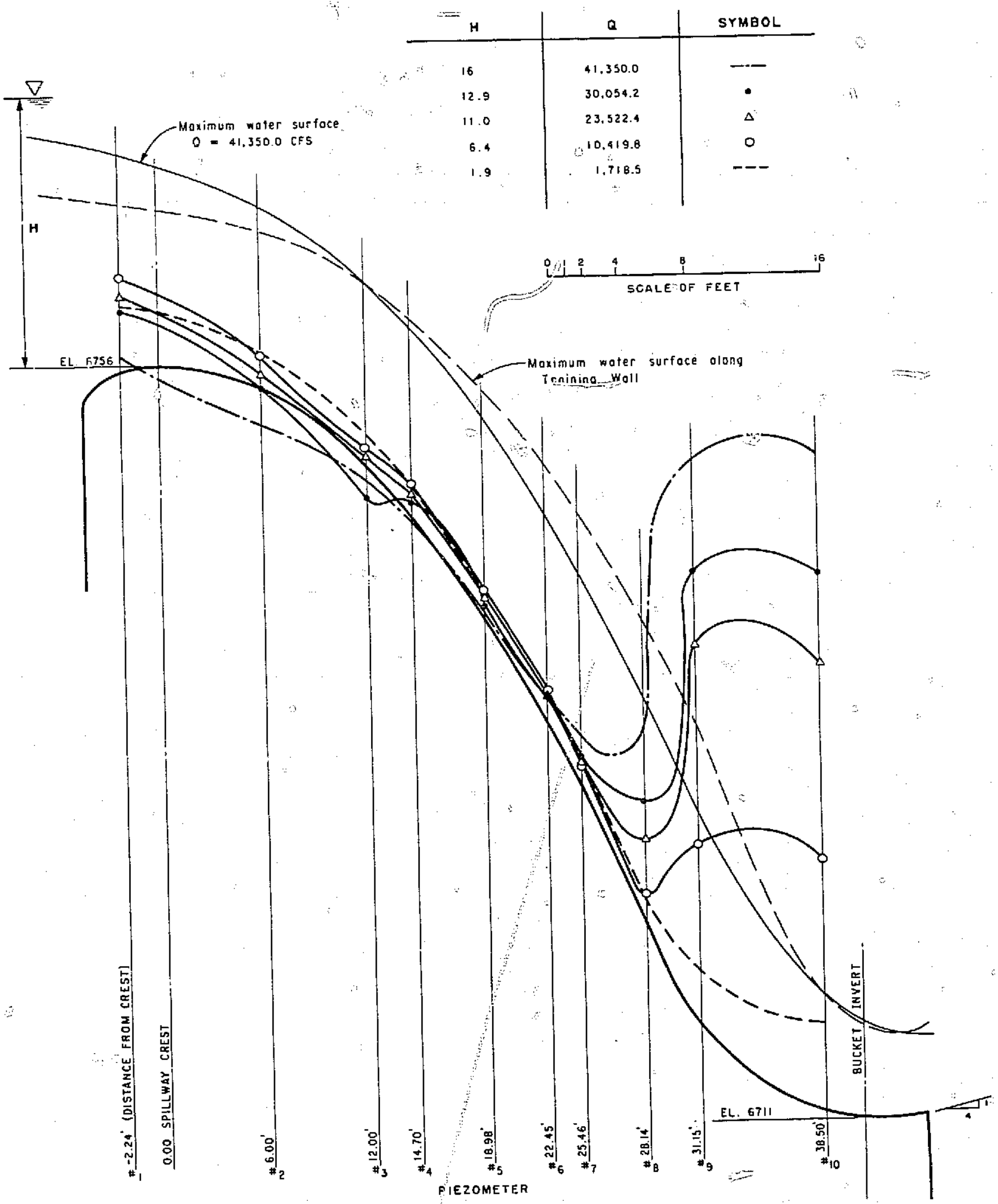


Figure 5. Water manometer pressures on spillway surface. 1:36–scale model.

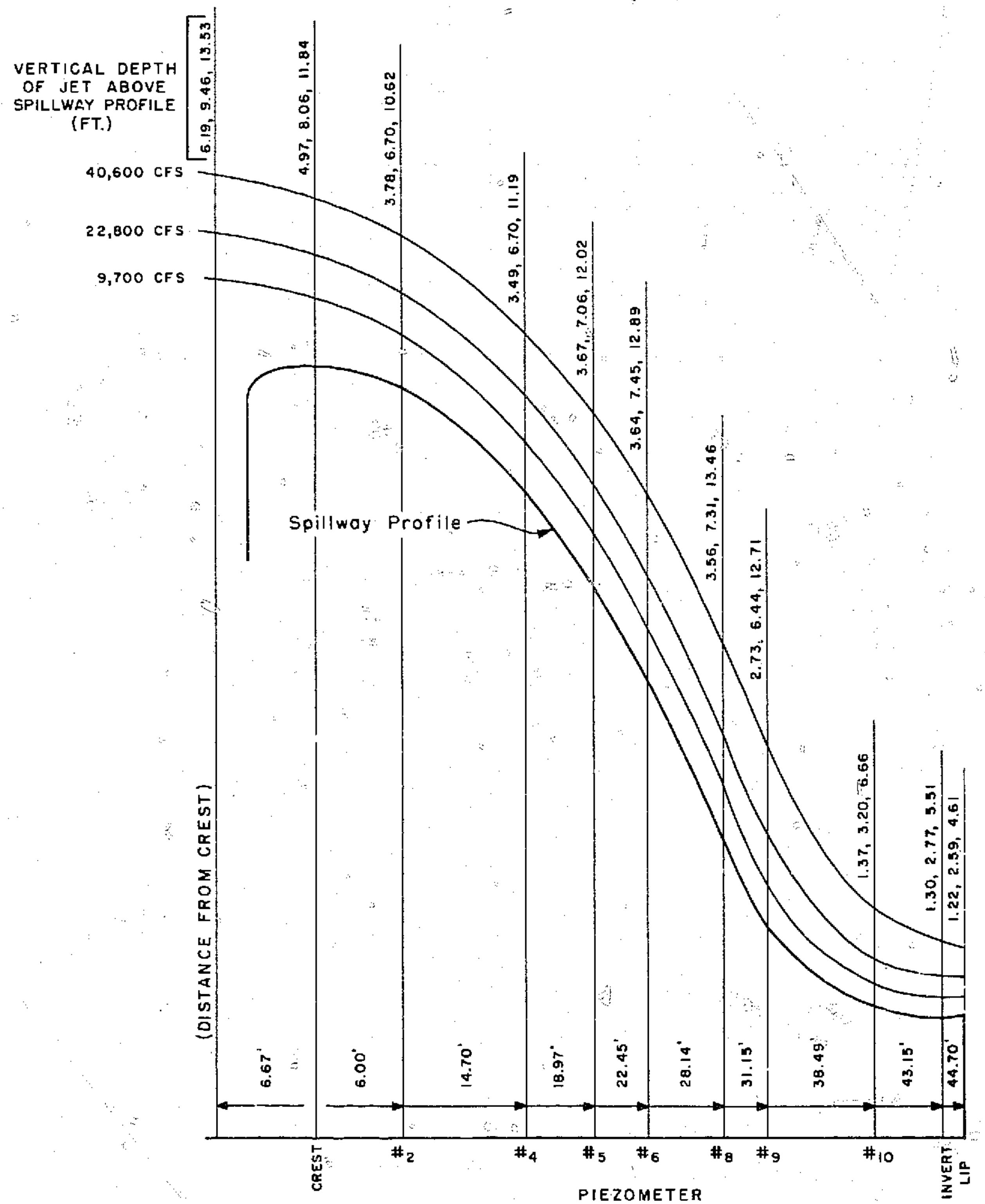
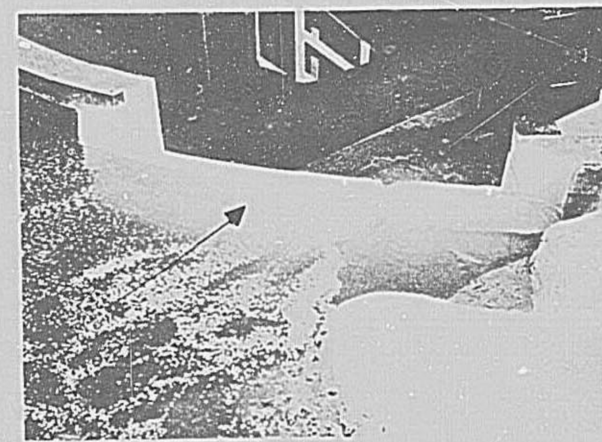


Figure 6. Depth of spillway jet. 1:36–scale model.

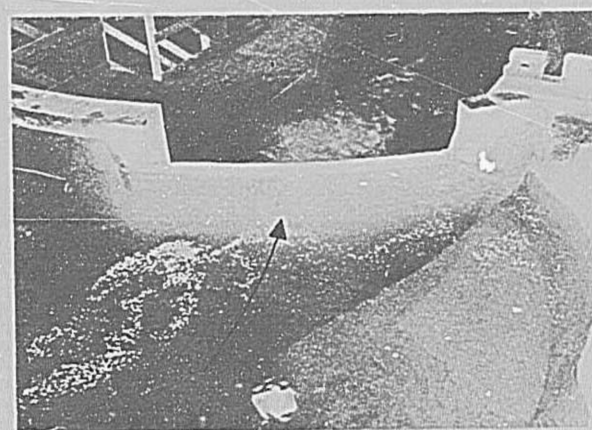
A. Spillway discharging 41,350 cfs. Initial plunge pool design. Photo P622-D-74325



B. Reservoir approach condition. Note the canyon wall protrusion in the center foreground. Photo P622-D-74327

Figure 7. Initial reservoir approach.

A. Spillway discharging 41,350 cfs. Initial plunge pool design. Photo P622-D-74331



B. Reservoir approach conditions. Note improved flow over the spillway. Photo P622-D-74329

Figure 8. Improved reservoir approach.

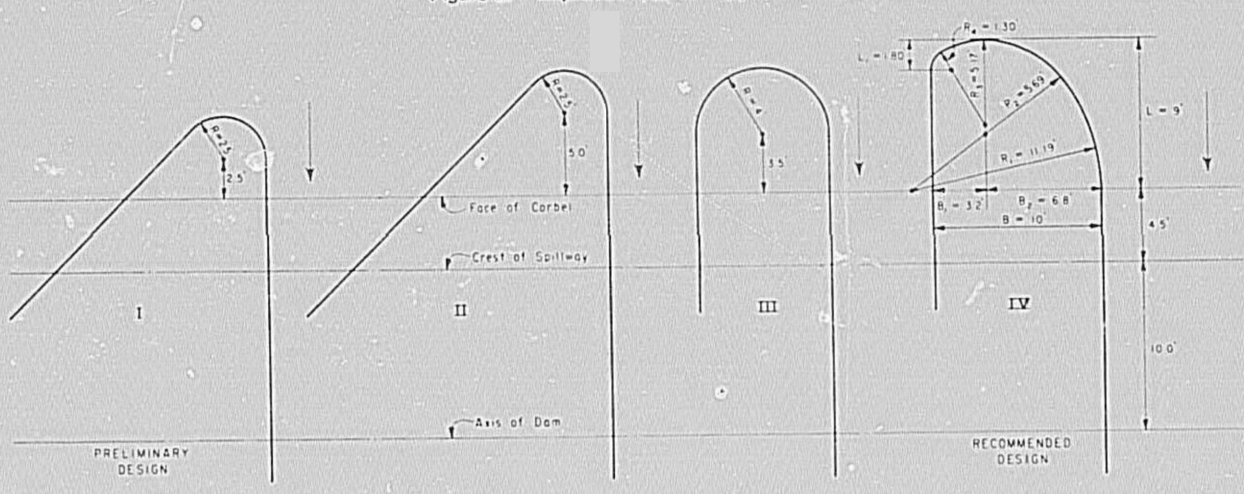
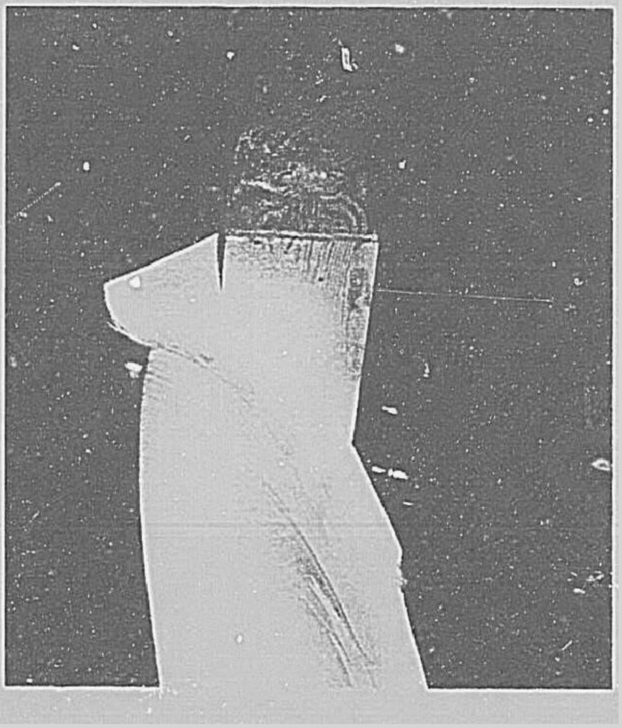
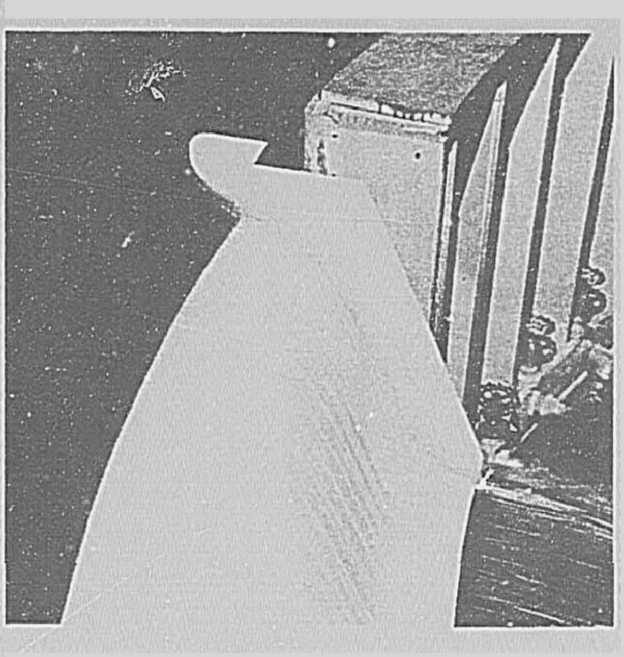


Figure 9. Progressive pier designs. 1:36–scale model



A. Preliminary pier design. Note abrupt draw down at pier and resulting rough flow surface along training wall. Photo P622-D-74332



B. Recommended elliptical pier design. Photo P622-D-74341

Figure 10. Spillway pier design.

to insure protection against rock erosion and undercutting of the right abutment and base of the dam. To flip the spillway jet further into the pool, the radius of the bucket was extended from the bucket invert, elevation 6711.0 to elevation 6711.45, where the bucket lip terminated at a 4 to 1 tangent. The spillway jet impinged well within the excavated plunge pool as a result of the modified bucket design and improved reservoir approach, Figures 11A and 11B.

The spillway jet sprang free from the bucket when the head was approximately 6.9 inches (175.3 mm) or at a discharge of 250 cfs (7.08 meters^3/sec). The spillway jet impinged on the downstream right canyon wall at discharges below approximately 500 cfs (14.2 meters^3/sec), Figure 11C. This impingement will occur only at low flows over the spillway.

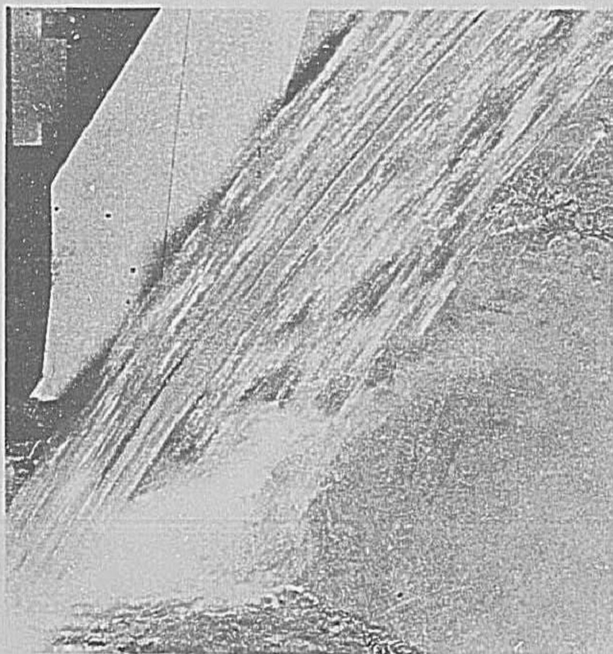
Plunge Pool Studies

Initial and recommended designs.—The initial plunge pool design resulted in a surface boil near the powerplant access road. The model study indicated that a considerable amount of expected excavation on the right side of the downstream canyon could be eliminated but that more excavation was needed on the left side to avoid the high boil on the water surface. Figure 12 illustrates the initial and modified plunge pool designs looking down from the dam. Figure 13 shows the improved flow conditions in the downstream plunge pool and along the powerplant access road. The initial performance is shown in Figure 8A.

To test the action of the submerged jet on large riprap, the 3 to 1 slope at the downstream end of the plunge pool was made up of rock representing 1-yard (0.76-meter^3) riprap, Figure 14A. Riprap along the left bank near the retaining wall was not simulated in the model because it would have required a major, expensive, model modification.

The model was tested for 1 hour at a spillway discharge of 41,000 cfs (1,161 meters^3/sec). A considerable amount of the large riprap was carried up the slope and deposited along the left bank. Figures 14B and 14C show the plunge pool water surface and eroded riprap.

To deflect the submerged jet up, away from the riprap, a 15-foot (4.57-meter) high deflector wall with a 1:4 batter was placed on the floor of the plunge pool, as shown in Figure 15A. Rock representing ¼-yard (0.19-meter^3) riprap [average diameter = 1.89 feet (0.57 meter)], which would be more common in the vicinity of the dam, was placed behind the wall and on the 3 to 1 slope. The model was tested for 1 hour at a spillway discharge of 41,000 cfs (1,161 meters^3/sec). Figures 15B and 15C show the plunge pool water surface and the relatively undisturbed riprap after the



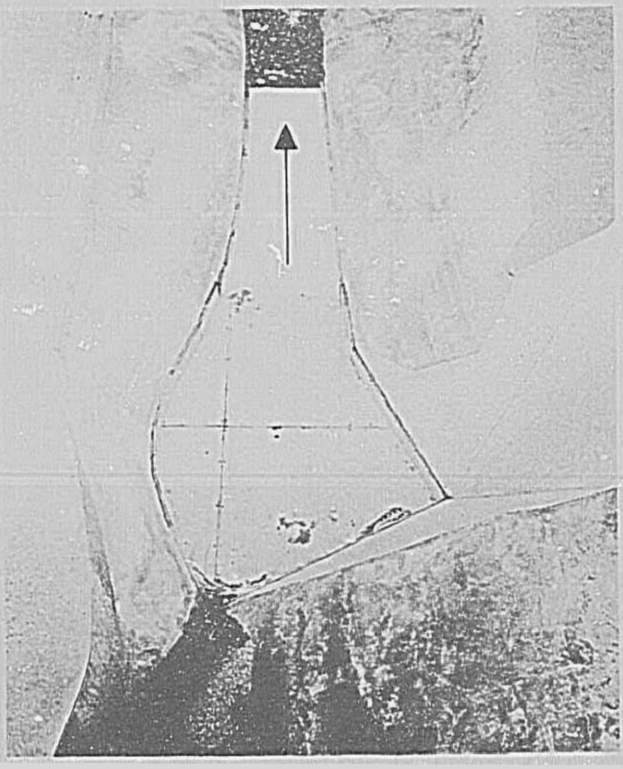
A. View of spillway jet missing the plunge pool—horizontal bucket lip. Photo P622-D-74324

B. View of spillway jet hitting the plunge pool—4 to 1 tangent at bucket lip. Photo P622-D-74330

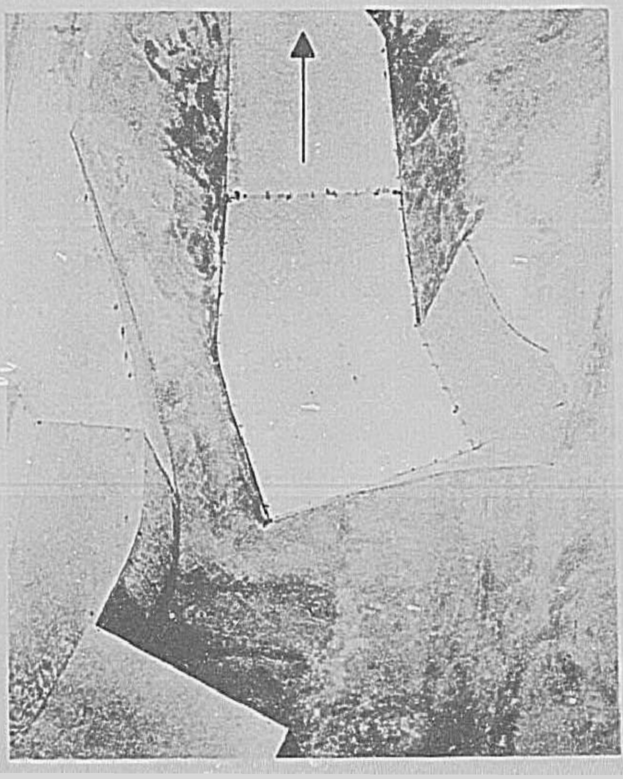
Spillway discharging 20,000 cfs

C. View of full spillway jet springing free. Photo P622-D-74351

Figure 11. The spillway jet

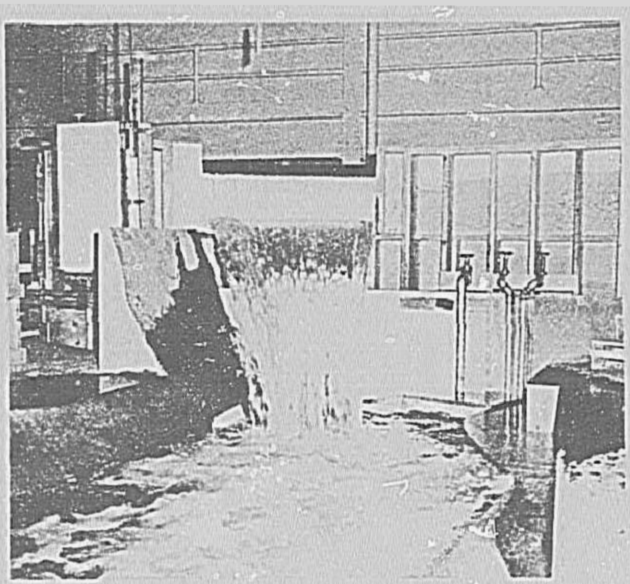


A. View from the dam looking toward the downstream end of the plunge pool. Initial design. Note excavation of right canyon wall. Photo P622-D-74333



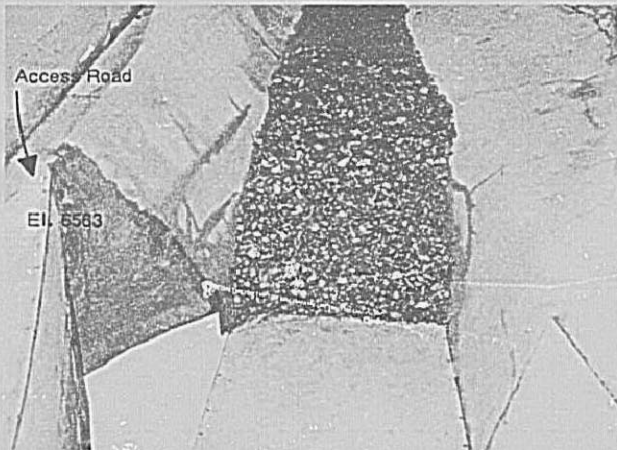
B. View from the dam looking toward the downstream end of the plunge pool. Modified design. Note the decreased excavation on the right side and modified left bank alinement. Photo P622-D-74335

Figure 12. Plunge pool designs.

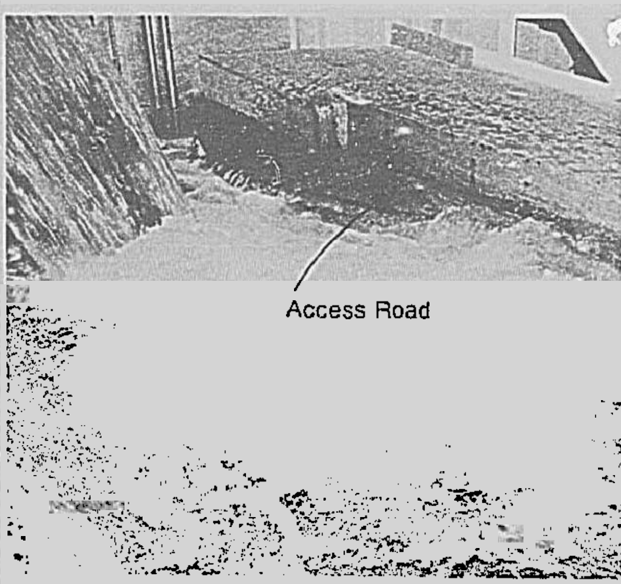


Spillway discharging 41,350 cfs. Photo P622-D-74334

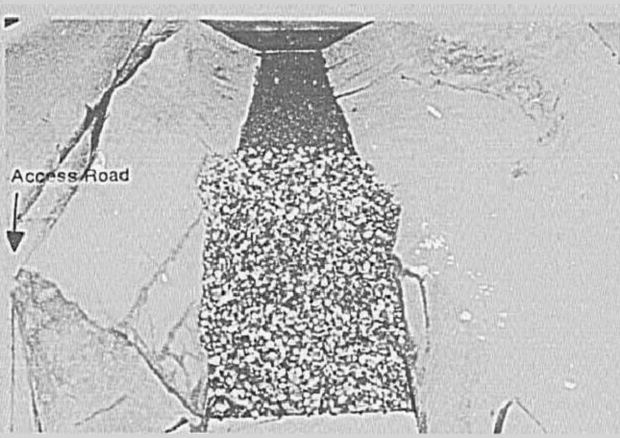
Figure 13. Improved water surface with modified plunge pool (Figure 12B).



A. View of 3 to 1 slope before the test. The rock represents 1-yard riprap. Photo P622-D-24336



B. Spillway discharging 41,000 cfs. Photo P622-D-74337

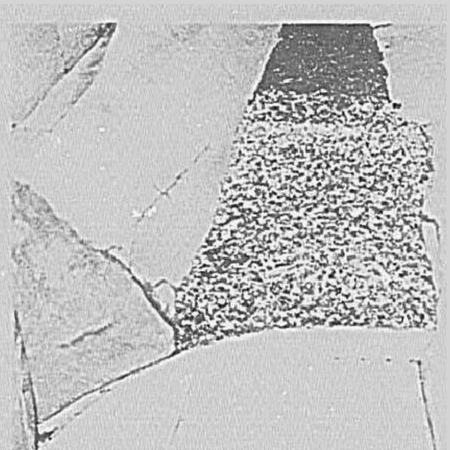


C. View of the 3 to 1 slope after the test. Note how some riprap has been moved to the top of the slope. Photo P622-D-74338

Figure 14. 3 to 1 riprap slope (1-yard riprap).

A. View of 3 to 1 slope and 15-foot deflector wall before the test. The rock represents 1/4-yard riprap. Photo P622-D-74339

B. Spillway discharging 41,000 cfs. Photo P622-D-74340



C. View of 3 to 1 slope after the test showing very little movement of the riprap material. Photo P622-D-74342

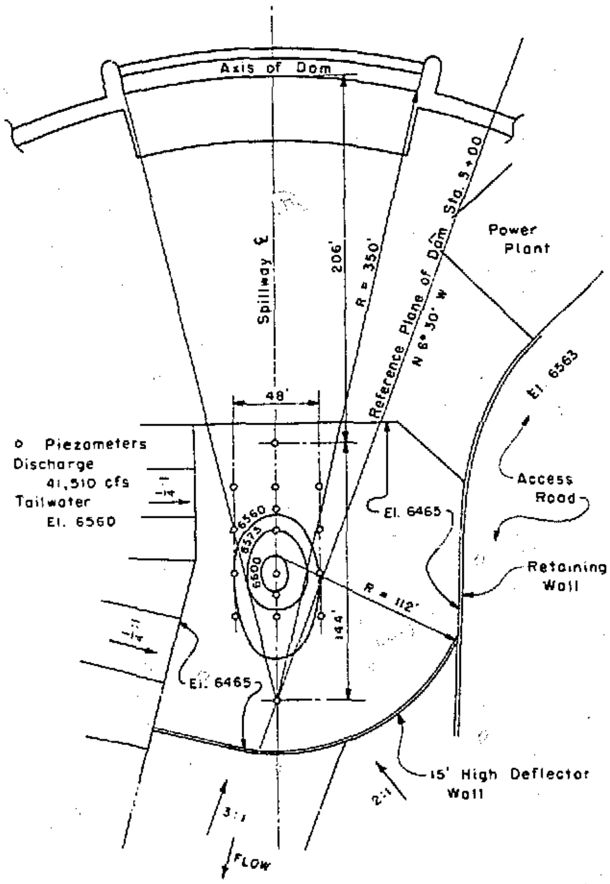
Figure 15. 3 to 1 riprap slope with 15-foot deflector wall (1/4-yard riprap).

1-hour test. The high boil near the powerplant access road in Figure B is due to the steep concrete slope in the model represented by the dark concrete in Figure 15A.

The recommended design for the side slope behind the deflector wall ranges from 3 to 1 to 2 to 1, as shown in Figure 1. The access road will be located some 60 feet (18.3 meters) further from the plunge pool and 5 feet (1.5 meters) higher than the design tested, Figure 15. Although this configuration was not tested because of insufficient space in the model, the increased size of the plunge pool should be more than adequate to contain the energy of the spillway jet.

For small spillway discharges there is a tendency for rock to be pulled over the deflector wall into the plunge pool. Therefore, it is recommended that the riprap slope start 2 feet (0.6 meter) below the top of the deflector wall.

Impact pressures.—To measure the impact pressures on the floor of the plunge pool, a grid of 16 piezometers was placed in an area 48 feet (14.6 meters) wide by 144 feet (43.9 meters) long, as shown in Figure 16. Maximum pressures (water manometer) were recorded for several discharges and corresponding tailwaters. The equipressure (equielevation) lines in Figure 16 indicate plunge pool floor pressures for Q = 41,500 cfs (1,174 meters^3/sec) with the recommended spillway bucket lip. Figure 17 illustrates the observed pressures on the plunge pool floor along the spillway centerline. Six more piezometers were later placed on the floor in a 3-foot (0.9-meter) by 6-foot (1.8-meter) area on the spillway centerline 278 feet (84.7 meters) from the axis of the dam, where the highest impact pressures were observed. These piezometers were equipped with pressure cells immediately below the floor of the plunge pool. Dynamic pressures were recorded for several discharges. The highest average value observed on the six pressure cells for a discharge of 41,000 cfs (1,161 meters^3/sec). represented a total pressure head of 147 feet (44.8 meters), or elevation 6612, Figure 18. The maximum instantaneous pressure represented an elevation of 6766, the reservoir elevation for a discharge near 41,000 cfs (1,161 meters^3/sec). This would indicate that at times the instantaneous energy level on the floor of the plunge pool reaches the potential energy level of the reservoir.



CRYSTAL DAM SPILLWAY AND OUTLET WORKS

Figure 16. Model plunge pool piezometer locations. 1:36—scale model

Outlet Works

The river outlet works consists of two 54-inch (1,371.6-mm) diameter, approximately 120-foot (36.6-meter) long conduits which run through the dam and powerplant. These outlet conduits are controlled by 48-inch (1,219.2-mm) diameter jet-flow gates located at the downstream end of each conduit, Figure 1.

In the preliminary design, each conduit had its own vertical intake tower located on the upstream face of the dam. The two intake towers were 160 feet (48.8 meters) in height with a 7.5-foot (2.29-meter) radius semicircular cross section. This design was later changed to a single 11.0-foot (3.35-meter) radius semicircular intake tower with two 54-inch (1,371.6-mm) diameter outlet conduits at the base, Figure 19.

The normal operating discharge of the river outlet works will be 1,600 cfs (45.3 meters^3/sec) with the reservoir surface elevation ranging from the top of inactive conservation storage, elevation 6700 to elevation 6755. Since the centerline elevation of the conduits and jet-flow gates is 6518, the gates will operate under a minimum submergence of 11 feet (3.35 meters).

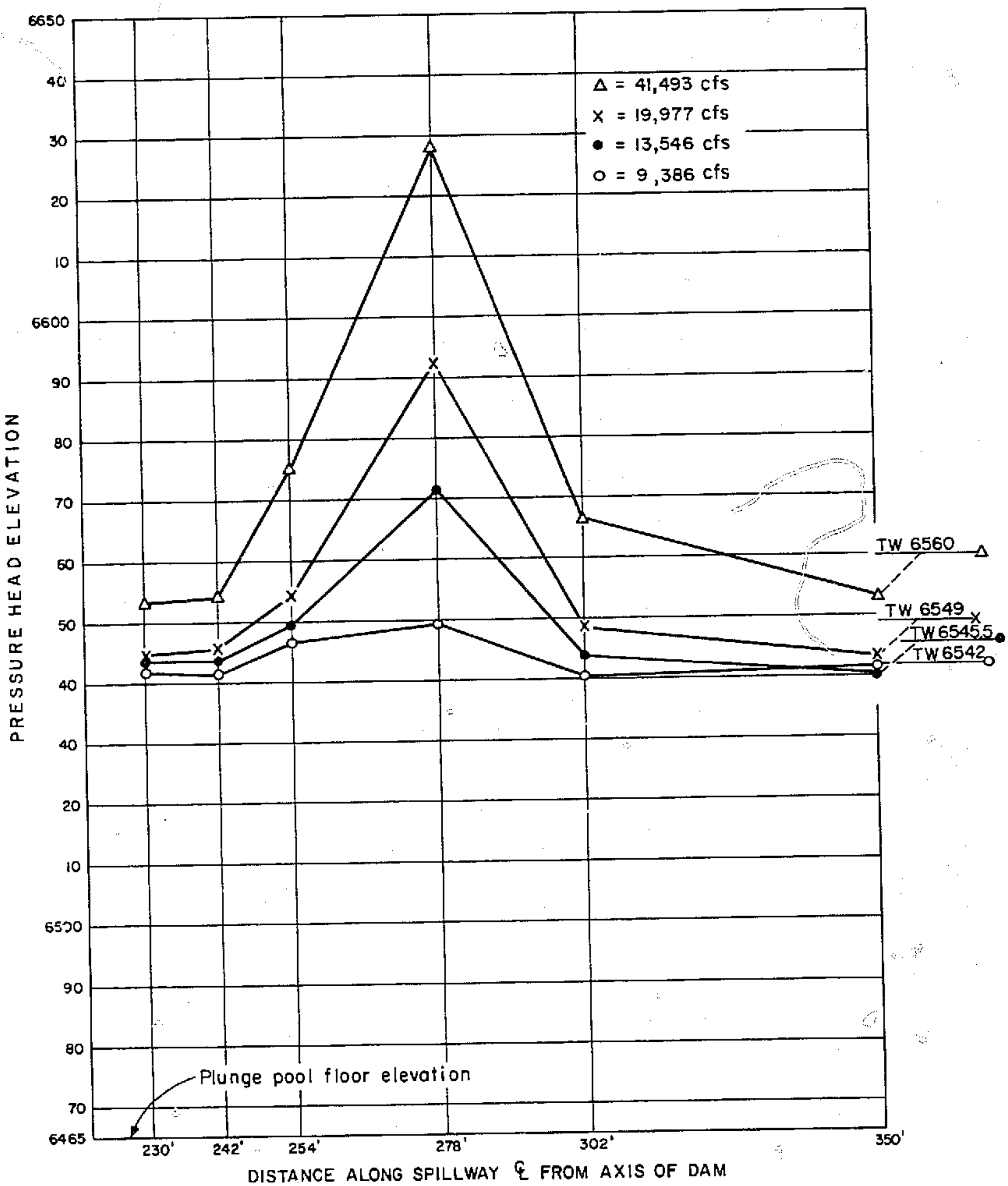
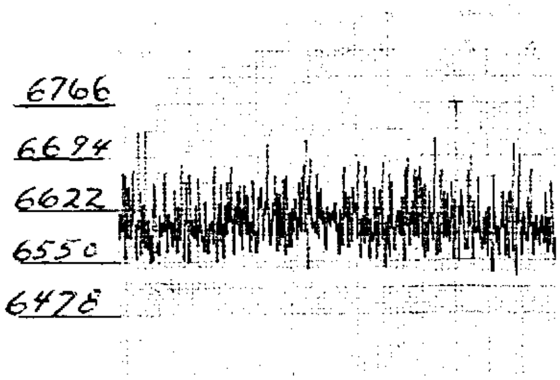


Figure 17. Plunge pool floor impact pressures. 1:36–scale model.



Q = 41,000 cfs
Tailwater elevation 6565
278 feet from axis of dam

Figure 18. Plunge pool floor impact pressure cell trace. 1:36—scale model.

The 1:13.60 model (preliminary design).—The model for the preliminary design was built to a scale ratio of 1:13.60 so that available 3.53-inch (89.7-mm) diameter model jet-flow gates could be used to represent the 48-inch (1,219.2-mm) diameter prototype gates. The two 54-inch (1,371.6-mm) diameter conduits and one of the 7.5-foot (2.29-meter) radius, semicircular, vertical intake towers were modeled with 4-inch (101.6-mm) pipes and a 6.62-inch (168.2-mm) radius model silo. One of these pipes was made of plastic and the other was made of sheet metal. The plastic model conduit included a bellmouth entrance and upstream from the model conduit the vertical, semicircular cross-sectional intake tower was built to simulate the prototype configuration. Only the lower 68 feet (20.7 meters) of the intake tower were constructed in the model and connected to the water supply pipe at an approximate elevation of 6586. The other conduit was connected directly to the water supply pipe. Figure 20 illustrates the model test facility.

The jet-flow gates were installed in a box 12 feet (3.66 meters) square by 12 feet (3.66 meters) deep to study the submerged conditions of the gates. A false retaining wall and a false bottom were installed to simulate the spillway plunge pool into which the outlet works discharges. A valve on the tailbox outlet pipe controlled the tailwater level. Water was supplied to the model by a portable centrifugal pump through an 8-inch (203.2-mm) diameter pipe. A calibrated 4-3/8-inch (111.1-mm) diameter orifice meter was installed in the 8-inch (203.2-mm) pipe to measure the discharge rate. Piezometers were installed at reference stations in the intake tower, upstream from the jet-flow gate, and at points within the bellmouth entrance to the conduit where low pressures were anticipated. The pressures along the bellmouth flow surface were measured by a pot-type mercury manometer and several U-tube mercury manometers. The tailwater elevation was measured using a staff gage attached to the inside wall of the tailbox.

The 1:19.85 model (recommended design).—The 6.62-inch (168.2-mm) model, semicircular intake tower used for the preliminary design was also utilized for the 11.0-foot (3.35-m) radius, single, intake tower system. Since this resulted in a change in the prototype tower radius from 7.50 feet to 11.0 feet, the model scale was changed accordingly from 13.6 to 19.85. To model the two 54-inch (1,371.6-mm) conduits, 2.75-inch (69.85-mm) diameter, standard plastic pipe was used.

Piezometers were installed at reference stations P_i and P_c in the intake tower and at the downstream end of each conduit similar to the earlier model, Figure 20. The resulting pressures were measured by the aforementioned pot-type mercury manometer.

Calibrated 4-3/8-inch (111.13-mm) and 2-3/8-inch (60.66-mm) diameter orifices were used to measure the small discharges.

To regulate the flow in each conduit, the downstream end of each 2-3/4-inch (69.85-mm) conduit was connected to the previously used 3.53-inch (89.7-mm) jet-flow gates.

Vortex formation.—The flow in the preliminary intake tower design was not stable, especially in the bellmouth transition from the intake tower to the conduit, as shown in Figure 21A. A violent vortex originated near the floor of the intake tower and developed in the bellmouth entrance to the conduit. A piezometer probe was placed in the core of the vortex by drilling a hole in the rear of the intake tower at the centerline elevation of the conduit. A pressure cell recorded model pressures of minus 23 feet (7.01 meters), indicating that the vortex core was at or near vapor pressure for a model discharge of 1.71 cfs (0.05 meter^3/sec). Pressures along the bellmouth flow surface were unsteady and pressure fluctuations as large as 200 feet (61 meters) prototype were recorded. The vortex formed near the entrance to the conduit and reached to the downstream end, Figure 21B.

To prevent this unstable flow condition, the floor of the intake tower was raised from elevation 6510.5 to elevation 6513.5, which decreased the area available for circulation, Figure 19. This allowed enough remaining space for the bulkhead gate seals on the upstream face of the bellmouth entrance.

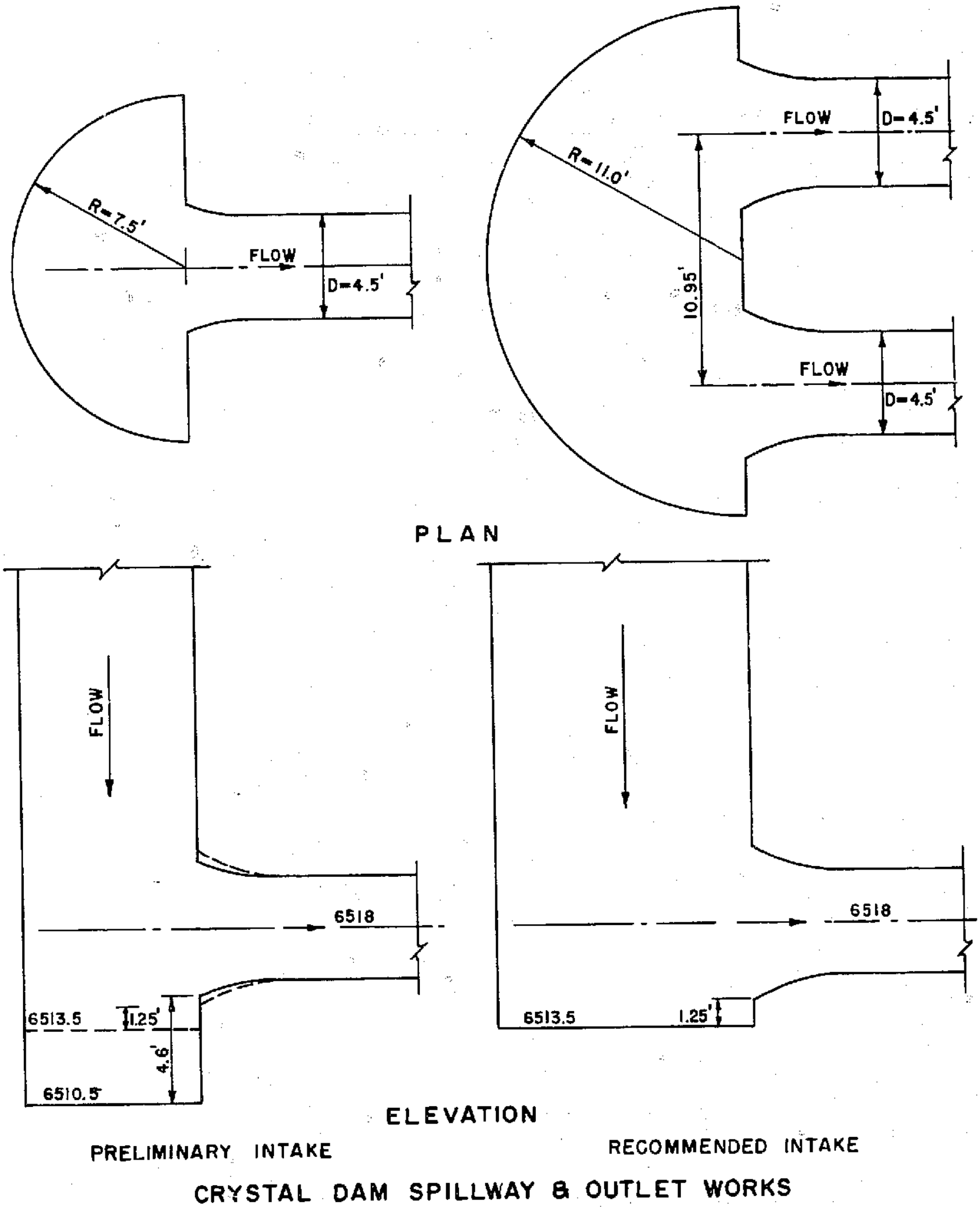
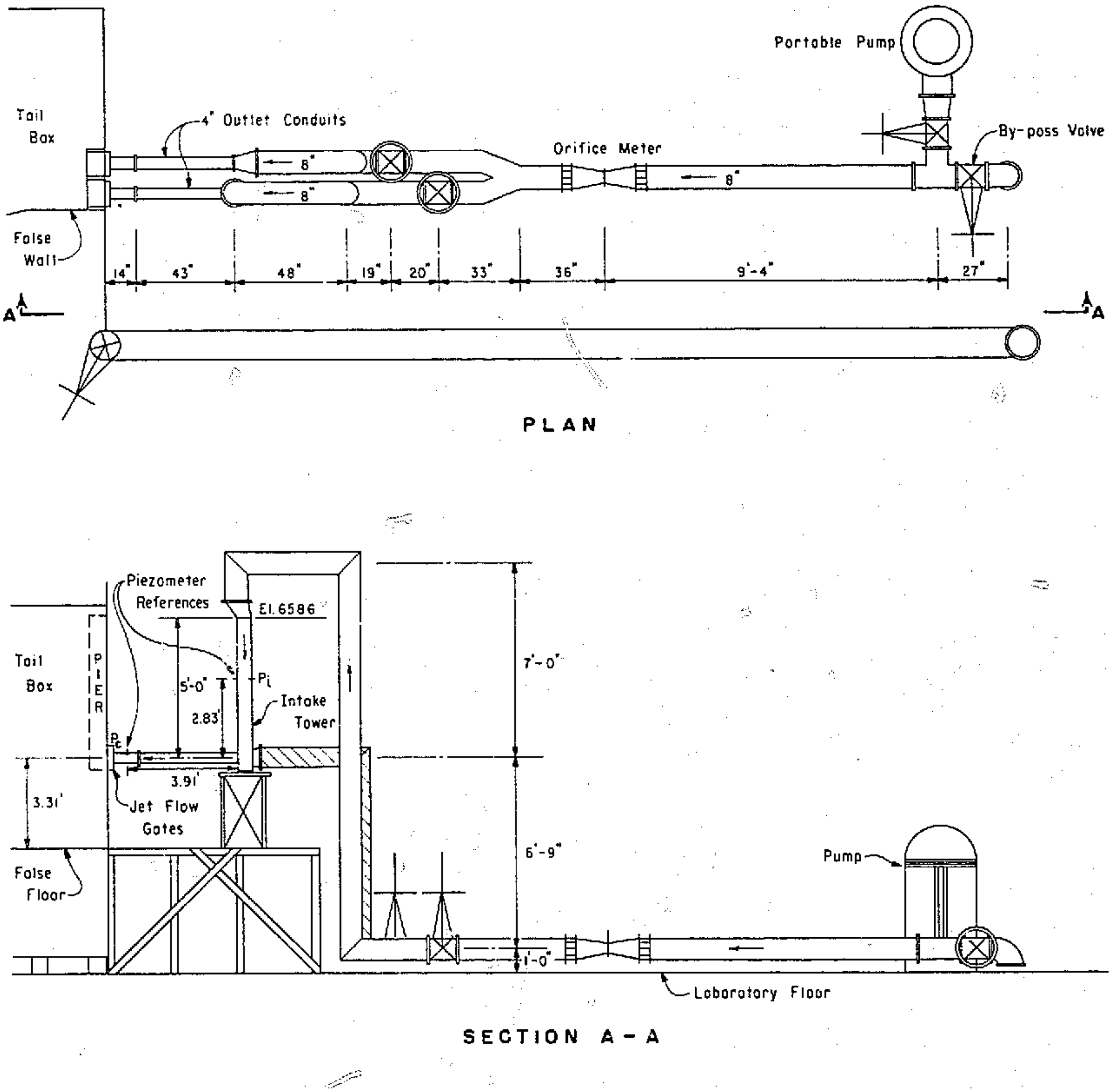
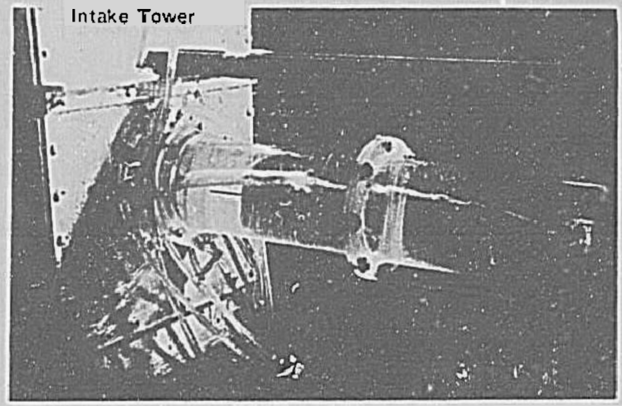


Figure 19. Intake tower designs.

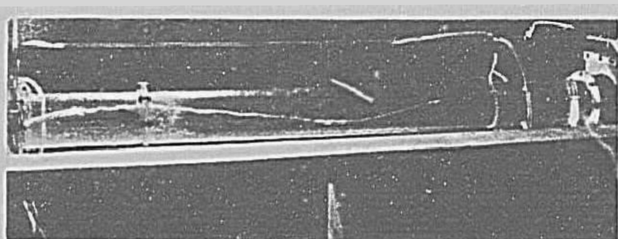


CRYSTAL DAM SPILLWAY AND OUTLET WORKS

Figure 20. Outlet works model layout (preliminary design). 1:13.6–scale model



A. Preliminary dual intake tower system. Floor of tower at elevation 6510.5, Q = 1160 cfs, 30 feet of submergence at jet-flow gate. Note vortex extending into conduit from the intake tower. Photo P622-D-74345



B. View of vortex extending length of conduit. Photo P622-D-74328

Figure 21. Vortex in horizontal outlet conduit.

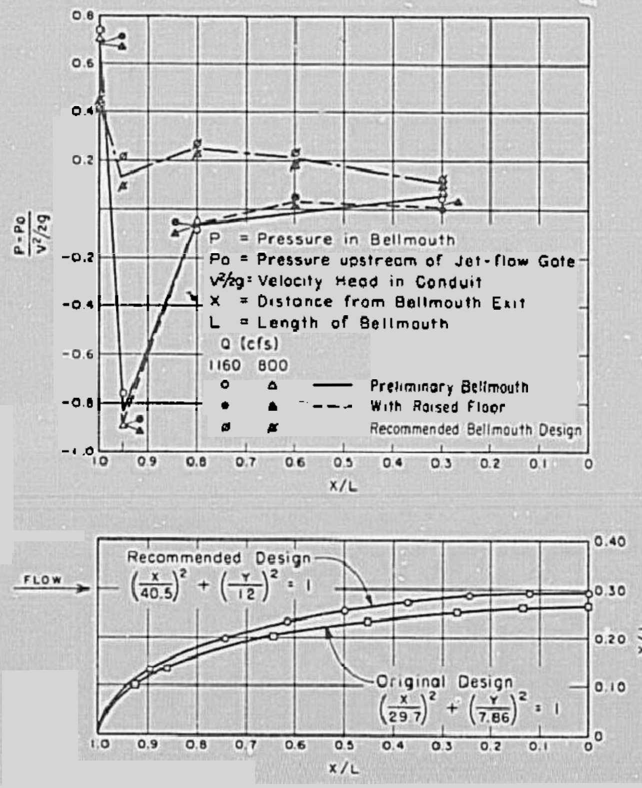
With this modification there was no vortex formation, and the pressure fluctuations along the bellmouth flow surface were very small.

Conduit bellmouth entrances.—In the preliminary bellmouth design, the pressure profile showed a sudden pressure drop in the entrance, Figure 22. Although raising the floor eliminated the vortex, the pressure distribution on the crown of the bellmouth remained approximately the same. A larger bellmouth shape was tested (recommended design) to achieve a more gradual pressure change in this region.

Figure 22 shows the two bellmouth profiles. The pressure profile in the recommended design was much improved over the preliminary design while the head losses through each were similar.

Bellmouth Head Loss Coefficients for Preliminary and Recommended Intake Towers

Preliminary intake tower design.—The bellmouth entrance head loss, h_e, in the preliminary intake tower



CRYSTAL DAM SPILLWAY AND OUTLET WORKS

Figure 22. Bellmouth profiles and pressure distributions. 1:13.6–scale model.

design was defined as:

$$h_e = h_L - h_f \qquad (1)$$

where

h_L = total head loss between the reference stations in the intake tower and conduit, and
h_f = friction head loss between these stations.

For the preliminary intake tower design, Figure 20,

$$h_L = \left(P + \frac{V^2}{2g}\right)_i - \left(P + \frac{V^2}{2g}\right)_c \qquad (2)$$

where

P = piezometric head at reference stations,
V = average flow velocity in intake tower or conduit, and
i, c subscripts referring to intake tower and conduit, respectively.

The extrapolated experimental results, Figure 23, indicated the following relationship between the prototype discharge, Q, and the difference in piezometric head between the intake tower and the conduit $\Delta P = P_i - P_c$

$$Q = 112.5\sqrt{\Delta P}$$

or $$\Delta P = \left(\frac{Q}{112.5}\right)^2 = \frac{2g\,A_c^2}{(112.5)^2}\,\frac{1}{2g}\left(\frac{Q}{A}\right)_c^2$$

where A_c equals the conduit area based on the 4-inch (101.6-mm) diameter, model conduit scaled to a 54.4-inch (1,382-mm) prototype conduit diameter.

$$\Delta P = \frac{64.4(16.14)^2}{(112.5)^2}\left(\frac{V^2}{2g}\right)_c$$

therefore $$\Delta P = 1.33\left(\frac{V^2}{2g}\right)_c$$

From equation (2) the total headloss, h_L, is therefore,

$$h_L = 1.33\left(\frac{V^2}{2g}\right)_c + \left(\frac{V^2}{2g}\right)_i - \left(\frac{V^2}{2g}\right)_c$$

$$h_L = 0.33\left(\frac{V^2}{2g}\right)_c + \left(\frac{V^2}{2g}\right)_i \qquad (3)$$

Since $A_c = 0.18\,A_i$

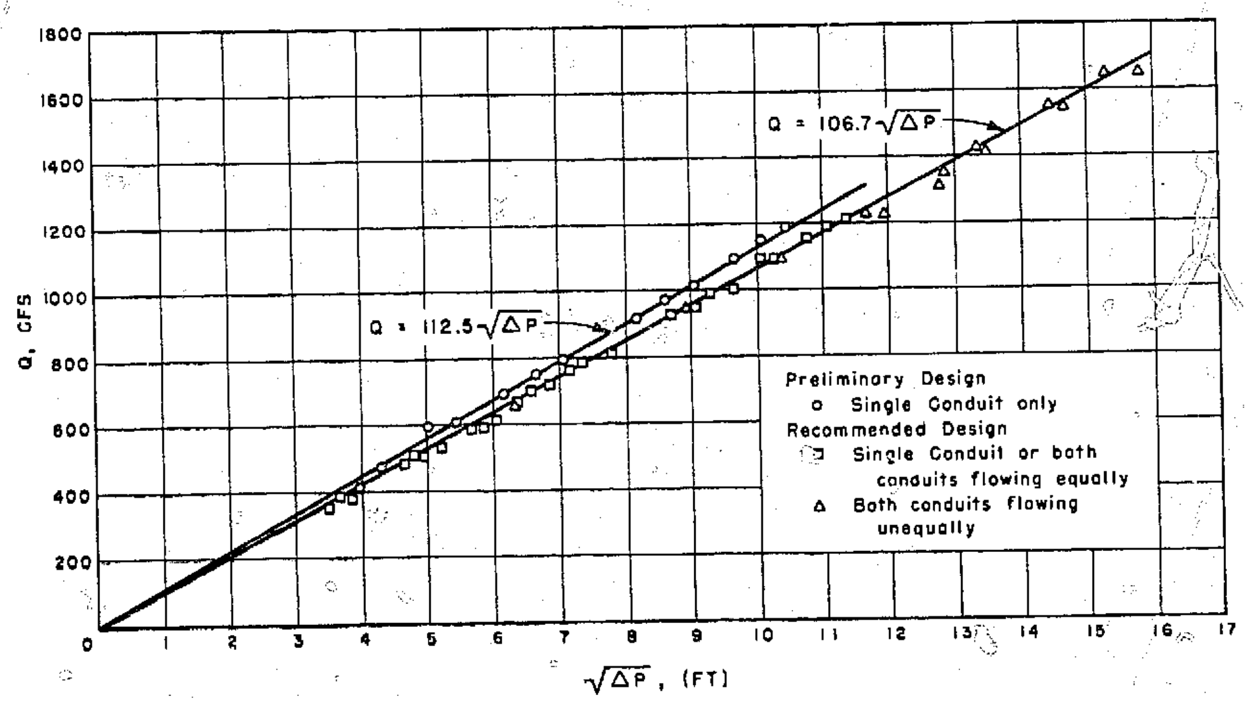
but $Q_i = Q_c$

therefore $V_i = 0.18V_c$

and $$\left(\frac{V^2}{2g}\right)_i = 0.03\left(\frac{V^2}{2g}\right)_c$$

therefore $$h_L = (0.33 + 0.03)\left(\frac{V^2}{2g}\right)_c$$

$$h_L = 0.36\left(\frac{V^2}{2g}\right)_c \qquad (4)$$



CRYSTAL DAM SPILLWAY AND OUTLET WORKS

Figure 23. Extrapolated model data (intake towers).

The friction head loss, h_f in the intake tower and conduit was estimated by the Darcy-Weisbach equation as follows:

$$h_f = \left(\frac{fL}{4R}\frac{V^2}{2g}\right)_i + \left(\frac{fL}{D}\frac{V^2}{2g}\right)_c$$

where

- f = friction coefficient,
- R = hydraulic radius,
- L = respective length, and
- D = diameter.

The friction head loss in the tower is negligible since the velocity head in the tower is only 3 percent of the conduit velocity head. L and D in the model conduit are 47 inches (1.19 meters) and 4 inches (0.10 meter), respectively. The friction coefficient, f, is expressed by using Manning's *n* as follows:

$$f = \frac{185\, n^2}{D^{1/3}}$$

Assuming a Manning's n of 0.008 for the 4-inch (0.10-meter) plastic conduit,

$$\frac{fL}{D} = \frac{185\, n^2 L}{D^{1/3}\, D} = \frac{185\, n^2 L}{D^{4/3}}$$

or

$$\frac{fL}{D} = 0.20$$

therefore

$$h_f = 0.20\left(\frac{V^2}{2g}\right)_c \tag{5}$$

Substituting equations (4) and (5) into equation (1),

$$h_e = (0.36 - 0.20)\left(\frac{V^2}{2g}\right)_c$$

$$h_e = 0.16\left(\frac{V^2}{2g}\right)_c \tag{6}$$

Recommended intake tower design.—The recommended design used the same bellmouth shape as the preliminary design. Extrapolated experimental results, Figure 23, indicated the following relationship between the conduit discharge, Q_c, and the difference in piezometric head, $\Delta P = P_i - P_c$:

$$Q_c = 106.7\sqrt{\Delta P} \tag{7}$$

This relationship was derived by operating either conduit open with the other conduit closed or operating both conduits with equal discharges. Further tests indicated that the total discharge through the intake tower equaled the sum of the two conduit discharges calculated from equation (7) when operating with unequal discharges through the conduits. Therefore, the total discharge,

$$Q_i = (Q_c)_R + (Q_c)_L$$

$$Q_i = 106.7\sqrt{\Delta P_R} + 106.7\sqrt{\Delta P_L}$$

$$Q_i = 106.7(\sqrt{\Delta P_R} + \sqrt{\Delta P_L}) \tag{8}$$

where

- ΔP_L = piezometric head difference between the reference stations in the intake tower and left conduit, and
- ΔP_R = piezometric head difference between the reference stations in the intake tower and right conduit.

From equation (7)

$$\Delta P = \left(\frac{Q_c}{106.7}\right)^2 = \frac{2g\, A_c{}^2}{(106.7)^2}\frac{1}{2g}\left(\frac{Q}{A}\right)_c^2$$

where A_c equals the conduit area based on the 2.72-inch (70-mm) diameter model conduit scaled to a 54.0-inch (1,371.6-mm) prototype conduit diameter

$$\Delta P = \frac{64.4(15.9)^2}{(106.7)^2}\left(\frac{V^2}{2g}\right)_c = 1.43\left(\frac{V^2}{2g}\right)_c \tag{9}$$

The total head loss, h_L, between reference stations in the intake tower and either conduit is,

$$h_L = \left(P + \frac{V^2}{2g}\right)_i - \left(P + \frac{V^2}{2g}\right)_c$$

$$h_L = \Delta P + \left(\frac{V^2}{2g}\right)_i - \left(\frac{V^2}{2g}\right)_c$$

or

$$h_L = 0.43\left(\frac{V^2}{2g}\right)_c + \left(\frac{V^2}{2g}\right)_i \tag{10}$$

Equation (10) is similar to equation (4). The relationship between the terms $\left(\frac{V^2}{2g}\right)_i$ and $\left(\frac{V^2}{2g}\right)_c$ is a function of the ratio $\frac{Q_i}{Q_c}$. However, in this instance the ratio $\frac{Q_i}{Q_c}$

is variable (for example, $Q_L = 0.1Q_i$ and $Q_R = 0.9Q_i$).

The friction head loss, h_f, in the intake tower and either conduit is,

$$h_f = \left(\frac{fL}{4R}\frac{V^2}{2g}\right)_i + \left(\frac{fL}{D}\frac{V^2}{2g}\right)_c$$

Under the same assumption of negligible tower loss and in this case substituting L_c = 37 inches (940 mm) and D_c = 2.72 inches (70 mm) and using Manning's n = 0.008,

$$\frac{fL}{D} = \frac{185\, n^2 L}{D^{1/3}\, D} = \frac{185\, n^2 L}{D^{4/3}}$$

$$\frac{fL}{D} = 0.26$$

Therefore, the entrance head loss is determined to be,

$$h_e = h_L - h_f = 0.43\left(\frac{V^2}{2g}\right)_c + \left(\frac{V^2}{2g}\right)_i - 0.26\left(\frac{V^2}{2g}\right)_c$$

or
$$h_e = 0.17\left(\frac{V^2}{2g}\right)_c + \left(\frac{V^2}{2g}\right)_i \qquad (11)$$

but
$$\left(\frac{V^2}{2g}\right)_i = \left(\frac{Q_i}{Q_c}\right)^2 \left(\frac{A_c}{A_i}\right)^2 \left(\frac{V^2}{2g}\right)_c$$

Therefore, equation (11) can be restated as,

$$h_e = 0.17\left(\frac{V^2}{2g}\right)_c + \left(\frac{Q_i}{Q_c}\right)^2 \left(\frac{A_c}{A_i}\right)^2 \left(\frac{V^2}{2g}\right)_c$$

$$\left(\frac{A_c}{A_i}\right)^2 = \left(\frac{15.9}{190}\right)^2 = 0.007$$

Therefore, for either conduit bellmouth,

$$h_e = \left[0.17 + 0.007\left(\frac{Q_i}{Q_c}\right)^2\right]\left(\frac{V^2}{2g}\right)_c \qquad (12)$$

Equation (12) indicates a bellmouth entrance loss slightly greater than that of the preliminary design, equation (6). If one conduit is open and the other closed, then $Q_c = Q_i$ and

$$h_e = 0.177\left(\frac{V^2}{2g}\right)_c$$

If both conduits have equal discharges, then

$$Q_c = 1/2\, Q_i$$

and $h_e = (0.17 + 0.028)\left(\frac{V^2}{2g}\right)_c = 0.198\left(\frac{V^2}{2g}\right)_c$

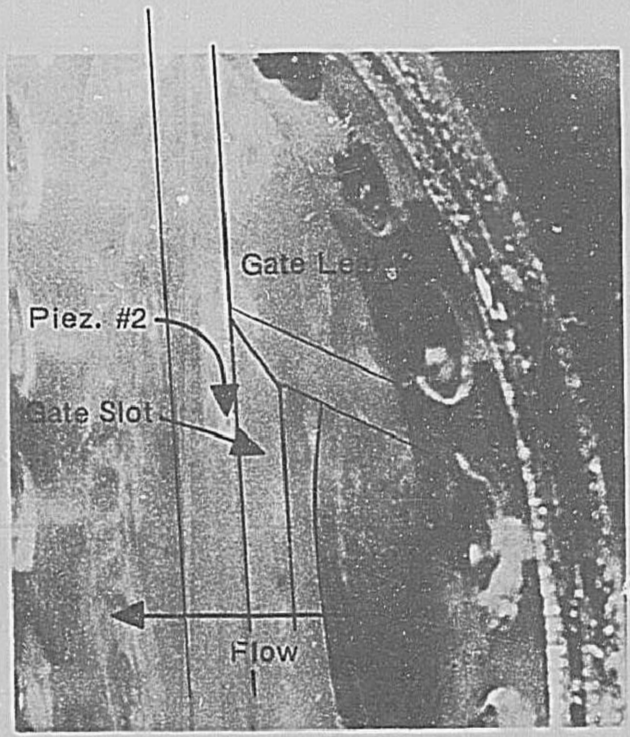
Cavitation of the jet-flow gate.—Cavitation of the Crystal Dam jet-flow gates was anticipated as a result of the Teton Dam hydraulic model study, which indicated inadequate circulation downstream from the submerged jet-flow gate for an enlargement less than 3 diameters. The Teton Dam jet-flow gate will operate with back pressures greater than 80 feet (24.4 meters). Figure 24B shows the cavitation cloud which formed with the Teton Dam submerged jet-flow gate in the range of 60 to 80 percent open under a relatively low back pressure, 15 feet (4.6 meters), discharging into a downstream chamber with a diameter three times the gate orifice diameter. The cavitation cloud originates at the intersection of the horizontal gate leaf and the circular orifice.

Pressures on the submerged downstream face of the orifice plate and gate frame of the Crystal Dam jet-flow gate were measured using four pressure cells to determine the possibility of subatmospheric pressures. Figure 25 shows the location of the four piezometers on the model jet-flow gate.

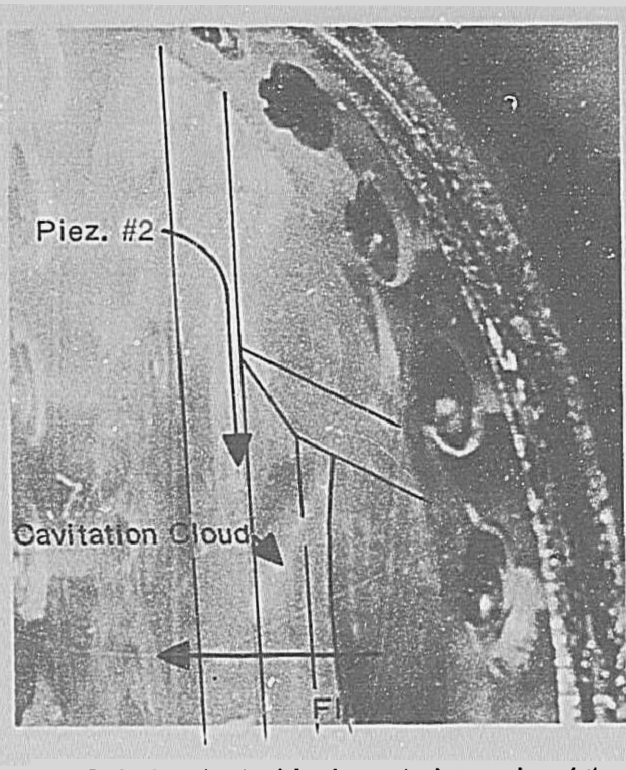
Tests of the preliminary outlet design, Figure 26A, with two jet-flow gates discharging into the tailbox, produced a minimum instantaneous pressure (Piezometer 3) of 13 feet (3.96 meters) subatmosphere at a gate opening of 70 percent (800 cfs). This minimum pressure occurred with a 19-foot (5.8-meter) submergence and an estimated reservoir elevation of 6750.

To improve the circulation on the downstream face of the submerged jet-flow gate, the 8.75-foot (2.67-meter) piers were cut back at a 30° angle below the minimum tailwater elevation 6529, and the sill was removed, as shown in Figure 26B. In this instance, the minimum instantaneous pressure (Piezometer 4) was 7.2 feet (2.19 meters) subatmosphere at a gate opening of 60 percent (650 cfs). It occurred under 13-foot (3.96-meter) submergence and an estimated reservoir elevation 6750.

It was concluded that with this configuration the jet-flow gate would be free from cavitation damage without further modification since the recorded minimum pressure was above that which would be considered a cavitation pressure. Tests run with less submergence had greater subatmospheric pressures, as would be expected. Therefore, a minimum submergence of 13 feet (3.96 meters) at 60 percent valve

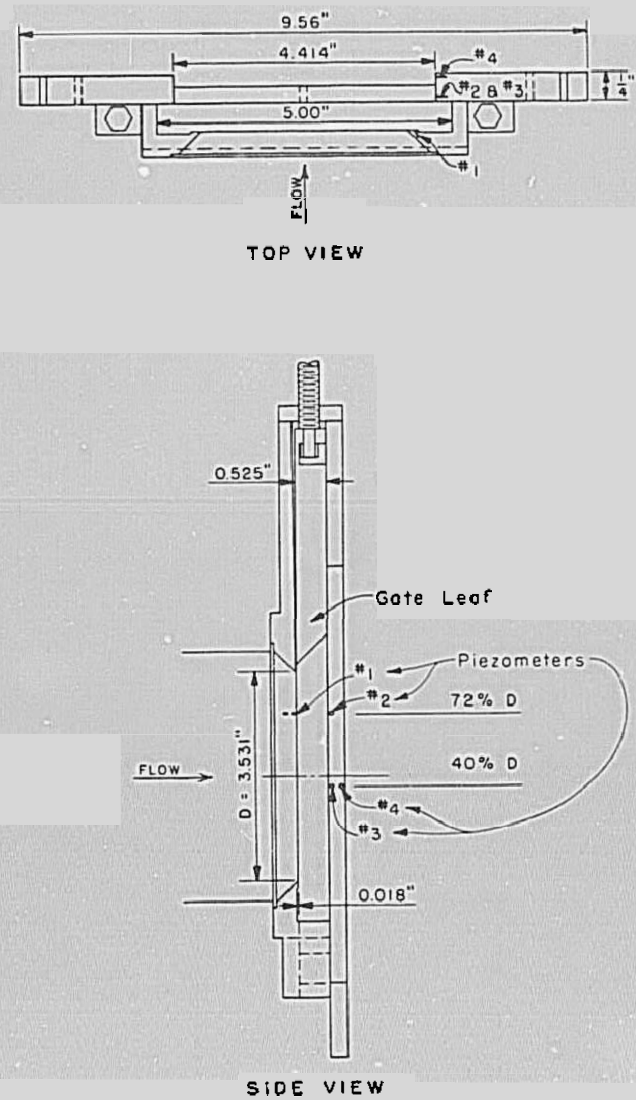


A. No cavitation (high back pressure). Photo P622-D-74328



B. Cavitation cloud originating at the intersection of the circular orifice and the horizontal gate leaf. (15-foot back pressure). Photo P622-D-74326

Figure 24. Typical cavitation cloud downstream from a submerged jet-flow gate (gate 75 percent open).



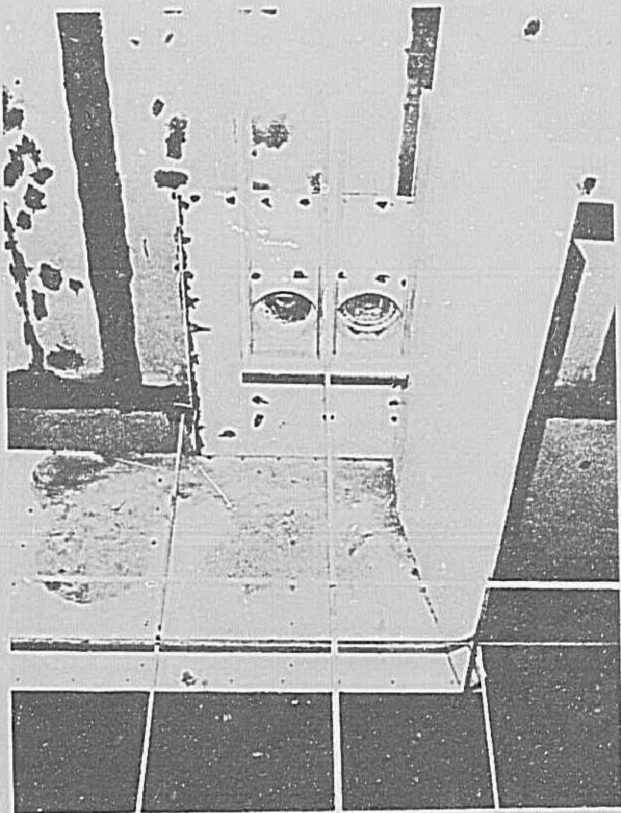
CRYSTAL DAM SPILLWAY AND OUTLET WORKS

Figure 25. Model jet-flow gate.

opening was arbitrarily set as the minimum tailwater criterion, where depth of submergence was defined as the distance from the valve centerline to the water surface.

Effect of submergence upon energy dissipation.— Several tailwater depths were tested to determine the effect of submergence of the jet-flow gate upon the water surface disturbance in the spillway plunge pool for normal operation with both valves 75 percent open, each discharging 875 cfs (24.8 meters^3/sec).

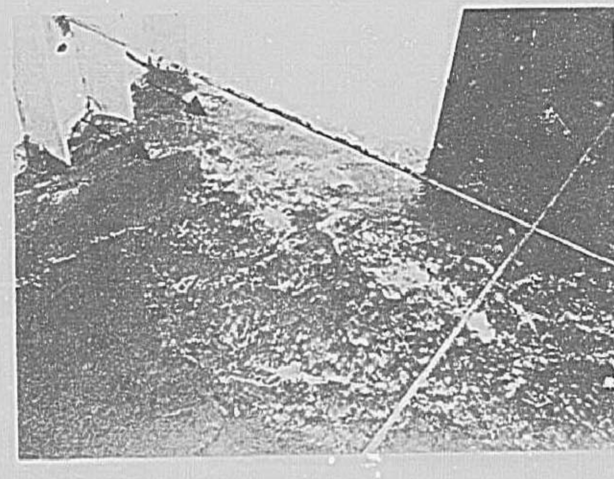
Tests were run with submergences of 6, 10, and 14 feet (1.83, 3.05, and 4.27 meters), Figure 27. The 6-foot (1.83-meter) and 10-foot (3.05-meter) depths resulted



A. Preliminary pier design with 8.75-foot piers and sill. Photo P622-D-74346

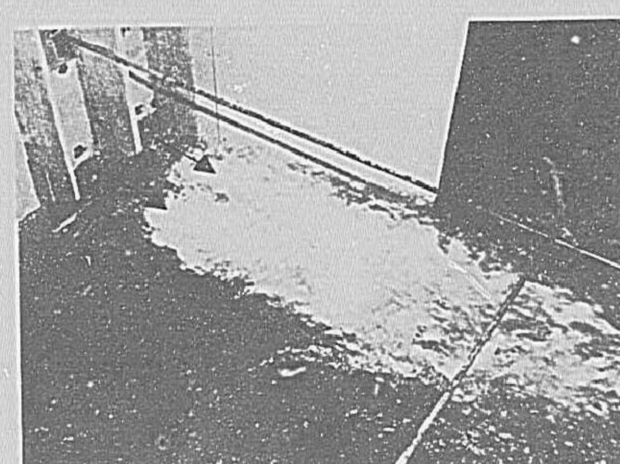
B. View of piers cut back at 30° below minimum tailwater elevation 6529.0. Note small 1-foot sill. Photo P622-D-74347

Figure 26. Plunge pool pier configurations for outlet works.



A. 14-foot submergence tailwater elevation 6532. Photo P622-D-74348

B. 10-foot submergence tailwater elevation 6528. Photo P622-D-74349

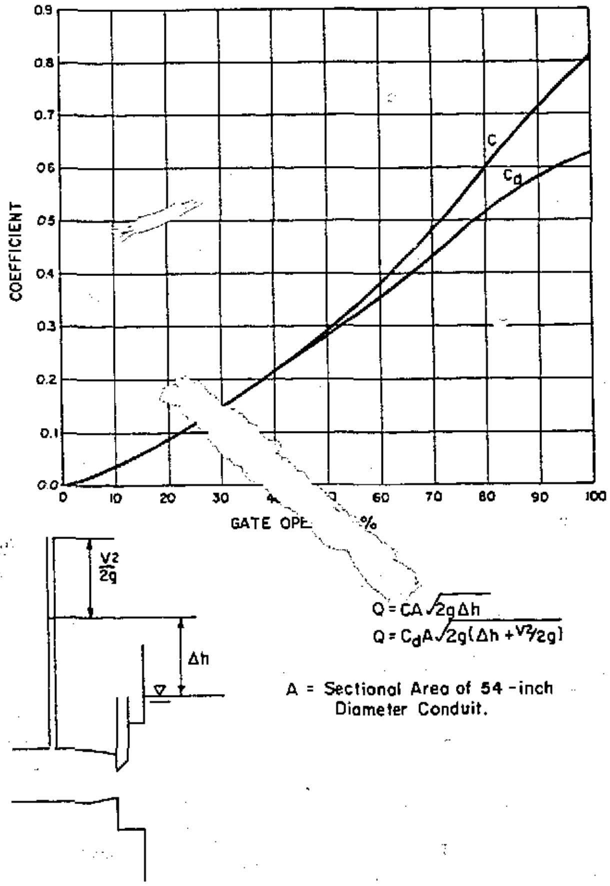


C. 6-foot submergence tailwater elevation 6524. Photo P622-D-74350

Figure 27. Jet-flow gate submergence (gates discharging 875 cfs each).

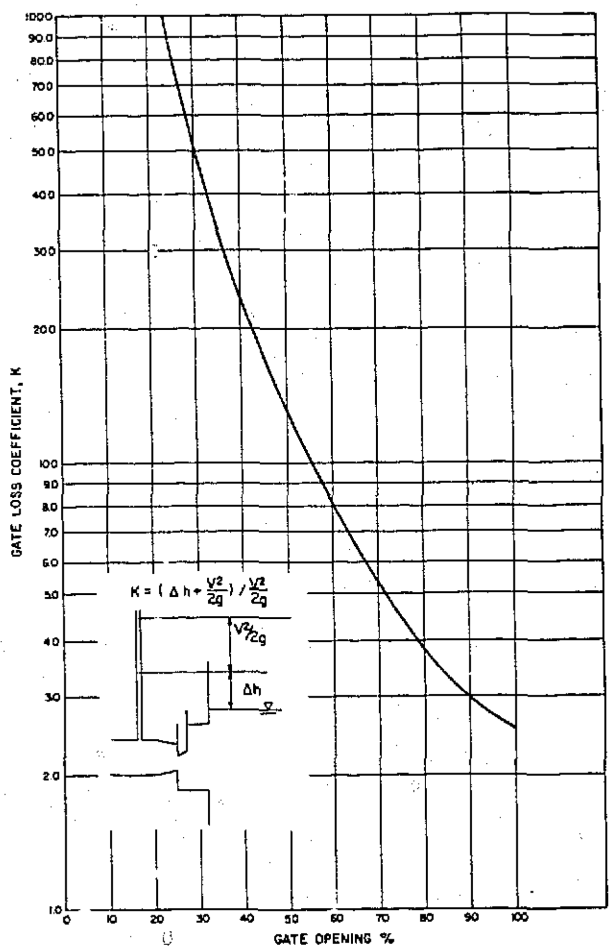
in a rough water surface and were inadequate. The 14-foot (4.27-meter) depth was sufficient, and the 13-foot (3.96-meter) submergence criterion for prevention of cavitation damage is adequate to minimize water surface disturbances in the plunge pool.

Jet-flow gate discharge coefficient.—The discharge characteristics of the submerged jet-flow gate were determined for various submerged depths and gate openings. The discharge coefficient, C (both with and without the conduit velocity head) and the head loss coefficient, K, of the jet-flow gate are shown in Figures 28 and 29, respectively. The discharge coefficient and the head loss coefficient are 0.628 and 2.536 at 100 percent gate opening. The head loss through the jet-flow gate for various gate openings is shown in Figure 30.



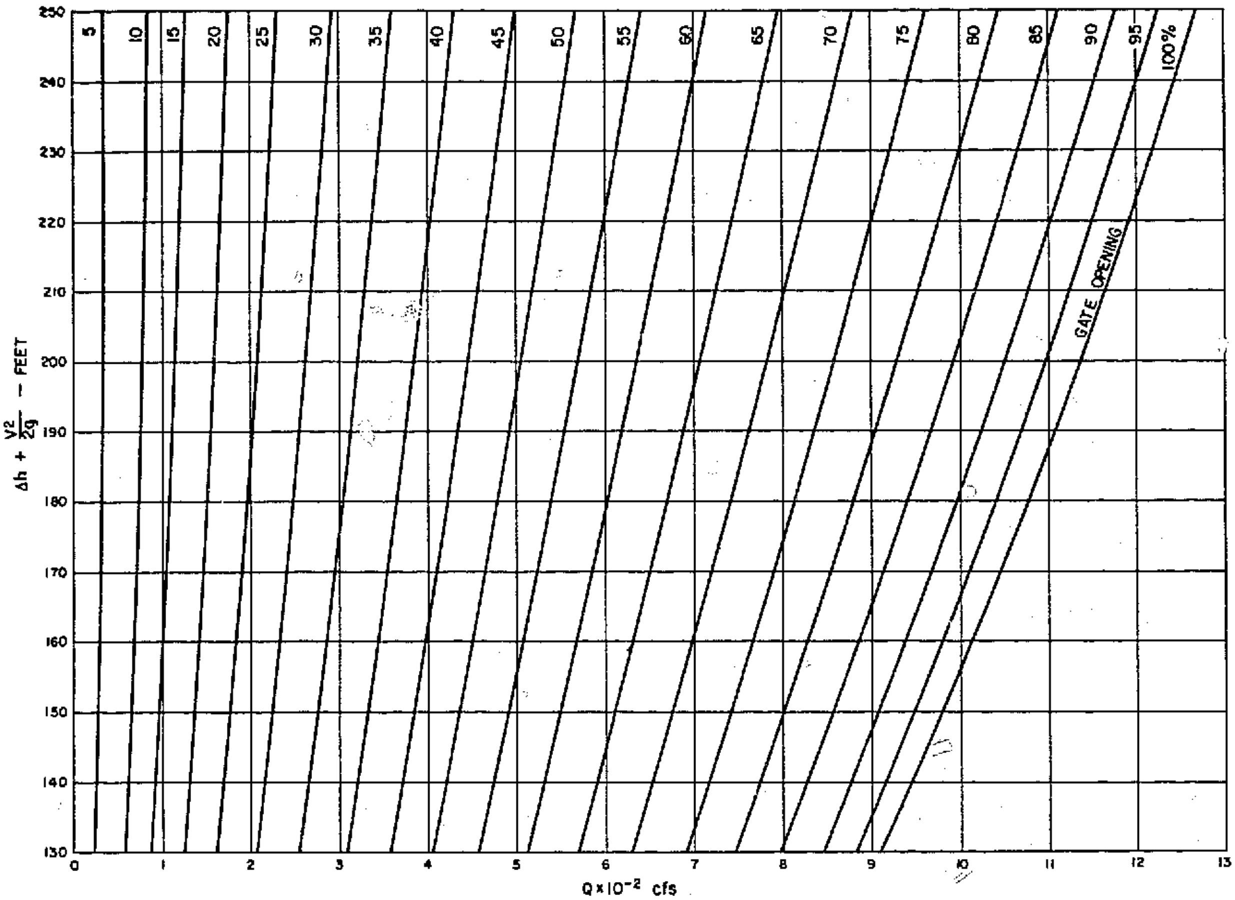
CRYSTAL DAM SPILLWAY AND OUTLET WORKS

Figure 28. Discharge coefficient of jet-flow gate.



CRYSTAL DAM SPILLWAY AND OUTLET WORKS

Figure 29. Gate loss coefficient versus gate opening.



CRYSTAL DAM SPILLWAY AND OUTLET WORKS

Figure 30. Head loss versus discharge for the jet-flow gate.

7-1750 (3-71)
Bureau of Reclamation

CONVERSION FACTORS–BRITISH TO METRIC UNITS OF MEASUREMENT

The following conversion factors adopted by the Bureau of Reclamation are those published by the American Society for Testing and Materials (ASTM Metric Practice Guide, E 380-68) except that additional factors (*) commonly used in the Bureau have been added. Further discussion of definitions of quantities and units is given in the ASTM Metric Practice Guide.

The metric units and conversion factors adopted by the ASTM are based on the "International System of Units" (designated SI for Systeme International d'Unites), fixed by the International Committee for Weights and Measures; this system is also known as the Giorgi or MKSA (meter-kilogram (mass)-second-ampere) system. This system has been adopted by the International Organization for Standardization in ISO Recommendation R-31.

The metric technical unit of force is the kilogram-force; this is the force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 9.80665 m/sec/sec, the standard acceleration of free fall toward the earth's center for sea level at 45 deg latitude. The metric unit of force in SI units is the newton (N), which is defined as that force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 1 m/sec/sec. These units must be distinguished from the (inconstant) local weight of a body having a mass of 1 kg, that is, the weight of a body is that force with which a body is attracted to the earth and is equal to the mass of a body multiplied by the acceleration due to gravity. However, because it is general practice to use "pound" rather than the technically correct term "pound-force," the term "kilogram" (or derived mass unit) has been used in this guide instead of "kilogram-force" in expressing the conversion factors for forces. The newton unit of force will find increasing use, and is essential in SI units.

Where approximate or nominal English units are used to express a value or range of values, the converted metric units in parentheses are also approximate or nominal. Where precise English units are used, the converted metric units are expressed as equally significant values.

Table I

QUANTITIES AND UNITS OF SPACE

Multiply	By	To obtain
	LENGTH	
Mil	25.4 (exactly)	Micron
Inches	25.4 (exactly)	Millimeters
Inches	2.54 (exactly)*	Centimeters
Feet	30.48 (exactly)	Centimeters
Feet	0.3048 (exactly)*	Meters
Feet	0.0003048 (exactly)*	Kilometers
Yards	0.9144 (exactly)	Meters
Miles (statute)	1,609.344 (exactly)*	Meters
Miles	1.609344 (exactly)	Kilometers
	AREA	
Square inches	6.4516 (exactly)	Square centimeters
Square feet	*929.03	Square centimeters
Square feet	0.092903	Square meters
Square yards	0.836127	Square meters
Acres	*0.40469	Hectares
Acres	*4,046.9	Square meters
Acres	*0.0040469	Square kilometers
Square miles	2.58999	Square kilometers
	VOLUME	
Cubic inches	16.3871	Cubic centimeters
Cubic feet	0.0283168	Cubic meters
Cubic yards	0.764555	Cubic meters
	CAPACITY	
Fluid ounces (U.S.)	29.5737	Cubic centimeters
Fluid ounces (U.S.)	29.5729	Milliliters
Liquid pints (U.S.)	0.473179	Cubic decimeters
Liquid pints (U.S.)	0.473166	Liters
Quarts (U.S.)	*946.358	Cubic centimeters
Quarts (U.S.)	*0.946331	Liters
Gallons (U.S.)	*3,785.43	Cubic centimeters
Gallons (U.S.)	3.78543	Cubic decimeters
Gallons (U.S.)	3.78533	Liters
Gallons (U.S.)	*0.00378543	Cubic meters
Gallons (U.K.)	4.54609	Cubic decimeters
Gallons (U.K.)	4.54596	Liters
Cubic feet	28.3160	Liters
Cubic yards	*764.55	Liters
Acre-feet	*1,233.5	Cubic meters
Acre-feet	*1,233,500	Liters

Table II

QUANTITIES AND UNITS OF MECHANICS

Multiply	By	To obtain
MASS		
Grains (1/7,000 lb)	64.79891 (exactly)	Milligrams
Troy ounces (480 grains)	31.1035	Grams
Ounces (avdp)	28.3495	Grams
Pounds (avdp)	0.45359237 (exactly)	Kilograms
Short tons (2,000 lb)	907.185	Kilograms
Short tons (2,000 lb)	0.907185	Metric tons
Long tons (2,240 lb)	1,016.05	Kilograms
FORCE/AREA		
Pounds per square inch	0.070307	Kilograms per square centimeter
Pounds per square inch	0.689476	Newtons per square centimeter
Pounds per square foot	4.88243	Kilograms per square meter
Pounds per square foot	47.8803	Newtons per square meter
MASS/VOLUME (DENSITY)		
Ounces per cubic inch	1.72999	Grams per cubic centimeter
Pounds per cubic foot	16.0185	Kilograms per cubic meter
Pounds per cubic foot	0.0160185	Grams per cubic centimeter
Tons (long) per cubic yard	1.32894	Grams per cubic centimeter
MASS/CAPACITY		
Ounces per gallon (U.S.)	7.4893	Grams per liter
Ounces per gallon (U.K.)	6.2362	Grams per liter
Pounds per gallon (U.S.)	119.829	Grams per liter
Pounds per gallon (U.K.)	99.779	Grams per liter
BENDING MOMENT OR TORQUE		
Inch-pounds	0.011521	Meter-kilograms
Inch-pounds	1.12985×10^6	Centimeter-dynes
Foot-pounds	0.138255	Meter-kilograms
Foot-pounds	1.35582×10^7	Centimeter-dynes
Foot-pounds per inch	5.4431	Centimeter-kilograms per centimeter
Ounce-inches	72.008	Gram-centimeters
VELOCITY		
Feet per second	30.48 (exactly)	Centimeters per second
Feet per second	0.3048 (exactly)*	Meters per second
Feet per year	$*0.965873 \times 10^{-6}$	Centimeters per second
Miles per hour	1.609344 (exactly)	Kilometers per hour
Miles per hour	0.44704 (exactly)	Meters per second
ACCELERATION*		
Feet per second^2	*0.3048	Meters per second^2
FLOW		
Cubic feet per second (second-feet)	*0.028317	Cubic meters per second
Cubic feet per minute	0.4719	Liters per second
Gallons (U.S.) per minute	0.06309	Liters per second
FORCE*		
Pounds	*0.453592	Kilograms
Pounds	*4.4482	Newtons
Pounds	$*4.4482 \times 10^5$	Dynes

Table II—Continued

Multiply	By	To obtain
WORK AND ENERGY*		
British thermal units (Btu)	*0.252	Kilogram calories
British thermal units (Btu)	1,055.06	Joules
Btu per pound	2.326 (exactly)	Joules per gram
Foot-pounds	*1.35582	Joules
POWER		
Horsepower	745.700	Watts
Btu per hour	0.293071	Watts
Foot-pounds per second	1.35582	Watts
HEAT TRANSFER		
Btu in./hr ft^2 degree F (k, thermal conductivity)	1.442	Milliwatts/cm degree C
Btu in./hr ft^2 degree F (k, thermal conductivity)	0.1240	Kg cal/hr m degree C
Btu ft/hr ft^2 degree F	*1.4880	Kg cal m/hr m^2 degree C
Btu/hr ft^2 degree F (C, thermal conductance)	0.568	Milliwatts/cm^2 degree C
Btu/hr ft^2 degree F (C, thermal conductance)	4.882	Kg cal/hr m^2 degree C
Degree F hr ft^2/Btu (R, thermal resistance)	1.761	Degree C cm^2/milliwatt
Btu/lb degree F (c, heat capacity)	4.1868	J/g degree C
Btu/lb degree F	*1.000	Cal/gram degree C
Ft^2/hr (thermal diffusivity)	0.2581	Cm^2/sec
Ft^2/hr (thermal diffusivity)	*0.09290	M^2/hr
WATER VAPOR TRANSMISSION		
Grains/hr ft^2 (water vapor) transmission)	16.7	Grams/24 hr m^2
Perms (permeance)	0.659	Metric perms
Perm-inches (permeability)	1.67	Metric perm-centimeters

Table III

OTHER QUANTITIES AND UNITS

Multiply	By	To obtain
Cubic feet per square foot per day (seepage)	*304.8	Liters per square meter per day
Pound-seconds per square foot (viscosity)	*4.8824	Kilogram second per square meter
Square feet per second (viscosity)	*0.092903	Square meters per second
Fahrenheit degrees (change)*	5/9 exactly	Celsius or Kelvin degrees (change)*
Volts per mil	0.03937	Kilovolts per millimeter
Lumens per square foot (foot-candles)	10.764	Lumens per square meter
Ohm-circular mils per foot	0.001662	Ohm-square millimeters per meter
Millicuries per cubic foot	*35.3147	Millicuries per cubic meter
Milliamps per square foot	*10.7639	Milliamps per square meter
Gallons per square yard	*4.527219	Liters per square meter
Pounds per inch	*0.17858	Kilograms per centimeter

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ABSTRACT

Hydraulic model studies were made to determine the flow characteristics of the Crystal Dam spillway, plunge pool, and outlet works. The initial flip-bucket spillway was tested, and the bucket exit modified by extending the 15-foot (4.57-meter) radius beyond the bucket invert to a 4:1 tangent at the bucket lip. The modified bucket was required to adequately flip the spillway jet into the plunge pool. A 15-foot (4.57-meter) high deflector wall was placed at the downstream end of the excavated rock plunge pool to deflect the high energy spillway jet from the 3 to 1 riprapped slope leading to the downstream river channel. A vortex appearing in the preliminary horizontal bellmouth transition design from the intake towers to the 54-inch (1,371-mm) outlet works conduits was eliminated by raising the floor of the intake tower closer to the invert of the bellmouth. This design was later improved by using one intake tower for both outlet conduits placed at the same elevation. Discharge coefficients are given for both the spillway and the submerged 48-inch (1,219-mm) jet-flow gates.

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Hydraulic model studies were made to determine the flow characteristics of the Crystal Dam spillway, plunge pool, and outlet works. The initial flip-bucket spillway was tested, and the bucket exit modified by extending the 15-foot (4.57-meter) radius beyond the bucket invert to a 4:1 tangent at the bucket lip. The modified bucket was required to adequately flip the spillway jet into the plunge pool. A 15-foot (4.57-meter) high deflector wall was placed at the downstream end of the excavated rock plunge pool to deflect the high energy spillway jet from the 3 to 1 riprapped slope leading to the downstream river channel. A vortex appearing in the preliminary horizontal bellmouth transition design from the intake towers to the 54-inch (1,371-mm) outlet works conduits was eliminated by raising the floor of the intake tower closer to the invert of the bellmouth. This design was later improved by using one intake tower for both outlet conduits placed at the same elevation. Discharge coefficients are given for both the spillway and the submerged 48-inch (1,219-mm) jet-flow gates.

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